CLASSIFICATION OF ROCK ACCORDING TO ITS MECHANICAL PROPERTIES

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by

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ABSTRACT

The thesis is in two parts. The first examines tests and observations in an attempt to select those most suitable for rock classification; it is assumed in this context that the most important role of such a classification is in the mapping of in situ variations in the mechanical character of rock materials. Geological maps provide insufficient mechanical information.

Criteria for the selection of classification tests and observations are established by considering their role in detail. Traditional geological methods of classifying rocks are reviewed to determine those observations that decide rock nomenclature, hence the mechanical implications of geological names. A range of physical and mechanical tests are then reviewed to determine which are necessary as a supplement to observation.

The system proposed is based on two properties, namely brokenness (average fracture spacing) and strength (assessed using a point load strength test). This two-dimensional classification is supplemented by durability testing, and by a modified and simplified form of textural and mineralogical description.

The second part of the thesis is concerned with the triaxial strength of intact rock materials, and attempts to classify a range of rocks according to their triaxial strength behaviour. In this case classifica--tion is intended not for routine rock description or mapping, but to demonstrate important differences in mechanical performance.

Equipment and techniques developed to simplify triaxial testing are described. The purpose and properties of strength criteria are discussed and an attempt is made to find an optimum curved criterion, one that gives improved strength prediction over the customary Coulomb-Navier straight line. A curvilinear criterion is selected and developed, and is shown to give practical improvements in strength prediction. Rock classifications are presented based both on the Coulomb-Navier parameters and on those of the curvilinear criterion.

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CONTENTS

PART I Rock classification for mapping

PAGE

1 Role and philosophy of rock testing and classification
- (a) Introduction 7
- (b) Role of testing and classification in rock engineering 7
- (c) Classification, classification criteria and rock 'character' 11
- (d) Optimum classification - the use of multivariate statistical methods 13
- (e) Practical criteria for the selection of classification tests and observations 20

2 Geological methods of classification; their application in engineering
- (a) Introduction 22
- (b) Sandstones and mudstones 23
- (c) Limestones and chemical sediments 27
- (d) Igneous rocks 31
- (e) Metamorphic rocks 36
- (f) Adapting geological methods to suit engineering problems 41

3 Rock tests - the extension of visual classification
- (a) Introduction 47
- (b) Aids to visual examination 49
- (c) Quantification of observed properties 53
- (d) Experimental determination of pore and density parameters 61
- (e) Strength testing 69
- (f) Durability testing 77

4 Field studies, conclusions and recommendations
- (a) Field studies 83
- (b) General outline of proposals for rock description 89
- (c) Proposal for a concise sample description and a two-dimensional classification 92
- (d) Fissure description 99
- (e) Textural and mineralogical description 101
- (f) Description using test data 105
- (g) Concluding remarks 110

PART 2 Triaxial strength of Rock

5 Development of triaxial testing techniques
- (a) Triaxial strength testing 113
- (b) The triaxial cell 113
- (c) Hydraulics 115
- (d) Instrumentation 119
- (e) Specimen preparation and storage 123
- (f) Triaxial testing procedure 124
- (g) Summary of results 127

6 Strength criteria and triaxial strength classification
- (a) The purpose of a strength criterion 129
- (b) Attributes of the strength surface 131
- (c) Mechanisms of failure 133
- (d) The selection of a triaxial strength criterion 135
- (e) Methods of estimating parameters and comparing prediction errors 137

(f) Comparison of linear and curved models 140
(g) Classification of rocks on the basis of their triaxial strength 140
(h) Mohr transformations of the straight line and curved criteria 145
(i) Alternative formulations of the curved criterion 146

BIBLIOGRAPHY 149

PART 1 ROCK CLASSIFICATION FOR MAPPING

1. ROLE AND PHILOSOPHY OF ROCK TESTING AND CLASSIFICATION

(a) Introduction

Philosphical discussions are rarely read, and some explanation is required to account for the inclusion of such a discussion as the first chapter of this thesis. This chapter is devoted to a search for criteria that will later be used to evaluate and select tests, observations and classification systems. Chapter 1 considers what is required of a test, observation or classification; chapters 2 and 3 review the avaible techniques and the selection is made in chapter 4.

Tests cannot be evaluated without a clear definition of their purpose so that the first part of this chapter discusses the role of testing, observation and classification in rock engineering. Often a classification is designed to meet the needs of a *particular* class of engineering problem or type of rock. This thesis attempts a more generally applicable rock description scheme to meet the needs of a broader range of problems; the range of problems to be considered will be outlined in the course of this introductory chapter.

In addition it will be necessary to consider in detail what is meant by 'classification'; the differences between 'natural' and 'artificial' classes of materials and the various stages in classification such as sampling, testing and observation, subdivision and nomenclature. The way in which a rock sample may be 'characterised' by means of a set of tests or observations particularly deserves closer study. The author hopes that he will be forgiven for discussing these 'philosophical' topics and also for including some, perhaps obscure examples, by way of illustration. These have been taken from the disciplines of behavioural psychology and pure geology where the science of classification appears to be rather more advanced than elsewhere.

(b) Role of testing and classification in rock engineering

A map showing the variation in rock quality over the extent and depth of an engineering site is essential prior to designing a rock structure, selecting rock material for aggregate, choosing for rock excavation, and for the many other tasks of the rock engineer. The rock map provides a 'feel' for rock quality, allowing comparison of one site with the next so that past experience may be directed towards the solution of current design problems. Construction of a map requires classification of the rock into mappable units, and classification according to mechanical properties requires both testing and observation. The part played by testing and observation, classification and mapping in the solution of engineering problems is summarised diagrammatically in Figure 1.

Conventional geological maps are perhaps less than adequate for use in the process of solving the problems of rock engineering. Geological maps are based on traditional systems of geological classification, these

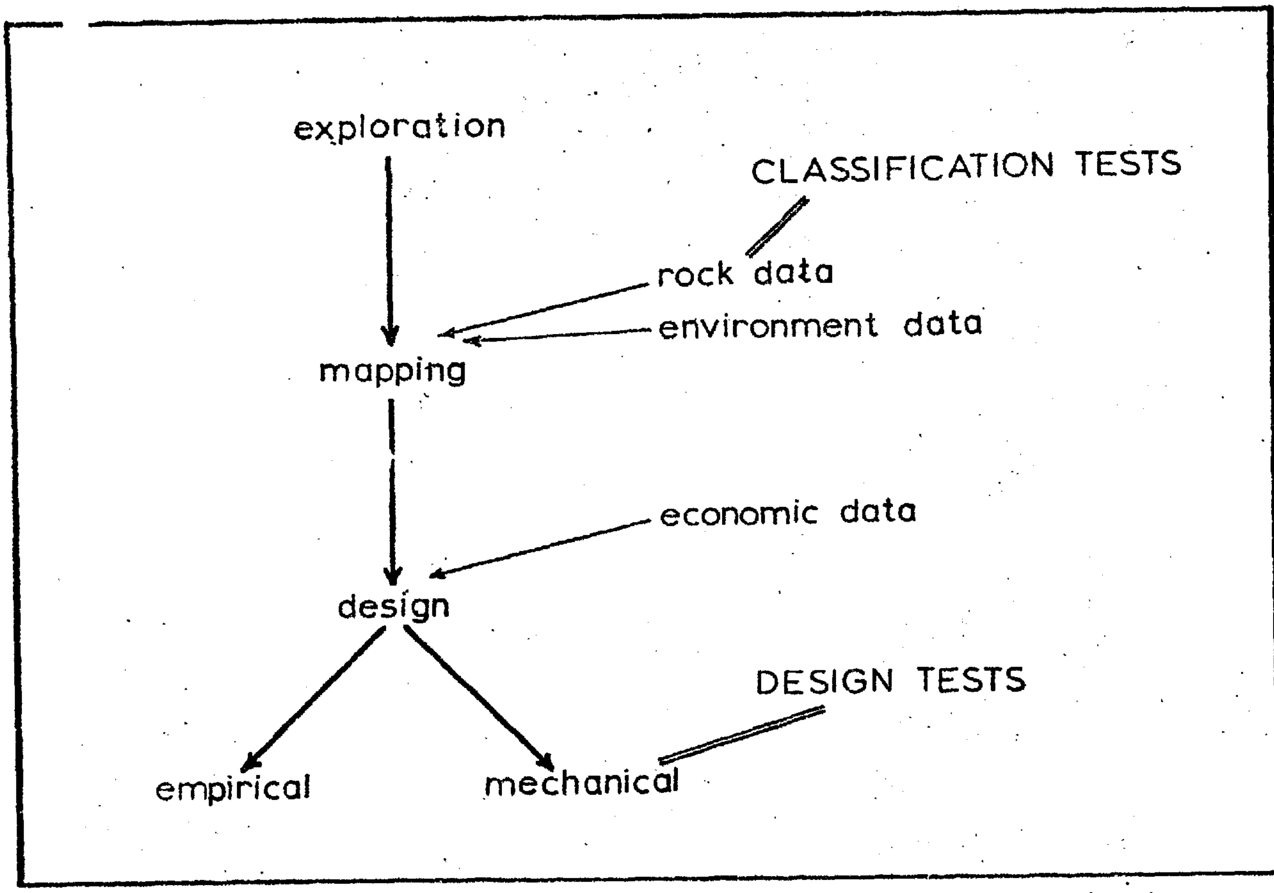


Figure 1: Role of Rock Testing
Tests are required to characterise the rock for mapping purposes early in the project. Later more sophisticated design tests may be necessary.

classifications in turn being based on observations of rock properties, particularly colour, texture, grain size and mineral composition. The geological classification is designed *to reflect rock history* rather than its future mechanical potential and when applied to problems that require a mechanical insight into the nature of the rock material, these functions often conflict with those of the engineer. A limestone or a mudstone stratum may possess mechanical properties over a very wide range of the total spectrum for rock materials. In these cases the geological map provides too little information for engineering purposes. Conversely, there exist over two thousand igneous rock names, many of such species being very similar in their mechanical nature. In these cases the geological map provides a surfeit of information.

Rock testing can be brought to bear on rock classification, and indeed simple tests have long been employed by geologists; tests such as scratching rock with a penknife or breaking it with a hammer. Mechanical tests such as these, tests that subject the rock to a mechanical stimulus and observe its response, are more direct methods of establishing the mechanical nature of the material, although it must be appreciated that one can also deduce mechanical nature from observations alone. There are as many pitfalls in adopting a classification based entirely on mechanical tests as there are in adopting one based solely on visual observations, and an optimum classification for rock engineering is likely to employ a combination of both observations and tests. Some properties of fundamental importance to engineering such as the 'hardness' of rock material, are best assessed with the aid of tests. Others such as fracture spacing can hardly be tested, and must be observed. Many properties fall in the intermediate categories and for these a choice must be made between observation and test techniques, or the two combined.

It is important to distinguish at this stage between *classification tests* and *design tests*, since the two types of test can have very different requirements. Classification tests are carried out at the stage of site exploration for the purpose of compiling the basic maps showing the nature of in situ rock, while design tests are carried out at a later stage to obtain information for use in design calculations. Ideally classification testing and mapping should precede design testing, since the rock map provides information necessary to determine the number and type of design tests required. Design tests (field jacking tests to determine deformability modulus, pressure tunnel tests, triaxial strength tests for example) are typically expensive and time consuming; so are usually carried out in tens rather than hundreds; it is most important that the few tests possible on a limited budget be adequately chosen and sited. Classification tests, on the other hand, place a premium on speed and simplicity since they must be carried out in sufficient number to allow use of the data for mapping the variation in rock character. Zones of rock are delineated on the map using these tests and observations, allowing results of design tests to be extrapolated in a rational and reliable way. Occasionally design tests are sufficiently simple to be employed for the purpose of rock classification and mapping, and there is no strict limit on the type of test that may be used for either purpose.

Referring again to Figure 1, there appear to be two alternative approaches to design, empirical or mechanical. Either may be adopted but often both are combined and supplement each other. Numerical and model techniques have been best developed in their application to the design of rock structures, while other rock engineering problems are usually tackled by empirical techniques based on the accumulated experience of previous projects. Exceptions to this rule exist, however, such as the design of blasting patterns using the principles of mechanics, but they are rare. Empirical design methods may be based solely on subjective judgement, or alternatively classification data may be used in emperical design equations that have been formulated from the accumulated data of previous projects. Classification data such as hardness and fracture spacing may, for example, be correlated with the ease of excavation of rock material so that plant requirements may be estimated from the map of rock character. Even in such cases 'design tests', here taking the form of full scale field excavation trials, may be used to supplement and confirm the empirical predictions. Empirical predictions of this sort rely heavily on the weight of accumulated data relating classification papameters to field performance, so that their reliability improves with experience in using a particular classification scheme. Mechanical, as well as empirical design, can only be adequately carried out with the aid of a rock quality map, which assists in the optimum choice of mechanical models to represent the engineering problem; such models are usually simplifications of the real situation, and the subjective judgements necessary for their formulation decide the relevance of results to the actual, as opposed to the idealised problem.

Such calculations must invariably be tempered by empirical judgement.

A wide range of rock engineering problems can be tackled with the assistance of maps of rock character variation, and usually an engineering project will encounter several such problems. A highway project, for example, is often associated with problems of cutting and tunnel stability, excavation, bridge pier foundation conditions and the selection of rock material for use as aggregate. If a different system of rock classification is to be used for each problem then much confusion and duplication of effort can arise. It would be of considerable benefit if the same classification system, hence the same rock quality map, could be used to tackle the whole range of problems. Due in part to a lack of communication, in part to a belief that a special problem deserves a special classification, there has been a trend towards the development of a multiplicity of classification and testing systems. A classification should to some extent be tailored to suit the application, but if classification criteria are carefully selected at the outset a change of emphasis from one application to the next, rather than a complete reorganisation, should suffice. Broad terms of reference are therefore necessary in designing a classification system. An attempt has been made to list typical engineering problems that call for rock classification (Figure 2), a list to be borne in mind when selecting tests and observations that will form the basis for classification, and in assessing the merits of such a classification once developed.

SLOPE STABILITY - shallow and deep slides; strength of slide surfaces; safety factors

STABILITY OF UNDERGROUND EXCAVATIONS - design of excavations- squeezing and convergence; rockburst alleviation

FOUNDATION STABILITY - bearing capacity and settlement of rafts, piers piles, bridge and dam foundations; performance of roadways under traffic; settlement due to subsurface excavation

WEATHERING & DURABILITY - ravelling of cut slopes and underground openings; deterioration of riprap and fill; settlement of embankments; swelling and heave of foundations and pavements

SUPPORT - design of rockbolting systems; design of tunnel linings and supports; design of retaining walls and cable anchors

EASE OF EXCAVATION - ripability and performance of excavating plant, tunneling machines, drillability; design of blasting patterns; design of crushing and grinding processes

ROCK MATERIAL SELECTION - bond strength, polishing and 'soundness' of concrete aggregate and roadstone; selection of riprap and fill; quality of building stone

FLUID FLOW - oil, gas and water storage and extraction; waste disposal; design of drainage systems, design of grouting operations; leakage from dams; uplift, pore and cleft water pressures.

Figure 2: Problems of Rock Engineering
Ideally a single scheme for rock description and classification would meet all needs. In practice the scheme must be to some extent modified to suit the application and the local rock.

(c) Classification, Classification Criteria and "Rock Character"

In order to discuss the role of rock classification in engineering it was necessary to talk loosely in terms of rock 'quality' or 'character'. Such terms should be defined more closely. Use will be made of an analogy between the character of rock and the character or personality of a human being, an analogy that may prove convenient if somewhat limited in scope. Systemmatic study and use of classification techniques has been taken further by the behavioural psychologist than in any other discipline, with the possible exception of geological science, where such techniques have gained increasing momentum in recent years. Hence the merit of comparing rock character with human character is greater than might be supposed.

The *personality profile* of the class 'homosexuals' is reproduced in Figure 3 (from Cattell's 'Scientific analysis of personality', 1965). Classification of each person is based on scores allocated for each of sixteen 'personality traits' by means of tests and observations. The scores of homosexuals in such tests are compared with those of the human population as a whole, the latter represented by the horizontal line on the profile of Figure 3, and it can be seen that homosexuals record high scores for certain attributes, while being deficient in other respects. The personality profile is, both in appearance and in practice, a 'key' to classification of human personality. Other samples of the human population would show a different profile.

The classification of rock can proceed in a similar manner, each rock sample being allocated a set of scores based on testing and observation, the character of the rock being reflected by high scores in some tests, low ones in others. The depth of characterisation achieved depends on the choice and number of attributes observed, and the design of a classification scheme involves the selection of the minimum number of attributes to reflect the maximum of relevant character information. The recognition of 'relevant' attributes and the design of testing and observational procedures for their measurement are two of the steps essential for rational design of a classification system.

The number of tests and observations that have been devised to reflect variation in either rock or human character is very large. Each may be used for classification, but many must be rejected if the scheme is to be simple. Classification may be simplified either by selecting individual 'index' observations to represent others that are closely related, or by compounding groups of closely related observations into single scores. The 'index test' approach has up to now been adopted almost exclusively in the field of rock engineering, while the compound 'attribute score' technique is prevalent in psychological studies. To examine either approach it is necessary to study the inter-correlation or 'clustering' of groups of related observations.

An observation, for example porosity, may be represented by a vector on a graph whose axes are those of observation scores for each of two samples of rock. The terminus of the vector is located by co-ordinates equal to the scores of each sample in the observation or test that the vector represents. Figure 4 demonstrates that the vectors representing closely related sets of observations tend to cluster together; a high Young's Modulus, for example is associated in a given

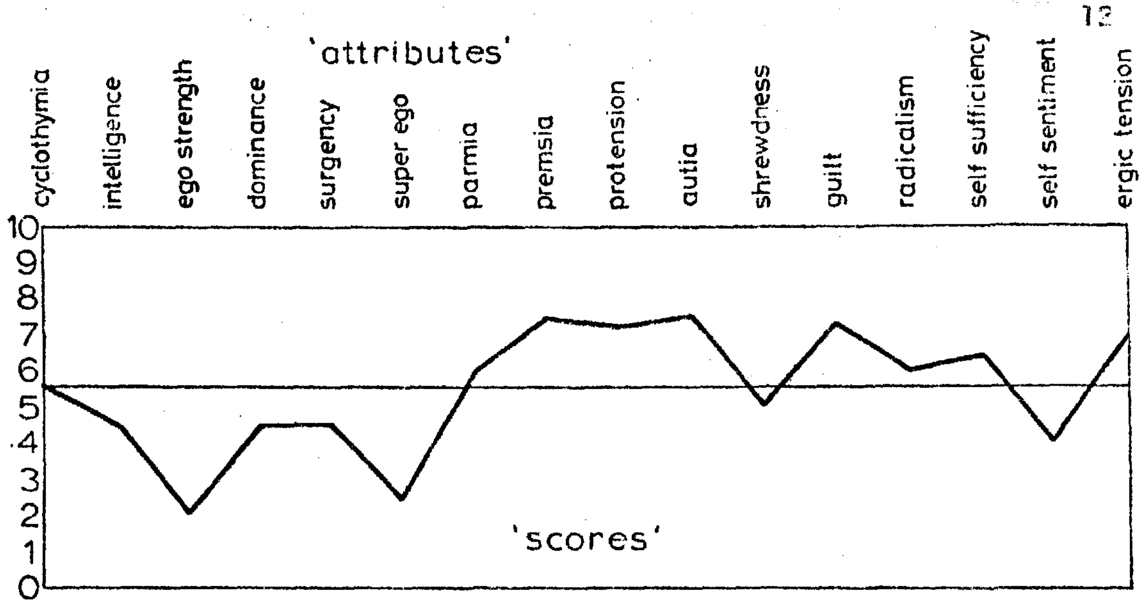


Figure 3: Personality Characterisation using a set of Observations (A personality profile of homosexuals after Cattell, 1965) Despite the obscure terminology of this example some useful parallels may be drawn with characterisation and classification of rocks.

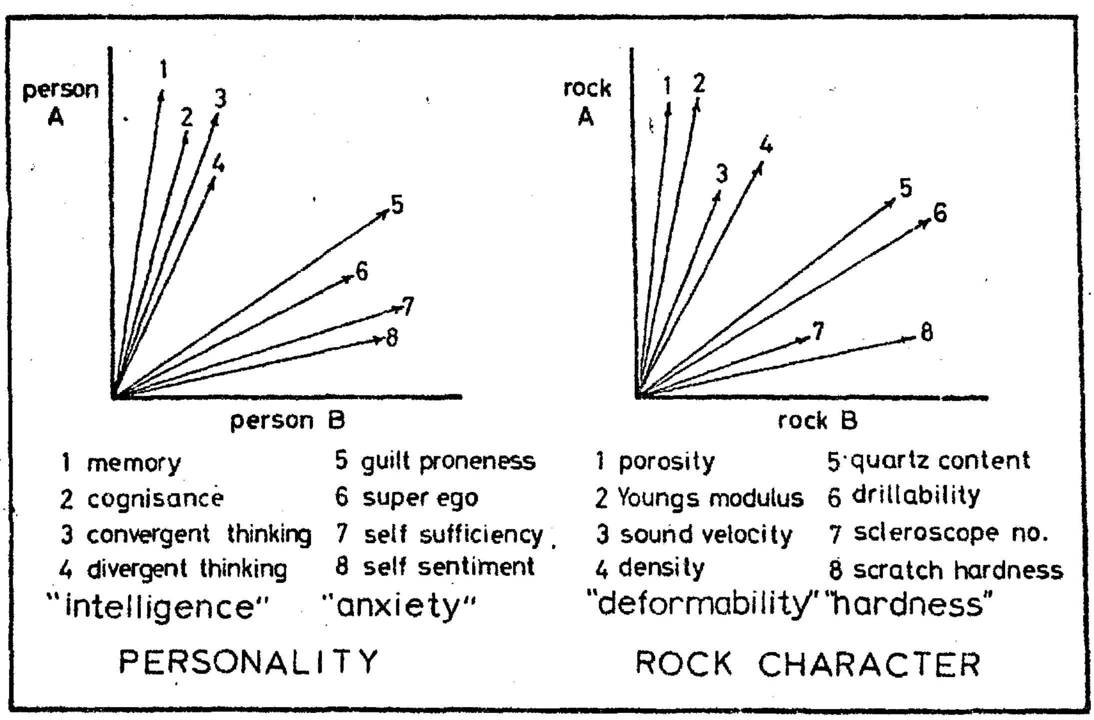


Figure 4: The clustering of groups of related observations Economic classification dictates use of a single index to represent each cluster, alternatively several closely related observations may be lumped together to form a single attribute score.

rock sample with a low porosity value, also with high values for density and sound velocity. The cluster of related observations may be represented by a single 'factor' which in this case we may, for convenience, term 'deformability'. A numerical value for the factor may be obtained by compo ding scores for the individual observations in the cluster in a way that will be outlined later. The recognition and compounding of observation clusters is more difficult when considering more than two samples, and computational techniques must then replace graphical methods. These techniques, outlined in a subsequent paragraph, can be of considerable assistance in helping to construct and to evaluate classification systems. Recognition of observation clusters and interpretation of their physical significance is of fundamental importance to the erection of a classification scheme, whether or not such sophisticated techniques are employed.

A further example of the application of 'observation cluster analysis' is provided by the scheme used to quantify the personality trait 'intelligence' by means of an 'intelligence quotient' (I.Q.) score. The first step taken by the psychologist was to recognise the existence of an observation cluster that could, on the basis of physical experience be termed intelligence. This process of isolating the basic personality traits or clusters from among the complete spectrum of psychological observations may be referred to as the 'first level' of cluster analysis. The next step was to carry out further cluster analysis using only the observations in the 'intelligence' cluster. This brought to light sub-groups of observations such as 'memory' and 'convergent thinking' which can be regarded as the attributes that go to make up intelligence. (Figure 4). An intelligence test could then be designed by careful selection of individual observations to represent each attribute adequately.

If two types of observation are closely related then one may be predicted from the other, and where economy of effort is of prime importance there is little to be gained from observing both. This motivates the use of index tests for rock classifications. The index test is usually selected from among the cluster of related observations on the basis of its simplicity and is used to predict values for the more difficult and time consuming observations within the cluster (values that may be required for the purposes of design). In terms of previous terminology the 'classification test' is used to predict results for the 'design test'.

Although the selection of a single index test is an attractive proposition particularly for practical rock classification and mapping, the accuracy of prediction of design test information can often be substantially improved by compounding more than one index observation, particularly if suitable index observations are far removed in the cluster from the quantity to be predicted. D'Andrea, Fischer and Fogelson (1965) used nine index observations to predict the uniaxial compressive strength of rock, employing multiple linear regression analysis for this purpose. Statistical methods may be used to decide which observations give best prediction, but the final selection of index tests must also be based on deciding which properties are easiest to observe.

(d) Optimum Classification - the use of Multivariate Statistical Methods

The selection of a set of tests and observations suitable for use as classification criteria is only the first step in designing a classification system. The set of classification tests is usually too extensive for practical use and must be simplified by selecting index tests to represent other closely related types of observation, or by compounding closely related clusters of observations into single classification 'factors'.

Following the selection and simplification of a set of classification tests it is helpful to subdivide the classification axes to form discreet compartments or categories. It is possible then to allocate a class name to each compartment, although a classification based on several observations may result in a large number of compartments that precludes the use of a simple nomenclature system. The stages in erecting a classification scheme may be summarised as follows:

(1) Selection of tests and observations

(2) Simplification - cluster analysis, compounding & index selection

(3) Subdivision

(4) Nomenclature

Subdivision into compartments or 'classes' can only be efficient if the compartment limits correspond fairly well with the boundaries of *natural families* or clusters of data points. Arbitrary selection of class boundaries can lead to compartments that are 'empty' (samples with the required properties are rare or non existent), or to the dissection of a natural class of materials into two or more sub-classes. Class subdivision should be adjusted to recognise and conform as closely as possible to the natural clustering of sample test scores. This concept, and its effect on mapping is illustrated in Figure 5, reproduced from the paper by Imbrie & Purdy, 1962. These authors posed the question "is the classification scheme being evaluated as simple as possible? By a simple scheme we mean a classification in which there is exactly one category for each group of samples found to be separated from other groups by discontinuities in the ranges of their observable properties". Even if the scatter of test results indicates a continuous gradation in properties rather than discreet clusters of results (as is often the case in practice) there will invariably be a tendency towards clustering. Some areas of 'observation space' will be more densely populated than others. Subdivision must take these fluctuations of population density into account.

Cluster analysis is used, whether consciously or intuitively, at two stages in the erection of a classification system. *Clusters of related observations* must be noted to simplify the classification, for example by the selection of index parameters. *Clusters of related samples* must be recognised if artificially positioned subdivisions are to correspond as closely as possible to the boundaries of natural families of materials. If observations (samples) are to be compared on the basis of only two samples (observations), then clusters or trends can easily be recognised with the help of graphical presentation. With multidimensional data it is possible to 'plot everything against everything else' in the search for clusters, but apart from the tedium involved this approach amounts to viewing the data only in directions parallel to the classification axes. Often the clusters overlap when viewed in these directions and only appear discrete when viewed obliquely. At this stage multivariate statistical techniques may be found to assist by revealing systematic patterns and trends in an otherwise obscure table of test results. It must be remembered, however, that the results of multivariate statistical analysis must be interpreted with the help of subjective judgement and that it is all too easy to draw incorrect conclusions from patterns that might emerge.

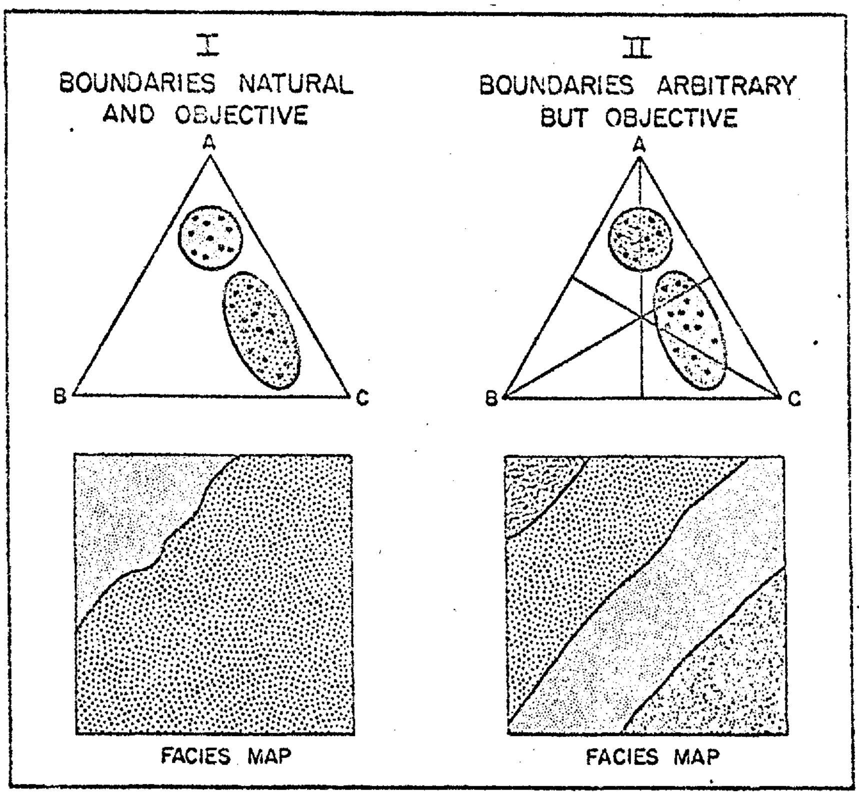


Figure 5: Natural and Artificial Subdivision (after Imbrie and Purdy 1962) Artificial subdivisions should correspond closely to natural subdivisions to avoid a false picture of rock variation.

Imbrie and Purdy's work provides an excellent example of the benefit gained by using multivariate statistical methods in classification and mapping. Two hundred samples of sea bed sediments were collected in an area of the Bahama banks, each sample being characterised by means of twelve observations relating to sediment composition in terms of molluscs, corals, pellets etc. By means of a multivariate technique known as 'factor analysis' the twelve observations were reduced to four compound 'factors' with the loss of only 11% of the original information value. The use of more than four factors was deemed unnecessary since the additional information gained would not have justified the loss of simplicity. Clusters of data points that were obscured by the complexity of the original data table could readily be recognised when the results were plotted on the factor axes. These clusters defined the mapping units used to construct a map of facies variation for the area of sea bed (Figures 6 and 7).

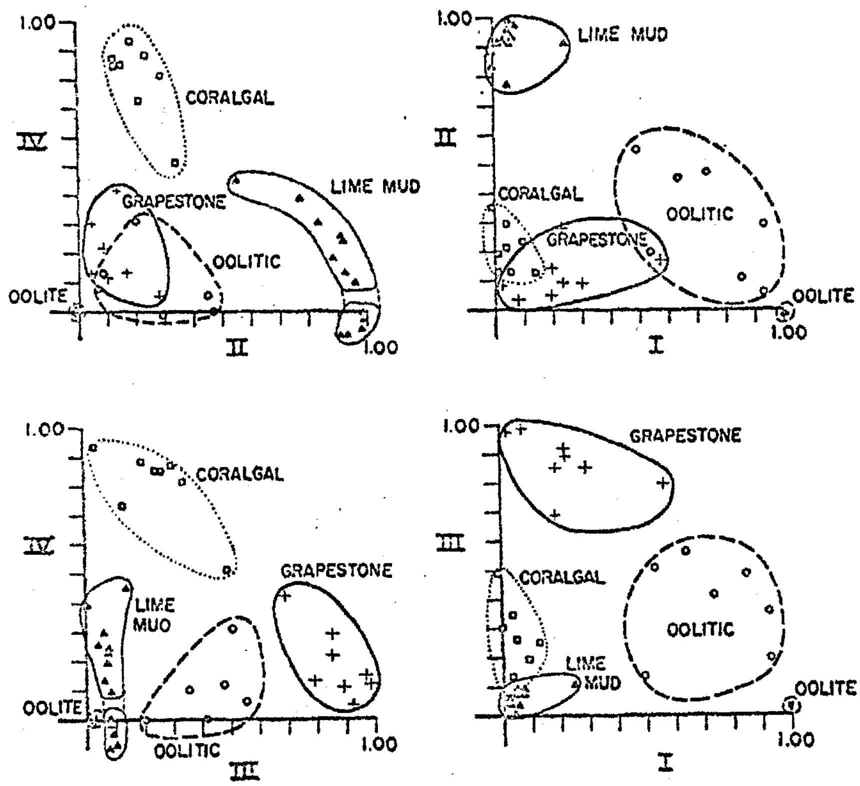


Figure 6: Natural clusters recognised by plotting on factor axes. Natural subdivisions are difficult to decipher from data based on more than two observations. Multivariate statistics can help. This example, also taken from Imbrie and Purdy, is a factual one; a map based on rock classes that emerged from the statistical study is presented in Figure 7.

The study described in this thesis did not employ multivariate statistical techniques, since it did not proceed beyond the stage of selecting tests and observations. This in itself proved a formidable task since criteria for the selection of tests had to be established; there were many types of test and observation from which to choose and little could be gleaned from the published literature as to the relative merits of these observations without an extensive and detailed study. The practical value of an engineering rock classification system must inevitably depend on the practical and theoretical merit of the set of tests and observations upon which it is based, so that these techniques were studied in detail before any attempt to collect and analyse data.

Further development towards establishing a system for engineering classification of rocks will require some further refinement of test and observation techniques, but mainly the application of these techniques to a wide range of rock materials and the study and analysis of the data obtained. Multivariate statistical methods will amost certainly prove of assistance at this stage, and these will now be briefly reviewed with reference to publications in which the techniques are described and developed.

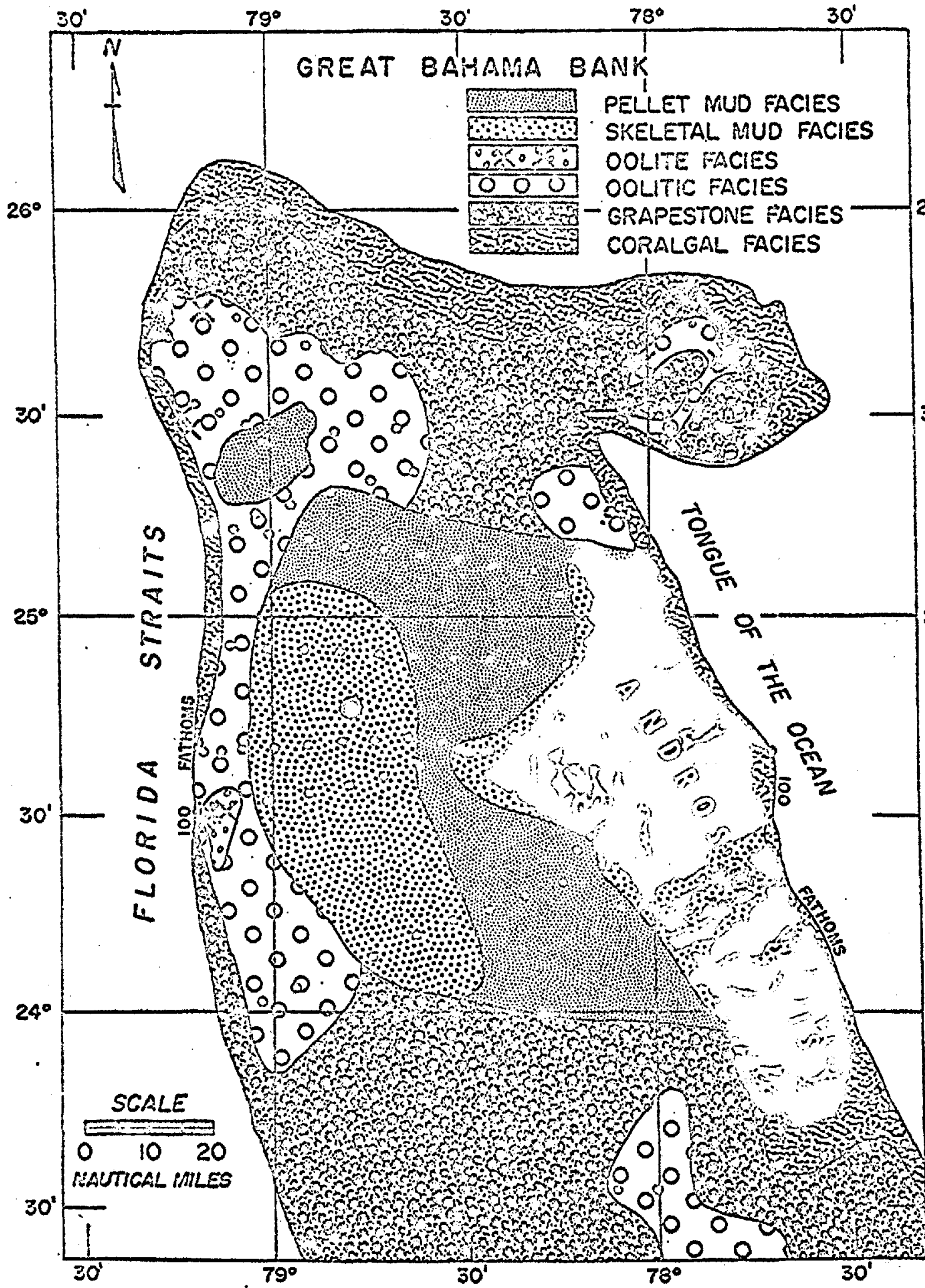


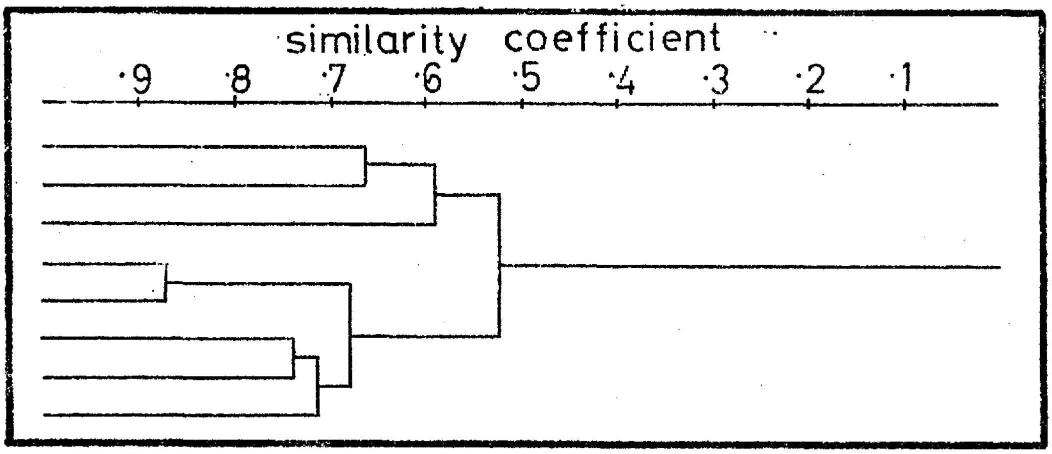
Figure 7: Map of the Great Bahama Bank based on a classification established using factor analysis.
Taken from Imbrie and Purdy, 1962. The mapping units are the classes or clusters of Figure 6.

The techniques have fortunately been in use over a period of several decades, particularly in the fields of behavioural psychology and pure geology, and have also been applied with considerable success by geochemists, mining and structural geologists at Imperial College. They have even been applied to analysis of mechanical property data for rocks, for example the paper by Mutmansky and Singh, 1967, but these studies have been based on a limited and possibly an inadequate set of tests and observations. The techniques are described in Harbaugh & Merriam's textbook 'Computer Applications in Stratigraphic Analysis' (1968), particularly in chapter 7 entitled 'classification systems'. A further textbook 'Multivariate Procedures for the Behavioural Sciences' by Cooley and Lohnes (1962) also provides a clear account of the techniques available. Papers relating to the development and application of these methods appear regularly in the 'Computer Contribution' series published by the State Geological Survey, University of Kansas, for example contribution No. 7, 'Proceedings of a Colloquium on Classification Procedures' (Merriam, 1966) and the recent contribution No. 31 by Demirmen (1969) on evaluation and improvement of classifications.

Similarity coefficients, the product moment correlation coefficient, the distance coefficient, cosine theta coefficient and the matching coefficient for example, allow quantitative comparison between samples or variables prior to grouping together those that are similar. The choice of similarity coefficient depends upon whether sample clusters ('Q-mode' analysis) or variable clusters ('R-mode' analysis) are to be studied, and also on the qualitative or quantitative nature of the data. A first step in cluster analysis is to compute a matrix of similarity coefficients from the raw data.

Dendograms provide the simplest form of cluster analysis. Comparison of 100 samples results in a matrix of 4950 similarity coefficients, so that some re-structuring and simplification is essential for interpretation. The dendogram, a graphical representation of the hierarchial relationship between variables or samples on the basis of their similarity, provides a simple aid to interpretation (Figure 8).

Factor analysis embraces a class of techniques whose aim is to discern simplifying relationships in complex data. It enables clusters to be identified in multidimensional classification space, and allows a reduction in the number of variables on which classification is based. The simplification is organised so that the new variables ('factors') contain a maximum of the information available in the original data table. Factor axes are positioned in multidimensional space to correspond as closely as possible to directions of maximum variance in the data. The axes are mutually orthogonal (independent or uncorrelated), the first factor delineating the most general pattern of relationships in the similarity coefficient matrix, the second factor delineating the next most general pattern and so on. To account for 100% of the original information it is usually necessary to use as many factors as there were variables, but a lesser number of factors results in simplification at the expense of some loss of information. It may be necessary to sacrifice some degree of representation to gain simplicity, or vice versa. A further step is to rotate the factor axes so that they correspond more closely with the centroids of clusters and therefore assume greater physical significance. The axes may remain orthogonal during rotation, or may be allowed to become oblique. Orthogonal rotation ensures that the final factors are uncorrelated, while oblique rotation results in a closer correspondence between factors and clusters, and the various schools of though favour one or other system.



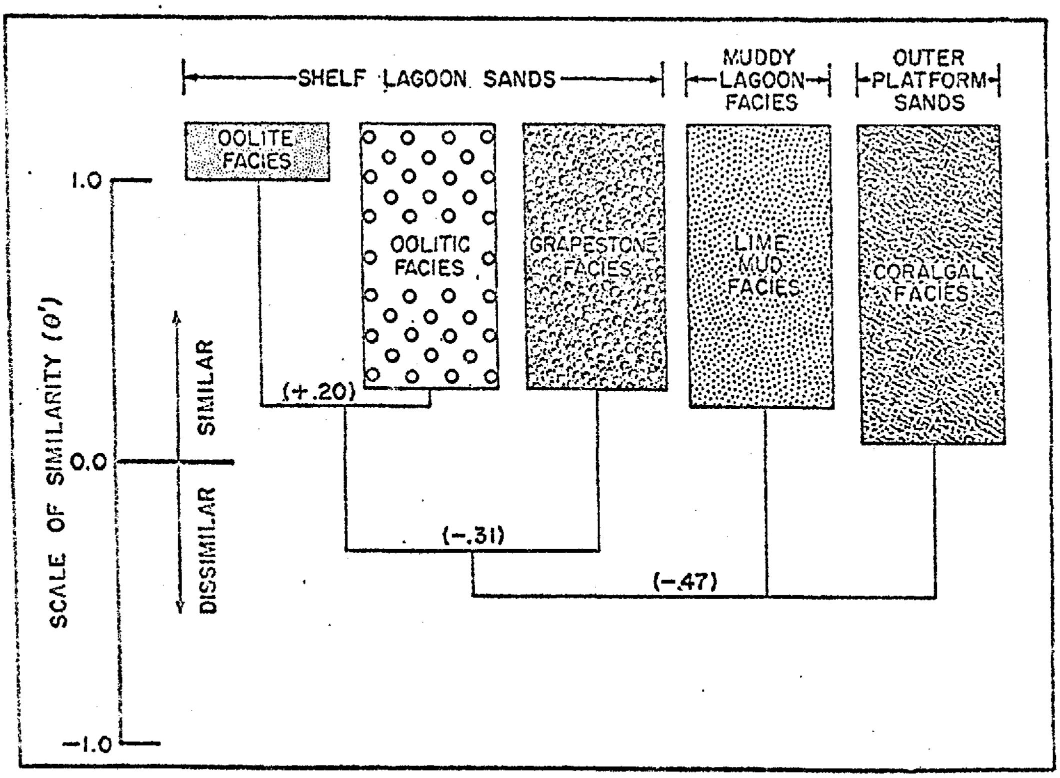


Figure 8: Examples of Dendograms
Dendograms offer a simple technique for illustrating the similarity or dissimilarity between classes of materials. The upper example is hypothetical; the lower relates to the data of Imbrie and Purdy. (See also Figures 6 and 7).

Discriminant function analysis has been proposed as one of the most powerful techniques in numerical classification. Clusters are separated by a (k-1) dimensional plane positioned in k-dimensional variable space. The equation of the hyper-plane is called a discriminant function and is used to allocate a new sample to one or other cluster, depending on the values of test scores for that sample. The probability of mis-classifying a sample, also the discreteness of clusters as measured by the separation of their centroids can also be determined.

Multivariate statistical analysis may appear complicated, but the application of these methods is facilitated by the availability of a number of well tried computer programs. The time and effort required in applying these techniques to the data is likely to be significantly less than that required to obtain the data in the first instance. Subjective skills are required to interpret the data, with or without the use of these techniques. Statistical and computational methods are likely to be of most use in the initial stages of erecting and evaluating the classification scheme, when the necessary skills are more readily available, rather than in the later stages of applying the classification scheme to practical engineering problems. A scheme that required the use of digital computation or statistics to classify a rock sample would have obvious practical disadvantages, whereas a scheme that had been thoroughly evaluated with the aid of these techniques would prove of more practical value than one based on intuitive concepts alone.

(e) Practical Criteria for the Selection of Classification Tests and Observations

In subsequent chapters the available observations and tests are to be examined with respect to their suitability for use in classification. It remains to set out criteria that may be used as a basis for including any given test or observation in a short list of suitable classification techniques. Some of these criteria have been previously mentioned, but at the risk of repetition a more or less complete set will now be outlined. Two major requirements present themselves. The tests and observations should be practical in terms of the available skills and resources of those who are to use and interpret the classification system; they should also adequately reflect the character of the rocks to be classified. The criteria may be itemised under these two major headings as follows:

PRACTICAL REQUIREMENTS

(1) *Quick test techniques* are required in order to test the large number of rock samples necessary for adequate mapping of rock variation. Hundreds rather than tens of samples dictate no more than two or three minutes for an individual observation. Specimen preparation time, in particular, should be minimised and it may be noted that in many weak or broken rocks it may not be possible to machine specimens whatever the time available. The total number of observations per sample should be small and it is essential to obtain a maximum of relevant information for minimum effort by optimum design of the classification system.

(2) *Simplicity of techniques* is required so that tests and observations may be reliably performed by non-experts. Tests requiring special skills to perform usually also call upon these skills for the interpretation of results, and may well detract from the practical value of the classifation system as a whole. Systemmatic standard procedures are in general to be preferred, rather than those that require personal judgement.

(3) *Robust, portable apparatus* (if any is required at all) will be required to allow testing in the field laboratory, or if possible on site. Transport of large numbers of samples to the laboratory is to be avoided on account of sample deterioration, loss of a 'feel' for the practical problems and in situ conditions and on account of the considerable time and effort required. Classification testing forms a logical extension of conventional core logging procedures, and the two should not be isolated.

REQUIREMENTS FOR ADEQUATE CHARACTERISATION

(4) *Relevance to rock properties*, both those that can be observed in the hand specimen and those observable on a larger scale, is essential. Certain types of rock in particular have peculiar characteristics and the scope of tests and observations should encompass all the important and fundamental aspects of rock character. A test that may be essential to characterise one rock type may be irrelevant when applied to another. In practice emphasis should be placed on those tests most relevant to the suite of rocks to be encountered on site. The most abundant and often encountered rock types should be emphasised in a classification scheme, rather than those of particular academic interest.

(5) *Relevance to engineering problems* is essential. The information provided by classification testing and observation should be tailored to meet the requirements of a given problem or set of problems. Unnecessary duplication of effort should be avoided, while ensuring that those aspects of rock character that are of particular relevance have been fully explored. Certain types of test and observation are of little relevance to engineering problems in general, and should be avoided.

(6) *Power of discrimination* between one sample of rock and the next is required. Tests that record the same or similar values when applied to different samples, those that have an inherently large error of measurement compared to the range of property values to be observed, should be avoided. In this context quantitative results are preferable, and teststhat require a change of technique or of apparatus in order to cover the complete range of rock materials are undesirable.

It was suggested that a classification scheme should be to some extent tailored to meet the particular needs both of the suite of rocks and of the engineering problems to be encountered on site. At the same time one should avoid a multiplicity of classification systems. These conflicting objectives may be reconciled by selecting from among the 'short list' of potential tests and observations those that are relevant to the widest range of materials and problems. These will be referred to as *basic classification tests and observations* and may be used to establish a universally applicable basis for engineering rock classification. Others of less general relevance may be termed *supplementary classification tests and observations* and may be called into operation to add depth of characterisation in directions dictated by particular engineering problems or rock materials. In addition to having a wider relevance, a basic classification test must meet more stringent requirements of speed and simplicity than must a test used occasionally to provide supplementary information. Basic classification tests, supplementary classification tests and design tests will usually rank in that order with respect to their complexity. When assessing a test against the criteria ennumerated above, its potential value in one or other role may be considered, but a more immediate task will be to place the techniques in context and to consider their capabilities and practical limitations.

2. GEOLOGICAL METHODS OF CLASSIFICATION; THEIR APPLICATION IN ENGINEERING

a) Introduction

Earlier it was suggested that an adequate engineering classification or description of rock is likely to require a combination of both testing and observation. This chapter is concerned with the search for suitable observations; tests will be discussed later. In the search for suitable observations it seems reasonable to start by examining the traditional geological methods of rock description since these are almost exclusively based on observation, and have been extensively studied and developed.

In predicting mechanical performance (the object of an engineering rock description) geological classifications and rock names are seldom as useful as the observations on which they are based. In some cases they are too complex for use in engineering. Over two thousand igneous rock names are in current use, and the average engineer knows little of the reasoning behind this diversity of nomenclature or of its relevance, if any, to the mechanical behaviour of the rock. In other cases rocks, from the engineering viewpoint, have been underclassified. Mudstones occur in great abundance (35% or more of all rocks) but are perhaps unattractive and difficult materials to study and have received little attention from the geologist. Geological classifications are *designed* to reflect rock history or genesis rather than to predict mechanical behaviour, so it is perhaps not surprising if they prove deficient when applied to engineering.

A common attitude of engineers towards geological terminology may be illustrated with a quotation from Harley (1926), although few would nowadays be prepared to put this attitude so bluntly: "The (ground classification) most generally used today is the local usage of terms such as hard, medium, soft, easy breaking and so on.... Another ground classification scheme, which however has had no general use, is that based on the rock name such as granite...... The faults of such a scheme should be obvious". There has been an unfortunate polarisation towards, on the one hand those who approve of geology and base rock description solely on geological observations, and on the other those who rely almost exclusively on the results of mechanical tests. An engineer adopting the first extreme attitude may express surprise when two limestones behave mechanically quite differently from each other, while a basalt and an andesite prove mechanically indistinguishable. The second attitude may result in the engineer discovering, after much expensive mechanical testing, that he could have forecast the test results by a careful visual examination of the rock without any testing at all.

It will be assumed that geological methods should neither be accepted nor rejected, but adapted to suit the needs of engineering. To make this adaptation it will be necessary to look beyond the geological classification and nomenclature systems at the observations on which they are based, and to select those observations that appear most relevant to mechanical behaviour. The geological literature gives an excellent guide to properties that *can* be observed, and it remains to decide those properties which, for engineering purposes *should* be observed.

The present chapter will consider in turn four major categories of rock, the sandstones and mudstones, the limestones and 'chemical' sediments, the igneous rocks and the metamorphic rocks. In addition to examining the criteria and structure of geological classifications (different systems are applied to each of the four categories of material) this study will also indicate the naturally occurring families of rock materials and their relative abundance. The chapter will conclude with a short review of some attempts to apply geological methods to engineering problems. Subsequent chapters will discuss tests that may be used to supplement observation, also the way in which tests and observations may be combined for the purposes of engineering rock description and classification.

b) Sandstones and Mudstones

These materials are characteristically fragmental (clastic) the fragments being derived from rocks of a predominantly silica or silicate composition; they have been termed 'siliciclastic'. The composition of the fragments largely reflects the source or 'provenance' while the texture is a good guide to the environment of deposition. Emphasis on provenance has led to the principal method of subdivision, that based on the mineral *composition of the coarser, hence visible, fragmental material.* Triangular classification diagrams such as those illustrated in Figure 9 may be used to assist in the subdivision. There is some controversy regarding the precise choice of end-members for this triangle but nearly all systems are based on quartz at one pole, feldspars at another and rock fragments at the third. Abundance of quartz is associated with the natural family of mature sandstones, the *orthoquartzites*, often themselves derived from earlier sedimentary rocks. Abundance of feldspars and igneous rock fragments is associated with another familty, the *arkoses*, derived from an igneous, granitic source while a third family, the *greywackes*, is characterised by a high content of rock fragments and mud. This rather oversimplifies the picture since these names are also associated with textural peculiarities that are often considered definitive of the rock name.

Grain size is also important in the nomenclature of this category of materials. The well known 'grade scale' (Figure 10a) is used by engineers to discriminate between conglomerates, sandstones, siltstones and claystones and the intermediate subdivisions such as fine sandstone. The grade scale is also widely used by geologists; it is not entirely adequate since it does not reflect the variation in grain size (sorting), only the most common, modal, size. Also the modal, size is easily overestimated since the larger grains are more obvious to visual inspection. The actual grain size distribution may be described using a classification triangle whose end-members are grade sizes such as sand, silt, mud. (Figure 10b). Alternatively the actual grain size may be

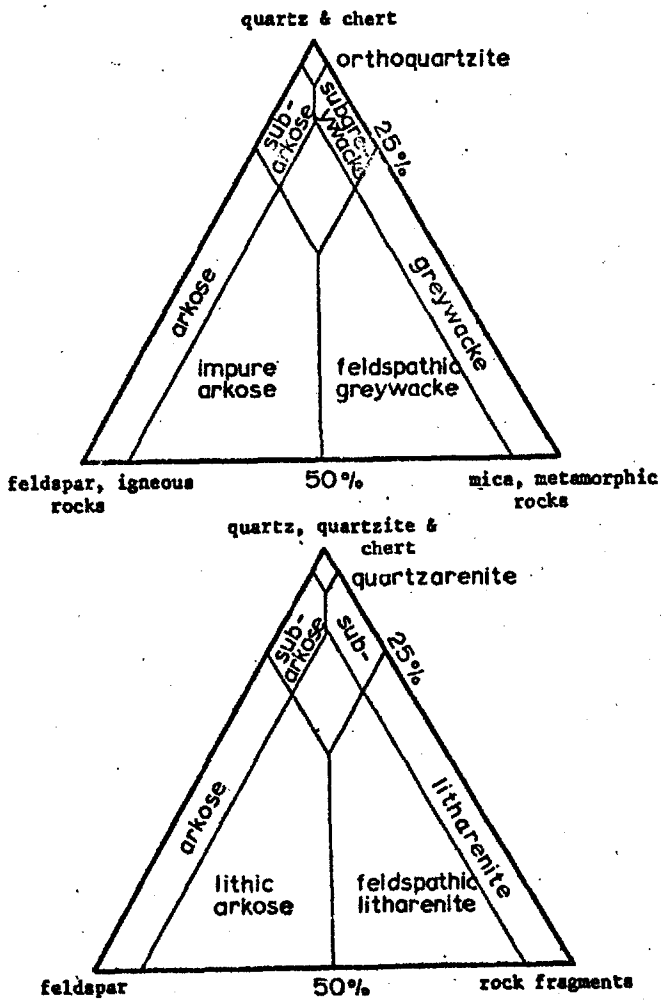
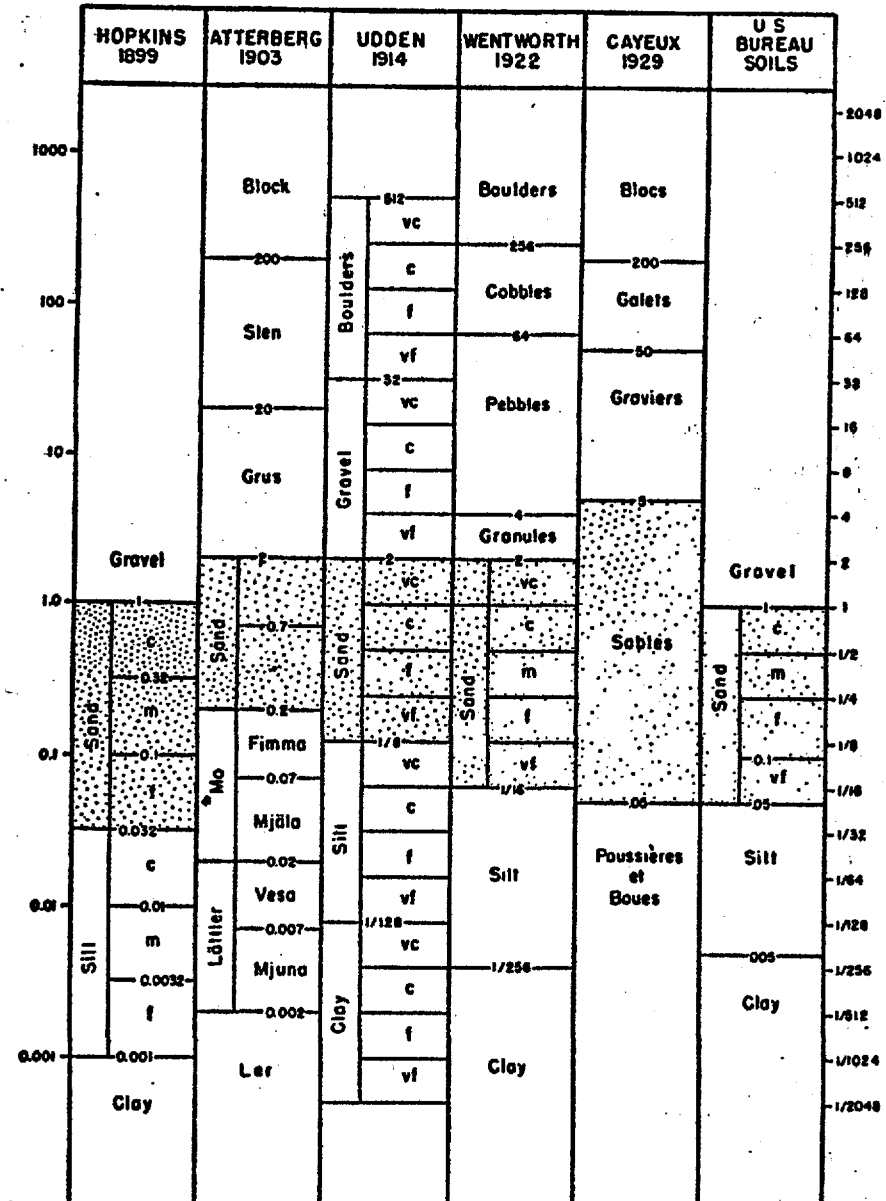


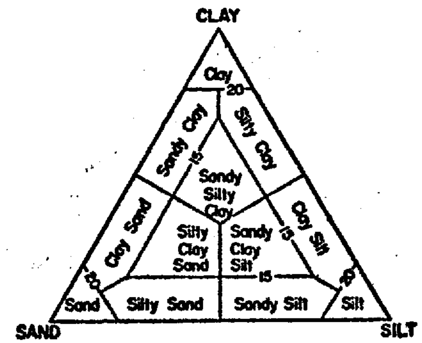
Figure 9 : Typical sandstone classifications (after Hatch, Rastell and Greensmith, 1965) Classification is according to mineralogical end members. Variations in the choice of end members are common. Triangle A was proposed by Folk (1954)and B by McBride (1963).

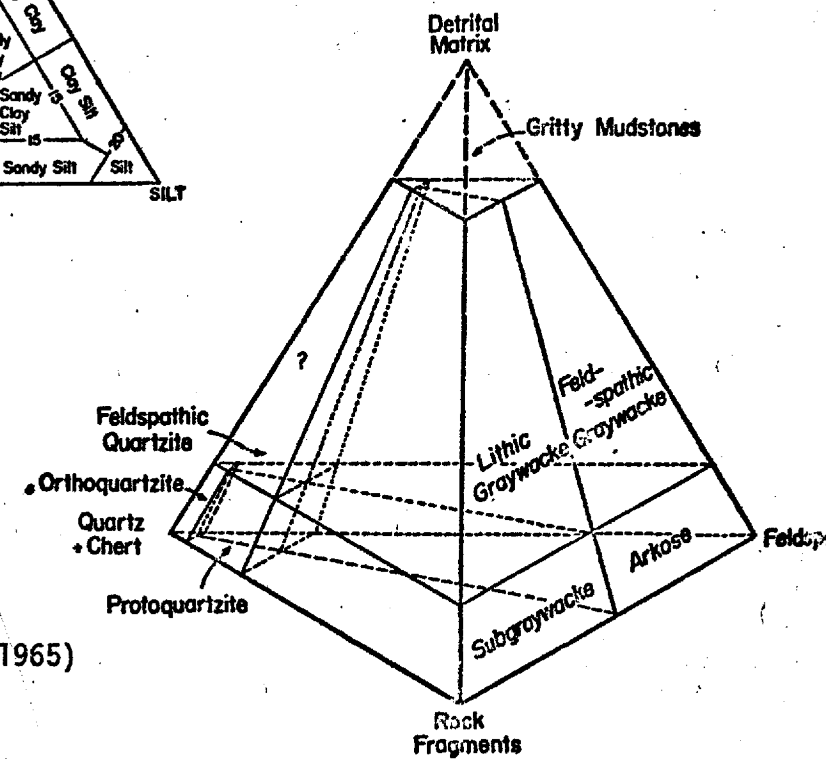
laboriously determined under the microscope and its distribution expressed using statistical parameters. Grain size as such, has perhaps only limited mechanical significance.

Quite often attempts are made to combine grain size and mineral end-members in the same classification triangle with the intention of relating closer to the complexities of rock genesis (Figure 10c). A particular point of controversy is whether to include the term 'matrix' in the classification scheme. "Because matrix is a size term it seems inappropriate to include it as an end-member in a classification based on mineral composition. Matrix does constitute a high percentage of the total volume of some sandstones, however, and it seems equally inappropriate to ignore it entirely". (Boggs, 1967).

If the presence of 'matrix' is often ignored in naming a sandstone, the presence of *pore space* is almost invariably ignored. Matrix and pore space assume much greater importance in







(after Hatch et al, 1965)

Figure 10 : Grain size classifications

10a (left) comparison of 'grade scales', a one dimensional classification system based on grain size

10b (top left) grain size end members used to provide a triangular classification

10c (top right) a grain size end member, namely 'matrix', is combined with three mineral end members to produce a classification that more closely reflects differences of provenance and environment of deposition.

the nomenclature of limestones, as will be discussed later. Both matrix content and pore content are likely to be of major mechanical importance. If, as seems likely, there are important engineering differences between the natural families orthoquartzite, arkose and greywacke this is perhaps more likely to be caused by textural differences in the materials than by the mineralogical differences definitive of these classes.

How does sandstone classification work in practice. Elliott and Strauss (1965) have reported some difficulties in applying the classical methods to 'field' identification. They compared careful microscopic measurements of grain size, made on coal measures sandstones, siltstones and mudstones, with the names given to the same rocks by twelve geologists from the National Coal Board and the U.K. Geological Survey. The results suggested that the "distinction between siltstones and sandstones was largely based on colour" and that there was "considerable difference of opinion..... on the demarcation between the mudstones and the siltstones...."One method ..... is to test the specimen for grittiness by scraping with teeth or a knife but unfortunately this test is very subjective". One or two geologists reported the need "to use a hammer on the specimens so that the sense of hardness assisted in their identification .... it is doubtful if any geologist in the test conciously related his 'field' identification to a particular size classification".

The authors recommended that practical classification should be based on granular quartz content. This parameter was easier to assess, and they reported "an approximately linear relationship between quartz grain percentages and the logarithm of the quartz grain sizes." They recommended that sandstones should be defined by a quartz grain content exceeding 50%, siltstones 50% - 20%, and mudstones by a content less than 20%.

There have always been difficulties in classifying the *finer grained materials* (igneous as well as sedimentary) since both mineral content and texture become difficult to assess. The three mineralogically defined families (orthoquartzite, arkose and greywacke) are often said to have analogies in the muddy sediments and probably some natural provenance subdivision does exist, but it is doubtful whether such a subdivision is practical or useful for engineering. The main subdivision used in practice is again one of grain size. 'Mudstone' is usually used to refer to the group as a whole, the coarser members being termed siltstone and the finer claystone. The term 'shale' is usually but not always retained for those materials showing flaky-fissility, e.g. silt-shale, clay-shale.

The fine sediments provide an excellent example to demonstrate the comparatively minor importance of the rock name when compared with the rock description as a whole. The modifying terms are all-important. The rock description is made up of the name and a selection from the many modifying terms available, for example 'red, current bedded, silica cemented fine sandstone'. Duncan (1968) points out that "indurated shales and mudstones may be very strong indeed as materials but the terms shale and mudstone can also be applied to other materials of quite different consistency". This observation has led to geological terms such as 'rock-like shale', 'soil-like shale' and to modifiers such as siliceous or calcareous (mudstone). Fissility, a further important observable property of some mudstone materials, is often used to modify the rock name. As a property it is difficult to quantify but attempts have been made (Ingram, 1953) to coin terms such as 'flaky-fissile', 'flaggy-fissile' and 'blocky-fissile'. The development of fissility depends both on the capacity to swell and on a lack of intergranular strength to resist swelling and it is perhaps best assessed by test rather than by observation.

c) Limestones and 'chemical' sediments

This group includes materials that have been "formed by chemical or biochemical precipitation *within the basin of deposition*" (Folk, 1959). The differences between these and the preceding 'siliciclastic', 'terriginous' (land derived) materials must be underlined since these determine differences in mechanical performance, also the rather different approaches to classification. Firstly they contain little or no fragmental silica or silicates. A large proportion are, however, fragmental although there are, in contrast to the silici-clastic rocks, several types of crystalline rock within this category. The bulk of such materials are carbonate rocks - lime-stones, dolomites etc., although we can include the evaporites such as gypsum, salt-rock and also materials such as coal and phosphate-rock in this group. The group from the engineering standpoint is characterised by 'soft', often soluble minerals.

The primary classification of these materials is *textural* rather than mineralogical. Three textural components are recognised, the fragments ('allochems', that is chemical/biological fragments), the 'sparry' (coarsly crystalline) cement and the microcrystalline or muddy matrix (Figure 11). The classification triangle reflects three natural families of material, the 'clean washed' limestones consisting of fragments, cement and a little mud, the 'muddy limestones' and the 'lime-mudstones'. Two important non-fragmental materials are omitted, the crystalline materials and the reef rocks which grow in-situ and which are often referred to as 'boundstone'.

It can be seen that grain size plays a small part in this classification, although grade scales similar to those previously described have also been applied to this class of materials (Figure 12). Grain size receives little attention in limestone classifications since, in contrast to the case for sands and muds of siliciclastic nature, it is a poor index to depositional environ-ment. This is the case because the source of fragments is often single sized; the fragments are often whole fossils or pellets whose size is controlled biologically. Also their behaviour in water depends on weight as well as size and the density of fossils that may contain trapped air is very variable.

The classification for fragmental limestones is supplemented by a prefix code that indicates the nature of the fragments. These include intraclasts (fragments of other limestones), oolites (rolled, 'onion' structured particles) fossils and pellets (probably excreta). The fossil type can also be indicated. The resulting terminology is illustrated in Figure 13.

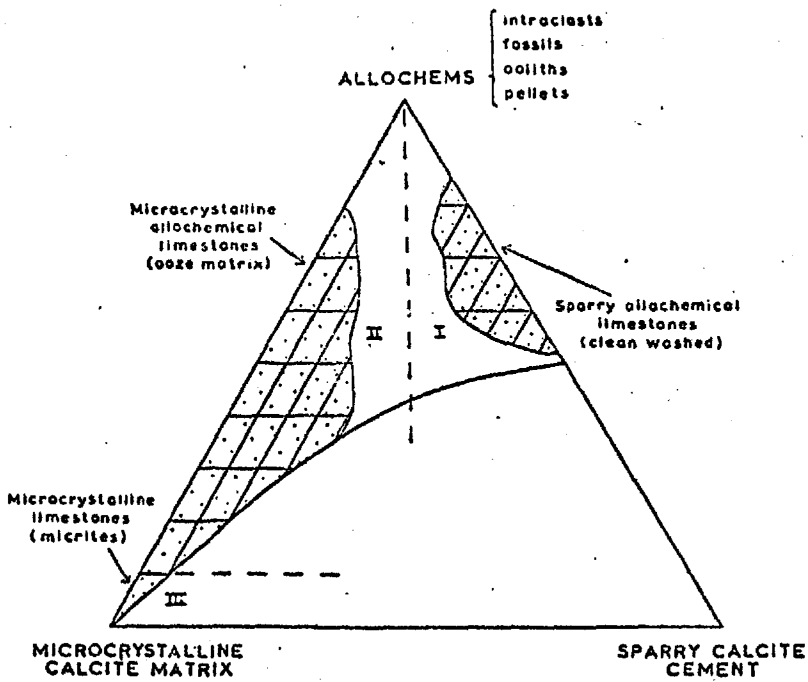


Figure 11 Limestone classification

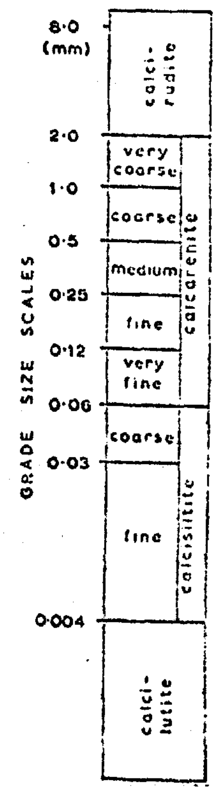


Figure 12

Limestone grade scale

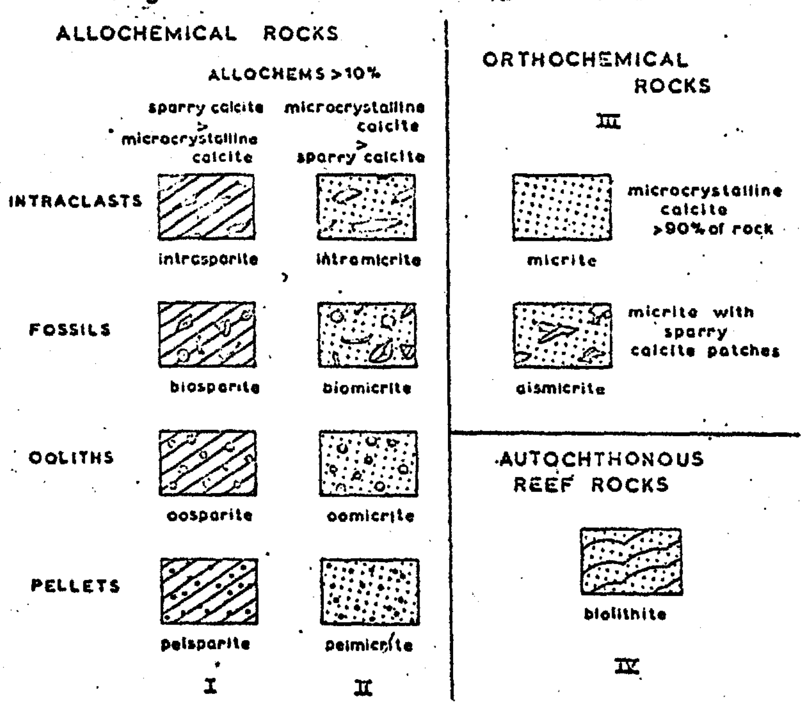


Figure 13

Limestone nomenclature

Figures 11-13 (after Folk, 1959) illustrate how limestone is commonly classified and named according to its crystal/mud/fragment proportions rather than on the basis of either grain size or mineral content. The grade scale of Figure 12, adapted from similar sandstone scales such as those of Figure 10a, plays no part in classification.

Any even greater emphasis was placed on texture by Dunham (1962) who made two further subdivisions between the lime-mudstone and the mud-free limestone that he termed 'grainstone'. He distinguishes between muddy limestones where the grains are in contact (his 'packstone') and those where grains 'float' in mud ('wackestone'). His classification is illustrated in Figure 14.

<table>
<tr><td colspan="5">DEPOSITIONAL TEXTURE RECOGNISABLE</td><td rowspan="4">NOT RECOGNISABLE</td></tr>
<tr><td colspan="3">CONTAINS MUD</td><td rowspan="3">Lacks mud and is grain - supported</td><td rowspan="3">Original components bound together</td></tr>
<tr><td colspan="2">MUD SUPPORTED</td><td rowspan="2">GRAIN - SUPPORTED</td></tr>
<tr><td>Less than 10% grains</td><td>More than 10% grains</td></tr>
<tr><td>MUDSTONE</td><td>WACKESTONE</td><td>PACKSTONE</td><td>GRAINSTONE</td><td>BOUNDSTONE</td><td>CRYSTALLINE CARBONATE</td></tr>
</table>

Figure 14 : Dunham's (1962) textural limestone classification.
His emphasis on the ratio of grains to mud matrix, the 'wacke' nature of the rock, is similar to the concept used to define greywacke sandstones and is likely to give a useful indication of the mechanical nature of the rock material.

The whole approach to limestone classification that has been outlined above has resulted from a fairly recent intensive effort in this field, largely associated with applications to oil technology, so that many of the terms may appear unfamiliar to the engineer. Ham & Pray (1962) remarked "it is a sad commentary that until recently the standard treatment of carbonate rocks was only to note by use of modifiers the colour, the 'crystallinity' and the presence of megascopically obvious fossils, and to record the dominant composition by the terms limestone or dolomite ......more research has been done on carbonate rocks in the period 1940-1960 than was accomplished in all the preceding years".

Other fields of engineering have undoubtedly much to learn from the treatment of texture resulting from this work. Archie (1952) developed, primarily for the oil industry, a limestone classification based on texture and particularly on porosity, that has been widely applied. His three 'types' namely 'compact crystalline', 'chalky' and 'granular or saccaroidal' are subdivided into four classes based on the size of visible pores. Although this treatment has been developed to the stage

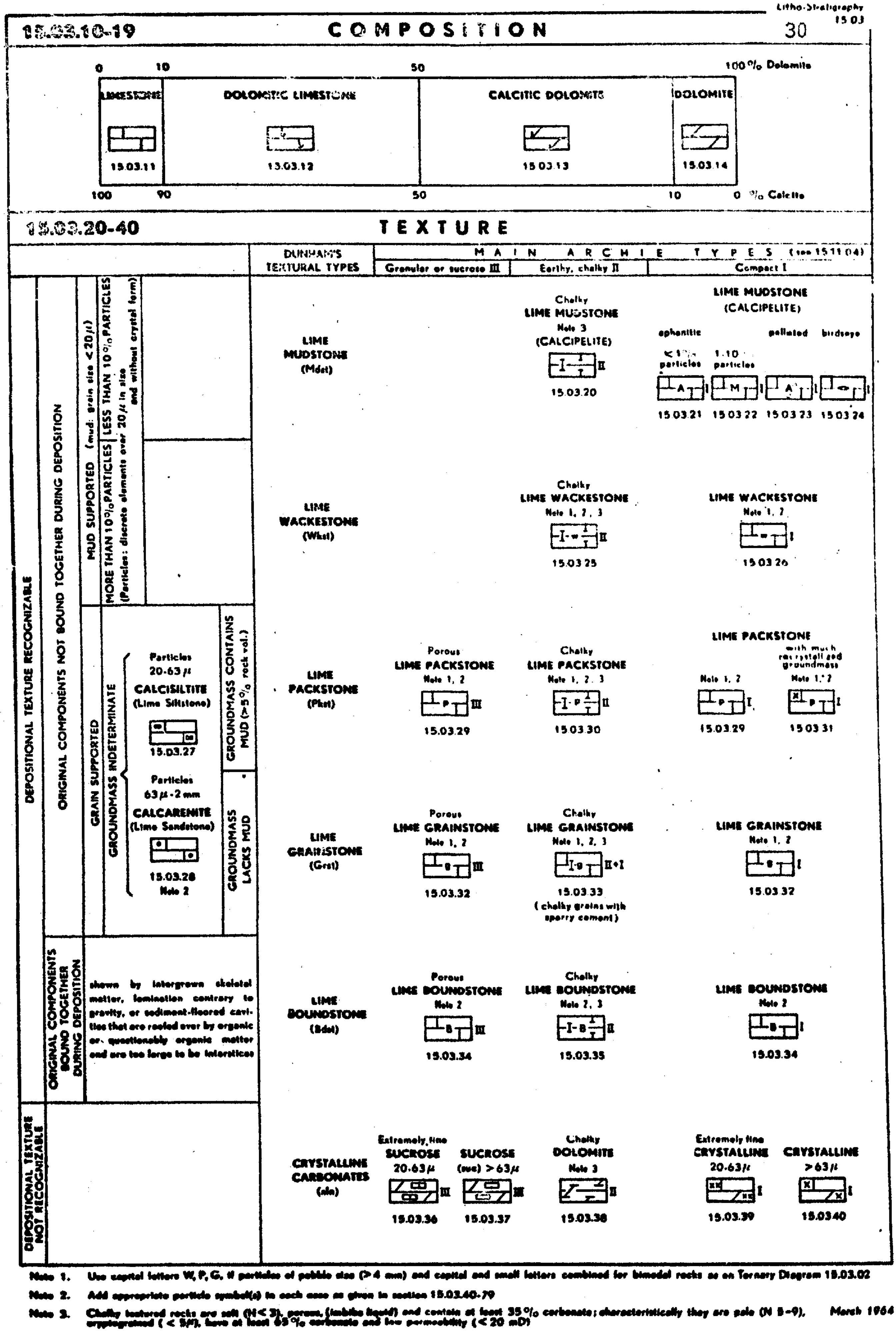


Figure 15 : Summarised limestone classification
(after Royal Dutch Shell, 1958)

where it is largely superceded by more quantitative pore description it represents one of the first attempts to recognise and describe pore space in rocks. Pores, to the engineer are perhaps the most important rock forming 'mineral' and the observation and quantification of porous textures will be discussed at greater length in chapter 3. Archie's pore classification appears as a criterion in the oil-technology classification of limestones reproduced in Figure 15.

Compared with texture, *composition*, (mineralogical and chemical) plays a subordinate role in the classification of the 'chemical' rocks although it is obviously instrumental in determing the main classes such as limestone, evaporite and salt rock. However, the limestones in particular are subject to alteration processes such as recrystallisation and dolomitisation, and mineralogical indices such as the ratios calcite:aragonite:dolomite or chemical indices such as calcium:magnesium ratio, are used extensively in tracing the history of such processes. These indices may have some, if limited, mechanical significance.

Alteration mechanisms play a large part in the formation of crystalline 'chemical' rocks although some are undoubtedly crystalline from the time of deposition. Hence there is a rather gradual transition from fully fragmental to fully crystalline materials and traces of fragments can be seen in most of the crystalline rocks of this category. There have been attempts to apply the crystalline-texture terminology of the igneous and metamorphic rocks (Friedman, 1965) although most authors appear to consider the term 'crystalline' to need no further amplification for these sedimentary materials. The subject of crystalline textures (or fabrics) will be more fully discussed in subsequent paragraphs.

Practical application of the limestone classification concepts previously discussed is illustrated by Figure 15, reproduced from the 'Standard Legend' of the Royal Dutch Shell group of companies (1958). This table is typical of the excellent concise tabulations of this document which acts as a standard for practical limestone nomenclature in the oil prospecting industry. For present purposes it also demonstrates how the various aspects of texture, composition, Archie classification etc., may be combined into a workable system.

d) Igneous Rocks

Igneous rocks are characterised by their magmatic origin; they have crystallised from a molten magma. They are *crystalline* (with a few glassy or amorphous exceptions) and are composed largely of silica or silicates. Their texture, rather less variable than that of sediments, is controlled to some extent by the magma composition, also by the mode of transport of molten or semi-molten magma, but largely by the rate of cooling. Their chemical and mineral composition (the basis of their classification) and the natural associations found among igneous rocks are governed by genetic criteria, best summarised using the concept of 'magmatic evolution' illustrated in Figure 16.

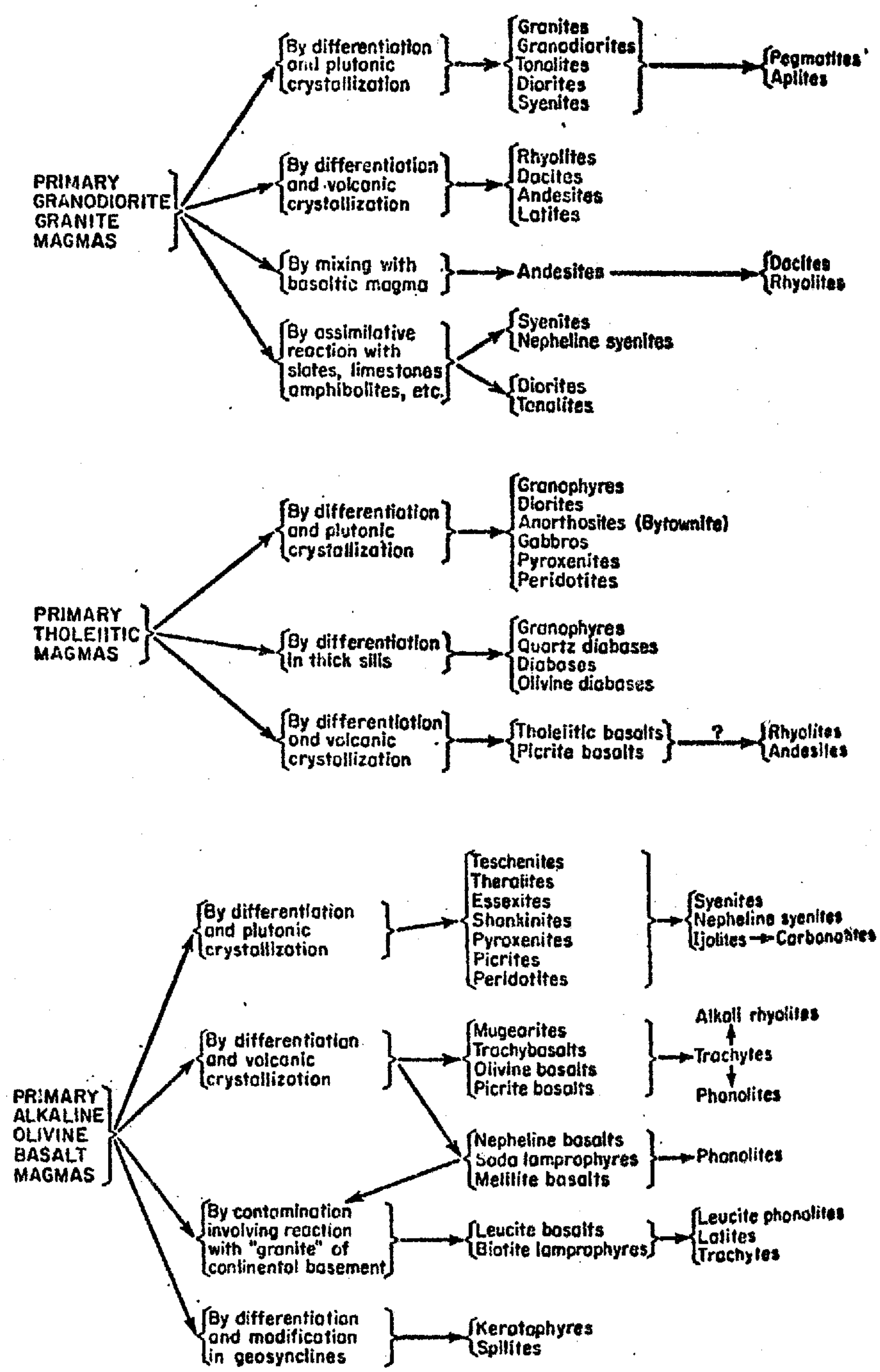


Figure 16 : Magmatic Evolution (after Turner & Verhoogen, 1960)

Theories of the processes of evolution of igneous rocks attempt to account for the common associations of rock types found in nature; they do much to explain the existence of natural families of rocks.

Texture plays a much lesser part in the classification of igneous rocks than of sediments, although *grain size* is used to make a primary subdivision between coarse, medium and fine grained varieties which then receive a further subdivision on the basis of mineral content. Igneous geologists are inclined to be evasive on the subject of quantifying the subdivision coarse:medium:fine. Cross et al (1906) recommend that the grade boundaries should be at 1mm and 5mm, but few would rigidly adhere to this suggestion. The primary subdivision corresponds more or less to a genetic one based on the mode of occurence (extrusive, intrusive or plutonic) of the igneous body; Hatch et al (1949) state that the grain size limits have been chosen "so that in each compositional series, most specimens from major (plutonic) intrusions fall into the 'coarsegrained' category; most of those from minor (hypabyssal) intrusions fall in the 'medium' group; while the majority of those with an extrusive mode of occurence are classified as 'fine-grained'." This recognises the control of grain size by rate of cooling, hence depth and size of intrusion, while keeping the actual classification a matter of subjective observation. Hence gabbros, usually associated with deep seated igneous intrusions, can sometimes be encountered in dykes or sills at shallower depths.

The three size groups are then subdivided on the basis of *mineral content*. The coarse and fine rocks are given separate subdivision and terminology. The medium grained rocks usually follow the same classification scheme as the coarser, using a prefix notation such as 'micro-granite', or their terminology may emphasise textural peculiarities e.g. quartz-porphyry.

Before considering the mineralogical classification it might be as well to discuss the *chemical classifications* that were particularly in vogue during the early part of this century. These are still used in parallel with mineralogical methods, particularly for the finer grained materials where the minerals may be difficult to identify microscopically. The prototype for chemical classification of rock was published in a book by Cross, Iddings, Pirsson and Washington (1903) and is referred to as the CIPW classification. It depends on the re-casting of chemical analyses (% oxides etc.) into amounts of largely hypothetical 'standard minerals'. These are known as the 'normative minerals' or 'norm' for the rock. This must be contrasted with the *observed* mineral assemblage known as the 'mode' for the rock.

A given composition of magma can yield several types of mineral assemblage depending upon the cooling conditions; the 'standard minerals' of the norm are indeed hypothetical. Chemical composition leads to an unambiguous classification but one that tells little of the processes of rock genesis. It has obvious practical drawbacks but itis auseful adjunct to the study of mineralogy particularly in the case of the fine grained materials.

Although mineral rather than chemical composition is the criterion used for further subdivision of the three grain size categories this subdivision retains a concept originally based on chemical composition; the *acidity* of the rock, or its 'silica saturation'. Rocks are subdivided into acid, intermediate and basic varieties. In theory this subdivision is based on the content of silica, both free (quartz) and combined, but since only the free silica can readily be estimated mineralogically the acid:intermediate:basic subdivision uses other mineralogical criteria that give an index to the silica saturation. The simplest form of such a classification is illustrated in Figure 17. It can be seen from Figure 17 that the principal indices of silica saturation are the % quartz and the type of feldspar. Igneous rocks of a given composition and hence acidity, are collectively known as a 'clan'. The clan is given the name of the coarser member, hence the 'gabbro clan' includes gabbro, dolerite and basalt.

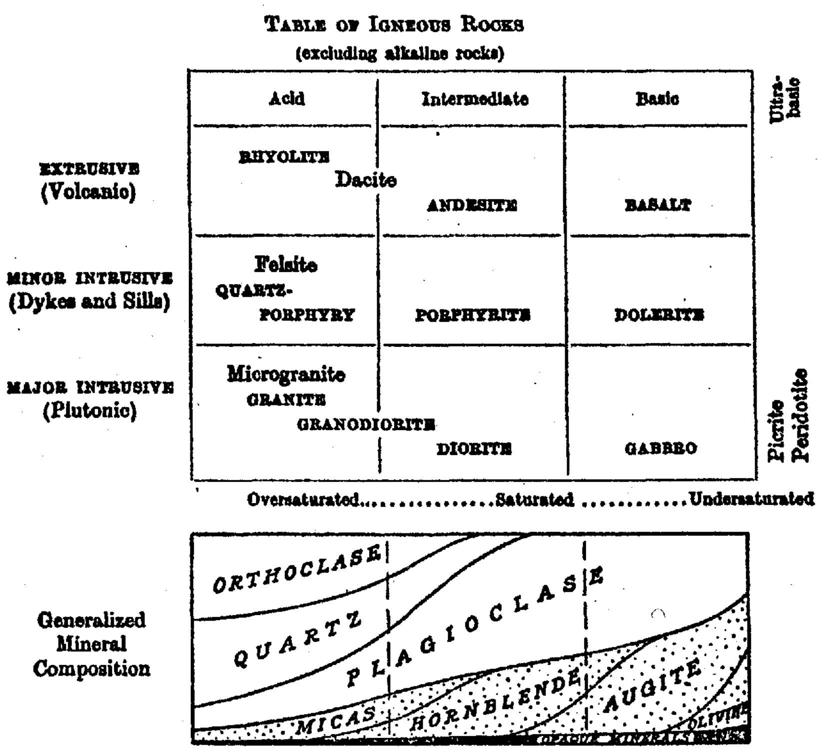


Figure 17 : Simple form of igneous rock classification (after Blythe, 1960)

Rocks are classified on the basis of 'acidity' (silica content) and grain size (or mode of occurrence). To estimate acidity the nature of the feldspars is examined together with the content of quartz, alternatively the 'colour index' (content of dark minerals) may be used. This simple classification corresponds approximately to the more precise and elaborate system illustrated in Figure 18.

The more sophisticated forms of igneous rock classification utilise more detailed mineralogical subdivision. Four mineral types are used as end-members namely quartz (Q), alkali feldspar (A), plagioclase feldspar (P) and feldspathoids (F), reflecting a decrease in silica saturation. The feldspathoids particularly, are under-saturated in silica so that rarely does a rock contain both feldspathoid

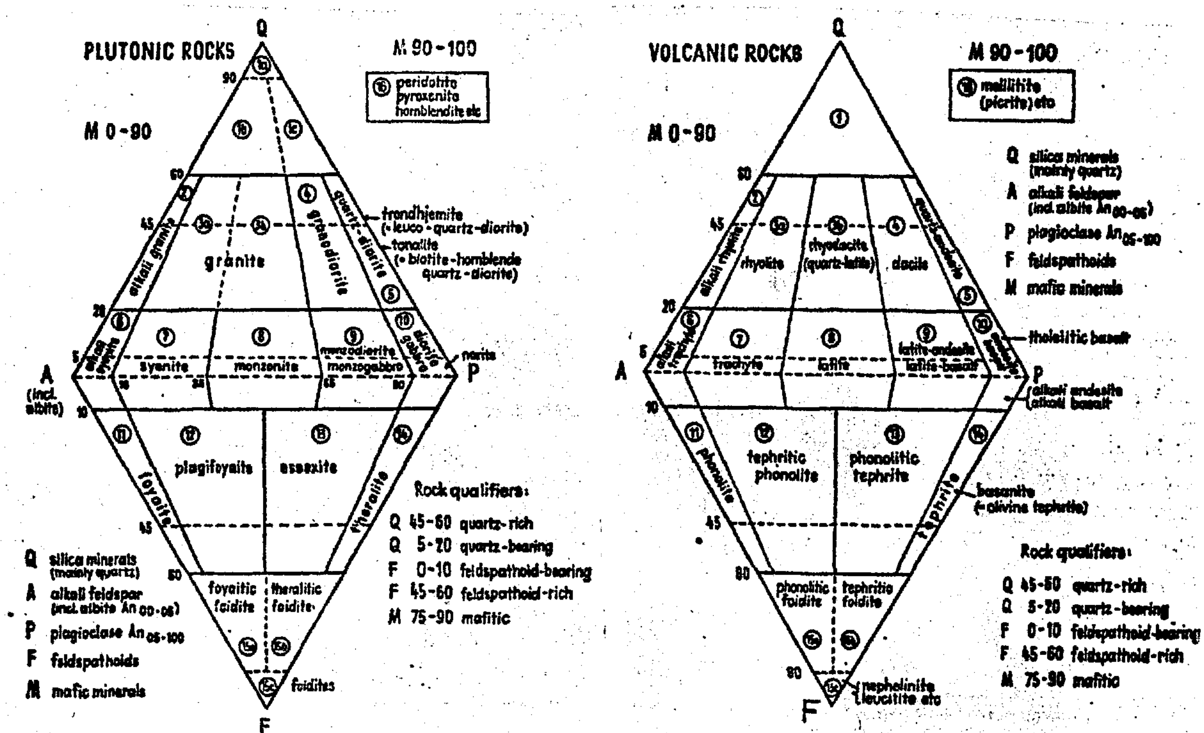


Figure 18 : Complete form of igneous rock classification (after Streckeisen, 1967)

Volcanic and plutonic (fine and coarse grained) rocks have different nomenclature systems but are in fact classified identically. The poles of the double triangles are such as to indicate the 'acidity' of the rock.

type minerals and free silica. The double triangle classification of Figure 18 is only possible since quartz and feldspathoids are in effect mutually exclusive, so that a rock plots in one or other triangle not both.

It is to be noted that only light coloured minerals are used in this classification. When the content of dark minerals is greater than 90% the rock is classed as 'ultramafic', without reference to silica saturation. The percentage of dark coloured minerals is known as the *colour index* of the rock. Figure 17 demonstrated that this easily observed property is an approximate index to silica saturation, but it is only used as such for field identification. Rocks with colour index greater than 75% may be given the modifier 'mafitic' (dark coloured).

A further feature is that only the larger crystals, those that can be microscopically identified, are used in classification. The fine grained (usually volcanic) rocks must therefore be classified on the basis of the mineralogy of larger crystals usually present in the fine grained matrix. This can lead to incorrect classification since the larger crystals are often unrepresentative of the mineralogy as a whole. Classification of these rocks must therefore be assisted by comparison with the composition of associated coarser grained rocks or by chemical analysis where this is practicable.

Textural descriptions of igneous rocks, since they do not play an essential part in classification, are usually qualitative only. The 'crystallinity' of the rock is observed along with grain size to give terms such as holo- (wholly) crystalline or glassy. The rocks are divided into equigranular and inequigranular varieties, the inequigranular being classified as porphyritic (large crystals in a fine groundmass) or as poikilitic (small crystals in a coarse groundmass). The main purpose of textural observations of igneous rocks is to interpret from the texture the sequence of crystallisation. This motivation is behind terms such as anhedral, euhedral, xenomorphic, idiomorphic, used to describe the extent to which crystals have been able to develop their full crystal outline during solidification.

Individual textures characteristic of a particular (mineralogically defined) rock are often given the name of the rock (e.g. granitoid, trachytic). It is of some mechanical significance to note that a rock of a given mineral composition (hence a given rock name) has a texture that is largely determined by its mineralogy, particularly since the sequence of crystallisation is with few exceptions the same. Also the shapes of individual crystals are largely predetermined by their mineralogy and impart a considerable influence on the texture as a whole. Hence if a gabbro behaves mechanically in a different fashion to a granite it may well be due to textural differences even though the rocks are defined minerallogically (according to their 'acidity'). Texture appears an important factor, but conventional geological descriptions of texture less than adequate, in determining the mechanical character of igneous rock materials.

e) Metamorphic rocks

Yet another approach to geological classification is adopted in the case of metamorphic rocks. This class of rocks is defined as having undergone recrystallisation due to elevated temperatures, pressures or a combination of both. The process of recrystallisation occurs while the rock, originally sedimentary or igneous, is still in the *solid state* so that its chemical composition remains largely unaltered even though the mineralogy may be radically altered.

The new, metamorphic, minerals that grow during this process of 'isochemical' reaction are often exclusive to the metamorphic rocks and impart *textures* that are characteristic both of crystal growth in a solid environment and of crystal growth under conditions of anisotropic stress (where this applies). Metamorphic nomenclature is usually based on rock texture rather than on composition, and mineral content is indicated by means of modifiers (for example quartz mica schist).

Underlying metamorphic rock descriptions, however, are the genetic concepts of metamorphic grade and of provenance, and it will be as well to consider these two aspects before a more detailed discussion of metamorphic textures and their use in classification. Barth (1959) favoured a *chemical classification* in terms of 'normative minerals' similar to that discussed earlier in the context of igneous rock classification. This approach can be useful in the interpretation of provenance since we have noted that metamorphic processes are usually isochemical; metamorphic rocks are likely to have very similar chemical composition to that of their parent materials. However, just as a change in cooling rate can produce a difference in mineral assemblage for an igneous rock, so a change in conditions of metamorphism can also give considerable mineralogical variation to rocks of the same chemical composition. For this reason chemical analysis can give little guide to the nature of metamorphic processes. The actual mineral assemblage in a metamorphic rock must be examined if conditions of metamorphism are to be established.

The concept of *metamorphic grade* is perhaps best illustrated by reference to the series 'shale-slate-phyllite-schist'. The provenance of a slate is a shale, the non-metamorphosed material. An increase in the metamorphic grade of this material is defined as an increase in the temperature and/or pressure of the metamorphic environment and is manifested by increasingly apparent changes in mineralogy that result from recrystallisation. These usually are accompanied by an increase in grain size and in other textural changes such as the development, or at later stages the loss, of foliation or fissility. A particular metamorphic grade is recognised by a characteristic mineral assemblage, particularly by the presence or absence of specific 'index minerals'. It is necessary to select different index minerals for metamorphic rocks of different provenance, and hence different original composition, since two rocks of the same grade but of different provenance can have quite different mineralogy.

A group of rocks of the same metamorphic grade but of different provenance is termed a metamorphic facies. The recognition of metamorphic facies from a hand specimen or thin section can be difficult since it requires assumptions as to the provenance of the rock and also the interpretation of mineralogy in terms of temperature and pressure conditions. A possible source of ambiguity should be pointed out. Several metamorphic rocks names, defined on the basis of texture, have also been used to name facies of metamorphism; amphibolite and granulite are examples. The problem of metamorphic terminology might at this stage be clarified by reference to Figure 19 which lists the principal provenance types, facies and textural varieties for these rocks.

The *textures* of metamorphic rocks are characteristic and are the most important factor in defining metamorphic rock terminology; they are also likely to be a most important factor from the engineering standpoint. In sedimentary rocks the texture and mineral composition are largely unrelated. Igneous and, particularly, metamorphic rocks on the other hand have a texture that is strongly related to their mineral composition. Metamorphic minerals grow in place, and their size, shape and orientation is largely controlled by their solid environment of growth and by the stress field that pertains at the time. With the appearance and disappearance of these minerals as metamorphic grade advances, the texture can be radically altered.

The first important subdivision in metamorphic rock textures is between those rocks that display anisotropic textures, particularly fissility, and those that do not. Anisotropic textures result usually from 'regional metamorphism' a type of metamorphism where an anisotropic stress field plays a dominant role. Schists and slates are typical of these rocks. The products of 'thermal' metamorphism', on the other hand, are usually more isotropic in texture. Thermal 'contact aureoles' are rarely more than a few hundred feet wide, so that these materials are of far less abundance than the regionally altered rocks that may develop over areas of many thousand square miles (Williams et al, 1954). The more isotropic type of metamorphic rock is not as rare as might be supposed, however, since some regionally altered rocks, the marbles and quartzites for example, also belong to this category.

The anisotropic metamorphic rocks fall into several categories depending on the cause of anisotropy. Slates are not, strictly speaking, metamorphic rocks since they comprise clay minerals that have not recrystallised. Their 'slaty cleavage' is caused by a parallel arrangement of clay minerals combined with a fine grain. A coarsening of grain and the development of recrystallised micas, characteristic of low grade metamorphism, produces the schists. The main cause of schistosity is in this case the abundance of parallel mica flakes. Important differences in the nature of this schistosity can arise from one type of schist to another, depending on the particular mineral assemblage present. Amphibolites show yet another type of anisotropic fabric, their weaker schistosity being due to preferentially oriented prismatic amphibole minerals rather than to platy minerals such as micas.

In general, fissility tends to disappear with advancing grade of metamorphism although it is also strongly influenced by the composition of the source rock. A rock of high metamorphic grade that originated as a clayey (pelitic) rather than a sandy or granitic rock with high quartz content will tend to contain more platy minerals. These, however, tend to seggregate into bands producing a 'gneissic' texture. The quartz and feldspar that predominate in such high grade materials is usually also oriented in a preferred direction but contributes little to fissility because of the more symmetrical crystal shape.

METAMORPHIC FACIES (Grade of metamorphism)

Increasing metamorphic grade
Increasing temperature
Increasing pressure
Decreasing water pressure

CONTACT FACIES - 1. Albite-epidote-hornfels 2. Hornblende-hornfels 3. Pyroxene-hornfels 4. Sanidinite

REGIONAL FACIES - 1. Zeolitic 2. Greenschist 3. Glaucophane-schist 4. Almandine-amphibolite 5. Granulite 6. Eclogite

Mineral composition reflects the grade of metamorphism. The choice of grade index minerals depends to some extent on the nature of the parent rock.

PARENT ROCK TYPES (Provenance)

PELITIC - derivatives of aluminous sediments (muds)
QUARTZOFELDSPATHIC - derivatives of rocks rich in quartz & feldspar (granites, sandstones)
CALCAREOUS - derivatives of limestones and dolomites
BASIC - derivatives of basic & semi-basic igneous rocks, and some sediments rich in Ca, Al, Mg, Fe
MAGNESIAN - derivatives of peridotites and montmorillonite sediments (minor abundance)
FERRUGINOUS - derivatives of cherts & ironstones (minor abundance)

Chemical composition reflects the parent rock type, since metamorphic reactions are usually isochemical

ROCK NAMES are primarily based on texture, e.g. Schist (abundant parallel mica), Gneiss (segregated quartz & feldspar), Slate (perfect planar schistosity + fine grain), Hornfels (mosaic of equidimensional grains)

Figure 19 : Facies, Provenance and Texture of Metamorphic Rocks

The close connection between mineral composition and texture allows metamorphic rocks to be classified mineralogically in much the same way as were the igneous materials. Figure 20 reproduces such a classification scheme from Winkler's textbook (1965). A primary division is made on the basis of facies, and then mineralogical end members are used for further subdivision. It is to be noted that the rock names are, even then, textural and that mineral content is indicated in the traditional manner, by means of modifiers. The modifying mineral terms may not necessarily imply that the mineral is dominant in the rock; they may only indicate its presence. For example amphiboles are significant by their absence from the classification of the amphibolite facies!

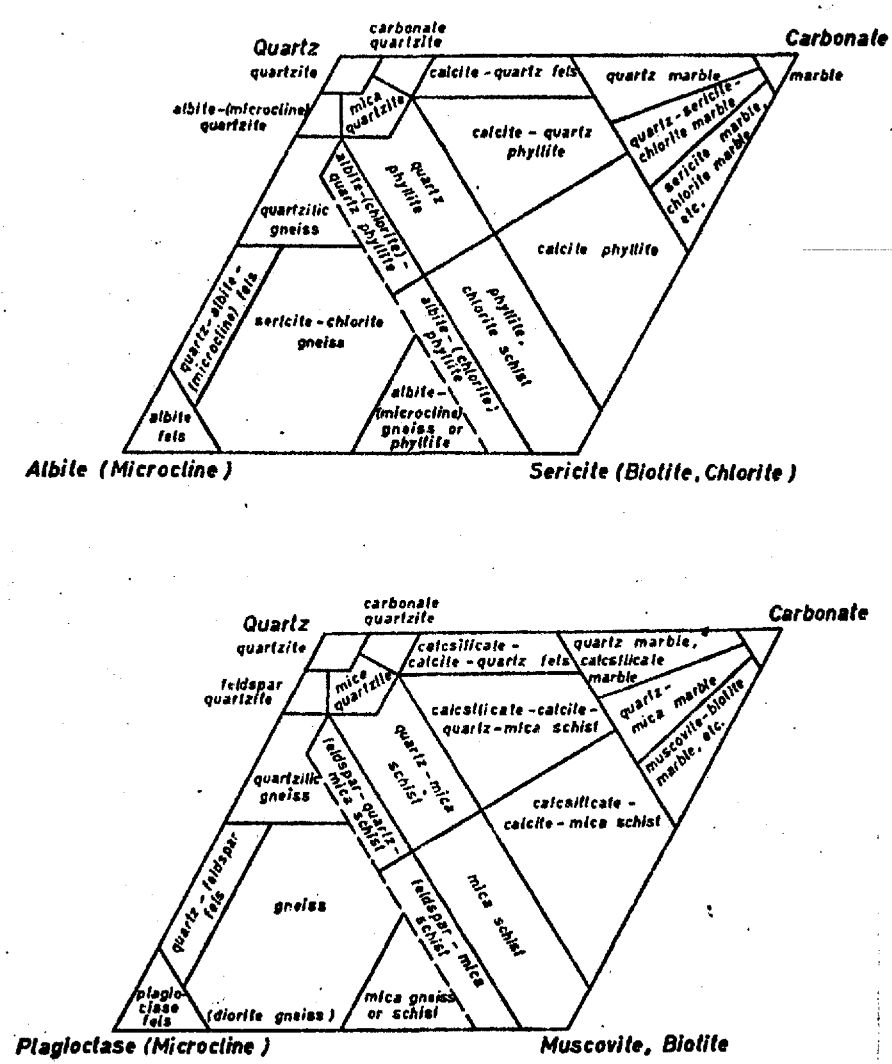


Figure 20 : Classification of metamorphic rocks (after Winkler, 1965).

The upper diagram relates to the 'greenschist' facies, the lower to the 'amphibolite' facies of rocks. Although the classification is mineralogical, the nomenclature still places emphasis on rock texture (e.g. gneiss, schist).

There is a further group of rocks that are better referred to as metamorphosed, altered or weathered rather than metamorphic. These retain much of their original igneous or sedimentary texture and composition and are classified accordingly. The classification of rocks that have been altered by diagenetic processes (weathering, cementation, compaction etc.) presents a problem in that the variations are gradual, and it would be impossible to devise a classification to cover all types. Their existence is perhaps best taken into account (for engineering purposes) by the use of modifying terms to indicate the nature of the alteration, these being supplemented by mechanical test data.

f) Adapting geological methods to suit engineering problems

The simplicity and yet considerable power of purely observational techniques for rock description are in little dispute, neither is the fact that traditional geological methods are inadequate for engineering. It is then surprising that there have been so few attempts to improve upon the geological methods and to apply observational techniques to rock classification in a way better suited to engineering problems. A few such attempts will be outlined in this section prior to presentation of the author's proposals for such a classification in Chapter IV.

In 1946 Terzaghi outlined a 'ground classification' intended for use in connection with tunneling operations. It is based on observations of the simplest sort yet is of obvious practical value. It serves as a reminder that most engineering problems require classification of bulk (rock mass) properties rather than of hand specimen properties. His grouping comprised the following 'rock types';

Intact rock (joint free)
Stratified or schistose rock
Moderately jointed, massive, rock
Blocky & seamy rock (weakly interlocked)
Crushed rock (sand-like particles)
Squeezing rock (flows without swelling)
Swelling rock

Terzaghi used this classification to outline tunnel design principles peculiar to each type. Interest in the factured nature of rock, hitherto neglected, resulted in a tendency to forget that rock material properties contribute to the properties of the mass, even to the properties of the individual joint or fissure. Both chalk and granite could fall into Terzaghi's 'intact rock', but their different character, despite possible similarity of jointing, is obvious. Terzaghi's classification is therefore not without its limitations although its simplicity is a most important point in its favour.

Most other attempts to apply geological observations to engineering employ a simplified geological nomenclature rather than devising, as did Terzaghi, a new engineering oriented system of terminology. A brief study of the Petrographical-Lithological Code, compiled by the Institute of Geological Sciences (1968) is sufficient to show the full complexity of geological terminology and at the same time the main barriers to its engineering application. The Code, as yet unpublished, proved a useful document in

COL.i:1. IGNEOUS ROCKS page 1.1

COL.ii	COL.iii	COL.iv
0 UNDIFFERENTIATED	0 UNDIFFERENTIATED	0 UNDIFFERENTIATED
A GRANITE-DIORITE GROUP	A GRANITE B MICROGRANITE C GRANITIC PEGMATITES AND APLITES D DIORITE E MICRODIORITE F DIORITIC PEGMATITE AND APLITE	Page 1.2
B RHYOLITE-ANDESITE GROUP	A RHYOLITE-DACITE B RHYOLITE-DACITE PORPHYRY C ANDESITE D ANDESITE-PORPHYRITE	Page 1.3
C SYENITE GROUP	A SYENITE B MICROSYENITE C UNDERSATURATED SYENITE D UNDERSATURATED MICROSYENITE E SYENITIC PEGMATITES AND APLITES	Page 1.4
D TRACHYTE GROUP	A TRACHYTE B TRACHYTE PORPHYRY C UNDERSATURATED TRACHYTE D UNDERSATURATED TRACHYTE PORPHYRY	Page 1.5
E BASIC GROUP	A GABBRO B DOLERITE C BASALT D SPILITIC ASSOCIATION E SYENOGABBRO (ALKALI-GABBRO) F MICROSYENOGABBRO (ALKALI-DOLERITE) G TRACHYBASALT (ALKALI-BASALT) H UNDERSATURATED GABBRO I UNDERSATURATED DOLERITE J UNDERSATURATED BASALT	Page 1.6-7
F ULTRABASIC GROUP	A PERIDOTITE B PYROXENITE C PICRITE D CHROMITE;ILMENITE-,MAGNETITE-,APATITE-RICH ROCKS E ANORTHOSITE F SERPENTINITE (SERPENTINE) G ULTRABASIC EXTRUSIVES H UNDERSATURATED ULTRABASIC ROCKS I UNDERSATURATED ULTRABASIC EXTRUSIVES	Page 1.8
G LAMPROPHYRE GROUP	A BIOTITE-FELDSPAR-RICH LAMPROPHYRES B HORNBLENDE-,PYROXENE-,FELDSPAR-,RICH LAMPROPHYRES C UNDERSATURATED,FELDSPAR-POOR LAMPROPHYRES	Page 1.9
H CARBONATITE GROUP	A SÖVITE B ALVIKITE C RAUHAUGITE D BEFORSITE E CARBONATITE EXTRUSIVES	Page 1.10
I GLASSES	A RHYOLITE-ANDESITE GLASS B TRACHYTE GLASS C BASIC GLASS D ULTRABASIC GLASS	Page 1.11
J PYROCLASTIC ROCKS	A AGGLOMERATE: VENT AGGLOMERATE B BRECCIA:VENT BRECCIA C LAPILLI-ROCK D TUFF:ASH E WELDED TUFF:ASH:IGNIMBRITE F SILICIFIED TUFF:ASH G PUMICE	Page 1.12

Figure 21 : Extract from the Inst. Geol. Sciences 'Petrographical-Lithological Code', itemising igneous rock classes.

the present study, acting as a concise and complete listing of rock terms and reflecting traditional concepts of rock classification.

It explicitly makes no attempt at re-classification, ordering rather than re-ordering the existing terminology and allowing a four digit code to be given to any rock type (so that it can be handled by automatic data processing techniques). Their 'igneous' code is reproduced in Figure 21 by way of illustration. Column (i) is reserved for the symbol for igneous rock while column (ii) shows the major subdivision into 'natural associations recorded in the field'. Column (iii) records the principal members of these natural families and column (iv) allows a further stage in subdivision. The complete listing of column (iv) occupies no less than eleven pages of the code. A 'prefix code', as yet unformulated, allows further modifying terms such as weathered, porphyritic, to be added to the four digit coding.

There is an evident need to simplify this terminology for engineering use. Many terms relate to materials of insignificant abundance, others to observable but mechanically insignificant differences in material properties. Many attempts at simplification do little to alleviate these two basic problems, including, in the author's opinion, the two examples about to be discussed.

The British Code of Practice CP2001 (1957, 'Site Investigation') reduces the complexity of terminology without greatly increasing its relevance to engineering properties. Two tables from this code are reproduced in Figure 22 and these are supplemented in the code by a glossary of geological terms.

Group	Rock type
Sedimentary	Sandstones (including conglomerates): Silicious, Calcareous, Ferruginous, Argillaceous Some hard shales and tuffs Limestone: Massively bedded (including chalk), Thinly bedded
Metamorphic	Some hard shales Slates Schists Gneisses
Igneous	Granite Dolerite Basalt

Class	Acid	Intermediate		Basic
Approximate silica content	80–65 per cent	65–60 per cent	60–52 per cent	52–45 per cent
Extrusive or volcanic (lava flows, glassy or fine-grained)	Rhyolites	Trachytes	Andesites	Basalts
Intrusive or hypabyssal (sills and dykes, medium-grained)	Quartz-porphyry	Porphyries	Porphyrites	Dolerites and diabases
Deep-seated or plutonic (large masses, coarse-grained)	Granites	Syenites	Diorites	Gabbros

Figure 22 : Simplified rock classification after British Code of Practice CP2001, 1957.

The traditional geological approach is used in a greatly simplified form, without any attempt to extract mechanical information from the geological observations.

British Standard 812 (1967) for sampling and testing of mineral aggregates and fillers contains a standard 'group classification of aggregates' reproduced in Figure 23. Before the introduction of this classification a purchaser of aggregate had to contend with not only the complex geological terminology but also an abundance of colloquial terms, such as trap, ragstone and cornstone. Quarrymen could hardly be blamed for having a vested interest in calling their rock by a favourable term such as granite or quartzite. The group classification provided a practical solution to this problem and rock aggregate is now classified by means of a group name, this being supplemented by a brief petrological description and by test data.

An important step in the direction of adaptation rather than straight simplification of geological methods has been made by Duncan (1966) whose classification system is reproduced in Figure 24. The most important point of divergence of his system from traditional geological methods is that it is a textural, rather than a mineralogical classification. The texture, specifically the crystallinity and intergranular coherence as tested with the blade of a penknife, is subdivided in Column I of his table and governs the 'group number' given to a rock.

The group number is supplemented by a letter C (calcareous) or N (non- or part-calcareous) indicating mineral composition. Although this is perhaps an oversimplication it lays emphasis on two characteristically different types of mineral assemblage, the 'siliceous' versus the 'chemical'. Others may be added. His approach to a simplified mineralogical grouping may be compared with that of Mendes et al (1966) who attempting to set up a mineralogical quality index for rocks comprising four compositional categories; 'sound minerals', 'sound minerals having an adverse influence or altered minerals', 'clean microfissures and voids' and 'infilled microfissures'. The 'provenance types' of the metamorphic rocks suggest further possibilities for simplified mineralogical classification (Figure 19).

Duncan chose to classify rocks mainly on the tenacity with which grains stick together and on their hardness, and although one can disagree on points of detail his system bears close examination. It requires only objective observation and so rocks may be classified by the engineer with little or no geological training; yet it could well prove more relevant to engineering problems than more elaborate geological descriptions due to its emphasis on texture rather than detailed composition, grain size or concepts of genesis.

In suggesting various approaches to the adaptation of geological methods for engineering, only a few examples have been cited. It is doubtful whether further discussion along these lines would serve a useful purpose at this stage. Observations alone, however well chosen, provide only limited mechanical information and the following chapter sets out to discuss how observations might usefully be supplemented using data from physical and mechanical tests. The two approaches, both observation and testing, will be brought together in Chapter four, in an attempt to combine the advantages of each in an engineering rock description.

Artificial group
Crushed brick
Slags

Basalt group
Andesite
Basalt
Basic porphyrites
Diabase
Dolerites of all kinds including theralite and teschenite
Epidiorite
Hornblende-schist
Lamprophyre
Quartz-dolerite
Spilite

Flint group
Chert
Flint

Gabbro group
Basic diorite
Basic gneiss
Gabbro
Hornblende-rock
Norite
Peridotite
Picrite
Serpentine

Granite group
Gneiss
Granite
Granodiorite
Granulite
Pegmatite
Quartz-diorite
Syenite

Gritstone group
Agglomerate
Arkose
Breccia
Conglomerate
Greywacke
Grit
Sandstone
Tuff

Hornfels group
Contact-altered rocks of all kinds except marble

Limestone group
Dolomite
Limestone
Marble

Porphyry group
Aplite
Dacite
Felsite
Granophyre
Keratophyre
Microgranite
Porphyry
Quartz-porphyrite
Rhyolite
Trachyte

Quartzite group
Ganister
Quartzitic sandstones
Re-crystallized quartzite

Schist group
Phyllite
Schist
Slate
All severely sheared rocks

Figure 23 : Simplified rock classification after British Standard 812 (1967)

The aggregate "trade groups" are designed to ensure uniformity in descriptions of quarry rock to be used as concrete aggregate. They bear some relation to mechanical performance but there is room for wide mechanical variation within each group.

FIELD IDENTIFICATION AND CLASSIFICATION OF ROCK MATERIALS

Apart from group VI, group number is based upon texture only.

GROUP	TEXTURE	STRUCTURE	COMPOSITION	COLOUR	GRAIN SIZE
I.	**Crystalline** (Fig. 41) Consists entirely of interlocking crystal grains or crystals. No particles freed when sample scratched with blade of penknife. Term restricted to strong rocks.	**Homogeneous** Grains and crystals apparently arranged randomly. No visible linear or planar structure. (Figs. 41, 42, 43, 44 and 45).	**Non-Calcareous** Calcium Carbonate is absent: the material will not react with dilute H.Cl. (i.e. no effervescence). In this event place (N) after the group number.	The specific colour of the rock should be noted as an aid in assessing its composition and also for purposes of correlation.	**Coarse Grained** Particles larger than 2mm in diameter or in largest dimension. Particles will easily be visible to the naked eye—this size corresponds to gravel-sand boundary in the soil classification.
III.	**Indurated** (Fig. 42) Strong rock material; grains cannot be freed when rock scratched with penknife blade. Crystals and crystal grains are not visible in the sample.	**Lineated** Particles show a preferred orientation but of a LINEAR rather than a planar nature. (See Fig. 46).	**Part-Calcareous** Non-calcareous material, (e.g. quartz grains) present in substantial amounts but the material reacts with dilute H.Cl. In this event place (N) after the group number.		**Medium Grained** Particles between .1mm and 2mm and visible to naked eye. .1mm corresponds with the sand-silt boundary of soil classification.
II.	**Crystalline-Indurated** (See Fig. 43) Strong rock material; a combination of the two above textures, i.e. some crystals or crystal-grains only are visible.	**Intact-Foliated** A planar structure is visible in the rock but closed or incipient fracture is absent. The intact foliation may be produced by the alternation of dark and light layers. (Fig. 47).	**Calcareous** Calcium carbonate is the main constituent. The material reacts with dilute H.Cl. (i.e. effervesces). In this event put (C) after group number.		**Fine Grained** Particles less than .1mm and invisible to the naked eye.
IV.	**Compacted** Cement bond between grains is absent, the particles being held together purely by tightness of packing. Usually applicable to fine grained materials where particles are invisible to naked eye. (Fig. 44).				
V.	**Cemented** (Fig. 45) A bond is present between the grains which are normally visible to the eye. **Note;** Groups IV and V—particles freed when samples scratched with penknife blade.				
VI.		**Fracture-Foliated** See Fig. 47 right of fig. A planar structure exists associated with closed or incipient fracture, e.g. bedding planes, cleavage.			

(1) For a full description of any rock material, select ONE term only from each of the columns above.
(2) Note that Crystalline, Crystalline-indurated and Indurated rock materials, i.e. Groups I, II and III do not free particles or grains when scratched with blade of penknife. Rocks of Groups IV and V, when scratched with knife blade, free grains.
(3) The Texture governs the GROUP NUMBER which is ascribed to the sample.
(4) If the rock is calcareous add (C) to the group number.
(5) If the rock is non- or part-calcareous add (N) to the group number.

Example: Crystalline, homogeneous, non-calcareous, light brown, medium grained rock material of Group I (N).

Figure 24 : Duncan's (1966) rock classification

He supplements this classification with measurements of porosity and density.

3. ROCK TESTS - THE EXTENSION OF VISUAL CLASSIFICATION

(a) Introduction

A framework for a rock classification based on observed properties was presented in the previous chapter. Not all aspects of rock character that are of engineering importance can be directly observed, or even inferred from observation however, without some form of mechanical test. Mechanical tests may be described as observations of the *response* of rock to a change of environment. This change of environment may take the form of stress application to the boundary of a specimen, as is the case with strength tests, or may involve the change of conditions within the specimen, wetting, freezing, heating or exposing to weather for example, as is the case with durability testing. Strength and durability testing are two major categories that may prove useful in classifying rock, and they will be discussed in detail below.

Not all tests involve observation of the response of rock to a mechanical stimulus. Some, such as the 'physical tests' to measure porosity or density, or the 'geophysical tests' (to measure sound velocity in rock, resistivity or thermal conductivity for example) measure virgin rock properties that involve only minor changes in specimen environment, or none at all. Other tests such as the use of stains to accentuate rock fabric or mineral content are merely a direct extension of observation techniques. These types of tests also are important in their potential to assist rock classification.

A summary of 'laboratory' rock tests in common use is presented in Figure 25. There are many tests, and many ways of categorising these tests in addition to those described above. They may be grouped as destructive or non-destructive, static or dynamic, or into classes based on their common sphere of application, aggregate tests for example. Many tests are available to classify rock. Criteria that may be used for the selection of classification tests were outlined in Chapter 1 and were summarised in Section 1(e) of that chapter. It may be emphasised that the object is to select a few simple tests that will together form a rock classification that is practical, and reflects rock character sufficiently well to be useful for a wide range of engineering problems.

Some properties, however, are better observed than tested. The fracture spacing in a rock mass in one such property, since although we would prefer to measure the influence of such fractures mechanically the required quantity of large scale testing would be prohibitive. Some properties may be measured by either observation or by testing, and the choice must be made on practical grounds.

Many of the tests listed in Figure 25 may at once be discounted as classification tests on the grounds of complexity, particularly those tests whose main function is to estimate rheological properties of rock materials. Tests to estimate strength as a function of stress, or to evaluate elastic, plastic or viscous parameters usually require many minutes or hours for the preparation and testing of specimens. Parsons and Hedley (1966) suggested the use of viscosity, and Voight (1968) proposed a classification based on elastic anisotropy . Even a classification such as that of Deere and Miller (1966) based on Young's

1. *Physical tests and aids to observation*

   Techniques to assist the observation of colour, grain size, mineralogy, surface roughness etc. Tests to quantify permeability, porosity, pore size, density and specific surface. Geophysical assessment of rock character using electrical, sonic, radiation, thermal and other techniques.

2. *Tests for surface 'hardness'*

   Rebound, scratch and indentation estimates of 'hardness'; abrasion and polishing tests; microdrill tests; friction tests on rough and smooth surfaces.

3. *Tests for crushing strength, impact and attrition resistance*

   Impact tests such as employing a pendulum or falling weight; attrition tests on aggregate in revolving drums; crushing tests on single lumps or containers full of aggregate; simple tests 'borrowed' from Group 4, such as uniaxial strength tests, Brazilian or bending tests, and used as an index of rock strength.

4. *Tests to explore rock strength as a function of stress state*

   Triaxial thick, thin or solid cylinder tests employing combinations of internal or external pressure, axial stress and torsion to achieve a variety of stress states; tests to achieve a variety of tensile stress conditions, direct tension, extension, bending, theta or ring specimen tests, point load tests on discs, cubes, plates etc.; direct shear, double shear and 'punch' tests; tests for thermal spalling and cratering with explosives; tests for strength anisotropy.

5. *Tests to explore rock deformability*

   Tests as in Group 4 but using deformation or strain measurement to estimate rheological parameters, elastic, plastic or viscous; creep tests; tests at elevated temperatures and those meant to explore the 'brittle-ductile transition'.

6. *Durability tests*

   Tests to determine response of rock material to a change of environment; slaking tests and wetting and drying cycles; salt crystallisation tests; freeze/thaw tests; acid immersion tests; ultrasonic cavitation; natural exposure.

Figure 25: Some 'Laboratory' rock tests

There are many tests available, and those most suited to practical rock classification must be identified. The selection may be based on criteria listed in Chapter 1.

modulus and uniaxial strength would appear of little practical value since the classification of just one rock sample would require the testing of several specimens, and a day or so of work in the laboratory. Such properties are certainly of considerable interest and relevance to engineering problems but their measurement can never form part of a basic rock classification system. Much of the latter part of this thesis is devoted to classifying rock on the basis of triaxial test parameters. This test is not at all suitable for basic rock classification, and should also be regarded as providing supplementary information.

The following discussion covers a range of potentially useful techniques by means of examples that are intended to give some perspective to the problems, rather than to provide complete coverage of all possibilities. Emphasis has been given to certain aspects, such as the measurement of pore parameters, that appear to have much potential and yet have received little attention in the field of rock engineering. Chapter 4 will contain proposals for a set of tests to be used, together with the observations described earlier for the basic engineering classification of rock materials.

(b) Aids to visual examination

One must be able to see features before they can be measured. Many textural or compositional features are obscure and difficult to observe, not to mention measure. The current discussion will relate to techniques and instruments that can be used to accentuate features of mechanical importance in the rock texture, allowing them to be readily observed and perhaps later quantified by visual methods that will be described.

Staining techniques may be used to improve the ease of identification of certain minerals, and were investigated in the hope that they might assist the rapid field estimation of mineral content. However, they were found to require rather more care and skill to obtain satisfactory results than one would normally expect of a field technique, and are also restricted in the range of minerals that can be stained. They are widely used to study rock mineralogy in the laboratory and appear to have certain limited, though important, uses on rock engineering studies; these will be outlined below.

Staining techniques are available to distinguish varieties of feldspar mineral (for example Barratt & Parslow, 1966), to distinguish the various carbonate minerals (Katz & Friedman, 1965) and to distinguish certain varieties of clay mineral (Mielenz & King, 1951). Feldspars are stained by etching with hydrofluoric acid fumes and applying sodium cobaltinitrite solution, when potassium feldspars appear yellow, sodium and calcium feldspars white, quartz remaning unstained. Carbonate rocks, or rocks containing carbonate minerals or cement, may be treated with Alizarin Red and Potassium Ferricyanide, either separately or as a mixture. Alizarin Red stains the calcite grains red, leaving dolomite colourless, while potassium ferricyanide imparts a blue colour to iron bearing calcite or dolomite. Hamblin (1962) describes the use of an Alizarin Red colour wash to accentuate the bedding in sedimentary rocks; concentrations of clay minerals that are often present along bedding laminations are coloured a dark red by this technique. The aqueous dye is probably adsorbed by these clay minerals, rather than there being any chemical reaction involved. The technique might also prove useful for accentuating weathered or altered minerals in granitic rocks, since the weathering products are also clayey and adsorptive.

A permanent record of the stained grains and also of the granular and pore texture may be obtained using the *acetate peel* replica techniques that are also described by Katz & Friedman. Acetate peels often provide a better clarity of textural features than displayed by the original rock specimen. Hamblin found *sandblasting* a useful method of accentuating obscure bedding in sedimentary rocks, reproducing the effects of natural erosion that often reveal features invisible in freshly cut specimens.

Dye penetrants may be used to accentuate the contrast between layers of greater or lesser permeability, and may sometimes assist in the observation of individual pores. Dye, as opposed to stain, techniques do not require a chemical reaction between the fluid and the mineral grains. Indian ink has often been used for this purpose. Pantin (1960) describes a method for softer sediments. The dye is applied and then the surface layer shaved off, revealing the more permeable parts of the rock stained darker than the less permeable zones. *Fluorescent dyes* may also be used in conjunction with ultraviolet light examination to reveal porous textures. Gardner & Pincus (1968) describe the use of this technique to reveal strain cracking in rock slices.

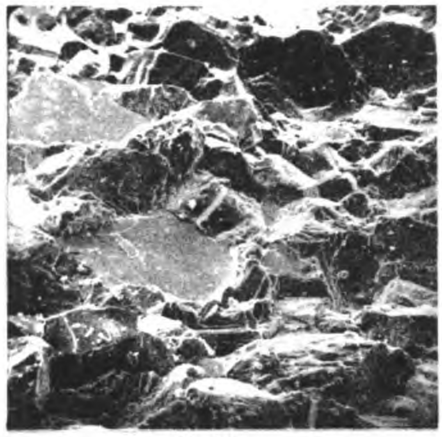
For detailed study, particularly quantification, of pore textures it is an advantage to fill pores with a solid rather than a liquid. A penetrant that solidifies inside the pores or cracks minimises disturbance during the preparation of specimens for microscopic study. *Rock impregnation* is essential when studying the more friable types of rock, although such complications are to be avoided if at all possible. Several impregnation techniques have been investigated by the author (Franklin, 1969) and a most satisfactory degree of impregnation was achieved using the monomers styrene or methyl methacrylate which have a viscosity less than that of water. The monomers were polymerised chemically to form polystyrene or perspex, and were treated with dyes prior to impregnation in order to produce good contrast between the pores, and the rock skeleton. The dying techniques were not entirely effective since soluble, non pigment materials had to be used to ensure adequate penetration and these were almost colourless in thin section. The techniques were developed primarily for use with the 'Quantimet' quantitative image analysis instrument that will be described later.

Nuss & Whiting (1947) describe a technique for reproducing rock pore space, the pores being first impregnated with plastic solidified by polymerisation, and the rock skeleton was removed by solution in hydrochloric or hydrofluoric acid, leaving a cast replica of the pore structure. Their impregnation technique is to be recommended, particularly the use of injection under high pressure where this does not damage the specimen. Satisfactory penetration is rarely achieved by vacuum alone, and the fluidity, rate of polymerisation and mechanical properties of the impregnating plastics are usually greatly improved by a combination of elevated temperature and pressure. The apparatus can be quite simple.

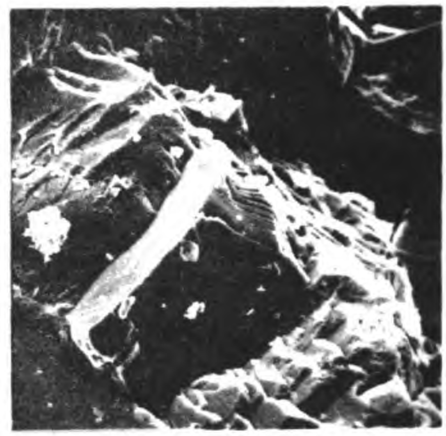
The observation of rock composition and texture may be assisted by any of a number of *microscopic techniques* ranging from a pocket magnifier to an electron microscope. Undoubtedly the most powerful technique is the study of thin sections of rock using a conventional petrological (transmitted light) microscope. Textures that are often obscure when viewed with reflected light become obvious, and the use of polarised light illumination readily allows mineral identification. A thin section may be prepared in a matter of minutes, and even in engineering applications will often justify the effort involved.

A simpler technique allowing the routine examination of hand specimens is to use a stereo-binocular reflected light microscope. The type used by the author had ample clearance for the manipulation of a fairly large hand specimen, and a zoom lens allowed points of interest to be first located and then examined under a higher magnification. A lower magnification, larger field of view and depth of focus than in the case of petrological microscope, allowed a better appreciation of coarse textural features; the porosity of the material, for example, could be better appreciated by observing the behaviour of a drop of water placed on the surface of a specimen. The magnification and image clarity were quite adequate to allow a rough estimation of mineral content, probably sufficient for most engineering purposes, and much more could be observed than with the unaided eye.

Scanning electron microscopy shows much potential for the study of rock materials. Although the equipment is expensive, the specimen preparation required is minimal. A lump of rock measuring perhaps one centimetre in diameter is mounted with aluminium paint on a metal stud. The only complication is that the specimen must be coated with a conductive layer, usually gold evaporated onto the surface under vacuum, to prevent the buildup of electron charge that would obscure the image. A wide range of magnifications, from x20 to x2500, are possible, together with a much better resolution and greatly improved depth of focus than can be achieved using light rather than an electron beam to form the image. Typical results for a marble are illustrated in Figure 26. Weinbrandt and Fatt (1969) describe the application of this technique to the study of pore structures in a sandstone, and it proved of valuable assistance in a recent study of the skid resistance of roadstones undertaken by Imperial College Geology Department for the U.K. Road Research Laboratory.

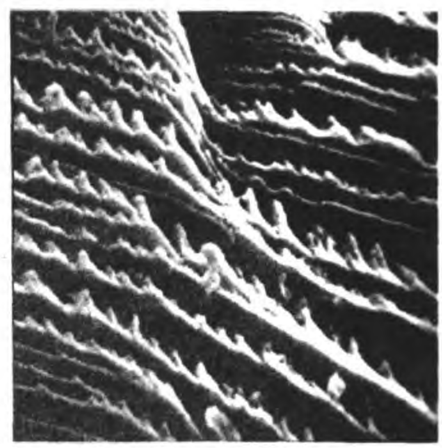


X 20



X 100

X 700



X 2500

Figure 26 : Scanning Electron Microscope.

A broken surface of Carrara Marble is viewed at magnifications of from x20 to x2500. Both depth of focus and resolution are superior to those that can be achieved using a light microscope.

(c) Quantification of Observed Properties

This section discusses some aspects of the quantification of a visual image, referred to elsewhere under various headings such as 'image analysis', 'granulometry' or 'grain mensuration'. The quantities or parameters used to describe a visual image are usually grossly inadequate when compared with the complexity of natural materials, but although much may be lost by quantification, much may also be gained. Quantitative measures of rock texture are essential if one is to relate texture to mechanical performanceon a quantitative basis.

Rock colour is a property that is most obvious and readily observed, yet is at the same time one of the most difficult to quantify. Although not of great value as a mechanical index, since colour changes can often be brought about by mechanically insignificant changes in rock chemistry, it should not be underrated as a descriptor. Colour changes can sometimes be used to identify marker horizons and be of value in stratographic correlation. Sometimes they are also associated with important changes in the mechanical character of a rock.

Quantification of rock colour requires that it be expressed in terms of three parameters, the 'hue' (a mixture of basic colours), the 'chroma' (brilliance or intensity of colour) and the 'value' (lightness of colour). The most recent standard for rock colour nomenclature is a colour chart published by the Geological Society of America (1963) this being based on the Munsell Colour Notation (1941). The chart is intended for field use. Hue, chroma and value form the axes of a 'colour solid' or 'colour sphere' reproduced in Figure 27. Rock colour is given a numerical designation based on these co-ordinates (e.g. 10YR 8/2) which may be supplemented by a name (for example 10G 6/8 intense light bluish green). It may be noted that wetting a rock changes the value, but not the chroma of a colour. Hosking & Ritson (1968) describe automatic colour measuring equipment.

When working in practice with a particular rock formation, it may prove easier to dispense with the colour chart by selecting representative rock chips as colour standards. The colours of these chips can be given their correct colour designation using the chart, to form the basis of a system of colour designation particularly suited to the local suite of rocks.

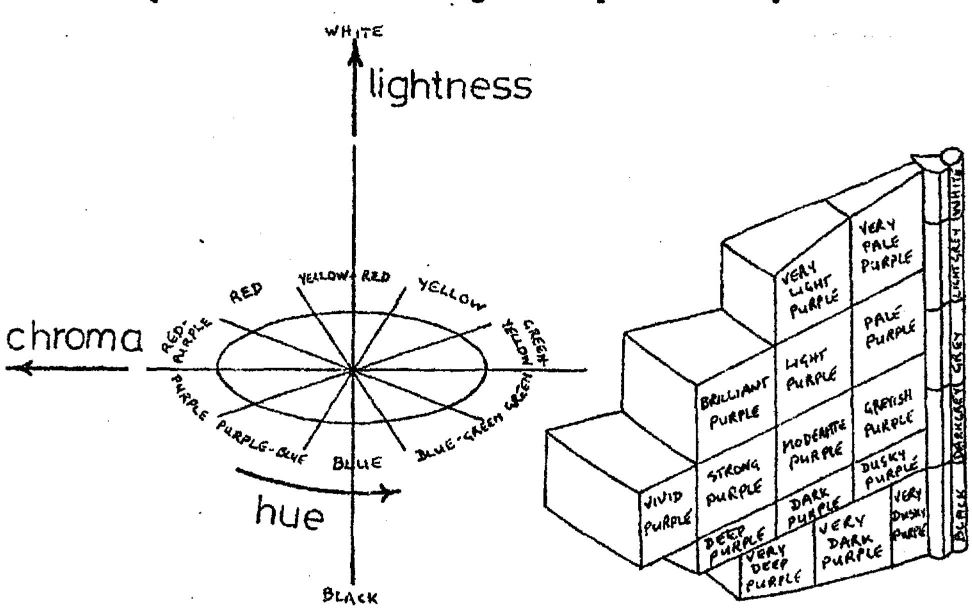
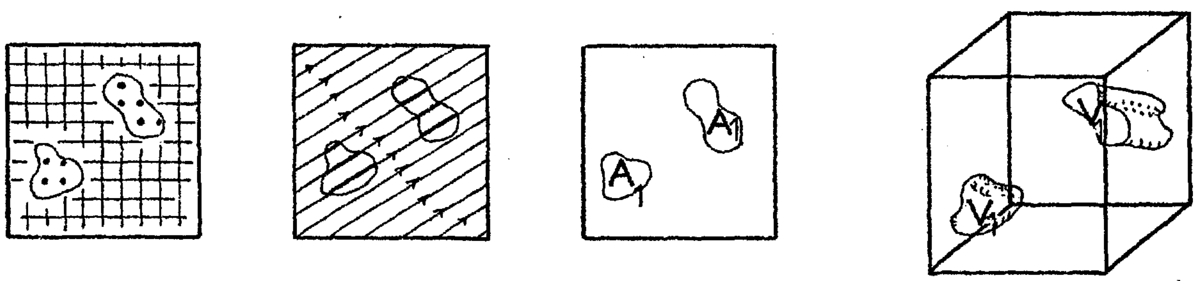


Figure 27: Quantitative Colour Description (from 'Rock Colour Chart', Geol. Soc. Amer.1963)

The dimensions of the 'colour solid' are hue, chroma and value. The 'purple' segment of the colour solid is also illustrated.

The *modal analysis* of a rock, the determination of mineral volumetric composition, is achieved by applying measuring techniques to a miscroscope image of the rock, and assumes that the various mineral or pore phases can be visually identified. Modal analysis is discussed in detail by Chayes (1956). The methods of estimating volumetric mineral content are typical of those used to quantify other types of textural observation, and will be outlined by way of illustration.

Volumetric percentages cannot be observed directly, since the observed image is planar. They must be derived from point counts, linear intercepts or area ratios. The derivation is based on an equality of observed ratios, illustrated in Figure 28, which may be demonstrated statistically.



$$\frac{\Sigma P_1}{\Sigma P} = \frac{\Sigma L_1}{\Sigma L} = \frac{\Sigma A_1}{\Sigma A} = \frac{\Sigma V_1}{\Sigma V}$$

Figure 28: Equality of Point, Line, Area and Volume Ratios in Granulometry.

Volumetric composition may be estimated by observing any of the three 'planar equivalents'.

The earliest attempts at modal analysis involved cutting around the sketched outlines of grains, and weighing the pieces. Hence area ratios were employed to predict volumetric proportions. This method was replaced by the use of planimetry. More recent and less cumbersome methods use point count or alternatively line integration techniques. The specimen is moved relative to a fixed cross hair in the microscope eyepiece, and a micrometer in the microscope stage is used as a measuring device, rather than a calibrated graticule in the optical system. The point count method employs a traverse in steps. A note is made of the phase lying beneath the cross-hairs after each increment of stage movement. The line integration method also involves a traverse, but the distance traversed across each phase is recorded by a series of dials, the dial corresponding to a particular mineral being brought into action by depressing a button as the phase comes under the crosshairs. Traverses may be random or systemmatic, in practice usually systemmatic, and sufficient parallel traverses are made across several areas of the specimen to ensure sufficiently accurate results.

Even with the assistance of devices such as the micrometer stage, manual methods of image quantification are extremely tedious, and progress has been made over the last fifteen years towards full automation. The *'Quantimet'* image analysing computer can reduce the time required for a modal analysis from many minutes to a matter of seconds, and its capabilities and limitations were investigated. The instrument combines a microscope, a television camera and a small computer, and is shown in Figure 29(a). Its capabilities are described in a series of articles in volume 16 of 'The Microscope', for example a paper by Fisher & Cole (1968). An image of the specimen is projected onto the screen of a television camera by a conventional or electron microscope or an epidiascope. This image is scanned by the 'flying spot' of the television camera at the intensity of 300 lines (traverses) per field of view. An electric signal is produced that is analogous to image brilliance, and the image may be divided into areas of greater or lesser 'greyness' by adjusting a brilliance threshold control. The parts of the image (the phases) to be measured are displayed as superimposed shading on a television monitor (Figure 29(b)) and phases of differing greyness may be selected by adjusting the brilliance threshold.

A selector switch allows the direct measurement of one of three quantities; the shaded area (integrated line traverse length), the projected length of the shaded area (total number of line intersections) or the number of shaded features. Also a size rejection control allows the measurement of area as a function of grain size (the grain size distribution of the material) or of area as a function of chord length (a shape parameter). Many other parameters may be derived from these basic observations, such as the 'form factor' or ratio of projected lengths in two orthogonal directions.

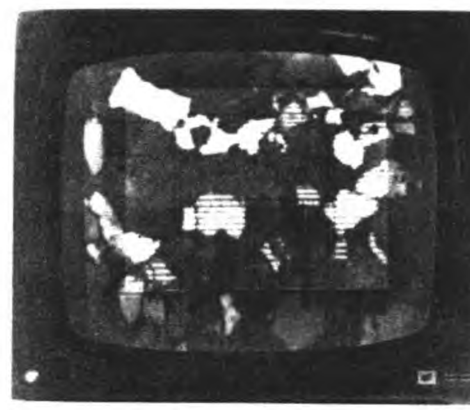
The quantimet's capability is, however, limited by its need to differentiate phases on the basis of brilliance or 'greyness' alone. The discerning power of the human eye is greater since more subtle considerations are used to discriminate between one phase and another. The instrument functions well, for example, on powders consisting on a suspension of discreet black particles in a white matrix, but when applied to thin sections of rocks can find it difficult to identify a grain boundary that might be quite obvious to the naked eye. Internal texture in some types of mineral grain (fossil textures, feldspar twinning for example) may often be misinterpreted as additional grain boundaries. Many of these misinterpretations can be avoided by careful adjustment of brilliance threshold, focus or resolution controls, and one can always observe mistaken interpretations since they are displayed on the television monitor. The instrument appears to have great potential, and the development of special specimen preparation techniques to accentuate phase contrast would seem justified. Some typical results obtained using this instrument are presented in Figure 29(c).

The *grain size* of a rock specimen may be estimated by using less sophisticated techniques than the one described above. Grain size measurements (see for example Allen, 1968) may be made using an eyepiece graticule such as that illustrated in Figure 30, a graticule that assists in the sizing of grains into logarithmic size categories. This graticule, described in British Standards 3406, (1961) and 3625 (1963), must be calibrated with a graticule on the stage of the microscope to allow for the change of scale due to magnification. A simpler method was used by the author, based on direct comparision of standard grain size slides with the specimen. These slides (Figure 31) were prepared by dividing a suitable sand into fractions on the complete range of standard sieves. The fractions were sprinkled onto blobs of adhesive on glass microscope slides. Several sprinklings ensured that a maximum of grains and a minimum of adhesive was visible, and the sand was chosen so that all fractions had approximately the same appearance apart from the size difference.

Quick and remarkably accurate estimates of grain size were possible, particularly with the aid of a stereo binocular microscope. The eye is very sensitive to size differences, and the use of comparison slides is perhaps the most practical solution to grain size estimation. The grain boundaries of crystal-net rocks are difficult to distinguish with the naked eye unless the specimen is very coarse grained, and it may be necessary to estimate grain size using a thin section. Comparison slides may be used in the same way, and Figure 32 reproduces the appearance of such slides as in A.S.T.M. standard E-112-63 (1966).

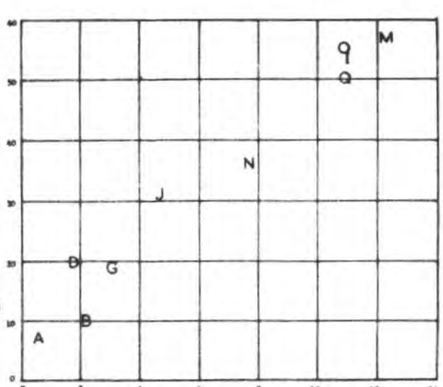
(a)

(b)



(c)

quartz % after optical modal analysis (Elliott & Strauss, 1965)



Quartz % (non-extinguished) using the Quantimet

Figure 29 : 'Quantimet' scanning television microscope.

Rapid image analysis is achieved and accurate quantitative results are possible provided the phases to be measured are in good optical contrast.

(thin sections for the study illustrated in 29c were lent by Elliott & Strauss)

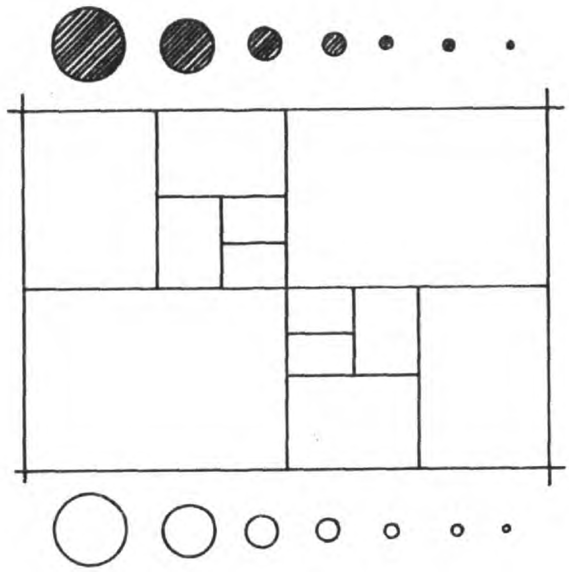


Figure 30 : British Standard graticule for grain size estimation. The size subdivision is logarithmic.

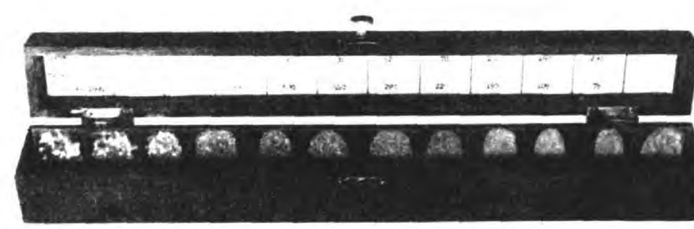


Figure 31 : Standard grain size slides for use with a binocular microscope

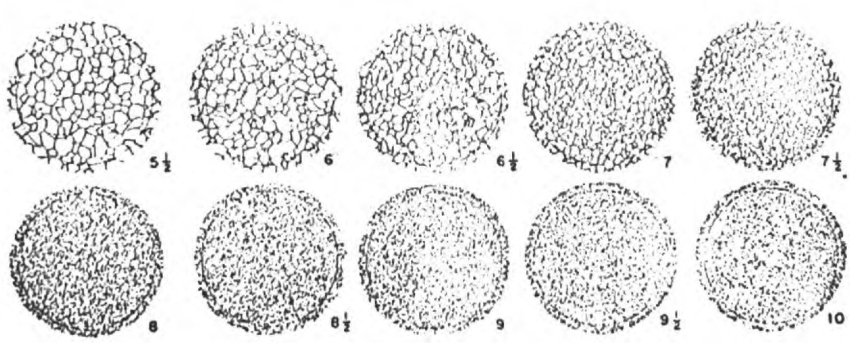


Figure 32 : Standard grain size graticules reproduced photographically, for use in the eyepiece of a transmitted light microscope (ASTM E-112-63)

Parameters such as grain size apply with precision only to very simple textural geometries, and are defined with reference to *simplified textural models*. The diameter of a spherical particle is precisely determined, whereas the grain size of an irregular particle can have several definitions. These simple models, although often unrealistic, are necessary for the definition and interpretation of the more complicated textural parameters and can often lead to a better understanding of the influence of texture on mechanical properties. The concept of textural models will be illustrated by referring to a model consisting of a bed of spherical particles of equal diameter.

Problems relating to the packing of particles are discussed by Gray (1968), who gives values for the coordination number (number of grain contacts per particle) and computed porosity corresponding to a given systemmatic packing of spherical particles with equal diameter. In the case of this idealised model there is a close relationship between coordination number and porosity. It proves interesting to compare this theoretical data with the experimental data available that relates porosity to strength for a wide range of brittle materials. The two types of data are illustrated side by side in Figure 33.

Packing Type	Coordination Number	Computed Porosity
	3	77.0
	4	66.0
	5	59.7
Cubic	6	47.6
	7	43.9
Orthorhombic	8	39.5
	9	32.0
Tetragonal	10	30.2
	11	28.2
Rhombohedral	12	26.2

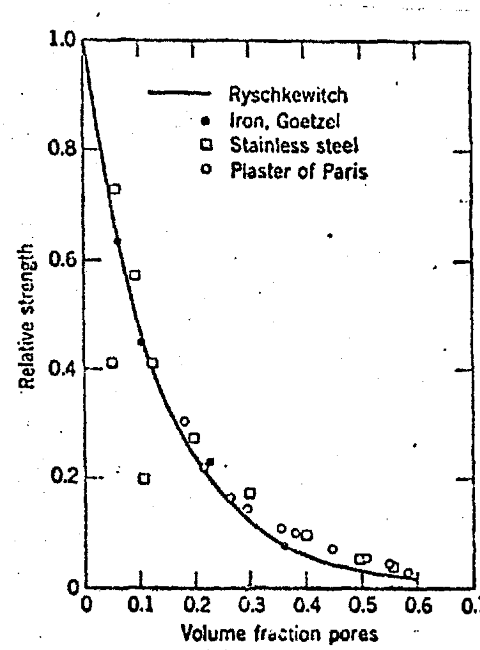


Figure 33: Porosity in theory; its effect in practice

Porosities calculated for ideal regular packings of spheres may be compared with the packing 'co-ordination number', the number of contacts per grain. The right hand diagram (Kingery, 1960) demonstrates that porosity plays a large part in determining mechanical behaviour.

Both the porosity and the co-ordination number of the model material are independent of grain size. Porosities encountered in practice are generally very much lower than those of the model, since grains are often far from spherical and are usually a mixture of different sizes. Porosity is by no means completely independent of grain size in practice, and a reduction in grain size is usually accompanied by an increase in porosity of fragmental rocks. A given porosity may be due to a few large pores or many small ones, and there is an obvious need for a second parameter in addition to porosity for the fuller definition of a porous texture. This second parameter may well be the pore size of the material, or its permeability. Many textural models have been proposed in attempts to relate porosity, pore size, 'tortuosity' of pores, 'bottleneck' size and other parameters to permeability, and some of these models are illustrated in Figure 34. Attempts to estimate values for some of these parameters from observation of rock specimens can give rise to considerable practical difficulty but the models can nevertheless assist in giving a deeper insight into rock behaviour. The quantification of rock textural observations is discussed in further detail by Pincus (1969) and by Haas et al (1967).

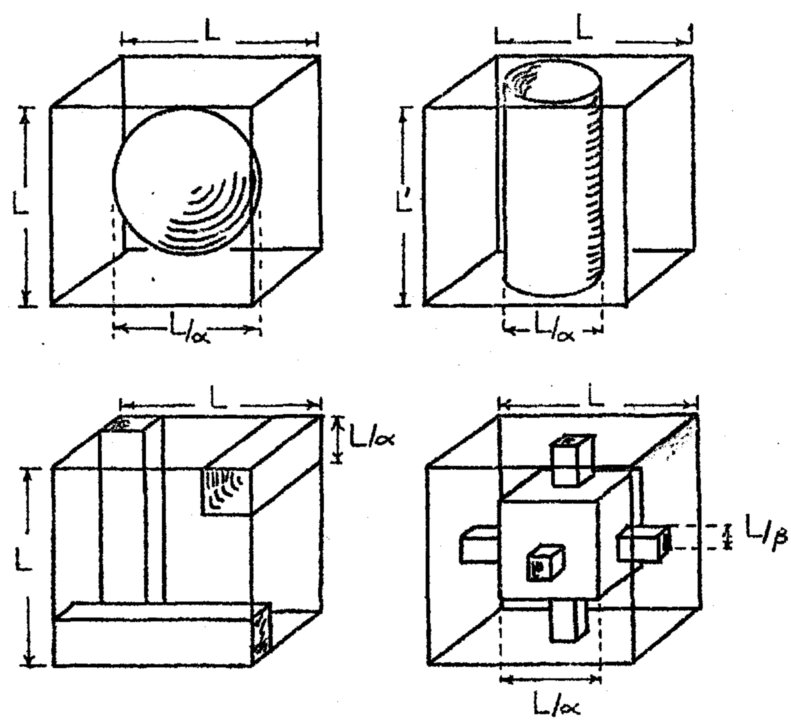


Figure 34: Idealised models of pore texture.

Many models are possible, and may be used to calculate the influence of pore shape on permeability. They may also be useful in studying the potential mechanical behaviour of a porous solid. The models illustrated here are those used by Flood (1968).

There is no reason why the granulometry of a rock mass should not be analysed in just the same way as granulometry on a microscope scale. Observations have an important part to play in predicting mechanical properties of bulk rock, since there is a limit to the amount of large scale mechanical testing that may be usefully and economically carried out.

Although parameters relating to large scale features are, strictly speaking, outside the scope of this thesis, the scale limitation is not to be given undue emphasis and many of the techniques apply irrespective of the scale of specimen. The *fracture porosity* of a rock mass will be discussed as a final example of the methods available for quantifying visual observations.

Using simple geometrical or statistical reasoning one can show that the contribution of a single set of parallel fractures (or joints) to the porosity of a rock mass is given by the ratio of their openess (width) to their spacing. A simple model where the fractures are equally spaced and have equal openness is used for this illustration. A model where three orthogonal fracture sets with the same properties of openness and spacing divide the rock mass into cubes, has a fracture porosity approximating to three times that for a single fracture set, and one can formulate a precise expression that takes fracture overlap into account. The dependence of fracture porosity on the ratio of fracture openness to spacing is illustrated graphically using both approximate and precise formulations, in Figure 35. The value of such a model can be seen since it allows one to estimate the importance of fracture overlap at various levels of fracture intensity. Also it is possible to estimate the likely range of fracture porosities that might be encountered (given data for fracture widths and spacings) or to assess widths and spacings if the fracture porosity can be estimated by other means.

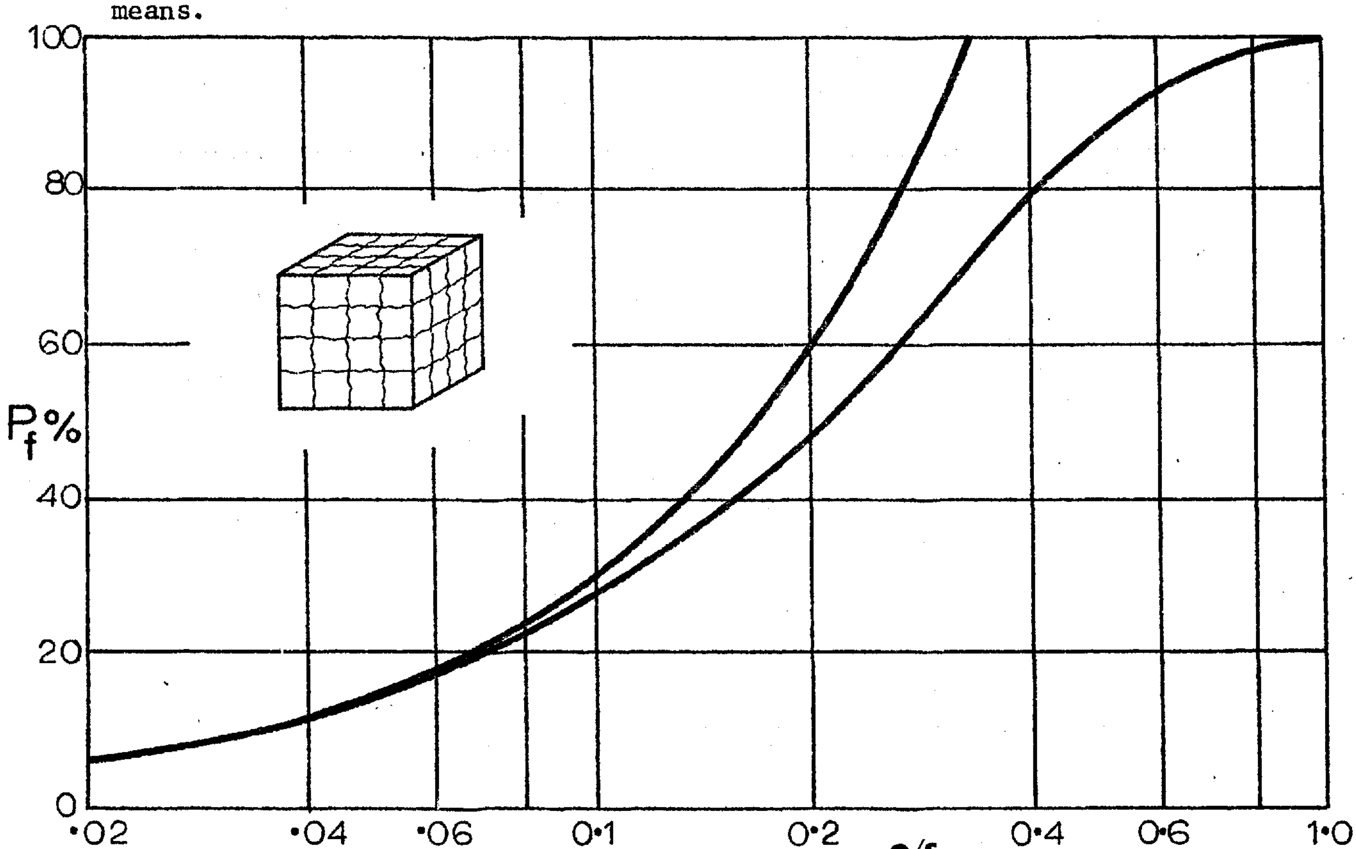


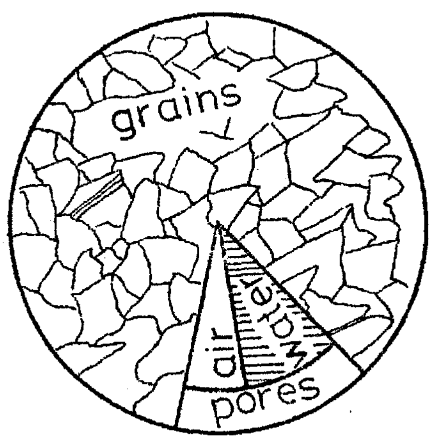
Figure 35: Rock mass fracture porosity

A simple idealised model of a cubic fracture system where each of three fracture sets has spacing 'S' and openness 'O', may be used to examine the relationship between fracture porosity P_f and the ratio O/S. Approximate and precise relationships have been plotted. For realistically low values of O/S the approximation proves adequate for engineering use.

Fracture porosities were estimated by Snow (1968) using indirect methods based on the measurement of permeability. They ranged between 0.04% and 0.0005%. The higher value, using the model of Figure 35, corresponds to a ratio of openness to spacing of 1:100. One can see that the contribution of fracture porosity to total porosity, even for this closely jointed and loosely packed rock, is rather small. The contribution of fractures to an increase of permeability or a decrease of strength is of much greater significance.

(d) Experimental Determination of Pore & Density Parameters

Basic definitions of porosity and density parameters are tabulated below in terms of the weights and volumes of the rock constituents. There is wide variation in the terminology and symbols used in the literature, and self descriptive terms and symbols have been chosen.



Rock Constituents

Grain weight or volume (G_w, G_v)

Water weight or volume (W_w, W_v)

Air volume (A_v)

Total weight (T_w) = $G_w + W_w$

Pore volume (P_v) = $A_v + W_v$

Total volume (T_v) = $P_v + G_v$

Water Density (γ_w) = W_w/W_v

The following parameters are commonly used:

DENSITIES (Specific gravity = density / γ_w)

1. Dry rock density (γ_r) = G_w/T_v
2. Saturated rock density (γ_s) = $(G_w + P_v\gamma_w) / T_v$
3. Grain density (γ_g) = G_w/G_v

POROSITIES

4. Porosity (P) = P_v/T_v
5. Void ratio (R_v) = P_v/G_v

SATURATIONS

6. Saturation (S) = W_v/P_v
7. Moisture content (M) = W_w/G_w
8. Water absorption or Saturated Moisture Content (M_{max})

$$M_{max} = \frac{W_w \text{ maximum}}{G_w} \simeq \frac{P_v\gamma_w}{G_w}$$

All parameters may be evaluated once the values for one parameter in each category have been given. The following are examples of the interdependence relationships; dry rock density and porosity have been taken as known, from which:

$$\gamma_s = P\gamma_w + \gamma_r$$

$$\gamma_g = \gamma_r/(1-P)$$

$$R_v = P/(1-P)$$

$$M_{max} = P\gamma_w/\gamma_r$$

The grain density does not vary greatly and for approximate calculations its value may be estimated and used together with porosity or rock density to compute the remaining parameters. Typical interdependence relationships are as follows:

$$\gamma_r = \gamma_g\gamma_w\ (1-P) \qquad \gamma_s = \gamma_w\ (P + \gamma_g(1-P)\ ) \qquad M_{max} = P/\ (\gamma_g(1-P))$$

γ_g/γ_w is approximately 2.65 for quartzofeldspathic rocks, 2.7 for carbonate rocks, 2.5+ for clay minerals and 3.2+ for dark minerals.

The ranges of porosity and specific gravity for some common rock types are illustrated in Figure 36. A rock whose grain density were constant but whose porosity varied would plot along a straight line given by the equation rock S.G. = grain S.G. (1-P), and a spread perpendicular to this line reflects a variability in mineral content, hence in grain specific gravity. Rocks intersect the zero porosity axis at specific gravity values characteristic of the mineral assemblage.

The *basic measurements* required for experimental determination of these pore and density parameters are discussed in detail by Manger (1966) and also in the Recommended Practice of the American Petroleum Institute (1960), and are described briefly below:

The total volume (bulk volume) of a specimen may be determined by dimensioning if a regular shaped specimen is available. A water displacement method may be employed, either by coating the specimen with wax or plastic and correcting the water displacement volume for the volume of coating, or by measuring the difference between saturated surface dry and saturated submerged weights, equal to the weight of water displaced according to Archimedes principle. Total volume may also be measured by forcing the specimen under mercury and measuring the resulting fluid displacement. The high surface tension of mercury prevents it from penetrating all but the largest of pores.

The second basic measurement required is that of grain volume. This may be determined by crushing the rock to a powder and displacing fluid in a pycnometer. This technique allows the calculation of 'total' porosity as opposed to 'effective' porosity; calculations based on all other techniques include closed pores in the estimate of grain volume. Grain volume may also be assessed using a Boyle's law method. The pressure-volume relationship is obtained for a container containing gas only, and then gas plus specimen, and the two results compared. The difference in compressibility is due to the volume of incompressible grains and this volume may be calculated from the results.

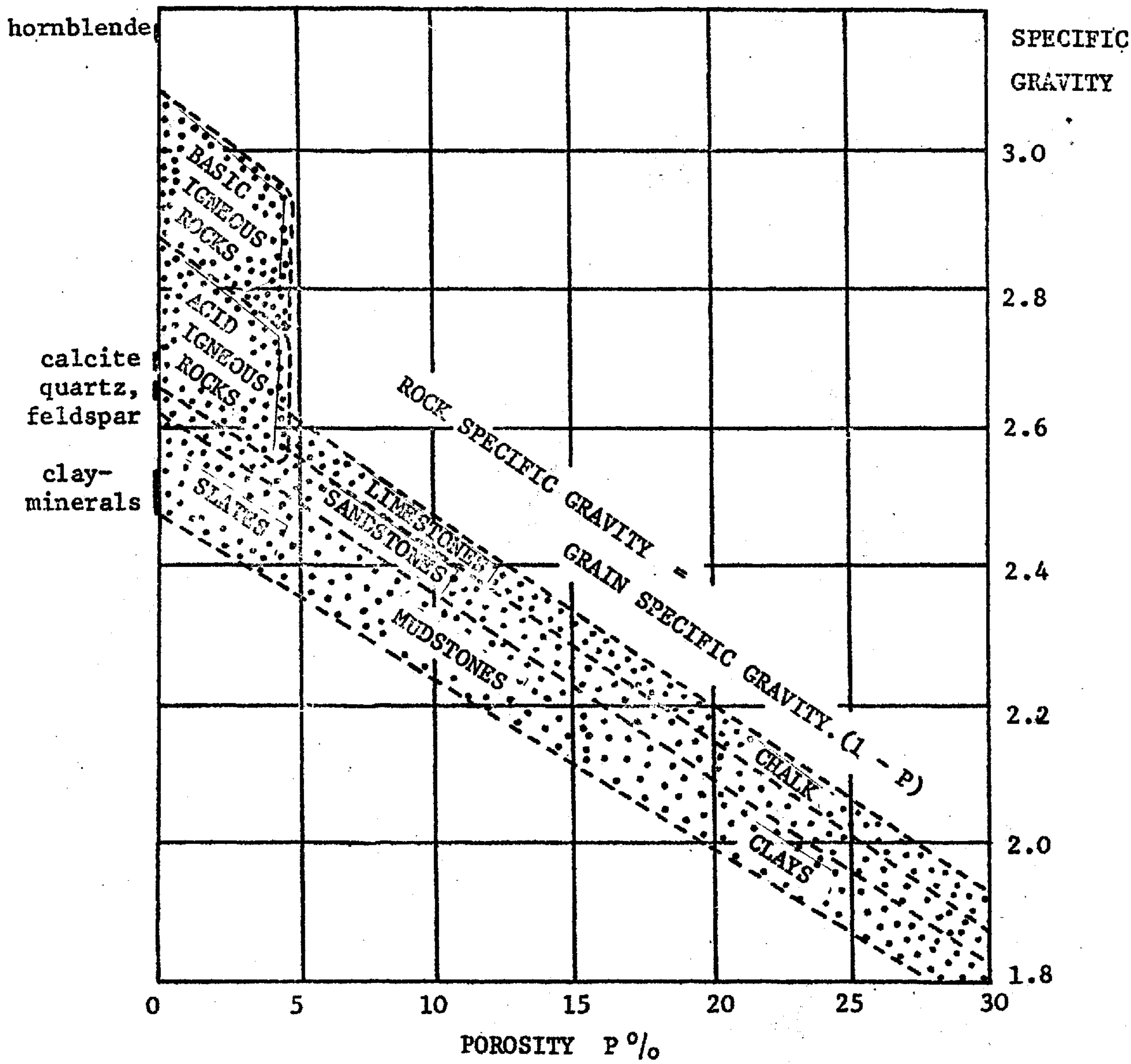


Figure 36: Typical porosity and specific gravity values for a range of common rocks.

Rock porosity and specific gravity are related by a linear expression provided that a unique mineralogy (hence average grain specific gravity) is assumed. Rocks with different mineralogy plot as different straight lines.

Measurements of pore volume offer an alternative to those that measure grain volume, provided that total volume has also been measured. Pore volume may be assessed by water saturation, the difference between saturated and oven dry weights giving the weight of water to fill the pores. Alternatively mercury may be forced into the rock pores at such a pressure that the volume of trapped air becomes insignificant. The volume of injected mercury then is a measure of pore volume. This method is expensive since the mercury is difficult to removed from the rock specimens. The 'Washburn Bunting' method of pore volume measurement removes air from the specimen by applying a vacuum. The air rises into a measuring cylinder, trapped by the mercury in which the specimen is immersed, and the pressure is equalised with that of the atmosphere so that its volume may be measured.

Finally, for density calculations the grain weight is required. This is obtained simply by removing pore water and weighing the specimen oven-dry.

Errors are involved in all these techniques but may be minimised if large volumes of rock are tested. Most of the errors are associated with surface effects, such as the difficulty of reproducing saturated-surface-dry conditions, and larger specimens may result in increased accuracy since they minimise the ratio of surface area to specimen weight. Porosity measurements below 1% should be regarded as within the range of possible experimental error unless special precautions are taken.

The basic methods may be used in any suitable combination. Two alternative techniques will be discussed, both in common use.

Wet method

Saturate the specimen; weigh saturated submerged; weigh saturated surface dry; weigh oven dry. (c.f. BS812 'Sampling & Testing of Mineral Aggregates')

Mercury method

Oven dry; Weigh; Displace mercury to measure volume; Use mercury to trap air and apply pressure giving P-V relationship and grain volume.

Although appearing complex, the mercury method is simpler and quicker. It requires no weighing for porosity determination whereas the wet method requires both surface drying and weighing to determine total volume. Morgenstern & Phukan (1966) present a simple technique using a U-tube. An alternative mercury method is discussed in detail by King (1966) whose cell is illustrated in Figure 37. The cell is portable and can test specimens of rather limited dimensional tolerance at rates of 20-30 per hour. With the outlet valve open the volume of mercury to fill the cell is measured, first with the cell empty, then when it contains the specimen. The difference gives the specimen total volume. The cell is then closed and a pressure-volume relationship measured with and without the specimen from which the grain volume may be determined. In practice the determination is simpler since certain values remain constant.

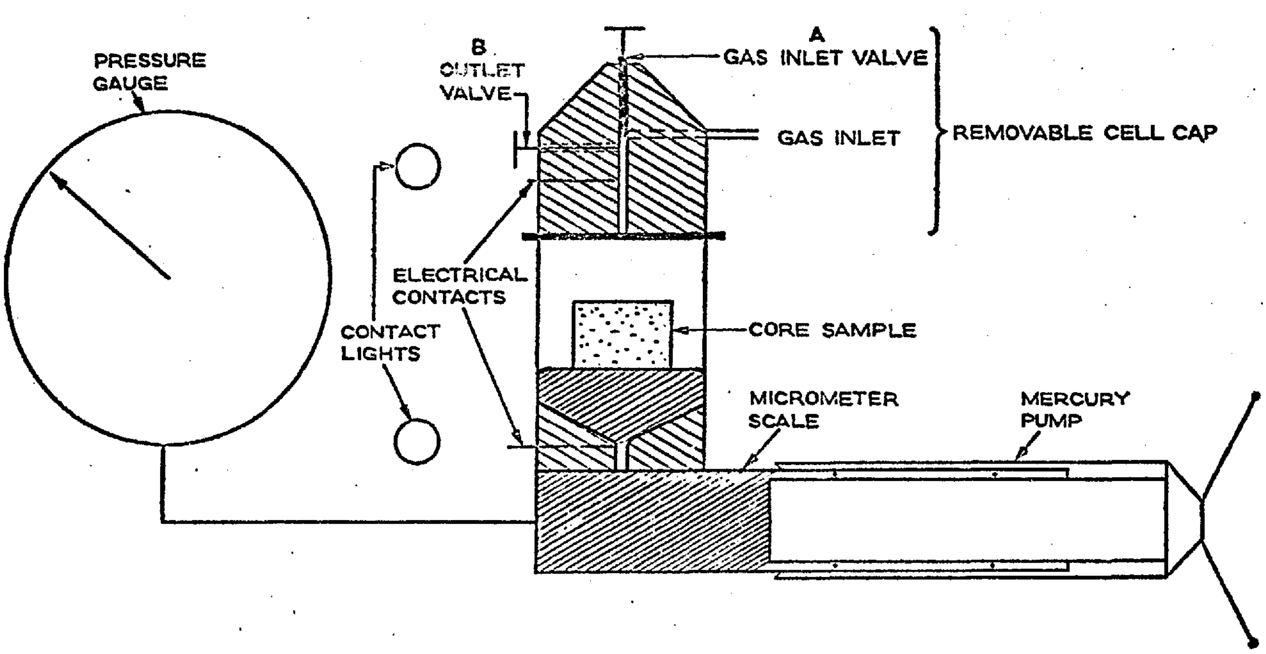


Figure 37: Mercury porosimeter (after King, 1966)

The water method has certain advantages; it is particularly suited for testing rock in bulk, allowing a representative sample of irregularly shaped lumps to be processed as a single batch. The technique is easily combined with durability testing where water saturation is also required. It was used for the Edinburgh project discussed in Chapter 4. A vacuum for saturation, an oven for drying and a balance for weighing are essential.

'Quick absorption' methods often used in engineering to act as an 'alteration' index or as a guide to rock 'quality' in fact measure porosity, although the saturation is not complete and result are also somewhat dependent on rock permeability. Results comparing water absorption 'porosity' after 1 hour and 48 hour immersion with that determined by the Boyle's law technique described earlier are given by Morgenstern and Phukan (1966) and are reproduced in Figure 38. One can see a correlation between results for the three techniques, but is is doubtful whether the 'quick' method offers any advantage. In practice it is slower and gives less reproducible results than porosity determination using vacuum saturation. Vacuum apparatus is usually readily available and is the only way of ensuring a high saturation in a reasonable time. Even using vacuum, some authors recommend several hours of immersion to ensure complete saturation, but for engineering purposes a vacuum saturation of about 15 minutes at 5 Torr should be sufficient.

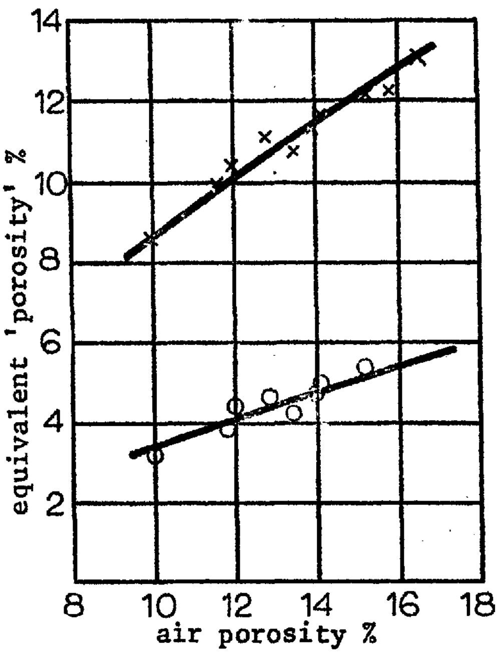


Figure 38: Comparison of 'quick absorption' porosity estimates with actual porosity values (after Morgenstern & Phukan, 1966).

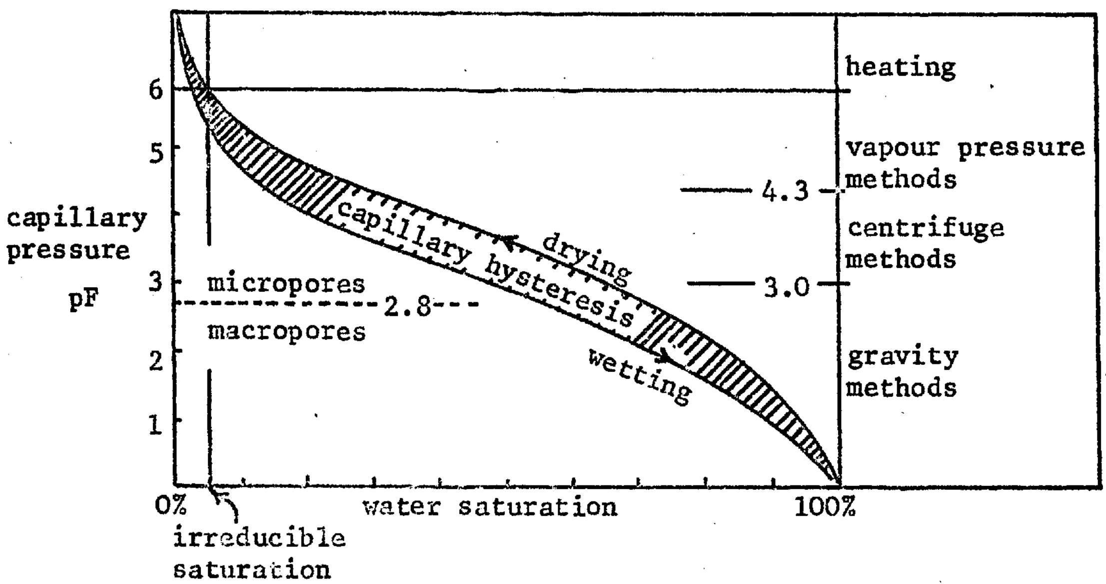
Water absorption volumes after 1 hour and 48 hours immersion are compared with the total volume of pore space (measured using a Boyle's Law porosimeter). The correlations are good, but whether the 'quick' techniques are in fact quicker than measurements of actual porosity may be questioned.

× 48 hour absorption
o 1 hour absorption

It was noted earlier that *pore size* is an important parameter in its own right; porosity may comprise a few large pores or many small ones, and pore size must be considered in studying the effect of pores on engineering properties. Pore size is most often studied in relation to problems such as rock permeability, the durability of building stones or the strength of ceramics, and much has been published on these topics. Several experimental methods have evolved for the study of pore size and size distribution. All must depend on the assumption of a simple model for pore texture, since actual measurement of pore size is only possible by visual methods described earlier.

The mercury injection technique used for measurement of total porosity may also be used to measure pore size distribution. Mercury enters pores of successively smaller diameters as the injection pressure is raised. A model for pore texture similar to a stack of capillary tubes of various diameters is usually assumed. Calculations based on the surface tension of mercury show that at 1 atmosphere mercury should enter pores of 7 x 10^{-4}cm in diameter, while at 700 atmospheres it should enter pores as small as 100 Angstrom. A graph of the injected volume of mercury against the injection pressure should therefore approximate to a graph of the pore volume as a function of pore diameter.

Other methods of estimating pore size distribution are based on the observation that the equilibrium level of water saturation of a rock specimen in a humid environment depends on the humidity or vapour pressure of that environment. The environment vapour pressure bears an inverse relationship to the capillary tension or 'suction' (negative pore water pressure) in the rock pores. Vapour pressure may be used to calculate capillary pressure or vice versa, if a capillary model for the pores is assumed; in practice it is usually capillary pressure that is plotted against saturation for the purpose of estimating the pore size distribution (Figure 39(a)).



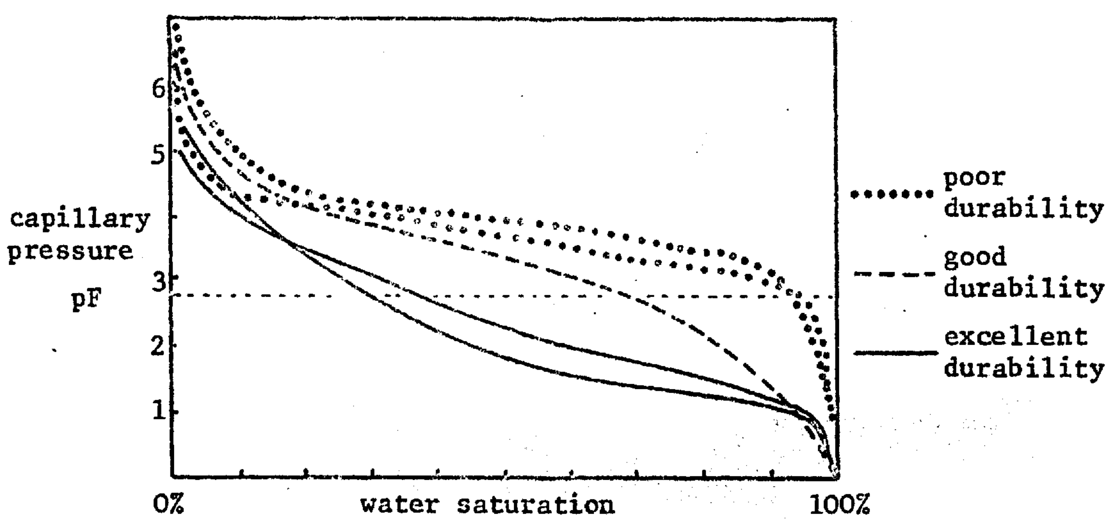


Figure 39: Pore size estimated from capillary pressure/saturation curves

The upper figure, 39(a) shows an idealised curve and the various practical techniques required to obtain data over different parts of the capillary pressure range. It also demonstrates the phenomenon of capillary hysteresis, possible definitions of 'micropores' and of 'irreducible saturation'. The lower figure, 39(b), illustrates actual data (after Honeyborne and Harriss, 1958) relating the observed durability of building stones to their capillary pressure/saturation curves, particularly to their 'microporosity'.

Capillary tension is most conveniently plotted on a logarithmic scale as the pF value, defined as the common logarithm of the water head equivalent to a given capillary tension. Various experimental techniques are necessary to produce the full range of capillary tensions required. For pF values less than 1.1 gravity methods may be used; one such method, commonly applied to soils, is to measure water saturation in a column of material standing with its base in water, as a function of the height above water. Higher capillary tensions may be produced by increasing the air pressure around the column, or by applying a vacuum below its base. Alternatively a centrifuge may be used to expel water from a saturated specimen. Above a pF of 4.3 the most convenient technique is to measure the saturation of a rock specimen brought to equilibrium in a controlled vapour pressure environment. Standard saturated salt solutions, or sulphuric acid at various levels of dilution, may be used to provide the standard humidity environment. The phenomenon of capillary hysteresis may be noted, and different curves are obtained depending upon whether the technique involves the wetting (water imbibition) or the drying of a specimen.

The techniques, and their importance in determining the durability of building stones, are discussed further by Honeyborne & Harriss (1958) and in the Building Research Station note A112 (1963), both of which emphasise the importance of *'microporosity'* as a determining factor in mechanical performance (Figure 39(b)). Microporosity is defined as the water retained after applying a capillary tension of 640cm water (equivalent to a pF of 2.8), and corresponds approximately to pores of diameter less than 0.005mm. Pore size plays an important part in determining the capacity of a rock to take up and hold water, and in controlling its ease of movement during weathering processes such as freezing and thawing. A further important parameter that is closely related to microporosity is the *'irreducible saturation'* of a rock specimen, defined as the water volume that can only be driven off by heating. This parameter, one that is much used in petroleum engineering as a measure of rock impermeability, may be measured in a reproducible manner by allowing the specimen to dry in a standard dessicating environment such as a packing of fine silica powder (Dobrynin, 1962) and determining the additional weight loss on heating.

Geophysical methods (sonic, electric, irradi ation and other techniques) do not allow direct measurement either of physical properties such as porosity, or of mechanical properties such as deformability. The use of sonic velocity to estimate Young's Modulus for engineering design purposes is based upon a misconception. The strain levels and strain mechanisms associated with the passage of a sound wave through rock are quite different from those encountered in most engineering problems, and the use of linear elasticity assumptions that apply to the complete range of strain levels is not justified. Geophysical techniques are best applied to the prediction of porous properties, with which they are closely related, but even then they require careful interpretation and the use of several complementary techniques, and often give qualitative rather than quantitative results. The proper (optimum) use of these techniques is discussed by Archie (1950). They are ideally suited to a comparison of one rock with another, particularly in determining the in situ boundary between two rocks of differing physical properties.

The techniques are often simple and require little or no specimen preparation. They are well suited to probing large volumes of rock, even up to the size of the Earth itself, and so can assist in field mapping of rock and in integrated rock classification from the smallest to the largest scale.

Resistivity techniques depend on the rock skeleton having greater resistance than the saline pore fluids to the passage of electric current. Resistivity therefore measures porosity, pore saturation and pore fluid salinity. Pore fluid salinity may be removed from consideration by characterising a rock or rock formation by its '(formation) resistivity factor', defined as the ratio of saturated rock resistivity to the resistivity of the pore fluid. This factor is constant for a given rock since there is a linear relationship between the two quantities. *Inductance* methods offer an alternative to direct current methods of resistivity measurement. An inductance coil is used to set up eddy currents in adjacent rock, these in turn creating magnetic fields that can be detected and used as a measure of resistivity. A further electrical technique, known as the *S.P. method* (self- or spontaneous-potential) is based on measurement of natural, mainly electrochemical, potential between an electrode in fluid adjacent to the rock and another in a remote reference position, such as in the ground at the tope of a borehole. Natural variations in potential largely reflect variations in the clay mineral content of rocks.

Radioactivity may be used in two distinct ways to chracterise a rock. Natural radioactivity is manifested by gamma ray emission from radioactive salts that are usually concentrated in the finer grained fractions of a rock. Detection of natural gamma ray intensity may therefore be used as an index to clay content. A second technique, known as the neutron-gamma (n-γ) method, excites gamma radiation artificially by bombarding the rock with neutrons from a high energy source. Gamma rays are liberated from pore fluids, since the hydrogen atoms that abound in these fluids are much closer in weight to that of a neutron, and therefore absorb most energy. The n-γ technique is therefore used as an index of porosity and pore fluid saturation.

Sonic and ultrasonic techniques are probably the longest established and best known of the geophysical methods. Elastic wave velocity or attenuation may be measured either by passing sonic or ultrasonic pulses through the rock, or by exciting a specimen of regular geometry to resonance. The velocity and attenuation of sound in rock depends to a large degree on the porosity (both fracture- and pore-porosity) and on the degree of saturation. Either parameter may be used as an index to these properties, but sound velocity (particularly that of the P-wave or first arrival) is the easiest to measure. The ratio of sound velocity in the field to that measured for smaller size specimens in the laboratory is usually less than 1.0, on account of the presence of open fissures in the rock mass, and has been used as an index to the intensity of fissuring of in-situ rock.

These and other geophysical methods are extensively used for logging the many thousands of feet of hole drilled for oil prospecting, particularly to log the sections of hole for which no core is available. They are described in great detail in the many publications of the oil industry (for example Schlumberger Well Surveying Corporation, 1958) and their simplicity may well allow their wider application for purposes of rock classification.

(e) Strength Testing

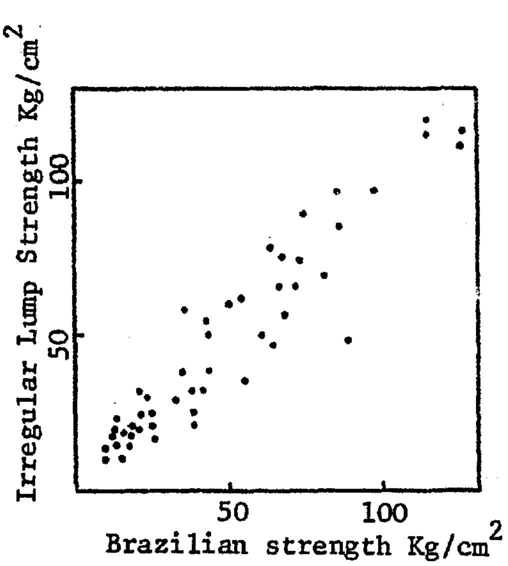
The *uniaxial compressive strength test* is long established as a basis for rock classification. It has been well documented (e.g. Hawkes & Mellor, 1969) and data is available for a wide range of rock materials. The test can be interpreted theoretically with little chance of ambiguity, and the results are reproducible.

It has, however, important disadvantages detracting from its use in rock classification, particularly the following:

A 2:1 length:diameter prism or cylinder must be prepared. This is time consuming and can influence specimen strength, particularly with regard to changes in moisture content.

Broken rock requires re-drilling to achieve the required dimensions. Weaker rocks require special precautions or preclude preparation altogether. In practice the test cannot be used for classification of heavily jointed or bedded rocks or weaker rock materials. Results are biased towards specimens that survived preparation.

To overcome these and other disadvantages alternative strength classification tests will be proposed. These are indirect tensile tests that allow testing of unprepared rock lumps or rock core. The type of test is selected to suit the shape of the rock specimens available. They require only one-tenth of the loading machine capacity of an equivalent uniaxial test so that portable testing equipment may be readily designed for use at the core shack or rock outcrop. There is also less room for operator error in this type of test.



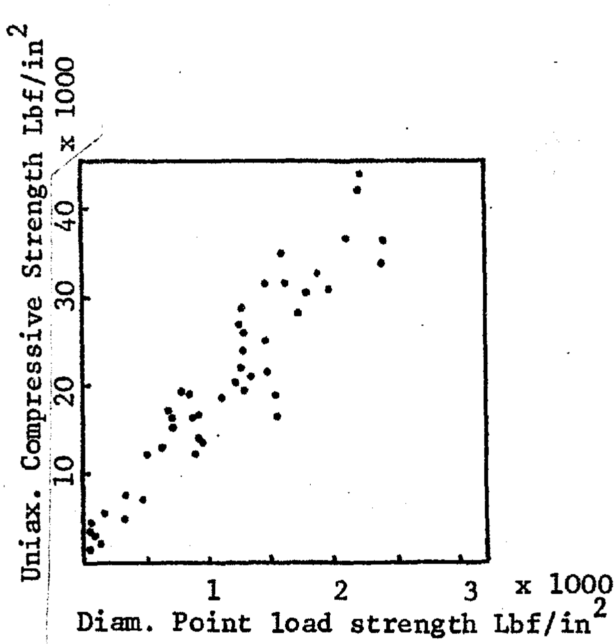


Figure 40: Intercorrelation of point load and other strength parameters

The left hand figure (after Hiramatsu and Oka, 1966) illustrates a close correlation between results of Brazilian and irregular lump strength tests. The right hand figure (after D'Andrea et al, 1965) demonstrates a close correlation between cylinder diametral point load strength and uniaxial compressive strength.

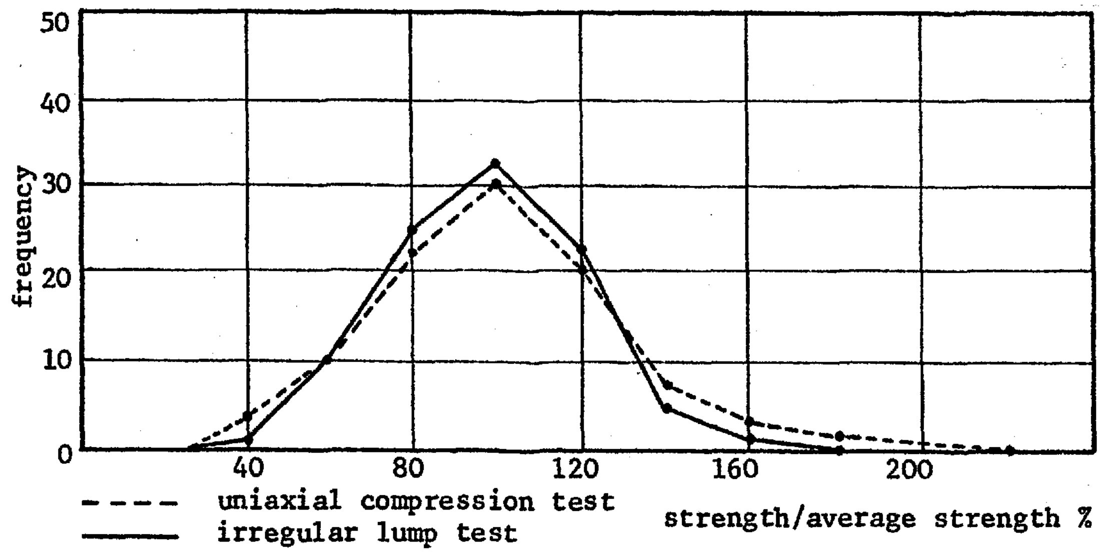


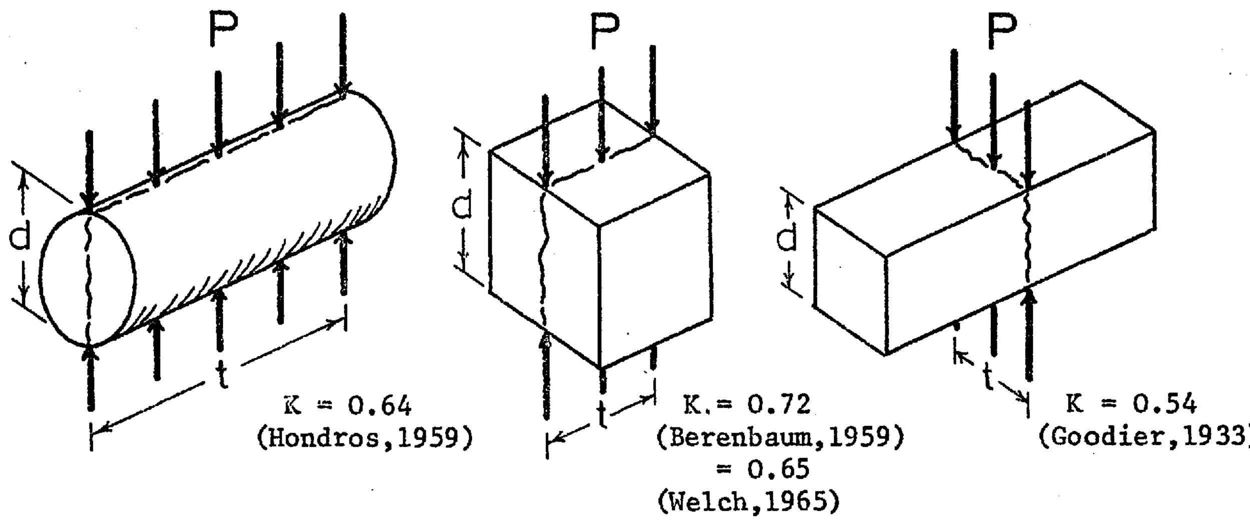
Figure 41: Comparison of reproducibility of results in the uniaxial compression and the irregular lump strength tests (after Wittke and Louis 1969, from data obtained by Protodyakonov).

The spread of results for the two tests is similar, and suggests that the irregular lump pointload test is a more repeatable strength index than might at first be supposed.

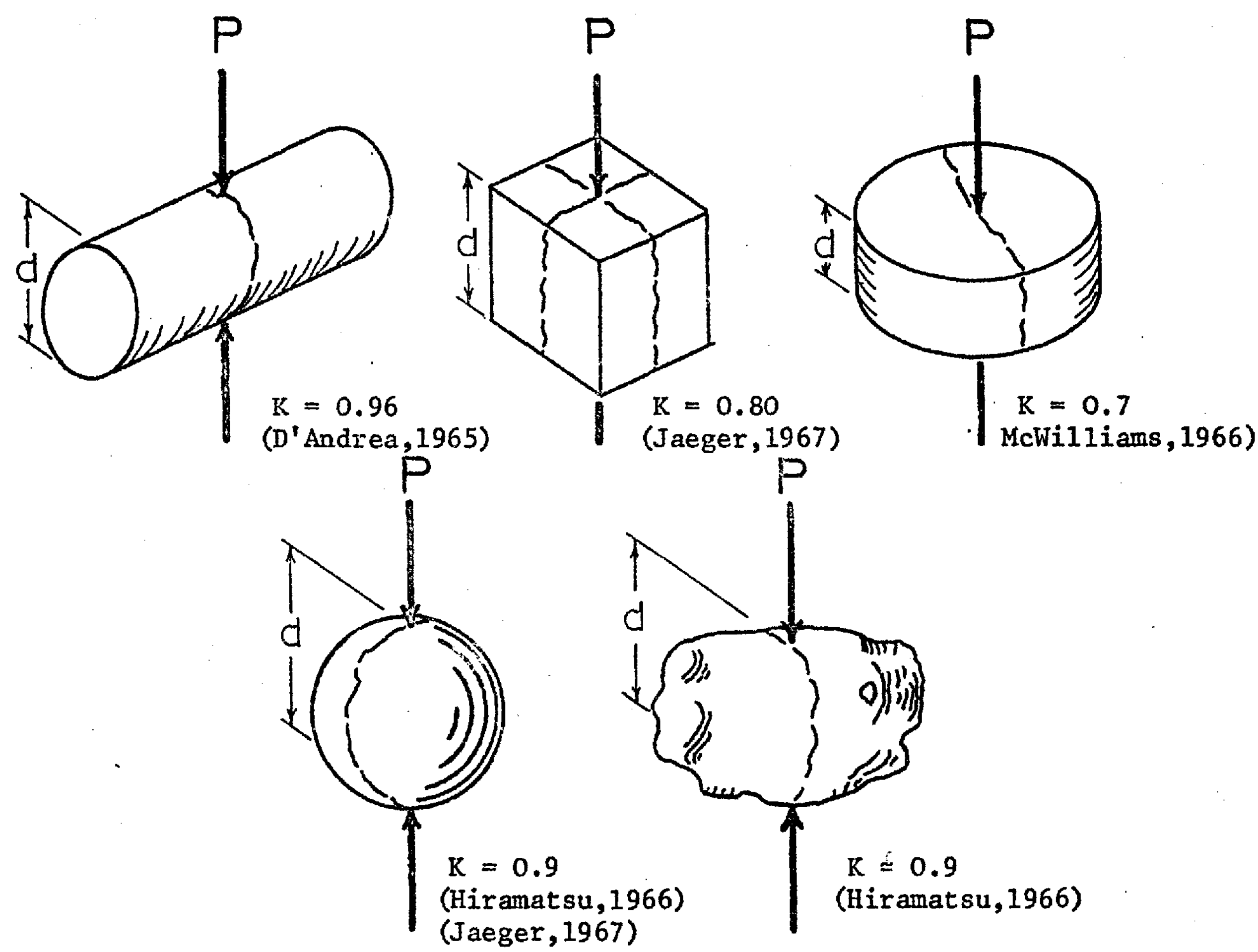
One might expect that more tests on irregular specimens than regular shaped ones would be required for the same precision of strength estimation. Figure 41, reproduced from the paper by Wittke and Louis (1969), demonstrates a similar spread of results in the two tests and suggests that the number of specimens required by each might well be of the same order. The spread in both cases is large, and several tens of tests per rock sample would be needed depending on the precision required. Since for a site of variable geology rock classification requires many samples, the simpler type of test would recommend itself. Uniaxial strength testing can provide useful supplementary information, particularly until such time as the simpler, more novel tests have been thoroughly developed. However, the specification of tests as complicated as the uniaxial test as a basis for rock classification will have the effect of prolonging the life of 'hammer and penknife' techniques, and methods of intermediate complexity are required.

Indirect tensile tests first became known following the work of Carniero and Barcellos (1953) who developed a cylinder splitting test for concrete, the well know "Brazilian' test. They have been used for academic purposes as a more easily performed method of assessing tensile strength than the direct tension test, but have also found practical application in the quality control of concrete (e.g. Mitchell, 1961) or ceramics (Spriggs et al, 1964). Mitchell proposed the Brazilian test as "much more satisfactory for field control of concrete than other test procedures". Indirect tensile tests have been used for some time in Russia for practical rock classification (Protodyakonov, 1960).

As a group the tests have much in common (Jaeger, 1967, 1969, Reichmuth 1968). They will be discussed as a group, although some of the tests are unsuitable for rock classification purposes. They may be broadly categorised into point-load and line-load types. Figure 42 illustrates typical tests together with formulae relating the maximum tensile stress in the specimen to the applied force and the specimen geometry. The specimens are heterogeneously stressed, so that the formulations depend on theoretical or experimental stress analyses, usually based on linear elastic assumptions.



LINE LOAD METHODS $\sigma_t = \frac{K\,P}{d\,t}$



POINT LOAD METHODS $\sigma_t = \frac{K\,P}{d^2}$

Figure 42: Point Load and Line Load strength tests
(σ_t = uniaxial tensile strength)

The stress distribution is also similar in each case. Maximum values of tensile stress occur along the loaded diameter, acting in a direction perpendicular to the applied load. They are accompanied by compressive stresses that act in the direction of loading. The stress distribution in a Brazilian disc may be used for comparison, and has been fully evaluated by Hondros (1959) and by Fairhurst (1964) (Figure 43).

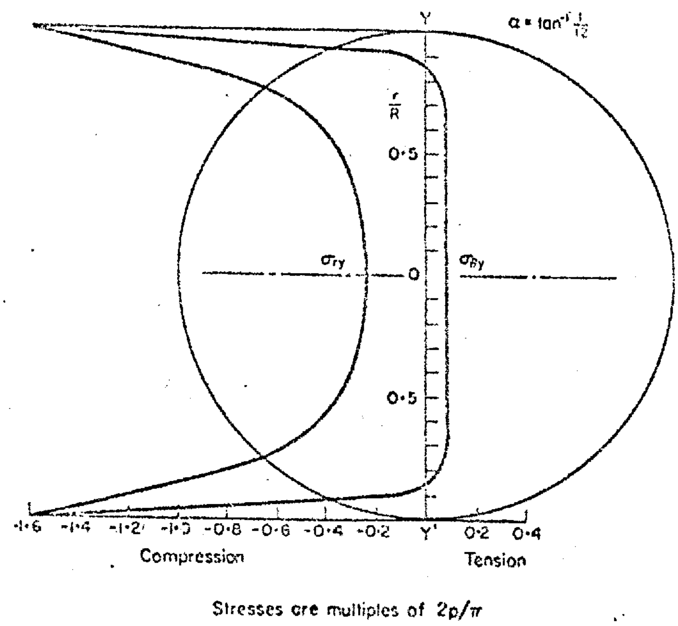


Figure 43: Stress distribution in point loaded specimens (after Fairhurst 1964).

The stress distribution in a diametrically loaded disc is similar to that in a point loaded irregular lump (e.g. Evans and Pomeroy, 1966). Critical stress conditions occur on the loaded diameter, where the horizontal stress is tensile and is accompanied by a vertical compressive stress of approximately three times the tensile value.

The *shape of specimen* has little influence on the appearance of the stress distribution along the loaded diameter. Several authors, for example Jaeger (1967) have underlined the similarity of stress distribution between line loaded cubes, line loaded cylinders, point loaded cubes and point loaded spheres. Slight quantitative variations from one test configuration to the next may be observed as differences in the coefficients in the formulations of Figure 42.

The *area of loading*, whether a theoretical point or line load or distributed over a finite area, has little influence on the value of maximum tensile stress. Peltier (1954), Colback (1966) and Welch (1965) emphasise that the width of strip loading in the Brazilian or cube splitting tests may approach one tenth of the specimen diameter without appreciably influencing the stress at the centre of the specimen. St. Venants principle indicates that the stress field should not be appreciably disturbed at depths below the platen contact that exceed the width of the loaded strip. In practice the specimen yields at the contact so that even the 'point' load is applied over a finite area. Some authors have suggested that more reproduc ble results are obtained if the load is artificially distributed, using concave platens or cardboard packing pieces.

High speed photography (e.g. Brown & Trollope, 1967) has shown that *fracture initiation* originates on the loaded diameter of the specimen, and propagates towards the platens. After the specimen has been split in this way, secondary fractures are often generated by further loading of the two specimen halves. Rudnick, Hunter & Holden (1963) used photo-elastic techniques to show that the strength of two half-discs could be as little as one eighteenth that of the original intact disc. Tensile stress in the central region of the specimen is more or less constant, so that although the fracture may originate off centre the tensile strength may still be calculated from the stress at the disc centre. The compressive component, although more variable with position, has little influence on the strength. This result is predicted by theoretical strength criteria and is substantiated since the tests, in practice, record very similar values for the strength of a given material.

There are definite *practical advantages* to having the critically stressed zone large, and in the interior of the specimen rather than on its boundary. A large critical zone is also likely to be large with respect to pores and grain boundaries, reducing the possibility of these stress raisers causing erratic results. Durelli & Parks (1967) pointed to an inverse relationship between strength and the volume of material subjected to 95% or more of the maximum stress. Having the critical zone in the interior of the specimen ensures that surface irregularities are of minor importance and that neither loading conditions nor specimen preparation are critical to strength. Also the specimen may be tested nearer to its natural moisture content since surface drying or wetting are less important.

The *ring test* or disc-with-hole test (for example Jaeger & Hoskins, 1966) was introduced in an attempt to improve on the Brazilian test by localising the origin of fracture initiation. Fracture originates at the hole boundary under pure tensile stress conditions, rather than the compression-tension combination characteristic of other indirect tension tests. However, this test suffers from the disadvantages outlined above. Specimen preparation is critical since the critically stressed zone is small and is concentrated at the free surface. Calculations of tensile strength are based on stress concentration factors that rely on linear elastic assumptions, of doubtful validity under the conditions of high stress gradient that exist near the hole. In practice the strengths are dependent both on the ratio of internal to external diameter and on the absolute size of the specimen. Tests such as this or beam bending tests, where the critically stressed zone is small, usually give tensile strengths about twice as high as those for direct tension tests on specimens of similar size. Results for various types of 'tension test' are compared in Figure 44.

	Gosford sandstone	Carrara marble	Bowral trachyte
Direct tension	520	1000	1990
Brazilian (15° contact	540	1265	1740
Pinching	450	670	1090
Disk with hole $\rho = 0.5$ external loading	1200	2500	3500
Disk with hole $\rho = 0.5$ internal loading	1100	2300	3700
Bending: 3 point loading	1140	1710	3650

Figure 44: Comparison of 'tension'test results (after Jaeger and Hoskins 1966)

Test results are correlated for the various types of direct and indirect tension test but there are systemmatic differences in the values of 'tensile strength' obtained, the strength increasing as the critically stressed volume of the specimen decreases.

The *irregular lump test* was developed in Russia (Protodyakonov, 1960) and has been further explored in Germany and France (Duffaut, 1968, Diernat & Duffaut, 1966; Wittke & Louis, 1969) and in Britain (Hobbs, 1963; Evans & Pomeroy, 1966) as a practical method for rock classification. To obtain a strength index having the dimensions of stress it is necessary to divide the applied force by an area. The author prefers Hiramatsu's formulation (Figure 42) where force is divided by the square of platen separation. Hiramatsu & Oka showed using photoelastic stress analysis that this formulation, exact for an elastic sphere, also applied well to the tensile strength of irregular lumps. It is also easier to measure platen separation than other alternatives described below.

Russian experiments indicated a linear relationship between the force required to break a lump, and the 2/3 power of its volume (a measure of cross sectional area). This led to the formulation of a strength index as fracture load/volume$^{2/3}$, and this formulation has been followed in France, although questioned on account of the influence of specimen shape on the results. Protrusions remote from the loaded diameter have little influence on strength but much on the volume of the specimen.

Extensive investigations in Britain by the National Coal Board led to yet a further definition of strength as measured by the lump test. It was found that a definition based on average contact stress at failure led to less scatter of results than one based on volume measurements. Contact stress was measured as the failure load divided by the average area of contact at the platens, the latter determined by placing graph paper and carbon paper between platens and specimen. The method was followed by Pigeon (1969) who was investigating the behaviour of rock-fill material, and he found close agreement between his results and those of the National Coal Board. However, it is doubtful whether the added complexity introduced by measurement of contact area is justified in practice. Comparison with other indirect tensile test formulations favours a definition based on the square of platen separation. It would also seem advisable to follow Hiramatsu & Oka by loading the specimen across its shortest diameter (contrary to Russian and French practice) and to load with spherical indentors rather than flat platens, since the latter can induce bending due to multiple contacts.

The *disc axial point load test* (Reichmuth, 1968; McWilliams, 1966) appears to be a valuable practical variation on the lump test, particularly suited to rock core that is broken into discs by stress relief or by bedding laminations. This test might be used without any machining of the disc faces. For less broken core, where the length of rock 'sticks' exceeds their diameter, the *cylinder point load test* can be used, again requiring no specimen preparation (D'Andrea, Fischer & Fogelson, 1965, Reichmuth 1968).

Reichmuth carried out an extensive study of a variety of point load tests. He recommended them for routine testing particularly on account of their rapidity and excellent reproducibility (standard deviations of the order of 8% of the mean). Figure 45a reproduces his results for cylinder point load tests on Bedford limestones re-plotted in Figure 45b to demonstrate the linear decrease in strength P/h^2 with cylinder diameter h.

This strength decrease with increasing specimen size is consistent with 'weakest link' theories (Durelli & Parks, 1967) and also with evidence from other types of strength test. Reichmuth attempted to obtain a constant value for strength irrespective of specimen size or shape, by fitting elaborate empirical formulae to the data. His simpler general expression for point load tensile strength, $\sigma_t = K.P/h^2$, identical to that of Figure 42, may be preferred if this test is to be used for classification, bearing in mind that K is only approximately constant.

In this application, to avoid the most undesirable expedient of standardising specimen size and shape, strength could be defined as the intercept h=0 (Figure 45b), provided that the linear relationship also holds for other rocks. Discs or irregular specimens would allow testing over a range of 'h' but rock core, in practice, would not. Alternatively, size correction factors could be evaluated. For field classification where errors of 30% might be tolerable (c.f. hammer evaluation of rock hardness) the ratio P/h^2 might be adequate as a strength index without adjustment.

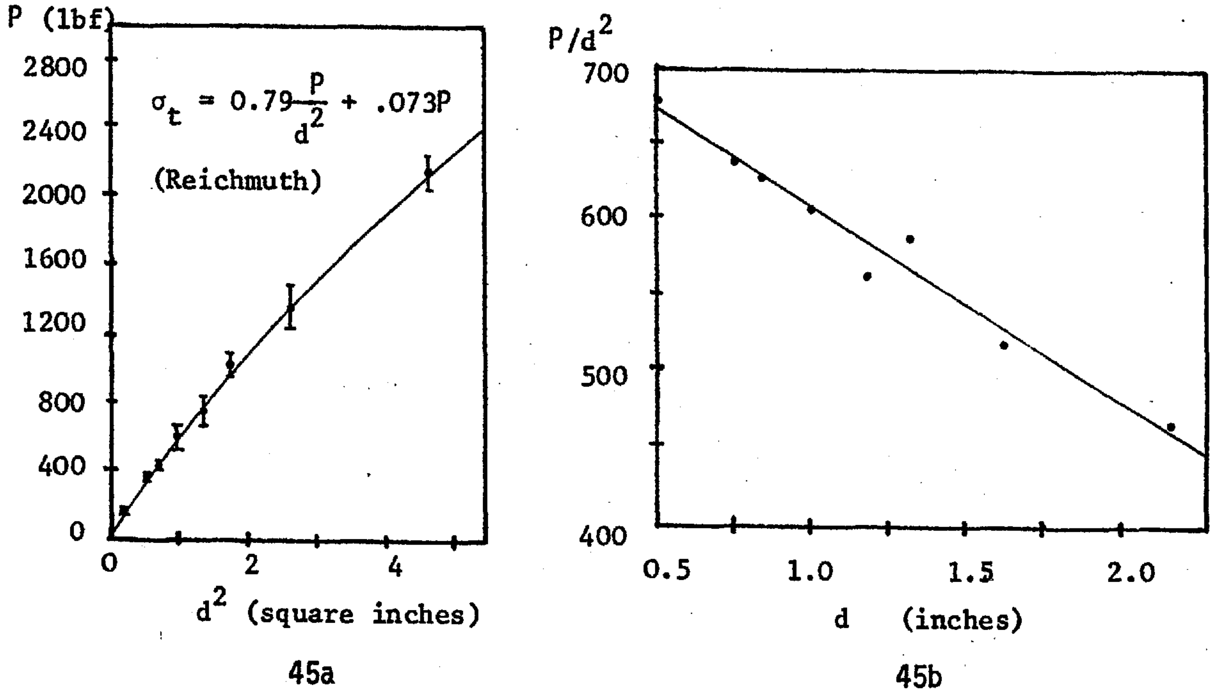


Figure 45: Size effect in the point load test.

Reichmuth's (1968) results, Figure 45a, demonstrate that the index P/d^2 is dependent on dimension d. If the results are re-plotted (Figure 45b) the relationship between P/d^2 and d is found to be linear. 'd' is the core diameter and 'P' the failure force in a diametral point load test.

Other strength and hardness tests deserve brief mention since circumstances might dictate that they be used to provide supplementary information, but the indirect tensile tests discussed above appear more suitable for use in classifying rock. *Surface hardness measurements* such as scratch, indentation or rebound tests may be criticised on the grounds that they generally do not discriminate well between different rocks; they give scattered results, the scatter being of the same order as the complete range of values attainable. In particular they do not adequately reflect a most important fundamental property, the intergranular strength of the rock. However, certain of these tests have much to commend them. The Schmidt rebound test is extremely rapid and simple and may be used in both field and laboratory. The test for 'polished stone coefficient' is typical of tests that may be particularly valuable in specialised applications such as, in this case, determination of the skid resistance of a material to be used for road surfacing. Such properties may not be adequately reflected by other types of test.

Tests on aggregate rather than individual lumps are widely used but require a 'specimen' in the form of graded aggregate. *Aggregate crushing tests* rely on a measure of size reduction when a quantity of aggregate is subjected to a standard crushing force. Initial grading, size and shape of container, method of pre-packing, magnitude and duration of applied load and other factors have to be rigidly specified to obtain reproducible results. Size reduction is perhaps not such a satisfactory index measurement of strength. *Attrition tests* also rely on a measure of size reduction, rock being subjected to impact by the rotation of a drum containing aggregate. *Impact tests* on single, regularly shaped specimens using a pendulum or a falling weight are necessarily complicated to perform. These tests and others of a similar nature are specified in most countries as standard procedures for aggregate testing (for example, B.S. 812, 1967) and are designed to approximate to 'natural conditions' of aggregate loading. They have, however, been shown to correlate strongly with each other and also with simpler tests such as those for indirect tensile strength. The simpler techniques seem preferable for rock classification.

In conclusion, it is proposed that one of three indirect tensile tests be used for the purpose of rock strength classification. The type of test should be chosen to suit the shape of rock lumps in the sample (Figure 46) and specimen preparation should be avoided. Time that would normally be taken in preparation should be utilised to allow further testing. It is suggested that the result might best be expressed as the ratio of fracture force to the square of platen separation, omitting any constant multiplier. The results should be correlated directly with uniaxial compressive strength to allow standardisation between tests of the three types, and further investigations to establish these correlations over a wide range of rock materials appear justified.

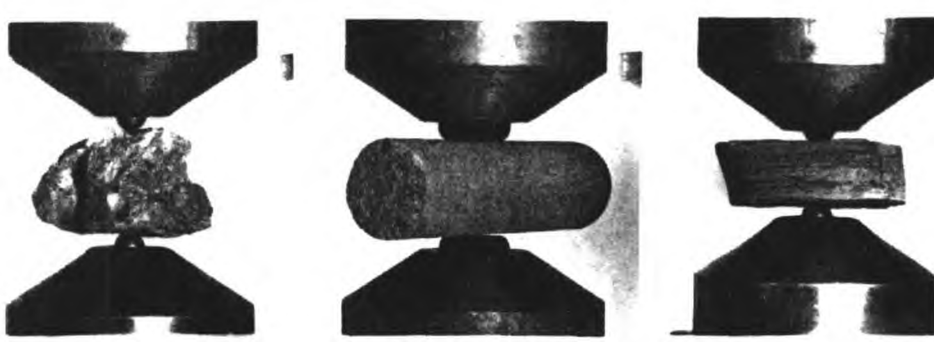


Figure 46: Three types of point load best suited for practical strength classification.

The 'lump' test (left) may be used when specimens are in the form of aggregate or when rock core is not available. 'Diametral' or 'axial' point load tests (centre and right) may be used on core specimens, depending on the length of core pieces that are obtained in situ.

(f) Durability Testing

A *definition of durability* must distinguish between the 'state of weathering' of a rock, and its 'weatherability' or potential for future degradation (Hamrol, 1961). Degradation of rock materials in an engineering structure may render the initial measurements of rock properties and the resulting design predictions meaningless, so that in classifying a rock there must be some measure of likely changes in properties due to weathering agencies. For the purposes of engineering, those weathering agencies that work over geological time spans, chemical and biological soil formation processes and thermal exfoliation for example, may be ignored although their effects will be apparent in measurements of 'weathering'. 'State of weathering', as opposed to 'weatherability' is reflected by properties such as strength, porosity and mineral content. Weathering agencies of more immediate engineering importance are frost, salt crystallising in rock pores, and the disruptive or weakening effects that can arise from day to day factors such as changes in rock moisture content.

Further testing is required to allow durability to be included as a factor in engineering rock classification. Tests have been developed for a wide range of applications including the design of rock structures such as slopes (Nakano, 1967), the durability of building stones (Honeyborne, 1965), roadstone and concrete aggragates (Bloem, 1966) and riprap materials (De Puy, 1965). The tests are similar irrespective of application and the most suitable may be selected for general use.

The *mechanisms of short term weathering* are associated with the action of pore fluids. These may weaken the rock, cause it to swell or may result in complete disruption of the rock material. The weatherability of a rock will depend firstly on its permeability and porosity, since these control the entry and retention of pore fluids and their mobility once inside the rock. Secondly, the action of fluids once they have penetrated the rock must be considered; their absorption, surface energy changes, solution of cement or disruption of bonds, and their power to set up disruptive forces by pore pressure generation or the formation of crystals. Finally the capacity of the rock to resist disruptive forces will decide the extent to which disruption, or merely weakening of the rock material will occur. A rock that is either impermeable, non reactive or has high intergranular strength will usually be durable.

Weathering mechanisms peculiar to clay bearing rocks have been the subject of detailed experimental studies by Nakano (1967) and Badger et al (1956) and are discussed in further detail by Boswell (1961). Clay bearing rocks, not only mudstones but some sandstones and weathered igneous rocks, may lose coherence and crumble or shale when dried or wetted. Among the several mechanism that may account for this behaviour, *ion exchange* appears to be dominant. Clay minerals may be flocculated by positive ions or dispersed by negative ones. The rapidity and completeness of dispersion depends both on the type of clay and on the nature of the fluid dispersant. Badger stated that sodium clays were the easiest to disperse, followed by potassium, magnesium and barium clays, and for nine samples of carboniferous shales demonstrated a strong negative correlation between durability and sodium content. The efficiency of a fluid or solution as a dispersing agent seems to be largely determined by its dielectric constant, a measure of ionic dissociation. Both Nakano and Badger investigated the comparative durability of shales immersed in liquids of differing dielectric constant and found a general correlation between the two properties, although Nakano found important exceptions to this rule that pointed to hydrogen bonding ability rather than dielectric constant as a determining factor. Arscott (1968) demonstrated the importance of hydrogen ion concentration by comparing the swelling of a mudstone in fresh water with that in salt water (Figure 47). Strains four times as large were experienced by the specimen in fresh water.

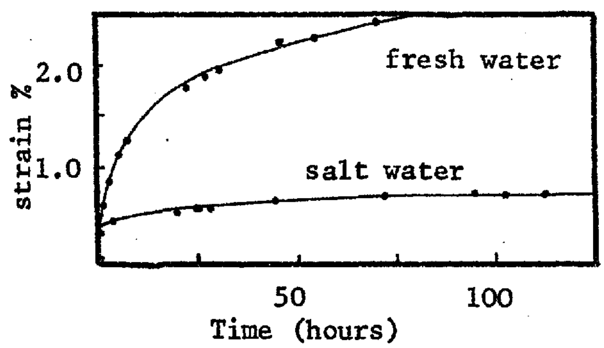


Figure 47: Swelling of mudstone in fresh and in salt water (after Arscott, 1968)

One of the factors influencing swelling and disintegration of mudrocks is the hydrogen ion concentration of the pore fluid.

The swelling of certain clay minerals, as well as their intergranular bond, is affected by the hydrogen ion concentration of the pore fluid. Illite and montmorillonite clay minerals in particular contain inter-layer potassium ions that favour hydration in the presence of water, causing swelling of the crystal lattice.

'Air breakage' has also been proposed as a disruptive mechanism. The action of water drawn into the rock under strong capillary suction forces may compress air in its path, resulting in disruption of the rock. Badger found that two of his nine samples disintegrated more rapidly when wetted in an air environment than in vacuum. Possibly related to this mechanism were the observations of Nakano, who found that some initial drying of the specimen below its natural moisture content was necessary before disruption could occur.

Stress relief is probably also an important mechanism. Many mudstones are overconsolidated; they were lithified under greater weights of overburden than their present depth of burial would suggest. They may well store elastic strain that could be released if intergranular bonds were weakened. One factor that may appear to detract from this hypothesis is the greater durability of older mudrocks that is commonly observed (Duncan, et al, 1968). Older rocks have commonly suffered a greater consolidation load; however they have also had longer to develop strong intergranular bonding that would more than offset any lack of durability due to overconsolidation.

Rocks free of clay minerals commonly weather differently from those that contain clay, although the mechanisms that affect clay free rocks may also disrupt clay bearing rocks. Loss of strength on wetting, and action of frost or salt crystallisation are two common mechanisms that will be outlined below.

Many rocks suffer a *strength reduction on wetting*, although the wetting alone is seldom sufficient to cause complete disruption of the material unless it contains clay minerals. The reduction in strength is commonly as much as 50% from the completely dry to the saturated condition, and the extent of weakening is a function of the moisture content of the material (Figure 48). Strength reduction due to wetting has been investigated in detail by Colback & Wiid (1965) who suggested that a reduction in free surface energy caused by water absorption was the prime cause of weakening. Crack initiation theories have pointed to a correlation between the stress required for crack initiation and the surface energy of the material in which the crack is to propagate. Another possible mechanism is the relief of capillary tension at grain contacts and at the tips of cracks. Water menisci in the rock increase their radius of curvature as the rock achieves a greater degree of saturation. The great strength loss from the oven-dry to the dessicator-dry levels of saturation lends support to this hypothesis and has been previously discussed (see for example the capillary suction/saturation diagrams of Figure 39). It is worth noting that additives to water can either increase or decrease its weakening effect (Robinson, 1967).

Frost or salt damage susceptibility is dependent on the growth of either ice lenses or salt crystals within the pores of the material. Rocks that are highly susceptible to salt damage may be frost resistant (Bloem, 1966) since the mechanisms are highly dependent on pore size. A high content of micropores (discussed earlier in this chapter) is necessary for frost susceptibility, whereas salt can effectively disrupt materials of much larger pore size.

Author	Material	Strength as a percentage of that when oven[2] or dessicator[1] dried	
		50% humidity strength[1] or air dry strength[2]	Saturated strength
(1) COLBACK & WIID, 1965	Qz Sst A	74%	50%
	Qz Sst B	65%	51%
	Qz Shale	63%	52%
(2) PRICE, 1960	Sst A	57%	-
	Sst B	68%	45%
	Sst C	51%	45%
	Sst D	80%	45%

Figure 48: Dependence of strength on moisture content

The figures relate to uniaxial compressive strength but all measures of strength are strongly dependent on moisture content. Capillary tension may be largely responsible for this effect (see Figure 39).

Tests for durability must provide both a means of causing disruption and a means of estimating its extent. Perhaps the most reliable means of causing disruption, if time is no object, is natural weathering and much may be learnt from observing the effect of natural exposure on rock material. *Accelerated weathering* is usually employed to give quicker results. A slight speeding up of natural processes is achieved using the 'tray test' (Building Research Station, described by Honeyborne, 1965) where blocks of rock are left standing in the open with their bases in water. After three years a rock may be classified as ' probably frost resistant' and after seven 'almost certainly frost resistant'. Faster methods, still requiring considerable time by engineering standards, employ cyclic changes of environment such as wetting and drying, freezing and thawing, or cycles of salt crystallisation. A variety of procedures are specified in various books of standards (e.g. American Society for Testing & Materials 1967) and two are tabulated in Figure 49. An alternative method to cycling for aggravating natural weathering mechanisms is to increase their severity by slight modifications of technique. Alcohol may be added to water in the freezing test to allow lower temperatures to be reached. Honeyborne describes a salt crystallisation test that had 'quite devastating results', in which blocks of rock were left to stand with their bases in a slurry of sodium chloride and calcium sulphate crystals. Mechanical attrition may also be used in conjunction with other methods to speed up the processes. It is most important that the acceleration of a weathering process should not alter its fundamental nature, hence salt crystallisation tests are not often of great value in determining frost resistance.

TEST	MATERIAL TESTED	DISRUPTIVE AGENT	CYCLE	DISRUPTION ASSESSMENT	REFERENCE
Soundness	Graded Concrete Aggregate	Sodium or magnesium sulphate solution	17 hour immersion dry to constant weight. Arbitrary number of cycles	Sieve for weight loss	ASTM C88-63
Freeze/Thaw	Concrete prisms in water	Ice	4^{o}C - minus 18^{o}C 3 hour cycle, 300 cycles	decrease in resonant freq. of block	ASTM C290-67

Figure 49: Durability tests

Durability tests involve accelerated weathering and assessment of its effect. Many are time consuming; the quicker tests sometimes fail to simulate natural weathering in a satisfactory manner.

The *extent of disruption* may be evaluated using a descriptive scale ranging from 'completely disrupted' to 'slightly disrupted' or may be measured in a more quantitative, though perhaps less meaningful manner. Rocks that have suffered strength loss but not disintegration may be strength tested before and after accelerated weathering treatment. Rocks that are disrupted may be subjected to swelling measurements (Murayama & Yagi, 1966) or alternatively the 'weight loss' after accelerated weathering may be evaluated by standardised sieving procedures. These ideally measure the change in grading and modal size of the material, but a single sieve may be used to give an estimate or index of the size degradation. Sieving appears both practical and suitably quantitative and sieving results have been shown to correlate well with durability values based on swelling tests (Nakano, 1967).

A durability test for mudstone was developed for use in the Edinburgh Roads project (described in Chapter 4) based on immersion of a few hundred grams of oven dried rock lumps in a dispersion solution of 2% aqueous sodium hexametaphosphate. The rock sample was shaken after 30 minutes, and after one hour was washed on a No.7 B.S.sieve. The 'slake loss index' was calculated from the oven dry weight of the original sample and of the fraction retained on the sieve, the weight loss being expressed as a percentage of the original weight. For comparison, the retained fraction was also classified on a descriptive scale based on fragment size and shape. Rocks that survived the slake test were given a slake loss index of zero and were further classified on the basis of strength and porosity. Results for this simple slake test (Figure 50) sufficed to allow a distinction to be made between mudstones of greater or lesser durability. Material with a slake loss index in excess of 20% was classified as non-durable (a soil) for the purposes of the project, and would normally be subjected to Atterberg Limits tests for further classification. Material of moderate durability was further described according to the nature of its fissility; blocky fissile or flaky fissile.

A similar durability test is described by Badger et al (1956) for use in colliery screening operations where shale lumps can clog the screens. It is reported to have been adopted as a standard by the National Coal Board. A shale sample of 100gm sized between 1/4 and 1/8 inch was placed along with ½ litre of distilled water in a 2lb 'Kilner' jar. The sample was subjected to mechanical attrition by rotating the jar at 40 r.p.m. for 30 minutes, then washed on a 36BS sieve. The finer fraction was then investigated by sedimentation size analysis. Durability was expressed as the inverse of the slake loss index defined above, based on the loss through the No.36 sieve or alternatively as the 30 or 10 micron size fraction. Several of the experimental sophistications employed by Badger could usefully be incorporated in the slake test technique used for the Edinburgh project, the object being to give more reproducible results without sacrificing a great deal of simplicity. Settlement analysis appears unjustified in view of the time required. Also routine field slake tests would require a coarser mesh sieve and larger fragments in the sample if they were to reflect the influence of rock fabric on durability. A tentative specification for such a test is set out in Chapter 4.

Specimens surviving the durability test may nevertheless have suffered a strength loss on wetting, and this may usefully be evaluated using the point load test described earlier. Strength is compared in the saturated state with that of a similar sample tested in a standard dry environment. Oven drying is undesirable since the

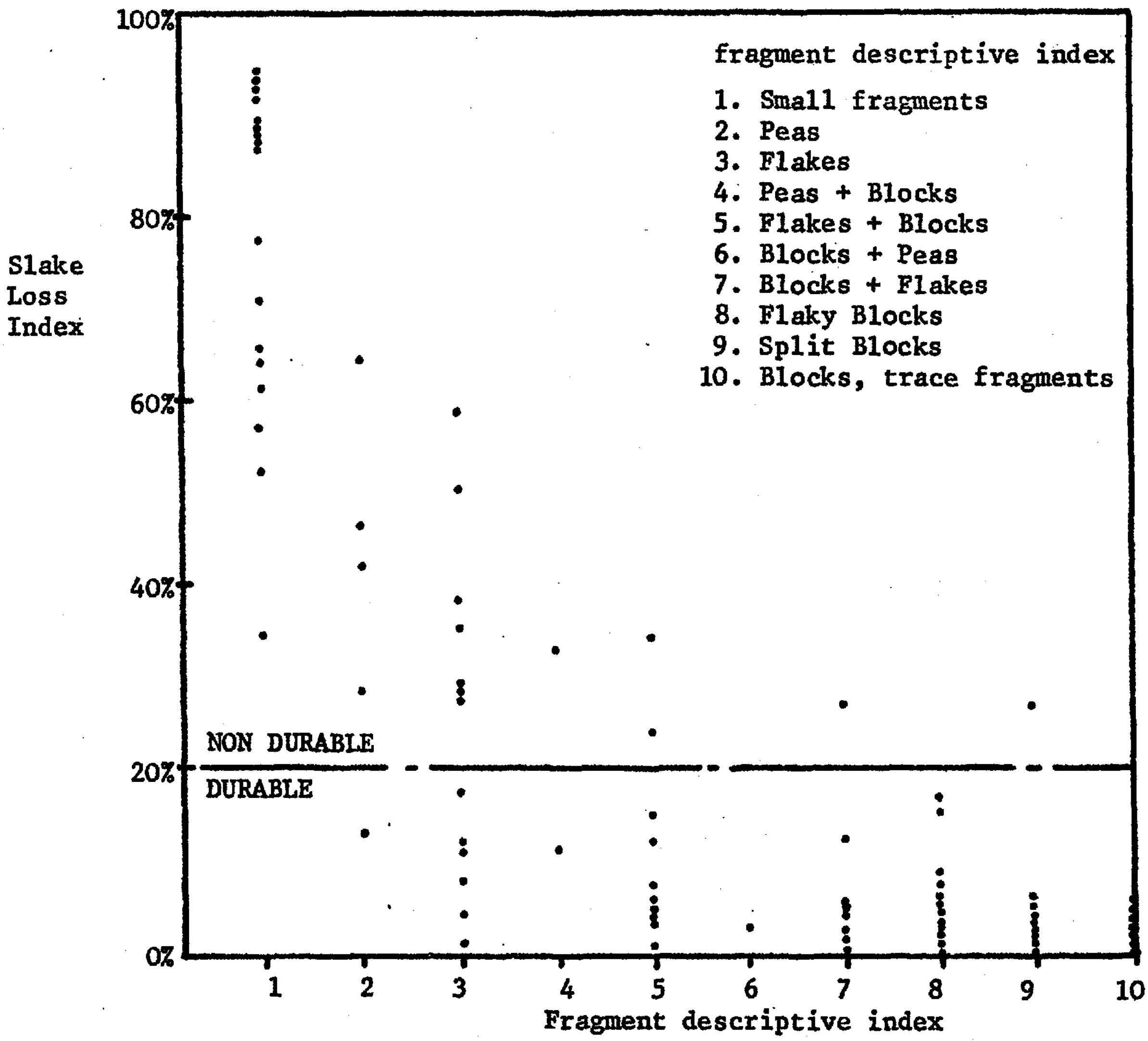


Figure 50 : Results of slake durability tests at Edinburgh

The effects of slaking were quantified in two ways, by means of a descriptive index and by loss on sieving. Some measure of correlation can be observed and the test allowed rock to be graded as durable or non-durable, the boundary being set at 20% slake loss on a No.7 B.S. sieve.

strengths obtained are unrealistically high. On the other hand it may take up to 30 days to reach moisture content equilibrium in a dessicating environment, although some shorter interval might be specified for practical purposes. Point load tests carried out by the author on a suite of Dartmoor rocks were triplicated, one subsample being saturated, a second oven dried and a third tested in an 'air dry' state with measurement of % saturation. In most cases there proved to be a systemmatic increase in strength with loss of water. For routine testing where moisture content effects are not to be investigated it is simpler to test in the air dry state, but for closer control specimens should be stored in an environment of controlled humidity, preferably one that is realistic.

4. FIELD STUDIES, CONCLUSIONS AND RECOMMENDATIONS

a) Field Studies

In this chapter a proposed system for engineering classification and description of rock will be presented, a system based on the criteria of chapter 1 applied to the observations and tests of chapters 2 and 3. Before outlining these proposals, however, two field projects will be briefly described. The author took part in these projects in an attempt to check the practicality of certain observations, tests and the proposed classification system as a whole. An *engineering* classification must be both *applied* and *interpreted* by the engineer, so that above all else it must be practical.

The first project involved the design and execution of a rock testing programme for the proposed Edinburgh inner ring road and trunk sewer scheme. The second, still in progress at the time of writing, involved a study of the weathering of a suite of Dartmoor rocks by means of in situ observations and laboratory tests. The lessons learnt from each project were invaluable, but since they have largely been incorporated in the preceding chapters the account of field studies that follows will be brief. The Edinburgh project is fully described in existing reports, and both are to be covered in future publications.

The Edinburgh project (Franklin, 1969 Drilling and Prospecting International, 1969) comprises three miles of road and half that of trunk sewer. Both road and sewer are likely to run partly in cut and partly in tunnel, and to raise a wide variety of geotechnical questions. Associated with the road, that has equal lengths to the East and to the South of the City centre, are a number of possible bridge and car park sites so that typical problems arising include the choice of plant for excavation and tunneling, the stability of tunnels, cut slopes and foundations and the suitability of excavated material for fill or roadstone.

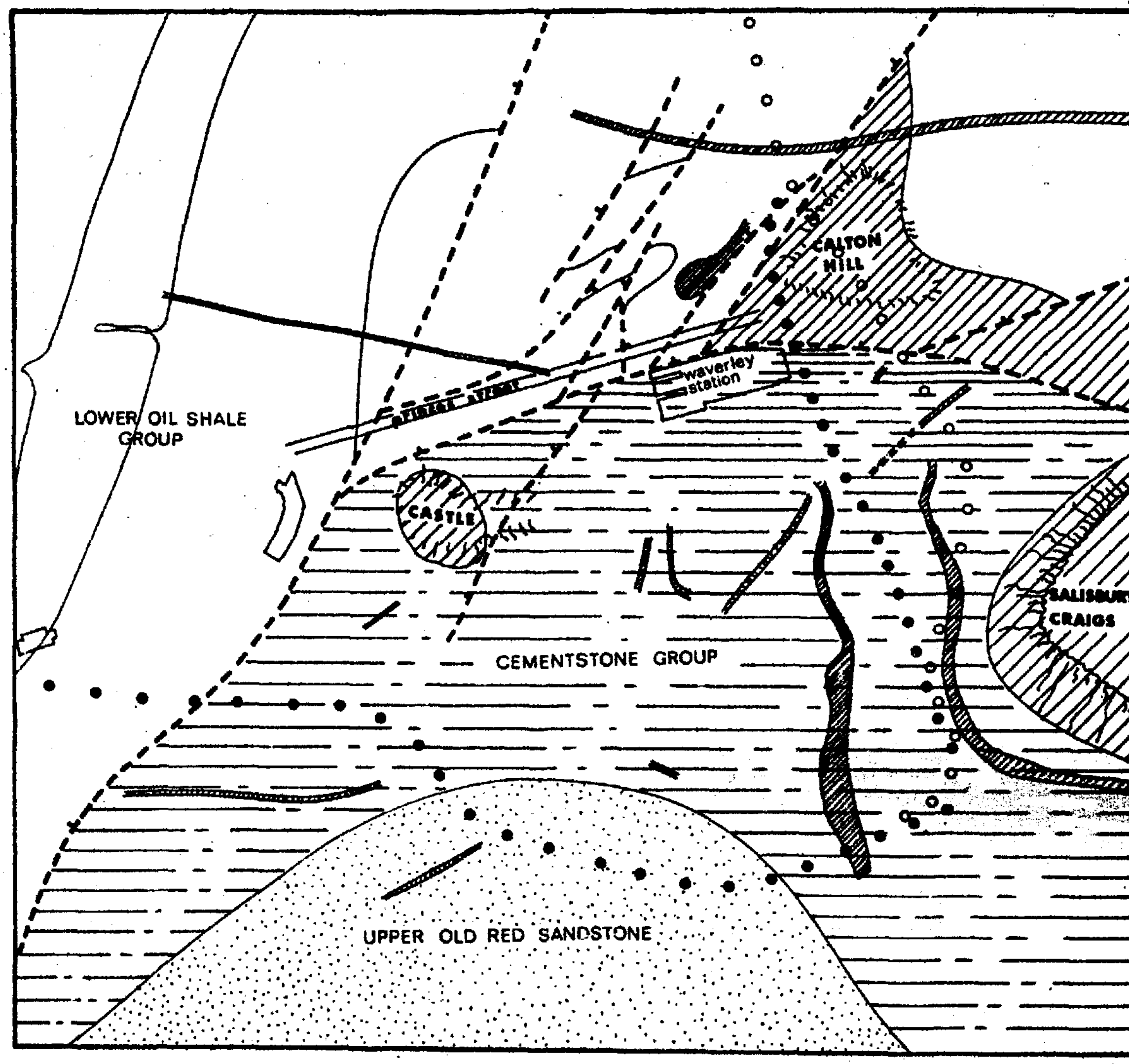
The Edinburgh City Corporation and Drilling & Prospecting International Limited (D.P.I.) agreed in October, 1968 to the author's co-operation in planning and conduct of a rock testing programme aimed at supplying the rock character information necessary to tackle this rangle of problems. The rock at Edinburgh also shows great variety. The majority of the site is covered by glacial or fluvioglacial material, fill and recent river, lake or beach deposits with thicknesses commonly up to fifty feet, but occasionaly much thicker. Exploration therefore depended on core samples from some 400 boreholes ranging in depth from 20 to 150 feet. Volcanic rocks (basalts and welded basaltic tuffs) outcropped at Calton Hill, the site of a proposed tunnel section. Bedrock to the South and West consisted of sedimentary rocks of the Lower Carboniferous and Devonian, including a large percentage of mudstones and flaggy siltstones, a number of thin but hard sandstone beds and the more massive red and pink sandstones of the Devonian inlier to the South of the City centre (Figure 51a). Sills and dikes of basic igneous material cut across the line of the road at several locations.

The variety of problems and of rock, by no means atypical of this class of project, suggested a broad coverage of the site using classification testing and mapping, rather than sophisticated testing at one or two locations. The author and D.P.I. were able to maintain a close liaison during the planning and execution of the testing programme. Tests and observations of more immediate relevance to the practical problems (including observations of 'lithology' and fracture spacing in the core, uniaxial and Brazilian strength classification tests, Schmidt hardness tests and 'wet' tests for durability porosity and density) were carried out by D.P.I. technicians at their Warwick laboratories. Supplementary work to improve the research value of the project such as thin section and stereo microscopic examination of the rock material, sound velocity measurements and further uniaxial testing, also compilation and analysis of all test results, were conducted by the author at Imperial College.

The disposition and character of individual beds could be presented on large scale sections and plans provided that suitable samples and test results were available in the vicinity. The extremely variable nature of the sedimentary rocks did not often allow correlation of one borehole with the next, however, despite the large number of closely spaced boreholes and the collection and testing of over 250 rock samples, each comprising several specimens. On the basis of test results the site could, however, be divided into five zones with each zone being characterised by its % composition in terms of five constituent 'rock types', namely basalt, very hard sandstone, hard sandstone, siltstone and mudstone. The rock categories themselves were characterised by means of classification tests and observations particularly with respect to their mineralogical and textural peculiarities and their 'brokenness' (fracture spacing), 'durability' (assessed by the slake test), and 'hardness' (assessed by compounding results of uniaxial, Brazilian, porosity, velocity and Schmidt tests). Each zone (Figure 51b) differs considerably from the next in terms of mechanical character. Zone descriptions, reproduced in Figure 52, offer information relating to engineering behaviour that is not available from conventional geological descriptions. Testing programmes along similar lines to that employed at Edinburgh have subsequently been applied by D.P.I. to a further five projects and the favourable response of clients encourages further development of this approach to rock testing and mapping.

The 'brokenness-hardness' classification that evolved as a result of this project appears particularly promising and will be discussed in more detail later in this chapter. It would appear that a single test to assess 'hardness' rather than compounded results of a set of tests is to be preferred, and the point load test is proposed for this purpose. The project also demonstrated the usefulness of simple slake tests to assess mudrock durability.

The Dartmoor project, aimed at relating mechanical properties of various rocks to their state of weathering, is being conducted by Dr. W. Dearman of the University of Newcastle and Dr. P. Fookes of Imperial College who invited the author to participate by suggesting and supervising a rock testing programme. Dr. Fookes is contributing a specialised knowledge of rock weathering, while Dr. Dearman has extensive practical experience of the local geology. In contrast to the Edinburgh project this study is an academic



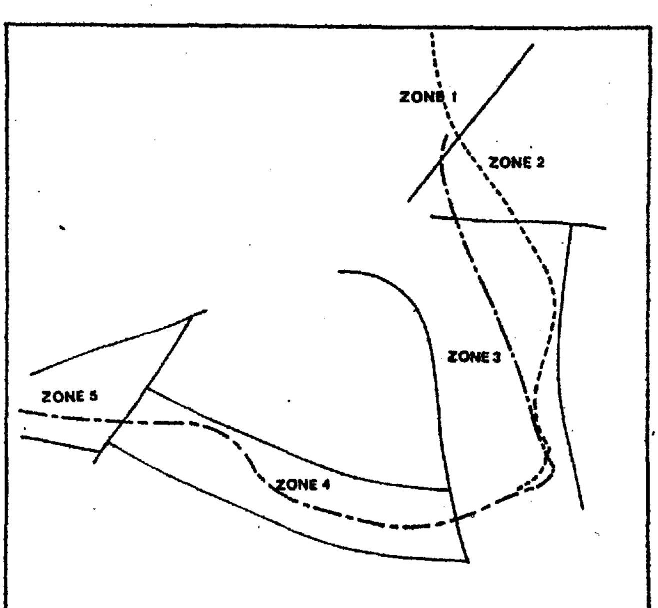


Figure 51a

Geological map of Edinburgh road and sewer scheme. The road line is denoted by closed circles, the sewer by open circles. Igneous rock is shaded obliquely.

Figure 51b

Key to subdivision of the Edinburgh site into zones of characteristic mechanical and engineering nature. The nature of each zone is apparent from the tabulation of Figure 52

ROCK TYPE		MUDSTONE	SILTSTONE	SANDSTONE	BASALT	TUFF
ZONE 1	Brokenness	Frequent	Frequent	Infrequent		
	Hardness	Very Weak	Med Hard	Med Hard		
	Durability	Low	Durable	Durable		
	COMPOSITION Length cored (m)	16 (55%)	11 (39%)	2 (6%)		
	Samples tested	3 (60%)	2 (40%)	Nil		
ZONE 2	Brokenness				Occasional	Infrequent
	Hardness				Very Hard	Weak
	Durability				Durable	Durable
	COMPOSITION Length cored (m)				275 (88%)	37 (12%)
	Samples tested				65 (96%)	3 (4%)
ZONE 3	Brokenness	Shattered	Frequent	Occasional	Occasional	
	Hardness	Very Weak	Very Weak	Very Hard	Ex. Hard	
	Durability	Low	High	Durable	Durable	
	COMPOSITION Length cored (m)	659 (67%)	205 (21%)	94 (9%)	25 (3%)	
	Samples tested	57 (54%)	29 (27%)	15 (14%)	5 (5%)	
ZONE 4	Brokenness	Abundant	Frequent	Infrequent	-	-
	Hardness	Very Weak	Med. Hard	Very Hard	-	-
	Durability	High	Durable	Durable	-	-
	COMPOSITION Length cored (m)	162 (26%)	145 (23%)	300 (48%)	5 (1%)	8 (2%)
	Samples tested	20 (21%)	27 (30%)	44 (48%)	Nil	1 (1%)
ZONE 5	Brokenness	Frequent	Abundant	Infrequent		
	Hardness	Very Weak	Med. Hard	Very Hard		
	Durability	High	Durable	Durable		
	COMPOSITION Length cored (m)	62 (70%)	24 (26%)	4 (4%)		
	Samples tested	4 (50%)	3 (37%)	1 (13%)		

Figure 52 : Edinburgh zone descriptions

excercise, although carried out with engineering applications in mind. Both P. Fookes and W. Dearman are members of the Engineering Geological Mapping Subcommittee of the Geological Society, hence interested in the engineering application of geological methods.

Rock samples were collected and outcrops observed in the vicinity of Okehampton, on the borders of the Dartmoor granite. A wide variety of rocks have been quarried in this area for roadstone and railway ballast, and there are excellent exposures of rocks at various stages of weathering. Samples were collected from the Dartmoor granite itself and from adjacent volcanics and intrusions of dolerite, aplite and tuff. A variety of sedimentary materials were also collected, noticeably hardened by contact with the granite magma but still retaining the majority of their original character. Samples tested included chert, quartzite, limestone, sandstone and baked mudstone.

The 'degree of weathering' was assessed in the field and each sample was assigned a weathering degree from 1 (fresh) to 6 (residual soil). Field observations included the fracture spacing and the ratio of soil to rock in the outcrop. Some of the more permeable rocks tend to weather right through, so that the whole outcrop is either fresh, moderately or severely weathered (granites are notable for this behavior). Most rocks however, weather by formation of a shallow decomposed skin along fissures that gradually thickens leaving fresh, strong 'lithorelics' or boulders in a matrix of decomposed material. If samples of these lithorelics are tested they may show little deterioration in mechanical properties even though the outcrop as a whole shows quite a high degree of weathering.

Test results, incomplete though they are at present, appear to demonstrate this behaviour (Figure 53). To supplement observations of fracture spacing and soil-rock ratio, tests for porosity and strength (the point load 'lump' test) were carried out. Porosity determination was by means of the water saturation technique, and lump testing was performed on saturated, air dry and oven dry subsamples, more to evaluate the test technique than to benefit the project. To this end over 2500 rock lumps are to be tested. It is worth noting that fewer than 50% of the samples could readily be machined into specimens suitable for uniaxial testing. It is much to be regretted that at the time of the field study a portable lump testing apparatus was not yet available. The apparatus used in the laboratory consisted on a straight adaptation of a 100 Ton uniaxial compression testing frame which, although it gave accurate results proved cumbersome and the technique inefficient. The author is co-operating with Engineering Laboratory Equipment Limited in the development of a portable point load testing machine that should overcome some of these problems.

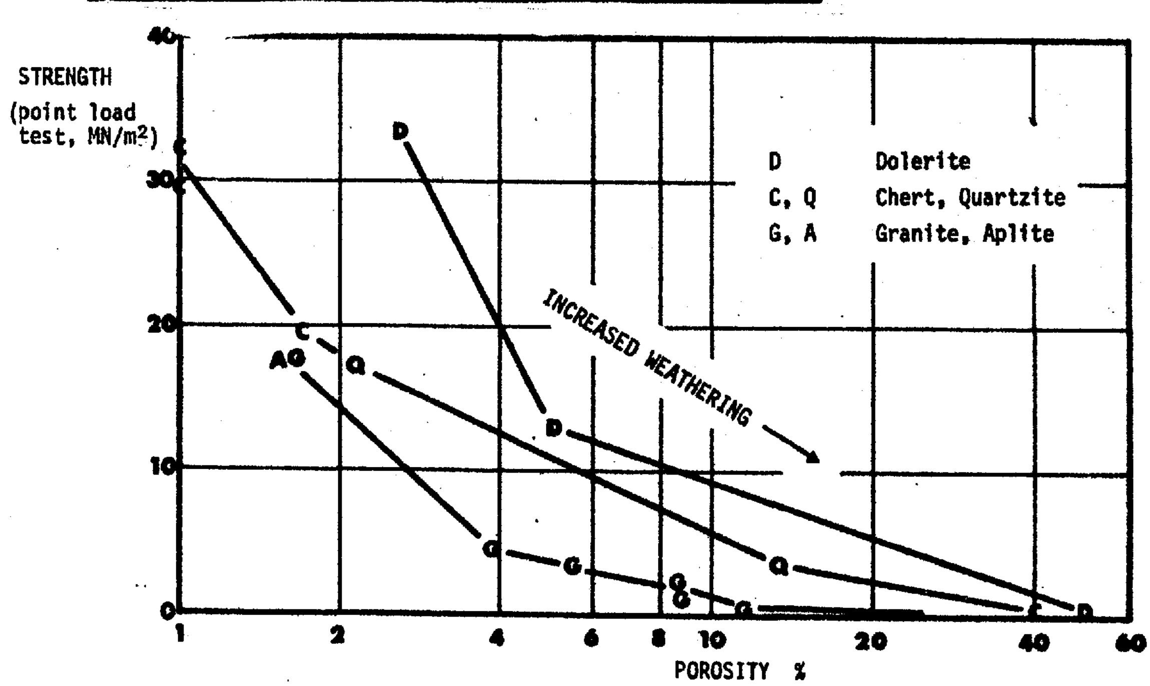
The work of the 'Commission on Laboratory and Field Testing of Rock' of the International Society for Rock Mechanics is relevant to future development of rock testing procedures and their standardisation. The commission circulated a questionnaire to members of the International Society to test opinion on the need for standard test procedures, the response indicating a general desire for standards to cover a wide range of rock test techniques. The Commission met

weathering index	strength MN/m²	porosity %	sample no.	rock type
1.5	17.0	1.65	27	granite
3	3.5	5.5	24	
3.5	1.2	8.85	25	
3.7	4.6	3.9	31	
3.7	2.2	8.7	32	
4.2	0.5	11.56	29	
5.2	sand	sand	26	
2	33.1	2.6	23	dolerite
4	13.0	5.1	15	
5	0.1	50.0	14	
2	15.7	1.45	1&2	tuff
2.5	17.0	7.7	30	
3.5	7.4	10.9	17	
3.5	5.9	13.45	28	
4	4.2	10.0	33	
1.5	29.5	0.65	13	chert
2	32.2	0.4	6	
3	19.9	1.7	22	
5	0.1	40.9	20	
2	40.7	0.55	7	limestone
3	34.9	1.1	8	
3.5	45.6	1.35	9	
5	0.1	58.8	21	
fault	15.8	0.9	4	baked mudstone
2	17.6	0.8	3	
2	16.6	0.9	5	
2	18.9	1.05	11	
2.5	17.6	2.1	10	quartzite
3.5	3.7	13.2	16	
2	29.4	1.5	19	sandstone
3	9.4	8.3	18	
1.5	17.6	1.55	12	aplite

Figure 53

Results of tests and weathering observations on a suite of Dartmoor rocks.

The weathering index, allocated on the basis of several observations such as the presence or absence of discolouration along fissures, corresponded closely to increases of porosity and decreases in strength. The high values of porosity attained even in the case of igneous rocks were unexpected, the graph below serving to illustrate the general correlation between strength and porosity. The point load test on irregular lumps of rock proved a satisfactory strength estimator, and no test requiring specimen preparation would have been possible in view of the weathered state of the rock.



recently in Oslo under the chairmanship of Professor D.U. Deere of the University of Illinois when an attempt was made to group together related test technques and to assign priorities for standardisation. Three major categories of test were proposed; 'research tests' whose function placed them beyond the scope of classification, 'design tests' and 'tests for classification and characterisation of rock'. Priorities for standardisation were assigned taking account of the fact that some classes of test could, given the present state of development of these tests, be more readily standardised than others. These tended to be given higher priority than other categories that might be of more practical value but would prove difficult to standardise. The order of priorities cannot therefore be taken to indicate an order of merit for the testing methods; the standards are to be tentative and will no doubt undergo modification in the light of experience, particularly since they represent the first attempt at standardisation on an international scale.

The categories of test as enumerated by this committee, listed in order of priority for standardisation, are illustrated in Figure 54. The Commission decided to attempt the drafting of tentative standards for five of these categories during the forthcoming year, the standards to be ready by the time of the Belgrade meeting of the International Society in September. Standardisation of only one category of design test was to be attempted, namely those tests that measure the modulus of deformability in situ. Initial drafting of standards for strength classification testing, including uniaxial strength testing and point load testing is to be undertaken by P. Duffaut of France; standards for hardness, drillability, attrition and abrasion tests are to be drafted by Professor Deere; standards for porosity and density tests and standards for durability and swelling tests are to be drafted by the author. Drafts are to be complete by the end of January 1970, when they will be circulated to other members of the Commission for critical review.

The existence of standards for testing procedures in no way compels use of the tests that have been standardised and controversy over the selection of tests to suit any particular problem or suite of rocks is likely to persist for some time. Standardised testing procedures should, however, allow the accumulation of a body of compatable test data and experience in using the tests allowing improved techniques to be recognised and incorporated in the standards as they become available. The evaluation of tests should prove a much simpler task when the variation in techniques used for testing has been reduced by the introduction of some measure of standardisation.

b) General outline of proposals for rock description **

A set of tests and observations is proposed for the purpose of rock sample description, and is set out as a 'sample description sheet' in Figure 56. The general form of this scheme will now be outlined, followed by a detailed description of the various rock characteristics to be observed or tested. The scheme is designed

** These proposals have been summarised in a paper presented for the 1970 International Conference on Rock Mechanics (Franklin, 1970)

International Society for Rock Mechanics

Figure 54 : List of rock tests and priorities for standardisation

CATEGORY 1 CLASSIFICATION AND CHARACTERISATION

A Intact rock material

Priority: (1) Unit weight, water content, porosity, absorption

(2) Strength and deformability modulus in uniaxial compression; point load strength

(3) Hardness abrasion, attrition and drillability (Schmidt hardness, Shore scleroscope, indentation, Los Angeles test, Daval test etc.)

(4) Swelling, slaking and durability

(5) Sound velocity; pulse and resonance (Lab.)

(6) Permeability (Lab.)

(7) Micro-petrographic description for engineering purposes (emphasis on mechanically important features)

(8) Anisotropy indexes.

B In-situ mass

Priority: (1) Joint systems; orientation, spacing, openness, roughness geometry, filling and alteration.

(2) Core recovery, rock quality designation and fraction spacing.

(3) Seismic tests for mapping and for rock quality index purposes.

(4) Geophysical logging of boreholes.

CATEGORY 2 ENGINEERING DESIGN TESTS

A Laboratory tests

Priority: (1) Determination of strength envelope and elastic properties (triaxial, biaxial, uniaxial compression and tensile tests) (direct shear tests)

(2) Strength of joints and planes of weakness

(3) Time dependent and plastic properties

B In-situ tests

Priority: (1) Deformability tests

(2) Direct shear tests (intact material, joints, rock-concrete interface)

(3) Field permeability, piezometric levels and groundwater flow

(4) Stress measurements

(5) Rock movement monitoring
Rock noise monitoring
Blast and groundmotions monitoring

(6) Uniaxial, biaxial and triaxial compressive strength

CATEGORY 3 RESEARCH

It was decided that research tests, including many of the rock physics tests, are beyond the scope of standardisation.

to meet the rock description requirements of an engineer, and represents a compromise between traditional geological and traditional engineering approaches, each having many points in its favour but neither proving entirely adequate as it stands.

The table contains sufficient information for a classification of the rock sample, but is not in itself a classification since no subdivisions or classes have been proposed. Data on which to base such a subdivision is not at present available. A form is suggested, however, for a simple classification system based on two of the more universally relevant properties listed in the table, namely 'brokeness' and 'hardness'. A simple classification system such as this can be useful since, although it offers less information than the complete table of properties, it is easier to visualise and to apply to rock mapping.

The rock description table is headed by a concise rock name, employing a simplified geological name with the basic classification observations as descriptors. Hence 'shattered, very hard basalt' would be used in preference to geological alternatives such as 'porphyritic trachybasalt', 'broken, hard, low durability mudstone' would replace 'compacted sandy silty mudstone' and this form of description may well prove of more value to the engineer particularly if it is supplemented by numerical values for the properties described. The concise rock name is followed by supplementary information relating to properties observed or tested in both the bulk sample (fissures, joints and structural relationships) and in the sample of intact material. The 'sample' should therefore be considered as a location within the rock mass rather than, more conventionally, as a lump of rock. No rigid demarkation has been proposed to separate those properties than can be observed in the hand specimen and those which cannot. Quite often fissures, for example, are present in the hand specimen both as micro-joints, cleavage or bedding planes and these are described similarly whether observed in situ or in the laboratory.

Although conventional techniques and terminology have been used wherever possible it proved necessary to deviate from current practice on several occasions, in order to ensure that the resulting rock description would be both practical and relevant. Mineralogical composition, while considered highly relevant to engineering behaviour, has been given a far simpler treatment than is usual. Only four mineral suites, made up of eight mineral components (including pores) are recognised. These components should be readily identifiable by the engineer and each imparts characteristic mechanical properties to the rock. Texture has been emphasised, rather than detailed mineralogy. Elementary geological classifications consider only a single textural feature, namely grain size, and far greater weight has been given to this property by engineers than is justified by its apparently rather limited mechanical significance. In place of grain size it is proposed to describe rock texture in terms of four components, namely fragmental material, coarse crystalline material, fine matrix and pore space. More recent practice in limestone, sandstone and mudstone classification in the field of pure geology utilises textural descriptions based on the proportions of these four components, and an approach such as this appears to be necessary to account for the influence of texture on mechanical performance. Further textural observations are required to account for properties such as anisotropy and fissility.

A further departure from common practice is that the same observations and tests are performed irrespective of whether the rock is igneous, sedimentary or metamorphic in origin. It may be recalled that these rocks are traditionally subdivided according to different sets of criteria in order to adequately describe the genesis of the material. It appears neither necessary nor convenient to treat each genetic variety of rock differently when it comes to mechanical classification. By way of example, a rock composed of quartz, feldspar and pore space may well be igneous (a granite) if its texture is isotropic-crystalline, sedimentary (an arkose sandstone) if its texture is isotropic-fragmental, or metamorphic (a gneiss) if its texture proves to be crystalline-segregated. A similar set of tests and observations would be carried out in each case to characterise the rock mechanically, for which purpose an interpretation of genetic origin would not be required. It should be emphasised, however, that an appreciation of rock genesis would in contrast be essential in order to construct a structural map of the site, but this exercise would call for the services of a structural geologist.

Properties that may be inferred from observation may often be more quantitatively and reliably assessed by means of direct mechanical tests. Observations are usually, however, quicker and more flexible by nature. Simple tests have been listed on the rock description sheet to supplement the visual observations. Some of the tests listed, the triaxial tests for example, would not form part of a routine classification testing programme but have been included on the data sheet for convenience. (The sheet is also designed to contain data obtained by Imperial College students as part of their course of laboratory exercises). Certain tests are to be regarded as 'basic' for classification purposes, and others as supplementary depending on the nature of the engineering problem and of the rock. It is likely to prove more convenient if the results of supplementary tests are appended to the sample description sheet rather than allowing space in the sample description for each of the many possibilities.

The table is intended as a check list of properties that should be noted, however crudely, for each rock sample. Precise description of an individual sample is not the first objective in rock engineering, where it may be far more important to obtain a reliable picture of the rock mass as a whole. The tabulation of Properties may, however, be completed to any level of precision in accordance with individual requirements. Some properties such as 'fissure roughness' can hardly be quantified other than by means of crude descriptive terms such as rough, smooth or undulating but they are nevertheless important and should not be neglected. On the other hand there is a danger that properties that lend themselves to quantitative description will be over-described, obscuring the more general picture of the rock sample and the mass as a whole. A balanced, rather than a precise description is required.

c) Proposal for a concise sample description and a two dimensional classification

The following discussion relates to the concise form of sample description appearing at the top of the sample description sheet of Figure 56, and to an even more cryptic two dimensional classification scheme that may be based on this information.

Three properties, brokenness, hardness and durability are suggested as 'basic' to an engineering description of rock. These properties are by no means the only ones that may be observed, and have been chosen for their relevance to a wide variety of rocks and to a wide range of engineering problems. In the concise form of rock sample description these properties are employed to assign qualifying descriptors to the geological name for the rock, so that less importance need be attached to the geological name itself. The geological name need no longer convey a great deal of mechanical information and fewer, simpler more widely recognised terms may be employed without overmuch regard for their mechanical implications. The geological name is also supplemented by a brief textural and mineralogical summary, adjusted to reflect the mechanical character of the material better than either the geological name itself or a more extensive geological description along traditional lines.

Observations and tests will be more fully described later in the chapter since each item appearing in the 'concise description' at the head of the table also appears in more detail lower in the table. The basic classification properties will now be defined and suggestions made regarding their use as a basis for classification of rock masses.

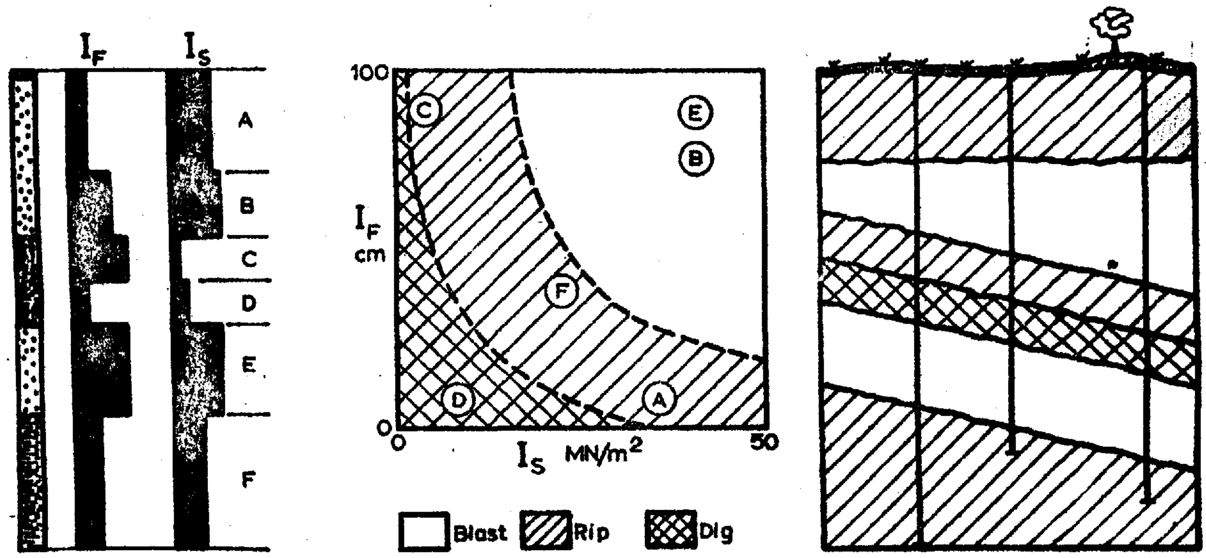
The property 'brokenness' has been defined by means of a parameter termed 'fracture index', the average linear size of blocks that constitute the rock mass. The important role played by fissures in the mechanical behaviour of a rock mass must somehow be taken into account even in the simplest of rock descriptions. A full description of fissures would involve several types of observation; fracture spacing has been taken as the most generally important since it controls the 'grain size' of the rock mass which at the one extreme may behave as a monolithic structure and on the other as a soil. It is also perhaps the simplest to observe provided that one does not look for extreme accuracy or attempt too precise a definition of the property. The fracture index has been defined so as to increase as the quality of the rock improves. A 'poor quality' rock may have a fracture index of 4cm compared with 400cm for a more massive outcrop. Fracture index may perhaps best be estimated by observing the size of lumps in a scree or a heap of excavated material, but can also be assessed as the average spacing of fractures in a rock core or in an outcrop.

A further property 'hardness' must be defined to account for the strength of individual blocks that comprise the rock mass. This property has been represented quantitatively by means of the 'point load strength index' which will be described in detail later in the chapter. The terms 'hardness' and 'strength' have here been used interchangably and this usage may require further explanation. Engineers and geologists commonly talk of hard and soft rocks implying strength rather than hardness in the strictly defined sense. Hardness sensu-stricto is a surface property, measured using scratch, identation or rebound tests and reflects the hardness of individual grains rather than the intergranular bond or coherence.

The point load strength test is a measure of coherence as well as of grain strength but the term hardness has been retained since it appears to have stronger practical connotations and to place less emphasis on precise mechanical concepts. The test may be regarded as a refinement of traditional methods of hardness estimation using a penknife or a geological hammer. It has nevertheless been shown to be closely correlated with other strength parameters so that 'hard' rocks according to this definition should also be strong in uniaxial compression, shear etc. It was selected as a measure of hardness in preference to these alternative tests on account of its greater practicality. It may be noted that the point load strength of intact material will also give some guide to the strength of fissures since the latter depends not only on fissure roughness but also on the strength of interlocking asperities on the fissure surfaces.

Estimates of both brokenness and hardness carried out at natural moisutre content may be misleading for those materials, the mudrocks in particular, that are water sensitive. Most rocks show some hardness reduction when wetted, but mudrocks may actually disintegrate when subjected to stress relief combined with moisture content fluctuations, so that both brokenness and hardness are affected. Such materials may perhaps be regarded as soils rather than rocks; however they commonly appear below, or interbedded with more durable materials, and since as a family they account for over 50% of all 'bedrock' their characteristics are of considerable importance to the rock as well as the soils engineer. The few geological names used to describe these muddy materials give little indication of their mechanical character and for these materials a further characteristic 'durability' should be determined to supplement the observations of brokenness and hardness. A 'slake durability index' is suggested for this purpose and will be described in detail later. It is a measure of the degree of disaggregation resulting from a controlled process of accelerated weathering. Following the convenient principal that 'the better the rock, the higher its index properties' this index has been defined so that a fully coherent rock will have a slake durability index of 100% (See Figure 61).

These three classification characteristics may serve to provide descriptors for a rock description but may also be used as a basis for a classification scheme. It seems expedient to select only two of the three for this purpose in order to simplify graphical presentation. Since some 50% of rocks will have a common durability of 100% the characteristics brokenness and hardness have been selected as classification criteria, omitting durability. Figure 55 illustrates schematically the use of a basic brokennes-hardness classification. In this example both brokennes and hardness have been observed in cored rock and are presented in the form of a core log (Figure 55a). The upper horizon A is broken and hard, horizon B is hard but is less broken; other horizons are described in a similar manner. It may be noted that mechanically defined horizons do not necessarily correspond to horizons of stratification. In this simplified example each horizon may be represented by a single point on the classification diagram (Fig. 55b) rather than by an area of the diagram, as would be the case more often in practice. Use of the diagram may be illustrated



(a) Core Log (b) Classification Diagram (c) Section Map

Figure 55 : Brokenness-Strength classification

Fracture Index (I_f) and Point Load Strength Index (I_s) may be recorded alongside petrological data, as part of the core log. This allows the identification of horizons with characteristic brokenness and strength; each horizon occupies its own location on the classification diagram which may be 'zoned' into mapping units. In the example above, these units are based on 'ease of excavation'.

with reference to a problem of rippability prediction. Soft rock may be easily ripped whether or not it is closely fractured, while hard rock may only be ripped if its block size is small; a less intensely fractured hard rock would require blasting. Three zones have been isolated on the diagram; a zone that can be easily dug without ripping, a zone that can be ripped without blasting, and a zone that will require blasting prior to excavation. The subdivided classification diagram may then assist in the mapping, either in plan or in section, of the site according to the techniques required for excavating each horizon. Such a map requires reliable borehole intercorrelation if mechanical properties are to be interpolated between boreholes; an alternative is to assess the site as a whole according to its excavation problems and plant requirements.

The example serves to highlight the shortcomings, as well as the advantages of this approach to classification. Ease of excavation can be predicted only to a limited extent, since it depends not only on brokennes and hardness but also on block shape (slabby or cubic)) and on fissure orientation. Loss of such information is the price paid for the simplicity of a two dimensional classification. Further applications of the proposed scheme, such as to the selection of rockfill materials, may readily be visualised. In other contexts the value of brokenness-hardness information is less obvious, for example in the case of slope stability prediction, but (in the author's opinion!) becomes apparent on closer examination. The two dimensional scheme is intended to apply across the whole range engineering problems, with the proviso that in many cases supplementary information will be required.

This approach to classification may be compared with others. Rock Quality Designation (R.Q.D. after Deere et al, 1966) has been widely accepted as a useful approach to rock quality description. It reflects both the brokennes and to some extent the hardness of the rock (the ease with which fractures are created during drilling) but perhaps has less direct mechanical significance then a statement of these two properties independently. R.Q.D. calls for observation of fracture spacing but the observed spacings are presented only as a proportion of blocks exceeding 4 inches in size, so that much valuable information is lost. A further technique for rock quality assessment is based on the measurement of sound velocity through the rock, and a comparison of laboratory and field values in terms of the 'velocity ratio'. This technique, like that based on R.Q.D. is difficult to interpret in terms of properties with direct mechanical significance. Velocity ratio results are also related to brokenness and hardness, also to the openness of fissures and the extent to which the rock is saturated, and to other factors that tend to obscure the picture. Despite its limiations a brokenness-hardness classification would appear to score over others available when it comes to simplicity of execution and of interpretation. Its limitations are obvious ones so that there is perhaps less danger of the scheme being used in the wrong way.

It should again be emphasised that before such a classification scheme can become of real practical value, development in two directions is required. The classification must be related to rocks and to problems. Firstly the properties brokenness and hardness must be evaluated for a wide range of naturally ocurring rocks so that observations made on a single sample or site may be put in perspective. This may be accompanied by subdivision and nomenclature of rock classes so that a 'hard, broken rock' on one site corresponds to a rock from elsewhere that has been similarly described. Secondly the characteristics noted in classification must be related to practical engineering performance so that they may be used to predict rock behaviour. From both points of view a classification is likely to become more useful as it is increasingly used. Selection of suitable tests and observations is only the first step in this process.

To conclude this section we may return from discussion of two dimensional classification to complete the account of the 'concise rock description' at the head of the rock description sheet . The purpose of the descriptors used to amplify the geological name may now be more evident. Two of these descriptors are in fact the parameters proposed for basic classification. Selection of the geological name itself is none too critical since other supplementary

ROCK SAMPLE DESCRIPTION — **SAMPLE** *207*

Site location *Darley Dale, Derbysh.*	Sample location X Y Z *Stancliff quarry* , BH depth
Fracture index; Strength index; Durability index; Stratigraphic position; Geol.name *60cm + 3.32MN/m² (moderately weak) durable carboniferous sandstone*	
Texture; %F, %X, %μ, %P; Isotropy Coherence *87F, 13P highly isotropic & uniform, coherent*	
Minerals; *83* %hard, *17* %soft *quartz-pore*	

FISSURES set	dip/dip direction		Spacing (S) cm	Openness	Persistence	Planarity/ roughness	Filling/ alteration
1							
2							
3							
4							

Block size (fracture index) $(S_1+S_2+S_3)/3$ cm	Block shape $(S_1:S_2:S_3)$
Other structural features *sawn blocks from stone mason*	

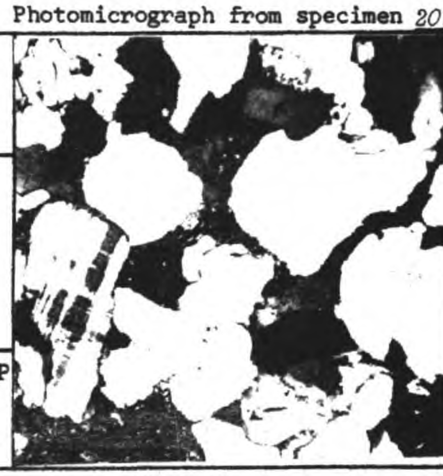
ROCK CONSTITUENTS	%	Size	Texture	
Dark grains	*2*	*3*	*F*	*83* % Hard
Quartz	*79*	*3*	*F*	
Fresh feldspar	*2*	*3*	*F*	
Altered minerals	*1*	*3*	*F*	
Salts				*17* % Soft
Platy minerals	*3*	*3*	*F*	
Clay minerals				
Pores	*13*	*2*		

Textural summary; *87* %F, % X, % μ, *13* %P

random/~~oriented/segregated~~

Coherent/~~friable/fissile~~ *slight friability*

Photomicrograph from specimen *207/7*



Magnified X *100*

Lighting *crossed polars.*

Direction of section

POINT LOAD STRENGTH (strength index P/d^2) *3.324 MN/m²* description of method; *diametral point load test*	
SLAKE DURABILITY (durability index %) Liquid limit Plastic limit description of method; *durable - not tested*	
POROSITY (hand specimen %) *12.6%* Dry rock density *2281Kg/m³* description of method *saturation, mensuration*	
SOUND VELOCITY (laboratory) *2614m/s* (field) SCMIDT REBOUND No.	
UNIAXIAL COMPRESSION strength *74.8MN/m²* Young's modulus *16800* Poisson's ratio *0.27*	
TRIAXIAL COMPRESSION parameters σ_c G ; A B ; c tanϕ No. specimens *27* *MN/m² units 93.3 4.70 2.44 0.796 21.5 0.853*	

GENERAL COMMENTS *Highly reproducible results - uniform material*

Figure 56 : Rock sample description sheets

Site location *Atalaya, Spain*	Sample location X Y Z , BH **depth** *Open pit, North face*
Fracture index; Strength index; Durability index; Stratigraphic position; **Geol.name** *100cm* *strong* *durable* *welded tuff*	
Texture; %F, %X, %μ, %P; Isotropy Coherence *20%4F,40%3X,39%2Xμ,1%P* *ill sorted, some size segregation, coherent*	
Minerals; *99* **%hard,** *1* **%soft** *Quartz with 5% iron minerals*	

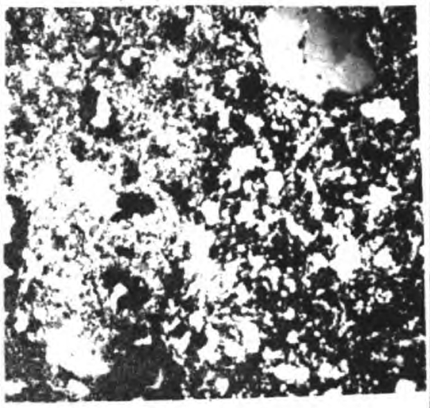
FISSURES set		dip/dip direction		Spacing (S) cm	Openness	Persistence	Planarity/ roughness	Filling/ alteration
joint	1	*90°*	*175°*	*100-300*	*tight*	*50%-80%*	*wavy and*	*slight-*
joint	2	*80°*	*95°*	*"*	*"*	*"*	*rough*	*weathering*
joint	3	*35°*	*250°*	*"*	*"*	*"*	*"*	*"*
	4							

Block size (fracture index) $(S_1+S_2+S_3)/3$ *100* cm	**Block shape** $(S_1:S_2:S_3)$ *1:1:2 (cubic)*
Other structural features *joint properties variable from one location to next*	

ROCK CONSTITUENTS	%	Size	Texture	
Dark grains *iron minerals*	*5*	*4*	*F*	*99* % Hard
Quartz	*94*	*4+2*	*F+X+Xμ*	
Fresh feldspar				
Altered minerals				*1* % Soft
Salts				
Platy minerals				
Clay minerals				
Pores	*1*	*1*		

Textural summary; *20* % F, *40* % X, *40* %*Xμ*, *1*% P
~~random/oriented/segregated~~ *slight segregn.*
Coherent/~~friable/fissile~~ *ill sorted*

Photomicrograph from specimen *214/2*



Magnified X *100*
Lighting *crossed polars.*
Direction of section

POINT LOAD STRENGTH (strength index P/d^2) description of method;
SLAKE DURABILITY (durability index %) Liquid limit Plastic limit description of method; *durable - not tested*
POROSITY (hand specimen %) *0.89%* Dry rock density *2910Kg/m³* description of method *saturation & mensuration*
SOUND VELOCITY (laboratory) *5230m/s* (field) *5000m/s* **SCMIDT REBOUND No.** *51.3 (20)*
UNIAXIAL COMPRESSION strength *65.0MN/m²* **Young's modulus** *49098* **Poisson's ratio** *0.16*
TRIAXIAL COMPRESSION parameters σ_c G ; A B ; c tanϕ No. specimens

GENERAL COMMENTS *Angular, ill sorted quartz fragments in low porosity crystal-net quartz matrix (welded tuff)*

Figure 56 : Rock sample description sheets

information is available to characterise the rock mechanically, texturally and mineralogically. Simple names in common use appear to be more desirable, terms such as those used in the 'trade grouping' of concrete aggregates, or those defined in the British Code of Practice for Site Investigation CP2001 (Figures 22 and 23 of Chapter 2). Such terms contain only limited mechanical information and may even be misleading, so that they should not be isolated from their mechanical, textural and mineralogical descriptors when making predictions of engineering behaviour of these materials. The meaning and relevance of the textural and mineralogical information contained in this portion of the sample description sheet will, it is hoped, be clarified in subsequent discussion.

d) Fissure description

This topic lay outside the scope of the author's research, and the observations suggested for fissure description are those that have been suggested by others. Nevertheless this aspect of rock description must be included to avoid a biased account. It would be unwise to make sharp distinctions between fissure properties, intact material properties and rock mass properties since all three are closely related.

A few comments are required regarding the definition of a fissure. A fissure may correspond to bedding, jointing, cleavage or may have a man-made cause. No clear distinction may be defined between a 'tight' fissure and a plane of weakness. Neither are fissures solely a field phenomenon since they are often present as cleavage, bedding or microjointing in a hand specimen. They may be more loosely referred to as 'discontinuities' since they dissect the rock mass giving it a blocky as opposed to a monolithic character.

Fissures have been tabulated (Figure 56) as *sets* of parallel or sub-parallel weakness planes, and space has been left to record the nature of each set, whether bedding, cleavage or jointing. It may not always be possible to recognise distinct fissure sets without extensive statistical observations, but major sets are usually prominent and are identified without much difficulty. The properties recorded are therefore average properties for each set rather than those of an individual fissure.

Orientation is recorded in terms of dip and dip direction. The more recent and practical dip observation instruments give results in this form rather than in terms of dip and strike.

Fissure spacing has been recorded as a direct linear measurement rather than in terms of the number of fissures per unit length (fissure frequency is the inverse of fissure spacing) since spacing appears to have more direct physical meaning in terms of the 'grain size' of the rock mass. Also when fissures are closely spaced it becomes far simpler to observe a representative spacing value than to count the number of fissures intersecting a traverse. The recorded value represents the average of a number of readings in the vicinity of the sample. The 'apparent spacing' must be corrected to give the actual, or shortest spacing between fissures if the line of traverse along which observations are made does not closely correspond to the normal to the fissure set. Again, however, only a modest level of precision is required.

For the remaining fissure characteristics that should be observed, no widely accepted quantitative treatment is available yet they are of undoubted mechanical importance. Fissure openness ('width') is extremely difficult to quantify although attempts have been made using feeler gauges etc. It must be emphasised that surface fissures are likely to be much more open than subsurface ones. Fissure openness has a major influence on permeability of the rock mass, also on strength, but only minor effect on rock mass porosity. To describe openness, qualitative terms such as 'tight' or 'open' are envisaged until more quantitative methods become available.

Fissures can often be traced over only a limited distance. They may be truncated by other fissures, or may become so 'tight' that they cease to exist. The term 'continuity' has been used to describe this characteristic, but since it leads to expressions such as 'the continuity of discontinuities' the alternative term 'persistence' has been used in its place. Fissure persistence, the percent of total length that can be regarded as fissure rather than intact rock, is extremely difficult to assess, particularly since a fissure may be truncated in two directions, not just one; it may be a 'penny shaped' crack. Nevertheless an estimate of persistence is often required for shear strength calculations. The persistance, and the openness of a fissure set are obviously closely related.

Fissure planarity (large scale undulations) and fissure roughness (small scale undulations) are properties of major mechanical significance since even the largest scale in situ test is likely to be small in comparison with the magnitude of such effects; an in situ fissure is likely to be stronger, possibly far stronger, than a smaller specimen of fissured rock. Strength test results might have to be upgraded accordingly, although techniques whereby this might be achieved are not yet available. Amplitudes and periods of undulations can be observed, but a system of coded descriptive terms is likely to be of more value given the present limited state of knowledge regarding the use of such data.

A further property of considerable mechanical importance is the nature, presence or absence of infilling material or of a weathered or altered zone adjacent to the fissure. Infilling material, strictly defined, has been transported into the fissure from outside, but soft fillings observed in joints and fissures may also be due to the weathering or alteration of local intact rock. Fissures usually present paths for the ingress of water, and weathering (depending on the permeability of intact rock, the water chemistry and the susceptibility of the rock material) may result in a weakened layer of considerable thickness adjacent to the fissure. Possible swelling and disruptive effects of certain types of infilling material or weathering product, their effect on permeability and reduction of fissure strength due to decreased interlocking are properties that should be considered. Certain types of filling material may better be described as gouge since they result from shear movement parallel to the fissure. The presence of gouge probably means that the fissure will have been considerably weakened, since interlocking effects may have been reduced to a minimum.

Hardly sufficient space has been left in the tabulation for a full description of each of these characteristics, but the tabulation may act as a check list and may be completed using a coded system of descriptive terms. The fissured state of the rock mass has been summarised using two indices, one for block size (the 'fracture index' described earlier and one for block shape. The latter may be described using terms such as slabby, cubic or columnar, but perhaps more quantitatively by means of the ratio of three orthogonal dimensions of a typical block (approximately the ratio of three principal fracture spacings). The fissure property description is completed with an account of other structural features that may be of mechanical importance. Comments such as 'intensely weathered zone', '3ft dike', 'similar to other sandstone beds in rhythmically bedded outcrop' are envisaged. Major structures such as folding and faulting would appear on the overall map of the site rather than in the description of an individual sample.

e) Textural and mineralogical description

Textural and mineralogical information has been included in the rock sample description despite the fact that such information is often regarded as academic and is little used by the engineer. This unfortunate situation has resulted from petrological descriptions that are lengthy and which concentrate on items of interest rather than of mechanical relevance. However, petrology determines rock behaviour, so that it should be possible to use a suitable set of petrological observations to predict this behaviour. Simplification is essential, firstly to place the observations at the disposal of the engineer armed with only a minimum of geological training, secondly to concentrate attention on those features considered to be of most mechanical relevance. The textural-mineralogical information contained in the tabulation of Figure 56 is the result of an attempt at simplification in both these directions, and will now be outlined.

Only eight rock constituents have been listed; this may represent an oversimplification of the very many rock forming minerals, but these eight are probably the most abundant and also of most mechanical importance. They are also quite easy to recognise without extensive geological training. Tabulation includes the percent by volume, size and 'texture' of each, terms that will be explained below. The tabulation is completed by a mineralogical and textural summary of the rock as a whole that appears to the right and below the list of rock constituents, also in the 'concise rock description' at the head of the rock description sheet. The reasons for selecting these particular eight constituents to represent rock mineralogy will, it is hoped, become clear from the following discussion.

Monomineralic rocks are rare; minerals occur as 'assemblages' in association with each other. Only five categories of mineral assemblage, it is suggested, occur with sufficient abundance to justify detailed consideration by the engineer, although exceptional rocks (such as coal) do exist and may well require special treatment. Each of the five differs considerably from the next both in appearance and in mechanical behaviour. They are tabulated in Figure 57, together with typical igneous, sedimentary and metamorphic rocks found in each category, and the peculiarities of each are discussed below.

CATEGORY	IGNEOUS	SEDIMENTARY	METAMORPHIC
I QUARTZOFELDSPATHIC	acid igneous rocks	quartz sandstone, arkoses	gneisses, granulites
II LITHIC/BASIC	basic igneous rocks	lithic sandstone, greywackes	amphibolites
III PELITIC (clay)		mudstones	slates, phyllites
IV PELITIC (mica)			schists
V SALINE/CARBONATE		limestones, salt rocks	marbles, calc-schists

Figure 57 : Common mineral assemblages in natural rocks

Most natural rocks fall into one of the five categories of mineral assemblage. The categories should be easily recognised and each is likely to have a characteristic mechanical behaviour.

The quartzofeldspathic category includes the acid igneous rocks (granites, rhyolites etc.) and their sedimentary and metamorphic derivatives namely the quartz and arkose sandstones and the gneisses. The properties of rocks in this group, characteristically strong, brittle and durable, are controlled primarily by the content of minor minerals such as pores and mica that are significantly weaker than the more abundant constituents, also by the ratio of quartz to feldspar. Quartz is mechanically stronger than feldspar on account of the strong cleavage characteristic of the feldspar family of minerals. Feldspars are also readily weathered to weak, clay-like minerals so that the state of alteration of the feldspars is an important mechanical index.

The lithic/basic category includes the basic igneous rocks (identified for engineering purposes by their 'colour index', their high content of dark coloured minerals), also the lithic sandstones and greywackes whose fragments are usually also dark in colour but comprise pieces of basic rock rather than single mineral grains. Also included are the amphibolites, metamorphic derivatives of basic igneous or lithic sedimentary rocks. The group as a whole is quartz deficient and therefore somewhat weaker than quartzofeldspathic materials, although considerably stronger, more brittle and durable than rocks in the remaining three categories. Dark minerals, like feldspars, weather and alter quite readily to clay like materials, so that their freshness should be considered in classification of this group of rocks. Metamorphic minerals are particularly prone to weathering since they are often of unstable form. Lithic sandstones may well prove more competent materials than quartzofeldspathic varieties, however, on account of their texture which is usually mud-filled rather than porous.

Pelitic rocks comprise the muddy sediments and the micaceous schists, usually derived from mudrocks by metamorphism. The category has been subdivided into clay bearing and mica bearing varieties. They are similar in that both clay and mica minerals are platy or tabular and tend to act as stress raisers, localised weaknesses in the rock material. Their mineral shape and their tendency towards segregation and subparallel orientation usually renders a clay or mica rock fissile, or at least anisotropic in mechanical behaviour. The difference between the two is that unlike the micas, the clay minerals have a strong affinity for adsorbed water. This renders mudrocks unusually sensitive to moisture conditions, and they are often non-durable giving low values for slake durability. The pelitic rocks as a whole, particularly the clay bearing varieties, are usually weaker and more ductile than either quartzofeldspathic or lithic-basic rocks. There is, however, a continuous gradation between these two categories and the pelitic rocks. An important index to mechanical behaviour is therefore the ratio between pelitic material and quartzofeldspathic or lithic-basic material in the rock, which may be expressed more simply as the clay, mica or quartz content of the material. The type of clay or mica may also be mechanically important, but usually not to so great an extent as its abundance and textural configuration.

The remaining category, the saline or carbonate rocks, are constituted of minerals with ionic character; these minerals deform by plastic mechanisms at comparatively low temperatures and pressures so that saline rocks are prone to ductile rather than to brittle behaviour. Perhaps of more mechanical importance is the dominantly biochemical origin of these rocks that has resulted in a great variety of textures, and a correspondingly large range in mechanical properties. The porosity and pore size of these rocks is particularly variable, and both are essential indices to mechanical performance (cf. chalk and marble). Crystallinity of these rocks is also of mechanical importance, and there is a more or less continuous gradation from fully fragmental to fully crystalline varieties that must be accounted for in textural description. Although most saline rocks are almost free from contamination there are some materials intermediate between saline and quartzofeldspathic, pelitic or basic composition and calcite cemented sandstones are particularly common. The relative proportions of such constituent materials should be noted.

The five mineral assemblage categories that have been described do not appear as such in the rock description table. They are, however, reflected in the choice of rock constituents for tabulation. The percentage by volume of each may be estimated after examining a hand specimen, thin section or both. The level of precision required for engineering purposes is unlikely to demand data to better than 5% accuracy. Pore space however, calls for special treatment and should be estimated by testing rather than by observation since porosity values are rather more critical to mechanical performance than percentages of other constituents. The sizes of pores, also both the sizes and volumes of other constituents, may be estimated visually. Rapid size estimation is assisted by use of predetermined size classification, and one proposed for this purpose is presented in Figure 58. This simple logarithmic subdivision corresponds to one of the standards for sediment size nomenclature, but its use should not be restricted to sediments; it may also be applied to pore size description and to description of grain size in igneous rocks. The 'texture' of each constituent is also tabulated, and a fourfold code is proposed for this purpose. The symbol 'F' is used to denote a constituent that occurs as visible fragments (detrital or clastic material). 'X' denotes a constituent that occurs as an interlocking crystal mosaic, whether in an igneous rock or as the crystalline cements of a sediment. 'μ' is used to denote a constituent that occurs as a matrix to a coarser fragmental or crystalline fraction, this symbol being prefixed by the upper size limit of the matrix material and by the probable texture e.g. 3Fμ 1Xμ. The fourth textural symbol 'P' is reserved for pore space.

SIZE CODE	SIZE	EQUIVALENT SEDIMENT
1		Clay
	.002 mm	
2		Silt
	.02 mm	
3		Fine Sand
	.2 mm	
4		Coarse Sand
	2 mm	
5		Gravel
	20 mm	
6		

Figure 58 : Proposed rock grain size subdivision.

The subdivision, identical to that used by the International Society for Soil Science, is simpler than that usually applied to sedimentary rocks, but more complex than that customary for igneous and metamorphic materials. It is proposed as adequate for engineering description of grain and pore sizes in all rocks, irrespective of origin. If a more precise description of grain size were required then actual grain sizes could be stated.

Having tabulated the properties of each constituent, the properties of the rock as a whole are summarised to the right and below the tabulation, and this summary is then included in the concise description at the head of the sheet. A simple index to the mineralogical nature of the rock is provided by comparison of the proportions of 'hard' and 'soft' constituents. This may well be improved after further investigation by adopting a system of weighting, so that for example pores are given greater weighting than salts as soft constituents, and altered minerals are weighted according to their degree of alteration. Texture is summarised in terms of four textural components

Fragmental, Crystal mosaic, Matrix and Pore and in addition two further textural observations are noted. These record the isotropy and the coherence of the rock. A rock may comprise constituents distributed at random in the material; alternatively it may be composed of grains which show preferred orientation of either external form or of crystallographic axes, alternatively it may be composed of bands or layers with differing grain size or mineral composition. The three alternatives have been termed 'random', 'oriented' (e.g. slates) and 'segregated' (e.g. current bedded specimens, flow banded or gneissose specimens). The quality 'coherence' is noted as a quick and useful supplement to the more quantitative mechanical indices described later. A coherent rock is one that cannot be split or crumbled with the fingers. A friable material is defined as one that can be crumbled, and a fissile material as one that can be split. The terms are qualified by adjectives so that they appear as follows; "highly segregated, moderately fissile" "isotropic, slightly friable", "Oriented, slightly segregated, highly coherent". Such terms could benefit from closer definition and the subject of quantitative observation of isotropy and coherence characteristics for engineering purposes seems to merit further study.

f) Description using test data

Tests have been suggested to supplement observations for the purpose of engineering classification and description of rock samples, since the observations themselves do not provide sufficient mechanical information. Of the tests listed at the foot of the sample description sheet (Figure 56), two, the point load and slake durability tests, are regarded as essential for 'basic' rock description. Supplementary tests may be selected from among the many available alternatives. Of these, some that are listed (the triaxial tests for example) would not normally form part of a classification testing programme, but are listed for the reasons given earlier in the chapter.

The supplementary tests require little further discussion beyond that given in Chapter 3. Tests to determine porosity and density are particularly useful; an estimate of porosity is required in the textural-mineralogical summary, and may also be regarded as 'basic' to classification. Any of the techniques outlined in Chapter 3 may be employed, and an international standard for porosity/density determination that will appear in the forthcoming year should give further guidance. Sound velocity and Schmidt rebound number determination also provide useful supplementary information although both are rather insensitive to anything less than a major change in the mechanical nature of the material. Both are quick to perform, although sound velocity tests require specimen preparation and this is only justified if regularly shaped specimens are needed for other types of test also. Further supplementary testing may well be called for; tests such as in situ sound velocity and permeability, aggregate crushing tests or attrition tests, depending on the nature of the problem and of the rock. This supplementary data would normally be tabulated on supplementary sheets to the basic rock description. More often an abbreviated rather than an extended sample description is required in which case supplementary tests could be omitted.

Records of test results should contain some reference to the technique employed, also to the number of tests performed and to the spread or scatter of results. The system of presentation that appears tobe most practical is to record for each test an average value, followed by the number of tests and the standard deviation of results enclosed in parentheses. Examples of this form of presentation are given in Figure 56. Other measures of scatter may be preferred to standard deviation. Usually the full test results together with any relevant graphs or histograms would be available to supplement the sample description sheet.

Ideally there would now follow an account of standard procedures recommended for the two basic classification tests, but since these are still in the course of development it would be premature to set out detailed specifications. Each of the two types of test is being given detailed study by a postgraduate rock mechanics student at Imperial College, and in addition, both types of test are shortly to be covered by tentative international standards. Instead, an account will be given of the present state of progress in developing these techniques and of possible lines for future research.

Figure 59 represents a tabulated summary of current ideas regarding point load test apparatus and techniques. The value of this test as a practical 'hardness' index depends greatly on the simplicity and portability of the equipment. Tests carried out in connection with the Dartmoor project described earlier used equipment that left much room for improvement. Apparatus is being developed by Engineering Laboratory Equipment Limited specifically for use in point load testing, and preliminary drawings of the first prototype, to be in service by the end of the year, are illustrated in Figure 60. Development ofthis apparatus is aimed at achieving a suitable balance between cost, simplicity of operation and reliability of results.

As well as the equipment, the techniques and result presentation both require further study. In particular it is necessary to reconcile the three variants of loading configuration required to allow testing of rock core and outcrop specimens of various possible shapes. These should produce a single point load strength index irrespective of the type of technique employed. A study of this problem amounts to an investigation of size and shape effects, and although work has been carried out along these lines it appears to be inconclusive. For the present a point load strength index given by the ratio of breaking force to the square of platen separation should be adequate, and should represent an improvement over more subjective and qualitative techniques of hardness assessment. However, further study may well improve the reproducibility of this test by, for example, the introduction of correction factors for specimen size and shape.

OBJECT

A simple strength index test applicable for all rocks without the need for specimen preparation.

APPARATUS SPECIFICATIONS

Loading
(a) Robust and small
(b) Rapid ram retraction to speed interchange of specimens
(c) Clearance to take specimens zero to four inches?
(d) Hydraulic or mechanical alternatives? Two ton capacity?
(e) Stiff machine to minimise violent failure. Protection?
(f) Loading rate controlled or arbitrary?
(g) Lateral rigidity - point load alignment must be maintained
(h) Possibility of flat or knife edge platens instead of points?

Load measurement
(a) Must hold a record of maximum force attained
(b) Small, robust & easy to operate
(c) Force from ram pressure? proving ring or beam? hydraulic load cell? electrical transducer?
(d) Lateral forces required to maintain alignment must not influence axial force measurement greatly.
(e) 1% of full scale (2 ton) is sufficient accuracy?
(f) Simple to calibrate

Platen separation measurement
(a) Accurate to 1/100 inch?
(b) Robust - fixture in apparatus for speed of observation - must not interfere with operation of apparatus
(c) Must have zero check and adjustment facility (platen interchange is envisaged)
(d) Direct reading scale or vernier?

TECHNIQUE

Three alternative specimen shapes must be accomodated - results must be mutually comparable, namely
(a) Diametral point loading on 'long' core
(b) Axial point loading on 'disc' core
(c) Point loading along shortest dimension of irregular lump

Evaluation should answer the following;
(a) Size and shape - can these be ignored - if not should limits be imposed, & what limits
(b) Can the three types of test be rendered compatible by applying predetermined correction factors or by presenting results in a particular way? (P/d^2, KP/d^2, P/d^3 etc.)
(c) Should moisture content be ignored or standardised?
(d) Should anisotropy be randomised or accounted for?
(e) How does scatter inherent in each type of test compare with that of the remaining two? with uniaxial test? with range of values for all rocks?
(f) As a result of (e) which tests are to be preferred and how many specimens, what measurement accuracy?

Figure 59 :

Point Load Strength Test - Assessment and lines for development

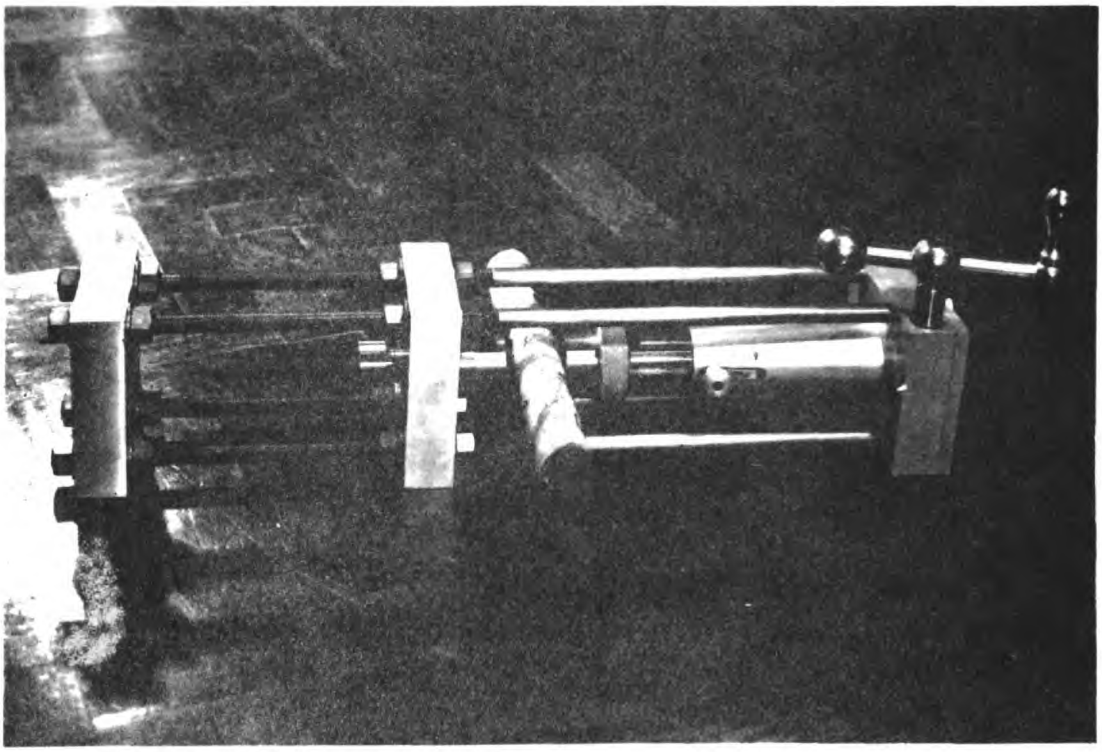


Figure 60 : Prototype Point Load Tester

The prototype, manufactured by Engineering Laboratory Equipment Ltd., is based on a mechanical ram developed for the California Bearing Ratio test. It has proved unsuitable due to insufficient lateral rigidity and a second prototype is being developed based on a shorter hydraulic ram.

OBJECT

A simple yet reliable method of predicting durability of mudrocks when exposed in excavation or used as fill etc. (Swelling tests, freeze thaw tests and salt crystallisation tests could be used to supplement this simple index).

Outline of possible alternative procedures

1. Nature of sample tested
   - (a) Cut prisms
   - (b) Irregular lumps of specified grading limits

2. Pre-treatment
   - (a) Natural moisture content
   - (b) Oven dry
   - (c) Dry in dessicating (etc.) atmosphere

3. Accelerated weathering
   - (a) Atmospheric exposure
   - (b) Humid-dry cycles (+ temperature cycles?)
   - (c) Saturation - air (or oven) dry cycles
   - (d) Immersion in distilled water, tap water, sodium hexometaphosphate solution, sodium oxalate solution, ethelene glycol (antifreeze) solution.

4. Attrition during accelerated weathering
   - (a) Avoid disturbing rock
   - (b) Controlled stirring or rolling at intervals
   - (c) Controlled stirring or rolling throughout slaking

5. Assessing the effect
   - (a) Visual observation + descriptive scale
   - (b) Full sieve analysis to determine size distribution before and after slaking
   - (c) Single sieve to indicate size degradation

A POSSIBLE METHOD

Sample 500-6000 gm of irregular lumps each 50-100 gm. Oven dry in standard mesh (7BS,8ASTM) cylindrical container (½ full)
Weigh

Cap container, mount axis horizontal, ½ immersed in tray with 2% sodium hexometaphosphate solution
Rotate 30 rpm for 1 hour (particles drop from mesh)
Remove cap, oven dry, weigh

$$\text{Slake durability index} = \frac{\text{Final weight of rock}}{\text{Initial weight of rock}} \%$$

Figure 61 :
Slake Durability Test - Assessment and lines for development

Possible alternative techniques for a slake durability test are also under investigation. The scope of these investigations is illustrated in Figure 61. Those tests carried out as part of the Edinburgh project were crude but simple and produced useful results. Their reproducibility might, however, be greatly improved by an increased measure of standardisation both of the form of pretreatment of the tested sample, of the nature of the accelerated weathering process itself and in the method of assessing the effects of this process. In neither the slaking nor the point load test is there any limit to the increase in reproducibility that may be brought about by increased control and standardisation. Since, however, inconvenience is caused by each measure of control introduced, there is a practical limit to the amount of standardisation desirable. For the slake test this is particularly evident when considering the control to be applied to specimen size and shape. Regularly cut specimens would, without question, give more reproducible results but would be impractical. Some control on the size and quantity of lumps in the sample tested appears, however, to be desirable to reduce the scatter of results with the minimum of practical inconveneience.

g) Concluding remarks

Part 2 of this thesis will be concerned with a detailed and rather specialised study of the triaxial compressive strength test, in contrast to the preceding chapters which have a more general application in engineering practice. Before leaving the subject of tests, observations and classifications suitable for rock mapping it may help to summarise some of the main points that have emerged.

It was suggested that geological rock names and descriptions are often inadequate for use in an engineering context. Neither are the more complicated and expensive engineering tests well suited for rock classification, nor are engineers often aware of the potential value of simple textural and mineralogical observations for predicting rock performance. It should be possible to combine simple tests and observations into a system for rapid and mechanically meaningful rock description, suitable for both use and interpretation by the engineer. Such a system would form a basis for rock mapping, and therefore must be simple enough to allow description of hundreds rather than tens of samples at an engineering site.

Rock mapping based on classification tests and observations should if possible precede any testing of a more complex and expensive nature that may be required, so that these tests may be selected and sited for optimum results.

Rock description, including any tests and observations that may be required, should be conducted in the field rather than in a laboratory remote from the site, and forms a logical extension of conventional core logging techniques. A single universal system would be preferable to rock classification systems purpose-made to suit each problem or rock type encountered, although some adaptation might be necessary to meet individual requirements. Careful selection of tests and observations is essential in view of the large number of alternatives and the limited amount of testing that practical considerations will allow. Criteria on which to base such a selection were proposed. However carefully observations are selected it is generally true that the fewer the observations, the less the depth of characterisation, and two-dimensional classifications are particularly suspect from this point of view.

Properties selected for 'basic' rock description, since they appeared to be relevant to the widest range of rocks and of engineering problems were *brokenness*, *hardness* and *durability*. These properties were defined and discussed together with techniques suggested for their quantification. A two -dimensional classification based on brokenness and hardness was proposed. Alternatively the properties could be used as descriptors added to a simple geological name. This name should then only be used as a convenient label and the descriptors, not the name, should be used to infer the mechanical nature of the material.

Additional 'supplementary' observations and tests were suggested to add greater depth to the rock description. In particular, a simplified form of mineralogical and textural description was proposed, selected to emphasise mechanical rather than genetic characteristics. Pores, both porosity and pore size, should receive particular attention on account of their mechanical importance.

Further study might be directed towards improving techniques and equipment for basic classification testing and observation. These techniques should then be applied to a representative selection of natural rocks in an attempt to explore and to subdivide into classes the natural distributions of properties. A major topic of research will be to relate the resulting classification to field performance of the materials.

PART 2 TRIAXIAL STRENGTH OF ROCK

5. DEVELOPMENT OF TRIAXIAL TESTING TECHNIQUES

a) Triaxial strength testing

One of the most important characteristics of rock, from the point of view of the design engineer, is the significant increase in strength with increasing confining pressure. An adequate description of this behaviour requires a number of triaxial tests over a range of confining pressures. The conventional triaxial test applies a *radially-symmetric* stress to the circumference of a cylindrical specimen, and hence provides only limited information as to the strength characteristics of a rock material. The interpretation of triaxial test results and the classification of rock materials according to their strength characteristics are to be discussed in Chapter 6. The present chapter is restricted to an account of the development of testing apparatus and techniques.

The number of tests required for a given sample of rock may be large. Twenty or more tests may be necessary for satisfactory accuracy of strength prediction, depending upon the homogeneity of the sample, hence the scatter of the data. Conventional triaxial testing apparatus is inclined to be cumbersome and slow to operate, and a simpler design of triaxial cell was developed in order to speed up the testing procedure. (Hoek and Franklin, 1968). This was particularly desirable since a large amount of testing was envisaged in connection with the fitting of strength curves for rock classification. A simpler cell was also required to allow routine testing by engineering and geology students as part of their course work, and for the routine testing of rock for practical design of rock structures.

Results are tabulated at the end of this chapter for eight rock samples, comprising about 250 specimens. These triaxial tests had the dual role of evaluating the apparatus and techniques, while at the same time providing data for the study discussed in Chapter 6.

b) The triaxial cell

The cell is illustrated in Figure 62. The version used for the current series of tests was designed to accept 1½ inch diameter specimens with a length: diameter ratio of 2:1, and to apply confining pressures of up to $69 MN/m^2$ ($10,000\ lbf/in^2$). The pressure capability was chosen as approximately equivalent to the vertical stress under 10,000 feet of overburden, the maximum pressure likely to be encountered in engineering practice. Cells suitable for testing a wider range of core sizes are now being manufactured by Engineering Laboratory Equipment Limited in the U.K., and by Terrametrix Inc. in the United States.

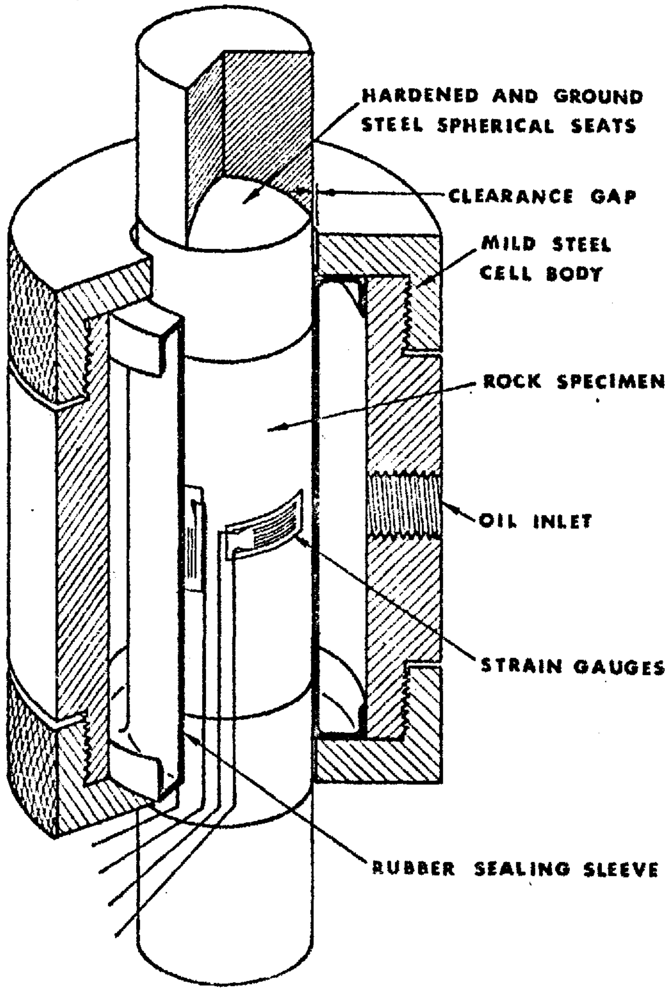


Figure 62 : Cutaway view of triaxial cell.

The specimen is shown mounted with electric resistance strain gauges, but strain measurement is the exception rather than the rule in routine triaxial testing.

The cell body is machined from mild or alloy steel and comprises a cylinder onto which screw two end caps. The cell weighs approximately 10 lb. The main feature of the design is the one-piece synthetic rubber sleeve that fulfils the double function of preventing leakage of fluid from the cell, and of preventing ingress of fluid into the specimen. A single sleeve proves sufficiently strong to withstand the testing of over 100 specimens. The cell is not dismantled between tests, and the sleeve retains an annulus of fluid while the specimen is inserted, tested and then extruded.

The specimen is compressed axially using a conventional compression testing machine, the axial load being transferred to the specimen via two spherically seated platens that minimise bending stressed. Cell pressure is provided from a hydraulic pump connected to an oil inlet in the cell wall. A further oil inlet, not shown in Figure 62, is used to provide a tapping for cell pressure measurement. Quick release self-sealing couplings are used at both hydraulic connections.

Rubber sleeves for the prototype cells were cast at Imperial College from liquid Adiprene L100, a polyurethane rubber, using a technique described fully in the previously cited publication. Sleeve extrusion pressure was tested as a function of clearance between the specimen and the sleeve bore. The maximum working pressure could be held without extrusion with a radial clearance of up to 0.025 inches, so that the cell could operate with a conveniently wide range of tolerance on specimen diameter.

The speed and convenience of testing was evident since up to eight specimens could be tested by students in the course of a half hour hour laboratory session. The portability of the cell should allow routine testing in the field, although its use has so far been confined to the laboratory. Field testing of rock specimens is much to be preferred to laboratory testing since the deterioration of specimens is minimised and a close correlation between testing and in situ geological observations may be maintained.

The design facilitates deformation measurement under conditions of triaxial stress, since leads from electric resistance strain gauges may pass between specimen and jacket. Most conventional cells require that such leads be passed through cell wall and confining fluid, with consequent problems of experimentation. Proving trials were carried out on Bunter Sandstone specimens and typical results are illustrated in Figure 63. The majority of tests have, however, been for the purpose of strength determination and have not been instrumented for strain measurement.

A modification of the design has been incorporated in a later version of the cell which has not yet been fully tested. In this case the cell is fabricated in two halves, both of which are self-sealing. The cell may be opened like a book, so that specimens may be removed without extrusion after testing. Instrumentation of the specimen should also be simpler using this design of cell. Extrusion of specimens can occasionally lead to damage of the sleeve, particularly when specimens are highly deformed, but this did not prove to be a significant problem during the current testing programme. The 'split cell' is illustrated in Figure 64(a). Figure 64(b) illustrates a particularly versatile biaxial loading machine developed for model testing by I. Ergun of the Mining Department at Imperial College. This machine also employs polyurethane rubber sleeves for pressure application, and demonstrates the versatility of this material.

Robertson Research Company Limited report use of the standard cell for creep testing at cell pressures of up to 4000 lbf/in^2 for periods of up to four months, during which time the cell continued to seal and to "perform satisfactorily". The cell also shows potential for use in permeability testing. One of very similar design has been used at British Petroleum laboratories, acting to seal the cylindrical surface of a specimen while water is forced through in the axial direction. The present design of cell seems well suited to this application.

c) Hydraulics

Two independent sources of hydraulic pressure are required, one to operate the hydraulic ram supplying axial force to the specimen, the other providing cell or 'confining' pressure. The axial force could be provided by a mechanically driven ram, but a loading capacity of at least 100 tons is required and mechanical rams in this range are unweildy.

A proprietary compression testing machine may be used to provide axial loading, but in the present test series axial load was applied using a simple four column reaction frame and a 100 ton

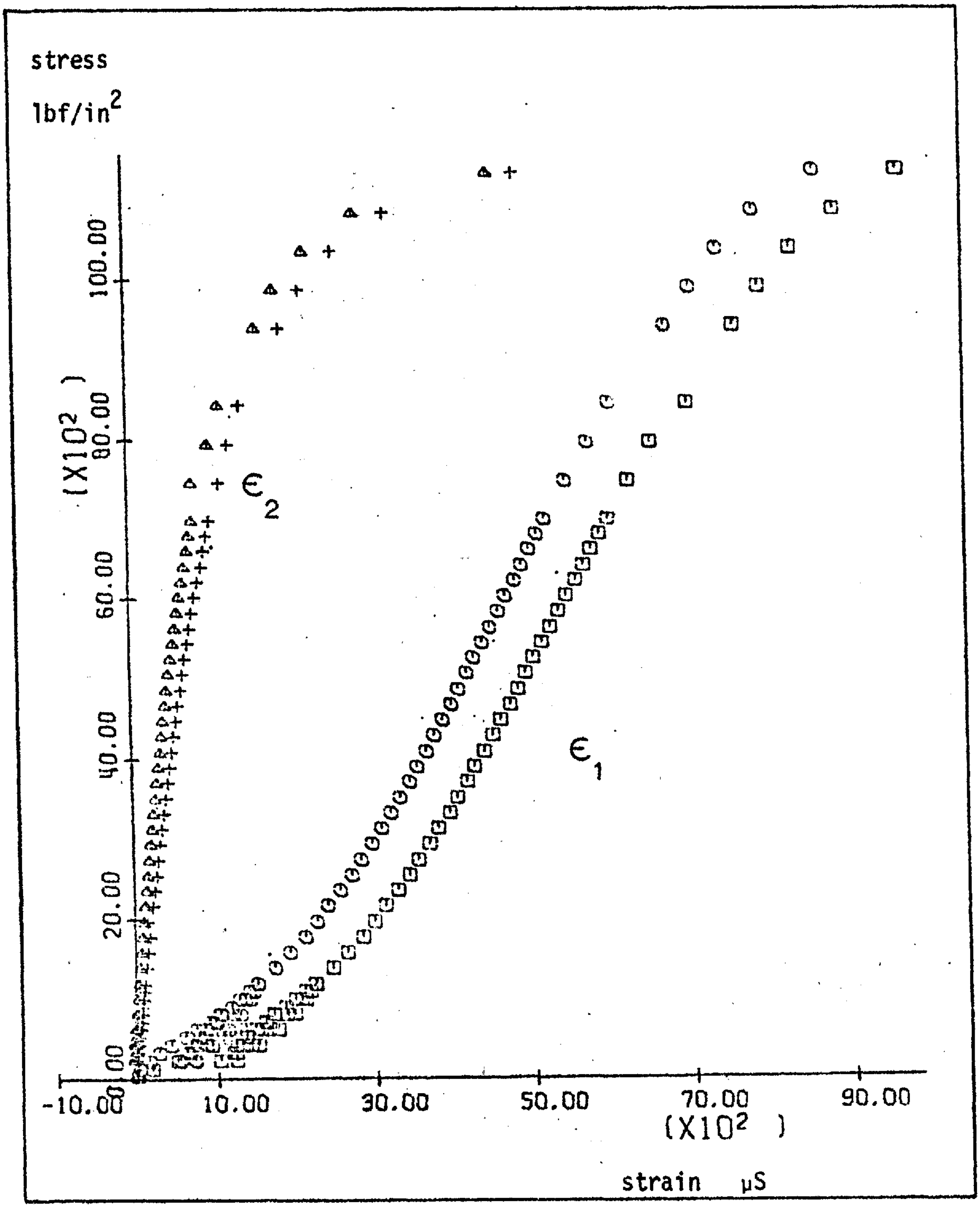


Figure 63 : Stress-strain curves

Computer produced graph of axial strain and lateral strain in a specimen of Bunter sandstone. Two electric resistance strain gauges were used in each case and the results show satisfactory freedom from bending stresses. The specimen was tested at 1000 lbf/in^2 in the triaxial cell.

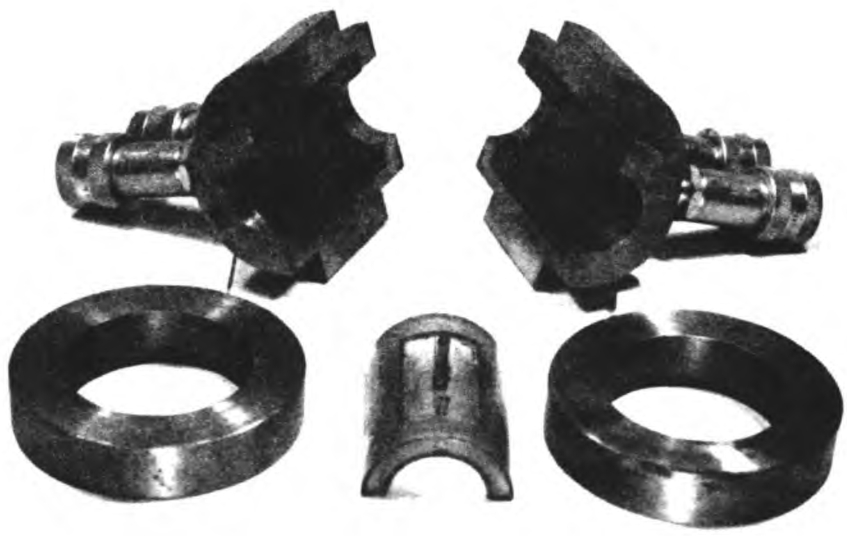
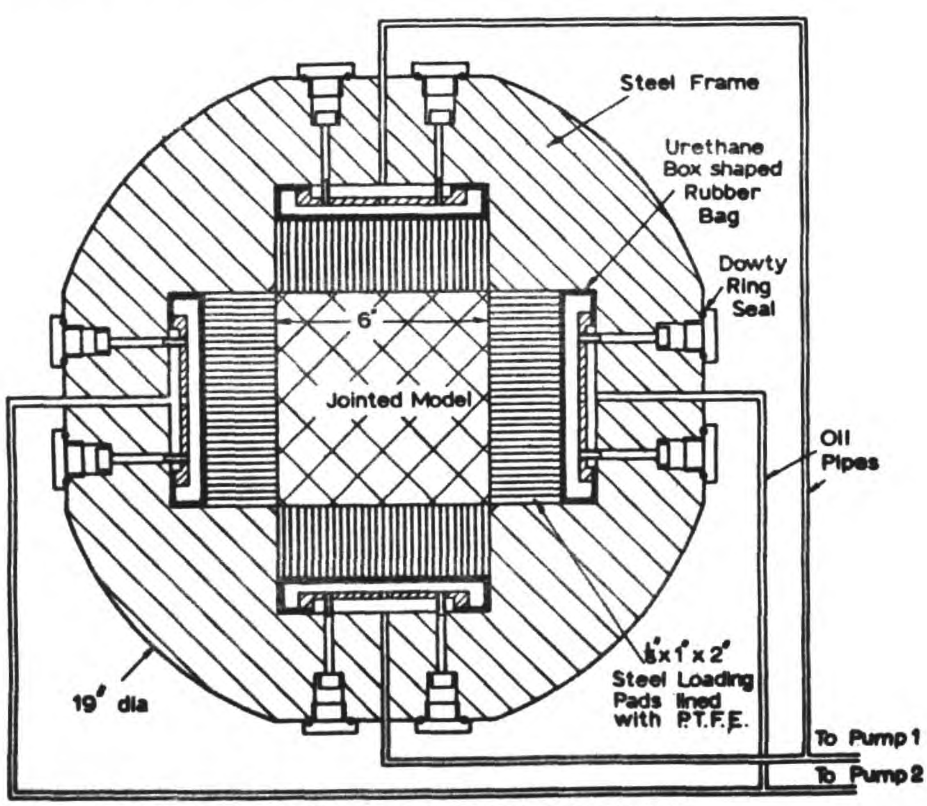


Figure 64a : 'split' version of triaxial cell
The polyurethane rubber membrane is shown in the centre foreground.

Figure 64b : Ergun's biaxial loading frame
This machine also uses polyurethane rubber membranes that allow displacements of up to ½ inch in the model



SECTION OF BIAXIAL COMPRESSION RIG TYPE 1.

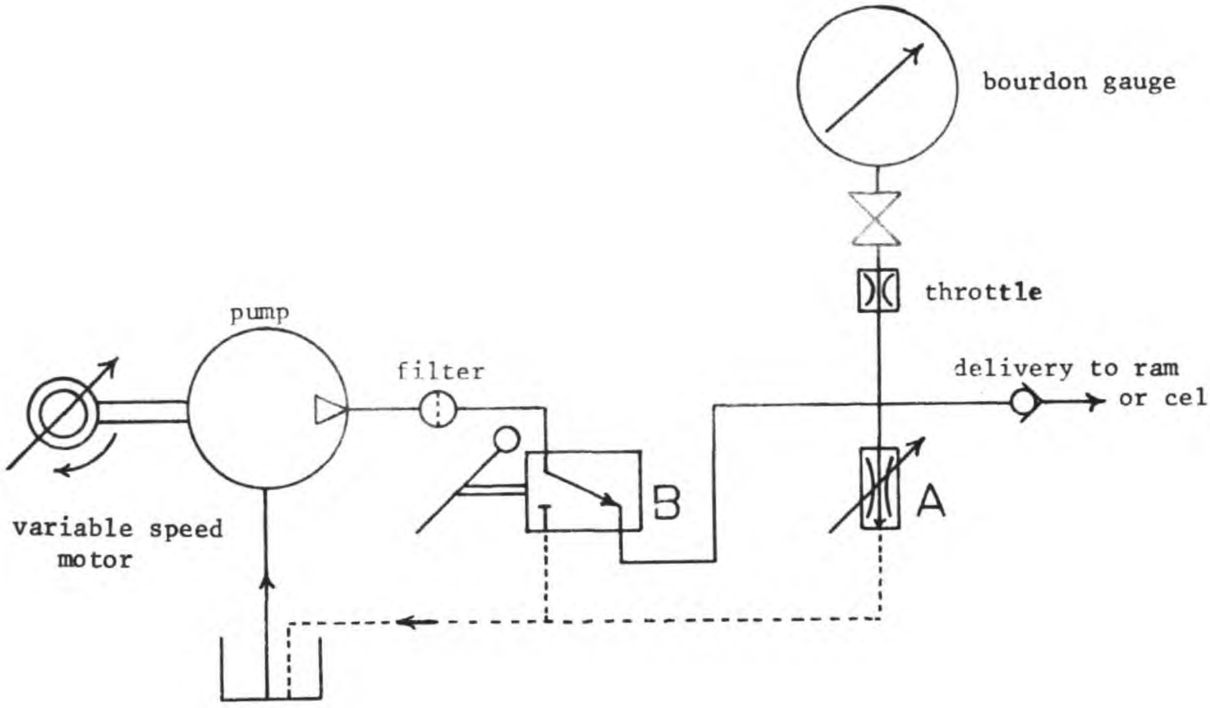
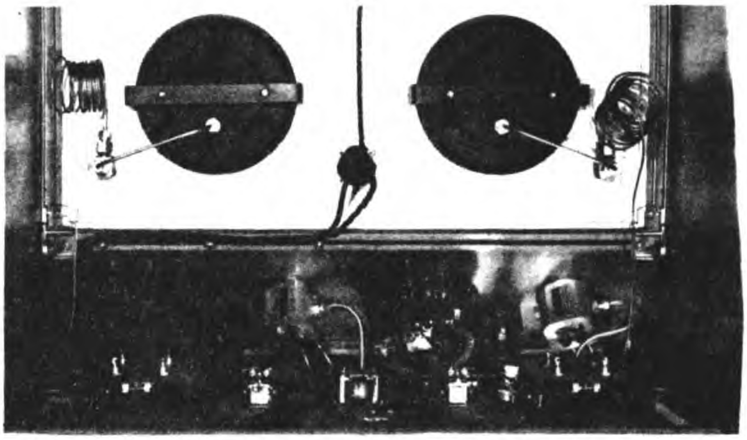
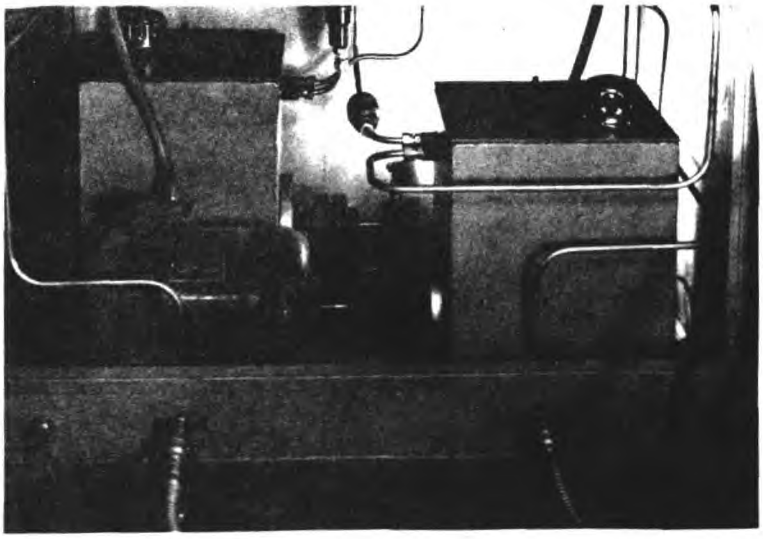


Figure 65 : Hydraulic pressure circuit & console





hydraulic ram. Both ram pressure and cell pressure were supplied from identical motorised pump units, with hydraulic circuits designed on the 'bleed off' principal (Figure 65). An Oswalds & Ridgeway swash plate axial piston pump generating 10000 lbf/in^2 was driven by a variable speed d.c. motor. Pressure could be controlled either by adjusting the motor speed, or by adjusting the needle valve 'A' to 'bleed off' or by-pass part of the delivery volume. A quick release valve 'B' allowed the full delivery to be discharged back to the pump reservoir in case of emergency.

As a result of experience in the present testing programme, the needle valve is to be replaced by an 'Amsler' spring loaded pressure release valve. This type of valve has been shown to give a much superior pressure control to that provided by a needle valve, and has been used extensively in the Soil Mechanics and the Concrete Materials laboratories at Imperial College. The Amsler valve employs a spring loaded piston for pressure control, the hydraulic force on the piston being balanced by tension in the spring. The piston is rotated back and forth to reduce friction, and pressure may be held constant to within 1% irrespective of changes in oil temperature and other disturbing factors. Whereas the needle valve requires many turns at low pressures but only slight adjustment at higher pressures in order to achieve the same pressure change, the Amsler valve exhibits a linear response between number of turns and pressure increase throughout its range. This characteristic greatly improves the ease with which pressure may be adjusted. It is still necessary to include a variable speed pump in the circuit so that flow rate may be adjusted to a suitable level, and to ensure that the frequency of pump pulsations is not such as to cause resonant oscillations of the valve.

Either ram or cell pressure (or both) may be provided using a simple hand pump. The hand pump is inconvenient for ram pressure application due to the large oil volume required during the 'approach stroke' of the ram. It is not to be recommended for cell pressure application on account of the pressure surge experienced during failure of a specimen. Most specimens increase their volume suddenly during failure, forcing pressurising fluid back into the hydraulic circuit. If, as is the case with a hand pump, the hydraulic circuit is a dead-end, the pressure may rise by as much as one or two thousand lbf/in^2 during the process of failure. This will lead to difficulty in interpretation of experimental results and might also endanger the safety of the experiment. Handpumps have additional disadvantages to those mentioned, but their greater simplicity may lead to their use in some triaxial testing applications.

d) Instrumentation

Instrumentation to measure both axial force and cell pressure is required. Either may be measured mechanically using Bourdon tube pressure gauges. Hydraulic pressure tends to straighten the curved dead-end metal tube that forms the sensing element of this type of gauge, the movement being transferred to a rotating pointer by means of a system of levers and gears. Cell pressure may be measured directly but axial force must be obtained from the relationship between ram thrust, ram fluid pressure and ram cross sectional area. Part of the thrust (given as the product of ram pressure and area) is absorbed as frictional resistance in the ram, and to obtain reproducible force-pressure calibration it is advisable to use a ram with low friction (p.t.f.e.) seals. If a conventional compression testing machine is used to supply axial force to the specimen, then the Bourdon gauge that is usually incorporated in this type of machine may be used to measure the force. This gauge is usually calibrated directly in terms of force rather than pressure, but should be re-calibrated periodically to account for possible changes in ram friction.

It is difficult to record maximum applied force using a Bourdon gauge unless the gauge incorporates a maximum indicating neede. This needle is carried around the dial by the pressure pointer and should neither impede the progress of the pointer, nor be so free running as to be carried round the dial by its own inertia.

Electrical (transducer) measurement of force and pressure allows automatic recording during the test, and was employed for the current series of tests. Electrical measurement is to be preferred provided that experimental conditions allow adequate calibration and operation of the instruments. Bourdon gauges are simpler and should be used where the expertise necessary for maintenance and operation of electrical instruments is not available.

Pressure transducers usually employ electric resistance strain gauges mounted on the outside of the dead end tube containing the fluid whose pressure is to be measured. As pressure increases the tube expands, disturbing the balance of the resistance bridge. The bridge is excited by an excitation voltage (usually d.c) and the out of balance output voltage is proportional to the pressure to be measured. Force transducers (load cells) usually also operate on the resistance strain gauge bridge principal, the resistance gauges being mounted on a steel column that carries the force to be measured.

In the current test series the outputs from force and pressure transducers were supplied to the X and Y axes of an X-Y pen recorder. Hence the 'stress path' of a specimen could be followed during the course of a test (Figure 66). This was helpful in allowing the reliable observations of the stress conditions at the instant of failure, particularly with 'quiet' or ductile specimens where failure might otherwise pass unnoticed. Failure is usually accompanied by a slight drop in axial force and by a surge in cell pressure, even when testing relatively ductile materials. Stress paths for the twenty or so specimens that comprise a sample for strength testing may be plotted on the same sheet, allowing stress paths for subsequent specimens to be selected to fill in missing points on the strength curve.

Calibration of force and pressure measuring systems was carried out prior to each testing session (Figure 67). It is most important that reliable methods of calibration are available and easy to operate, allowing calibration to be repeated at regular intervals, particularly with electrical instrumentation where 'drift' can change the calibration during the course of a series of

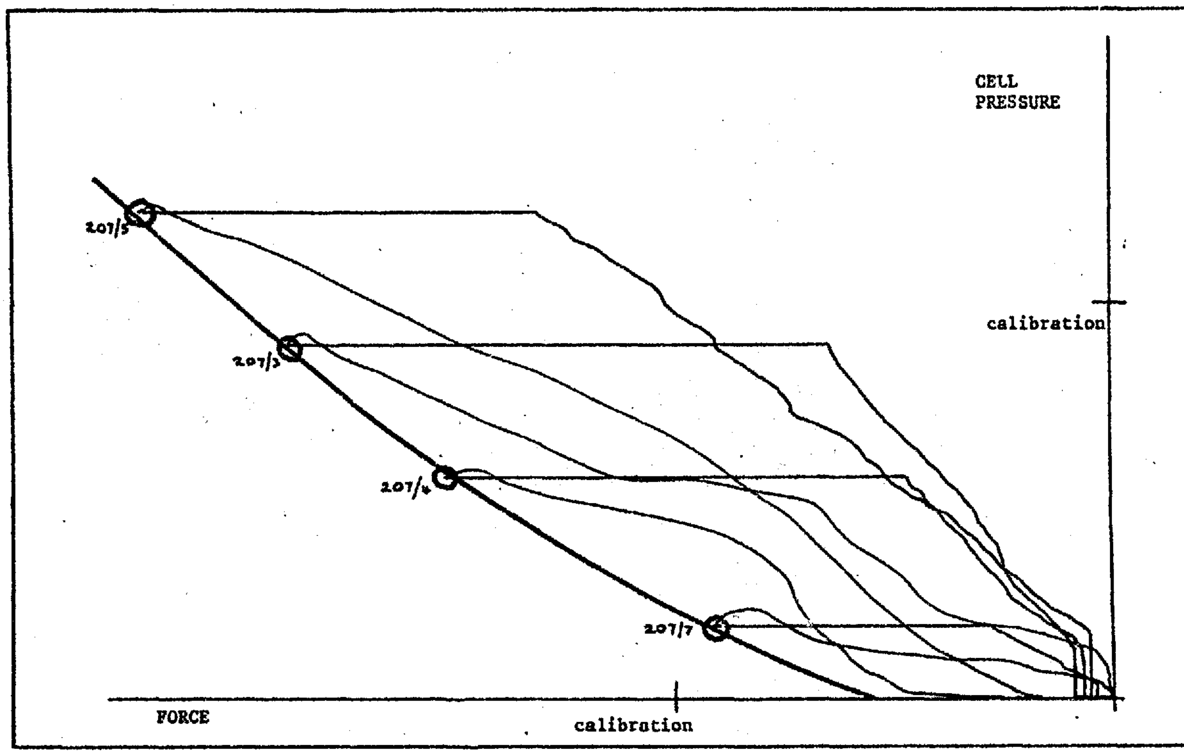


Figure 66 : X-Y plot of stress paths.

A series of specimens that comprise a rock sample are tested and the axial force-lateral stress paths for all specimens are recorded automatically during testing. The strength curve for the sample may then be constructed on the X-Y plot.

experiments. Force calibration was achieved using a proving ring installed between the platens of the loading machine. A known force increment was marked as a distance on the X-Y plot, eliminating errors that might arise if transducer and plotter sensitivity were calibrated independently. Similarly pressure was equated with plot distance by connecting the pressure transducer to a Budenberg dead-weight pressure calibrator. The pressure calibrator was provided with a quick release coupling to receive the pressure transducer.

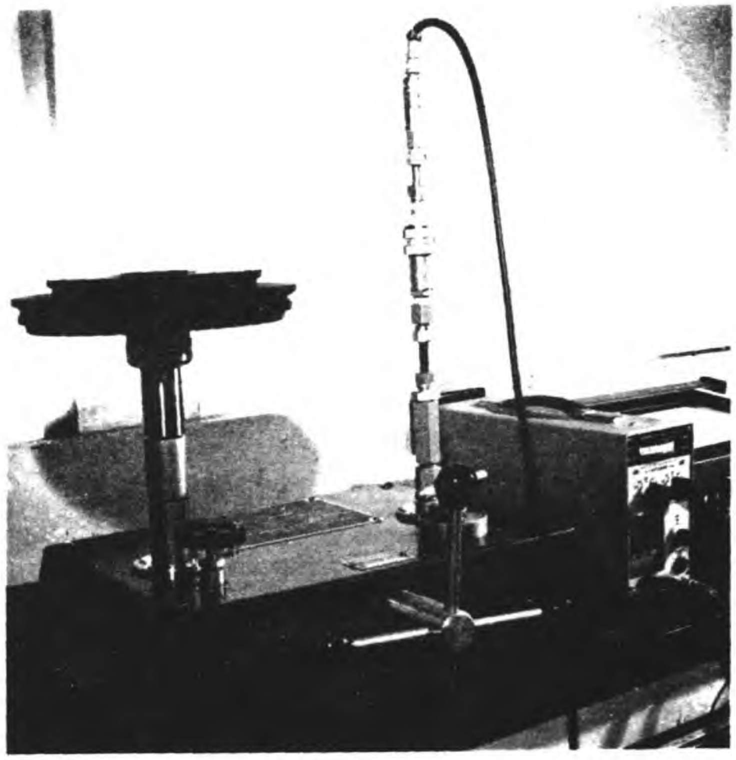
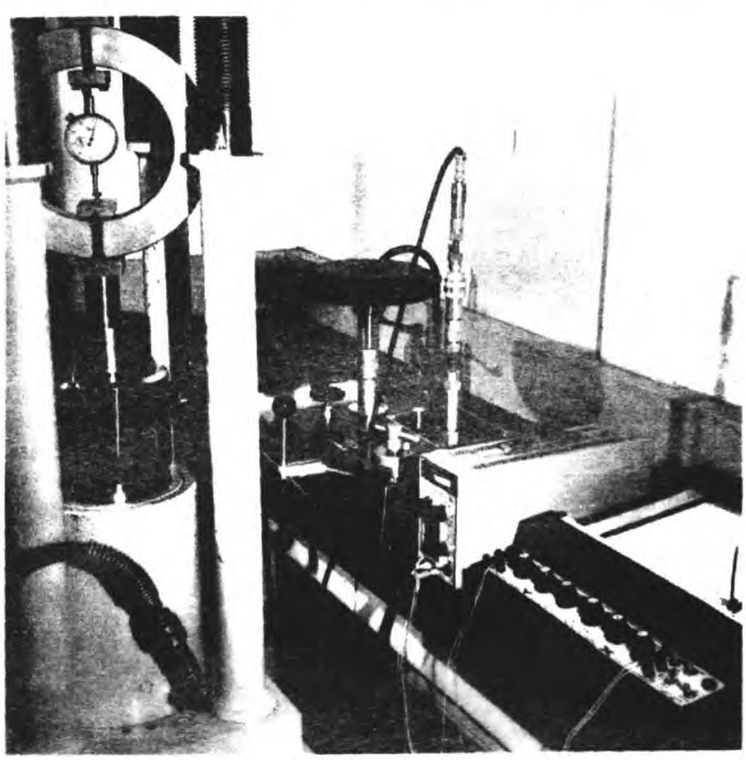


Figure 67 : Equipment for pressure and force calibration

Above - Budenberg dead weight pressure calibrator with pressure transducer plugged into quick release coupling prior to calibration.

Below - Proving ring installed between spherical seats in the loading frame, prior to force calibration. The X-Y plotter and stabilized supply for recording force and pressure are visible to the right of the picture.



e) Specimen preparation and storage

Rock specimens 1½ inches in diameter were diamond drilled from blocks of quarry rock. If the block is rigidly clamped and the drilling machine is robust, the quality of cylindrical geometry achieved is excellent and there is no need for additional cylindrical grinding. The end-facing of specimens is critical since specimen ends must be accurately flat to avoid non uniformity of stress. The end faces need not, however, be perfectly parallel since the use of spherical seats is sufficient to eliminate bending stresses. Nor need the ends be polished; lapping is not to be recommended as a method of end preparation since although a high degree of polish may be achieved, the degree of flatness is usually inadequate.

Ends of specimens were faced by mounting the cored stick in a lathe and by sawing with a diamond impregnated cut off saw. Both specimen and saw blade rotate during the cutting operation. The cut was fed with a continuous supply of cooling water. The cut faces were usually of adequate flatness, but where 'nipples' of rock remained after cutting, the face was dressed with a diamond impregnated cup wheel grinder. The length of specimens produced was 3 inches, with a tolerance of ± ⅛ inch (within that allowed by the design of triaxial cell).

Specimens were washed and labelled with a felt pen after preparation, and were stored in stackable nylon coated wire trays prior to testing (Figure 68). This form of storage minimised specimen handling and consequent damage, and the trays allowed free access of air during drying. Saturation, oven drying and also the measurement of saturated submerged weight could be carried out without removing specimens from the trays.

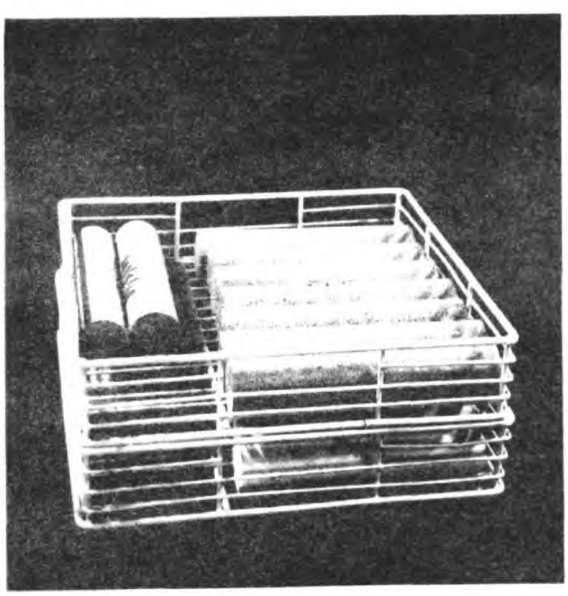


Figure 68 : Specimen storage.

Specimens stored in stackable nylon-coated wire trays to minimise damage caused by handling.

Specimens were allowed to reach air-dry equilibrium prior to testing. It is anticipated that for future test series the specimens would be brought to moisture content equilibrium in an environment of standard relative humidity. Index tests carried out prior to triaxial testing included the measurement of porosity and bulk density using the 'wet' method described in Chapter 3, and also the measurement of sound velocity in the air dry state. Specimen length was required for calculation of sound velocity, and specimen diameter for the calculation of strength. These dimensions were obtained using dial gauge comparators that allowed superior speed and accuracy of measurement than when vernier calipers are employed for this purpose. Observational parameters characteristic of each sample were measured using binocular and thin-section microscopy (Chapter 2) and the textural peculiarities of individual specimens were also noted.

f) Triaxial testing procedure

PRIOR TO TESTING

Cell assembly

Cell components are cleaned, one end cap screwed home and the sleeve inserted. The second end cap is then screwed home. End caps should be screwed fully home but only finger-tight, since they may otherwise be difficult to remove after pressure application (Fig.69).

Filling and bleeding the cell

The hose from the pressure system is connected to one quick release coupling, and a spare male connector to the other 'bleed' connection. The cell is held with the bleed connection uppermost and filled with oil (Figure 70). Both couplings are disconnected without allowing the entry of air. Small quantities of air are likely to enter the cell during the course of testing, and the cell should be re-bled if more than quarter full of air, since highly compressed air is a considerable experimental hazard. The oil level may be observed through the translucent sleeve if one end cap is removed. It should not be necessary to bleed more frequently than once every twenty tests, and if excessive leakage or air entry is experienced, the sleeve should bechecked for damage and cleaned to ensure that grit has not impaired the quality of hydraulic seal.

Axial alignment of the loading machine

The loading machine platens incorporate recessed bearing plates to receive the spherical seats. The alignment of these plates is adjustable and is checked prior to each test series using an accurately machined steel specimen.

Instrument calibration

The efficient functioning and accurate calibration of both force and pressure measuring instrumentation is checked prior to each test series. Force and pressure are cycled through the full range about twenty times and the instruments allowed to fully warm up prior to calibration, to minimise errors due to hysteresis and drift. Measuring systems should occasionally be checked for linearity of calibration.

TESTING

Setting up the axial loading system

The lower spherical seat is placed in its locating recess and the cell is lowered over the seat until it rests on the lower bearing plate. The specimen is taken from its storage tray, its number noted and is inserted into the cell. The upper spherical seat is placed in position on top of the specimen. Convex halves of the spherical seats should abut the specimen. The ram is extended until the top seat locates in its recess, and a *small* retaining force is applied. The alignment of the specimen with top and bottom seats is checked, the retaining force being removed to adjust alignment if this proves necessary.

Setting up the cell

The pressure hose and transducer are connected to the cell, a small axial force preventing loss of specimen alignment (Figure 71). The cell is raised to its operating position ensuring that spherical seats protrude equally top and bottom. A small pressure is applied to clamp the cell in position (Figure 72).

Testing

Axial force and cell pressure are increased from their initial bedding-in values until the specimen fails. The X-Y plot of force-pressure is observed during load application to ensure that the apparatus is functioning correctly and to detect the instant of failure, if this is inaudible. If a cell pressure is applied with zero axial force, either before or after testing, there is a danger that the sleeve will be intruded between specimen and platens and will be damaged. After failure axial force and cell pressure are removed simultaneously. The loading should be removed soon after failure since grossly deformed specimens are difficult to remove from the cell.

Extrusion

With cell pressure and axial force at zero, the pressure hose and transducer are uncoupled from the cell. The cell is placed on a bench and one end cap removed. The threaded portion of the cell is placed in the locating recess of the extruder and the specimen slowly extruded from the cell (Figure 73). The cell is replaced on the bench, wiped clean of grit (Figure 74) and the oil level checked. The cap is replaced and the cell is now ready for further testing. In rare cases where the specimen has been over-deformed and will not readily extrude, the cell must be drained of oil and the sleeve removed. The specimen is then broken up inside the sleeve, the sleeve washed, checked for damage, and replaced.

Figure 69

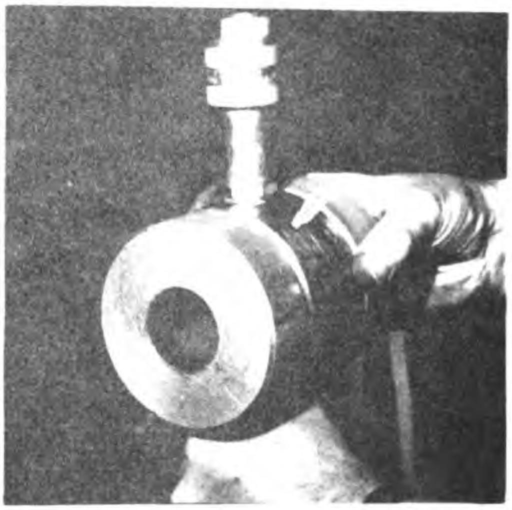


Figure 70

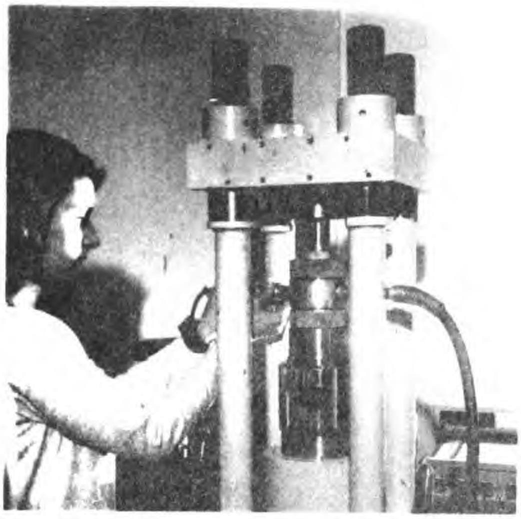


Figure 71

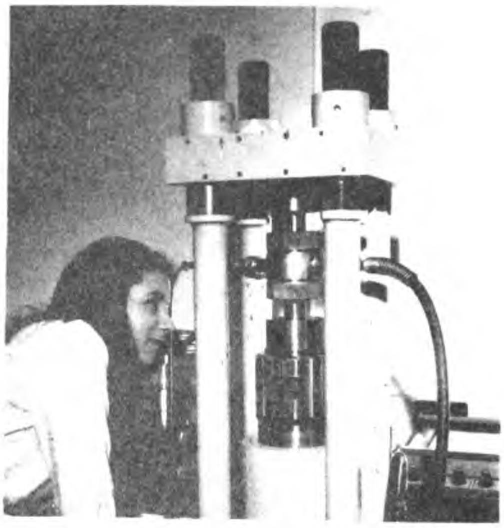


Figure 72

Figure 73

Figure 74

Triaxial testing procedure

g) Summary of results

Samples tested were, with the exception of sample 208, Carrara marble, typical and widely used British quarry rocks. Sample 204, quartz dolerite or 'Whinstone', is crushed for use as concrete aggregate or roadstone. The remaining rock types are more commonly used as building stone or for monumental or decorative purposes. The rocks are not typical of those encountered in engineering practice since their use as aggregate or for building dictates an unusual uniformity of lithology. Most commonly encountered rocks cannot be used as building stone since their properties are too variable.

Triaxial strength results are tabulated in units of MN/m^2. These results are analysed in the following chapter in order to derive triaxial strength parameters for each sample that characterise the shape of its triaxial strength curve. In the tabulation σ_2 refers to the cell pressure and σ_1 to the axial stress on the specimen at the moment of failure. Samples that were tested are referred to using the following sample numbers;

Sample Number	Rock Name
98	Granite, Blackingstone quarry, Devon
200	Limestone, 'Portland Stone' block 1
201	Limestone, 'Portland Stone' block 2
202	Sandstone, 'Millstone Grit', Derbyshire
204	Quartz Dolerite, 'Whinstone', Northumberland.
207	Sandstone, Darley Dale, Derbyshire.
208	Marble, Carrara, Italy.
209	Sandstone, Pennant, Swansea Wales.

Sample 98

σ_3	σ_1
9.341	309.26
0.0	171.45
43.515	540.98
38.082	543.72
29.375	485.83
21.730	415.95
43.232	539.85
6.460	273.25
0.0	179.30
22.916	409.92
12.816	269.90
33.043	440.06
55.841	569.45
0.0	135.87
0.0	111.35
27.135	392.80
49.340	488.43
16.387	340.43
2.77	249.00
5.02	293.10
1.59	234.30
13.21	359.02
17.96	410.69
8.72	318.48
0.0	197.3
7.8	276.8
19.7	406.5
28.5	486.8
0.0	195.9
9.9	312.1
21.4	407.6
33.9	453.9
0.0	201.5
7.8	283.0
28.8	458.0
45.5	566.0
10.7	316.0
0.0	213.6
25.8	431.1
39.2	480.8
0.0	196.1
7.4	284.4
34.8	512.7
51.7	523.9
0.0	193.4
15.1	330.1
5.3	270.5
17.7	362.3

Sample 200

σ_3	σ_1
39.21	211.99
9.72	126.19
32.39	182.07
0.0	42.30
14.04	115.44
43.69	249.25
31.73	194.41
19.52	165.28
39.54	270.24
4.98	118.85
13.46	155.05
20.02	201.76
5.65	116.75
30.49	231.93
1.58	98.56
35.64	216.18
15.95	192.57
30.90	207.27
28.07	194.15
2.65	115.83
17.11	156.11
26.33	186.85
29.07	192.90
46.60	234.16
22.09	171.32
41.78	207.27
0.0	76.18
38.54	220.39
9.13	126.19
33.47	206.77

Sample 201

σ_3	σ_1
35.72	195.27
18.52	114.12
9.55	118.06
23.25	143.77
20.68	143.77
13.37	119.90
16.61	127.53
8.89	82.38
30.73	146.76
5.81	81.47
33.47	141.93
34.80	151.91
11.71	99.87
15.78	102.06
25.34	132.87
14.12	109.28
36.55	138.76
16.36	107.57
4.82	58.18
30.73	144.04
20.19	98.91
6.48	76.35
10.63	99.17
12.79	70.57
7.47	86.84
20.02	97.34
10.38	94.19
3.16	49.93
7.39	72.15
15.12	89.46
5.07	59.56
11.21	95.76
2.91	69.91

Sample 202

σ_3	σ_1
14.47	153.16
25.00	204.48
31.05	220.15
42.11	249.13
19.47	178.23
5.00	104.98
8.16	125.35
11.45	144.94
0.0	55.62
0.0	58.76
49.34	285.56
38.29	236.60
51.71	263.33
0.0	54.06
2.89	82.26
0.0	37.23
0.0	45.50
0.0	41.09
34.48	203.72
0.0	51.36
24.13	185.45
0.0	67.7
3.45	79.07
11.03	127.0
0.0	58.6
0.0	57.72
46.88	260.94
0.0	51.64
6.89	106.73
31.71	217.42
0.0	49.81
0.0	55.36
28.26	253.01

Sample 204

σ_3	σ_1
5.02	333.75
1.32	328.30
34.74	498.91
42.93	514.76
0.0	331.78
2.38	340.97
0.0	315.04
0.0	315.65
20.21	410.85
13.74	380.65
7.40	344.59
36.98	512.71
23.91	453.60
28.27	474.63
0.0	305.14
0.0	210.68
3.45	284.72
0.0	214.4
0.0	311.39
31.02	496.37
10.34	341.94
0.0	275.76
0.0	267.49
13.93	364.0
0.0	314.37
17.24	422.60
34.47	497.75
0.0	312.02
0.0	299.74
0.0	273.69
6.89	390.45
42.05	552.15
0.0	272.86
21.72	461.21
27.58	489.05
0.0	278.93
20.68	457.07
44.12	561.17

Sample 207

σ_3	σ_1
22.5	201.9
41.2	287.3
0.0	74.8
6.9	120.9
21.5	199.2
42.5	290.2
0.0	80.3
9.8	136.6
3.2	94.8
46.9	298.9
0.0	79.7
4.4	117.3
35.5	264.3
50.2	319.0
0.0	83.2
21.7	210.4
30.2	240.8
52.8	315.5
0.0	82.4
7.8	136.0
24.4	228.9
44.8	301.7
14.07	169.2
15.64	177.2
14.27	183.52
20.34	210.25
28.53	243.20

Sample 208

σ_3	σ_1
30.86	205.92
16.15	156.44
39.13	234.41
10.51	131.1
35.19	217.17
25.21	188.14
3.94	119.09
2.23	111.59
21.84	179.16
47.50	263.1
51.74	262.2
0.0	93.76
0.0	93.07
0.0	90.31

Sample 209

σ_3	σ_1
41.87	439.65
28.40	377.99
15.72	305.59
4.89	248.00
21.53	342.0
43.98	438.32
24.70	362.01
7.0	244.47
35.40	405.95
42.27	422.37
44.91	428.84
49.93	440.22
20.87	331.81
51.51	458.86
15.58	312.21
7.66	237.68
33.02	398.82
42.26	441.63
25.62	345.95
2.90	211.25
12.15	298.11
19.41	322.28
30.51	378.18
37.24	401.38
10.30	290.55
5.15	251.78
1.32	205.22
50.05	452.92
0.0	197.17
0.0	197.86
0.0	195.79

Figure 75 : Triaxial Strength Results.

6. STRENGTH CRITERIA AND TRIAXIAL STRENGTH CLASSIFICATION

a) The purpose of a strength criterion

Any state of stress may be defined in terms of three *principal stress components* σ_1, σ_2, and σ_3 which may be used to represent the stress graphically as a point in *stress space* whose three axes are those of principal stress. A change of stress may be represented as the locus of the stress point moving through stress space, and this locus is termed the *stress path*. A line equally inclined to the three principal stress axes is known as the *space diagonal* and has equation $\sigma_1 = \sigma_2 = \sigma_3$. A stress may be analysed into hydrostatic and deviatoric components, measured respectively as a distance along the space diagonal and a distance away from this diagonal (Figure 76).

Under some stress states a rock deforms but does not fracture; it remains intact. Other stress states cannot be reached without fracture occuring. *Rock strength* may be defined as the point along any stress path at which a specimen ruptures. Other definitions are possible and are discussed by Bieniawski (1967).

Rupture of the specimen may occur under any number of principal stress combinations, and these 'strength points' may be joined to form a *strength surface* for a particular sample of rock. Stress paths may be followed without rupture of the specimen provided that they do not cross the strength surface. In practice the position of the strength surface may not be unique, its position may depend on factors such as stress path and rate of loading, the dryness of specimens and the precise way in which strength is defined.

The shape of the strength surface can be determined experimentally using strength tests, one or more specimens being required to determine the position of each strength point. Jaeger & Cook (1969) have discussed in detail the various types of test required to explore different regions of the strength surface. Regions explored using the more common laboratory tests are illustrated in Figure 77. Other regions require more elaborate testing techniques, so that the experimental data for these regions is scarce and often unreliable.

A *strength criterion* is an algebraic expression used to describe the locus of a strength point as it travels in the strenth surface. The role of a strength criterion is to allow strength prediction for the purpose of rock structure design. The algebraic expression allows interpolation, sometimes extrapolation from observed strength values. The parameters (constant terms) of the strength criterion characterise the shape of the strength surface for a given rock type and so may be used to classify the rock on the basis of its strength behaviour.

A strength criterion is at best only an approximate fit to the observed data. *Theoretical criteria* such as that of Griffith (discussed in detail by Hoek, 1968) are generally an inadequate fit to the data and must in practice be modified to improve their fit

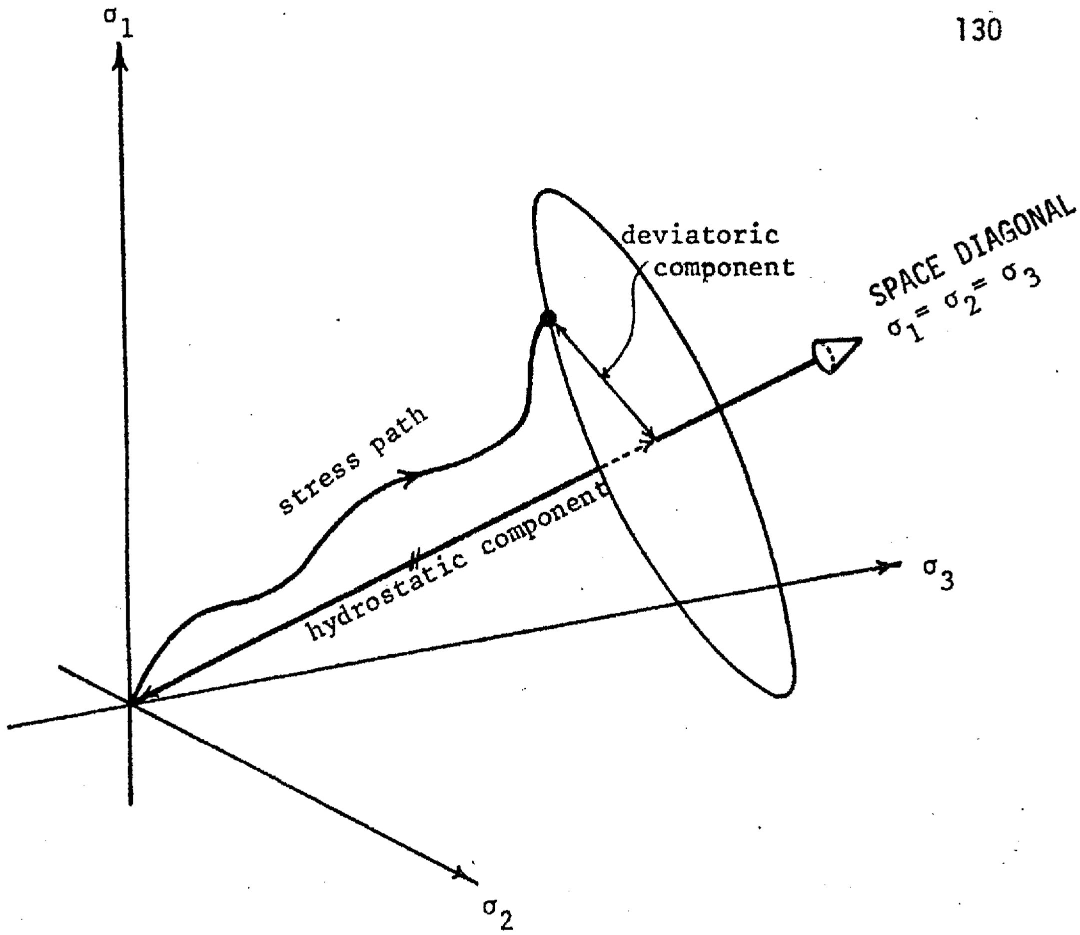


Figure 76 : Stress Space - definitions

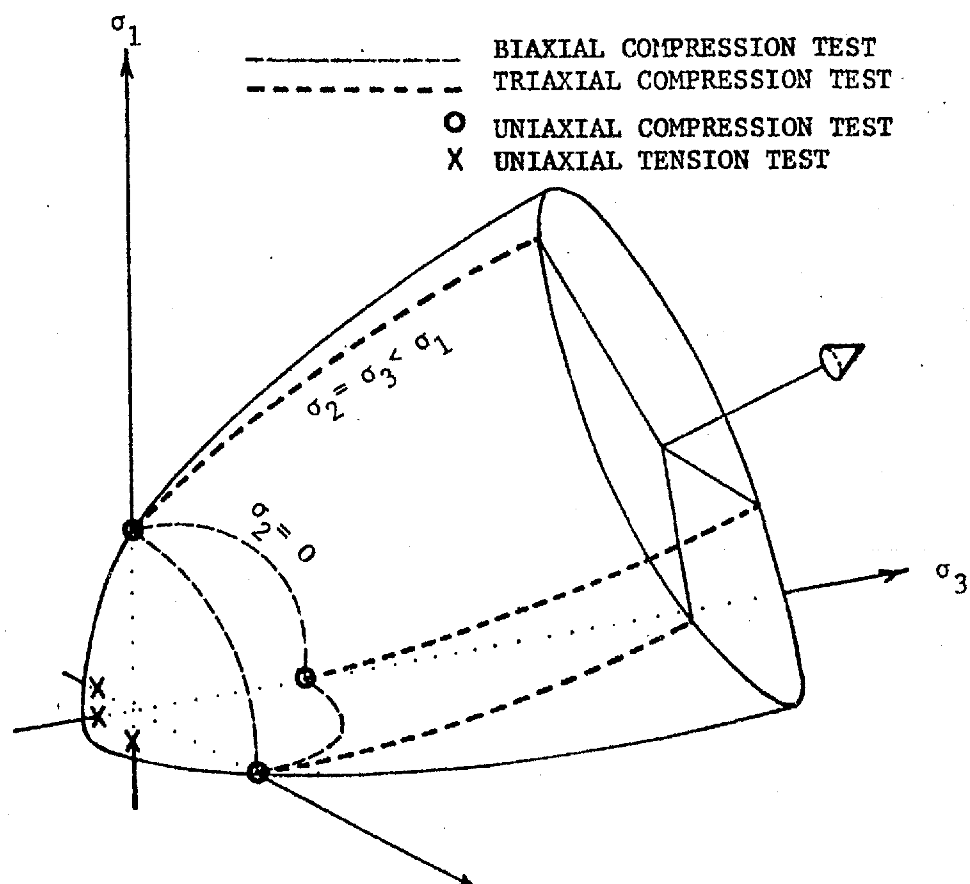


Figure 77 : Regions of the strength surface explored by common laboratory tests

at the expense of some loss of simplicity. Alternatively an *empirical criterion* may be adopted, having little or no theoretical basis, selected only to fulfil the practical requirements of accurate strength prediction and simplicity of use.

The choice of suitable *empirical* strength criteria and the use of these criteria to classify rock materials according to their strength behaviour form the basis of discussion in the forthcoming paragraphs.

b) Attributes of the strength surface

The strength surface for rock has certain properties that may be deduced from available experimental data, also from theoretical considerations. Any criterion selected to fit the strength surface is required to exhibit the same properties, as far as is possible. These attributes are listed below and are illustrated graphically in Figure 78.

(i) An isotropic material has a strength surface that is symmetrical about the 'Rendulic' planes $\sigma_1 = \sigma_2$, $\sigma_2 = \sigma_3$, $\sigma_3 = \sigma_1$. These planes intersect in the space diagonal, which may be regarded as the axis of the strength surface.

(ii) The surface is probably convex outward from the space diagonal at every point, hence an increase in hydrostatic stress results in an increase in deviatoric stress at failure.

(iii) The surface has both positive and negative intercepts on the principal stress axes. The positive intercepts (uniaxial compressive strength) are approximately ten times the negative intercepts (uniaxial tensile strength).

(iv) The surface closes at some finite hydrostatic tensile strength; it probably has a rounded 'nose' where the tangent plane at the nose is perpendicular to the space diagonal.

(v) The surface is probably open in the compressive quadrant; a hydrostatic compressive stress is unlikely to cause rupture. It should become asymptotic to lines parallel to the space diagonal at large values of hydrostatic stress ('plastic' behaviour).

A strength criterion defining the complete strength surface would necessarily be three dimensional, a function of σ_1, σ_2 and σ_3. Such criteria are mathematically complicated and also difficult to verify experimentally, owing to the scarcity of data over much of the strength surface. Most criteria, including the ones to be examined in this chapter, are restricted to defining a single two-dimensional curve in the strength surface, usually the curve where the surface intersects a Rendulic plane $\sigma_2 = \sigma_3$. This is the *triaxial strength curve*, the curve explored using the conventional triaxial test. This curve can then be extrapolated in order to define the complete surface, and the attributes listed above go a long way towards restricting the scope of this extrapolation.

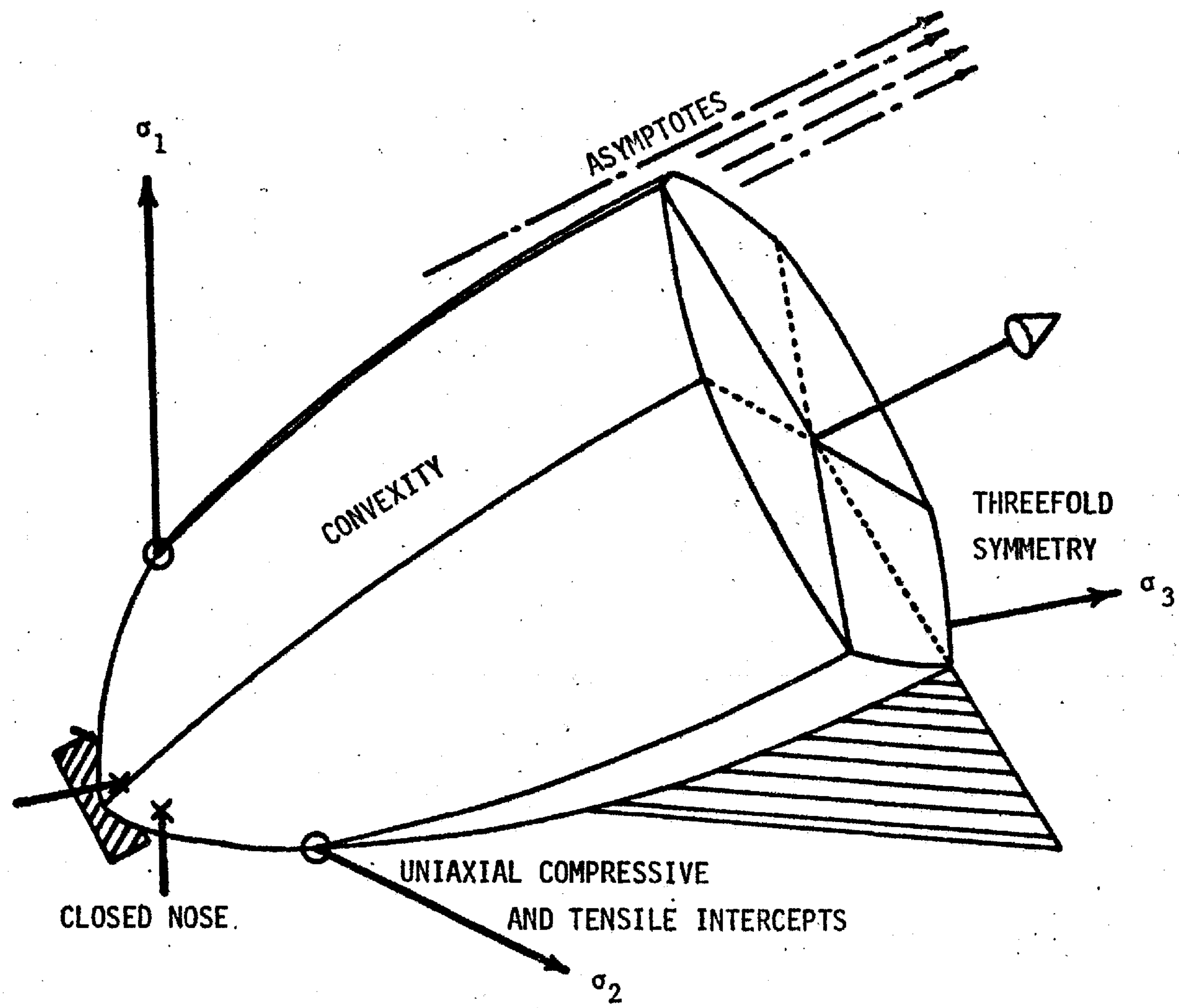


Figure 78 : Probable attributes of a strength surface.

A strength criterion should fit available data but should also conform to certain more general requirements.

If a further attribute is assumed, namely that *strength is independent of intermediate principal stress*, then the complete strength surface is precisely defined for a given triaxial strength curve. This assumption implies that the strength surface is generated by lines parallel to the principal stress axes. It may be observed that a different principal stress becomes intermediate in magnitude on crossing a Rendulic plane (Figure 79).

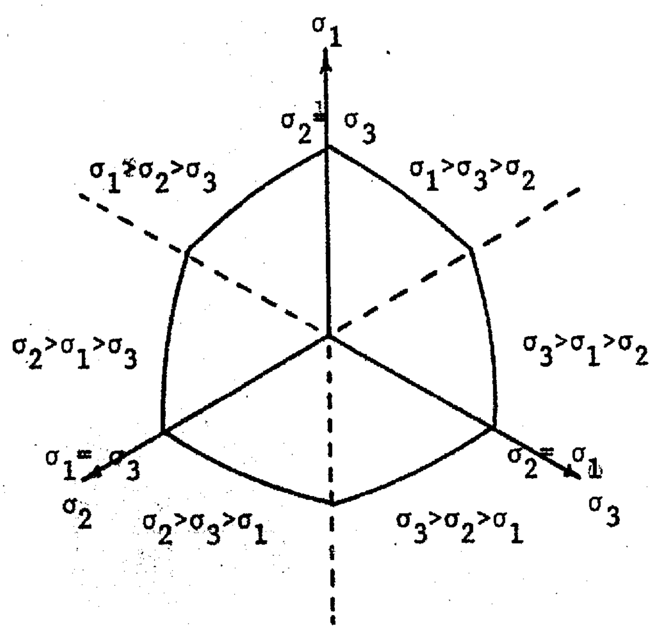


Figure 79 : 'π' cross section to the strength surface.

The Rendulic planes $\sigma_1 = \sigma_2$, $\sigma_2 = \sigma_3$ and $\sigma_3 = \sigma_1$ define regions of stress space in which different principal stresses become intermediate in magnitude.

Intermediate principal stress independence is a consequence of Mohr's hypothesis and although its strict validity has been questioned it may be justified as an approximation (Mogi, 1967). Mohr suggested that strength criteria should be formulated in terms of the normal and shear components of stress on the failure plane in a rock specimen, rather than in terms of the three principal stresses. This type of formulation has important practical advantages in some applications. A criterion once developed in terms of principal stress may be re-formulated in terms of Mohr's co-ordinates, and this approach will be adopted in subsequent paragraphs on grounds of simplicity and ease of interpretation.

c) Mechanisms of failure

Criteria generally do not fit equally well to different regions of the strength surface, and if fitted to data in one region may be grossly inaccurate when extrapolated. The processes or mechanisms that lead to rupture or failure of a specimen differ depending on the stress path and the region of stress space in which it crosses the strength surface, so that it is possible that strength criteria should also differ. Most empirical criteria that were studied gave poor extrapolation outside the regions of brittle or ductile compressive failure, the regions within which the criteria could be fitted to abundantly available data. Since these criteria are therefore of restricted validity it is necessary to define more carefully the regions over which they are likely to be valid, and also the criteria and mechanisms of failure that might apply outside these regions.

Mechanisms of failure have been discussed at greater length by Franklin (1968), and by Hoek (1968). Various failure mechanisms and stress regions over which they operate are illustrated in Figure 80 and are ennumerated below.

I Pure tensile failure

Griffith's theory predicts that flaws parallel to σ_1 are the first to initiate cracks. Cracks, once initiated, are unstable and propagate parallel to σ_1. Since σ_1 acts parallel to the propagating crack it has little influence on strength, hence Griffith proposed a strength criterion $\sigma_3 = \sigma_t$ (the tensile strength of the material).

II Cleavage failure

Griffith's theory predicts that flaws oblique to σ_1 are the first to initiate cracks. The region may be further subdivided by a line $\sigma_1 = 5.83 \ \sigma_3$; for smaller σ_1 the stress normal to the critical flaw is tensile, and for larger σ_1 compressive. In practice crack propagation throughout region II is unstable. Cracks curve towards σ_1 so that the final plane of rupture is parallel to the $\sigma_1$direction, as was the case for region I.

III Brittle compressive failure

At failure the specimen suffers a drop in load carrying capacity. Failure is usually abrupt and audible so that strength may be easily defined. Cracks initiated throughout the specimen stabilize, and the specimen fails by shearing movement on a plane oblique to σ_1. Towards the onset of failure crack initiation is concentrated in the vicinity of the rupture plane, weakening the specimen. Once generated, this shear plane is usually weaker than adjacent material and the specimen collapses.

IV Ductile failure

The strength of shear planes generated in the rock approaches or exceeds that of adjacent material, so that multiple shears result. Failure is usually gradual and inaudible so that it may be difficult to define. The load carrying capacity (strength) usually approaches a constant value, although for practical reasons it is generally defined as σ_1 at 1% strain in the σ_1 direction (if rupture has not occured by the time this strain has been achieved). Mogi (1966) attempted to define the brittle-ductile transition boundary (Figure 80) by experiment, but in practice the transition is gradual and difficult to locate precisely.

V Plastic failure

At high values of hydrostatic stress, rock crystals deform by plastic mechanisms such as twinning and gliding. Griggs & Blacic (1965) report these mechanisms operating in quartz at 400 MN/m^2, and in calcite at 1000 MN/m^2. It is likely that at these stresses the triaxial strength curve has become sub-parallel to the space diagonal, a condition for 'perfect plasticity'. Plastic mechanisms should not, however, be confused with plastic (irreversible) strains which are manifest at much lower stress levels.

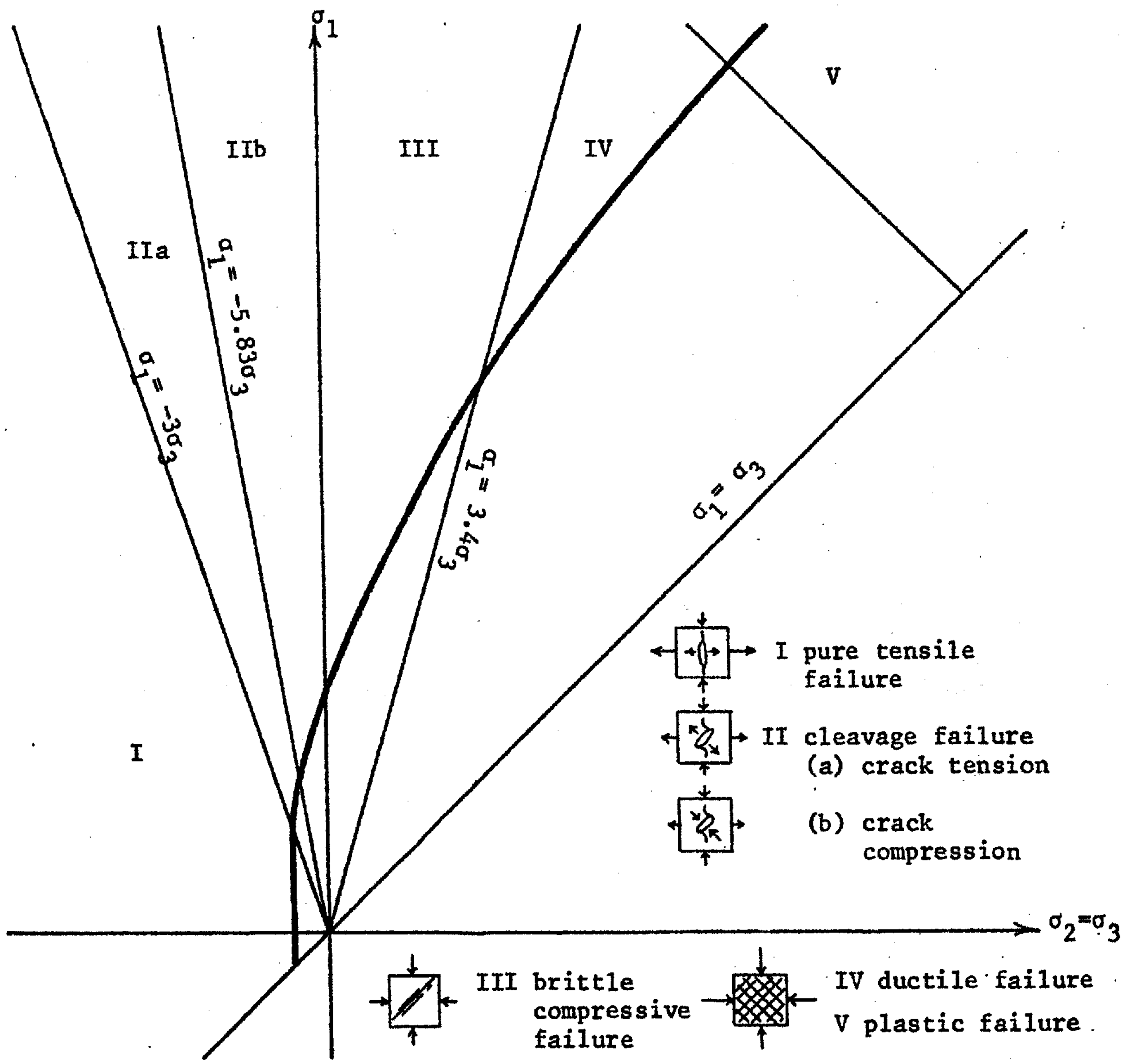


Figure 80 : Failure mechanisms operating over different regions of the strength curve.

d) The selection of a triaxial strength criterion

One arrives at an empirical strength criterion in the same way as one would any other empirical design formula, by collecting experimental data and by searching for an algebraic expression to fit these observations. Usually not one but several expressions can be found that fit the data with a satisfactory accuracy. Selection is made from among these expressions firstly by considering which gives the minimum prediction error (which is the best fit). The criterion must also be as simple as possible so that parameters of the equation may be easily estimated and so that the expression will not present difficulties when used in design. Finally, when the available data applies only to a limited range of values the validity of the expression outside the range of observation must be taken into account.

In the present study data was available only for brittle and to some extent ductile triaxial compressive failure, regions III and IV of figure 80. Criteria were examined for their fit to this data, for their general form in relation to the attributes of the complete strength surface that were described in paragraph (b).

177 samples of published triaxial strength data were collected and supplemented by 8 samples of data from tests carried out by the author and by students at Imperial College. The data, consisting of some fifteen hundred specimens in all, covered a wide variety of rocks. It was used to determine the most suitable form of triaxial strength criterion, and also to compare (or classify) rock types, defined geologically, on the basis of their triaxial strength behaviour. The earlier attempts at curve fitting and classification have been described in publications by Franklin (1968) and by Cockell (1969), a statistician assigned to the project by the mathematics department at Imperial College.

Models studied as potentially suitable for triaxial strength prediction were as follows;

1. $\sigma_1 = A + B\sigma_3$

2. $\sigma_2 = A + B\sigma_3{}^C$

3. $\sigma_1 = A \log (B + \sigma_3)$

4. $(\sigma_1 - \sigma_3) = A + BC^{\sigma_3}$

5. $(\sigma_1 - \sigma_3) = \dfrac{A(\sigma_1 + \sigma_3) + B}{(\sigma_1 + \sigma_3) + C}$

6. $(\sigma_1 - \sigma_3) = A + B(\sigma_1 + \sigma_3)^C$

7. $(\sigma_1 - \sigma_3) = A(\sigma_1 + \sigma_3)^B$

A, B and C are 'parameters' of the criteria, characteristic of a given rock sample.

Model 1 has the form of a straight line and is the principal stress formulation of the 'Mohr Coulomb' criterion, used almost exclusively at present for strength prediction purposes. It has long been recognised that triaxial strength data for rock is in fact curved. The amount of curvature had not been established, and one of the objects of this study was to determine whether a criterion could be found that gave a practically significant improvement in strength prediction. In addition the simplicity of the alternative, curved, criterion had to be sufficient to justify its use in place of the more conventional straight line.

Model 7 was found to best meet these requirements, often halving the prediction errors resulting from the use of a straight line criterion. It is at the same time simple, incorporating only two parameters both of which can be readily estimated from the data. Subsequent discussion will be directed towards a comparison of models 1 and 7, and to their use in classification of rock materials. First, however, some features of the remaining criteria that led to their rejection will be outlined.

The models exhibit a variety of the required attributes; models 4 and 5, for example, have asymptotes parallel to $\sigma_1 = \sigma_3$. All but model 3 gave improved strength prediction over the linear Mohr Coulomb criterion, as no doubt would other forms of criterion that were not investigated. The complexity of models 4 and 5 could not be justified despite their possible advantages. Their parameters could only be estimated using complex procedures and making assumptions that might not be justified. Model 2 provided a good fit to the data and was preferred by Cockell (1969) and by Newman & Newman (1969). However this model also was rejected since it involves three rather than two parameters, and is symmetrical about $\sigma_1 = 0$ rather than $\sigma_1 = \sigma_3$ making extrapolation of doubtful accuracy. It cannot be fitted to uniaxial strength data and requires a prior estimate of A, the uniaxial compressive strength. As a result this curve is forced to pass exactly through the uniaxial strength point, an undesirable state of affairs.

Both models 2 and 6 allow the material to possess tensile strength, but tensile strengths extrapolated from compressive strength data prove unrealistic. Model 7 was adopted despite the fact that it predicts zero uniaxial tensile strength (it passes through the origin of stress space), since it fits compressive strength data well. Unlike model 2 it can readily be fitted using both uniaxial and triaxial compressive strength data, giving equal weight to all test results.

e) Methods of estimating parameters and comparing prediction errors

The two criteria to be further examined and compared may be re-stated as follows:

$$\sigma_1 = \sigma_c + G.\sigma_3 \qquad \text{Linear criterion}$$

$$(\sigma_1 - \sigma_3) = A(\sigma_1 + \sigma_3)^B \quad \text{Curved criterion}$$

G and σ_c are the parameters of the linear criterion, respectively the gradient and intercept (uniaxial compressive strength) of the straight line fitted to data points plotted in the plane σ_3, σ_1. Parameters of the curved criterion are obtained as the gradient B, and A the antilogarithm of the intercept of a straight line fitted to $\log(\sigma_1 - \sigma_3)$ vs. $\log(\sigma_1 + \sigma_3)$. Figure 81(a) demonstrates that this double logarithmic plot is indeed linear, a result that appears to hold for all the rock samples studied. Figure 81(b) shows the same data plotted in principal stress co-ordinates, together with the fitted curves whose parameters have been estimated from the logarithmic plot.

The parameters may be estimated by fitting a straight line by eye to a graphical plot of raw or log-transformed data, or by using the method of least squares. The statistical formulations for the gradient and intercept using the least squares technique are standard, but for ease of reference are given below:

If $y = mx + c$; N observations ; $\bar{y} = \frac{\Sigma y}{N}$; $\bar{x} = \frac{\Sigma x}{N}$

then the gradient m, and intercept c of this best fit line are given by

$$m = \frac{\Sigma xy - N\bar{x}\bar{y}}{\Sigma x^2 - N(\bar{x})^2} \qquad \text{and} \qquad c = \bar{y} - m\bar{x}$$

Stresses quoted in this chapter are in S.I. units of Mega Newtons per square metre (MN/m^2). Inspection of the criteria shows that parameters B and G are dimensionless, while A will be shown later to have dimensions of $[\text{stress}]^{1-B}$. A computer programme was developed to convert stress units, also to estimate strength parameters and prediction errors as defined below. For the purposes of classification of rock materials the Mohr transformation of the straight line criterion was used rather than the principal stress formulation, since this is probably the more familiar form of expression. Hence the parameters C and $\tan\phi$ were computed from σ_c and G, and used in their place. The relationships between these parameters will be demonstrated later.

The adequacy of fit of the two models was compared by studying the deviation of observed data points from the fitted line or curve.

$$\text{Strength prediction error} = \left| \frac{\sigma_1 \text{ observed} - \sigma_1 \text{ predicted}}{\sigma_1 \text{ predicted}} \right|$$

All errors were taken as positive in sign. Errors for all specimens within a sample were averaged to obtain the *overall prediction error* E_o. In addition the errors for uniaxial strength observations were averaged to obtain the *uniaxial prediction error* E_u. When a straight line is fitted to curved data E_u will be greater than E_o, and will give an estimate of the maximum likely prediction error (Figure 81 (c)).

The curved criterion cannot be solved explicitely for σ_1, so that predicted strength was estimated using an iterative procedure. An estimate of σ_1 in the right hand side of the criterion equation was used to obtain a better estimate on the left hand side. This value was then transferred to the right hand side and the procedure repeated. The

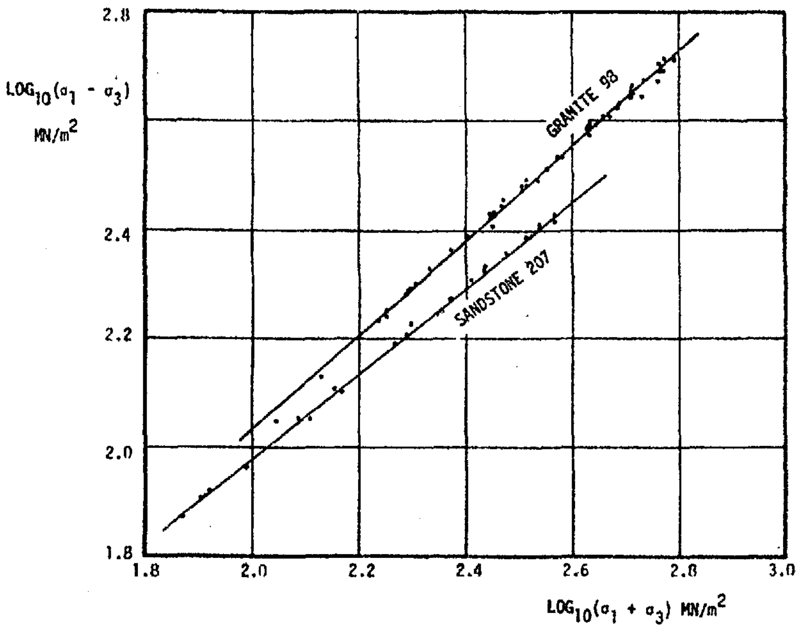


Figure 81a

Double logarithmic plot of triaxial strength results for granite and sandstone samples. A linear relationship is found to hold for materials as different as London clay and basalt.

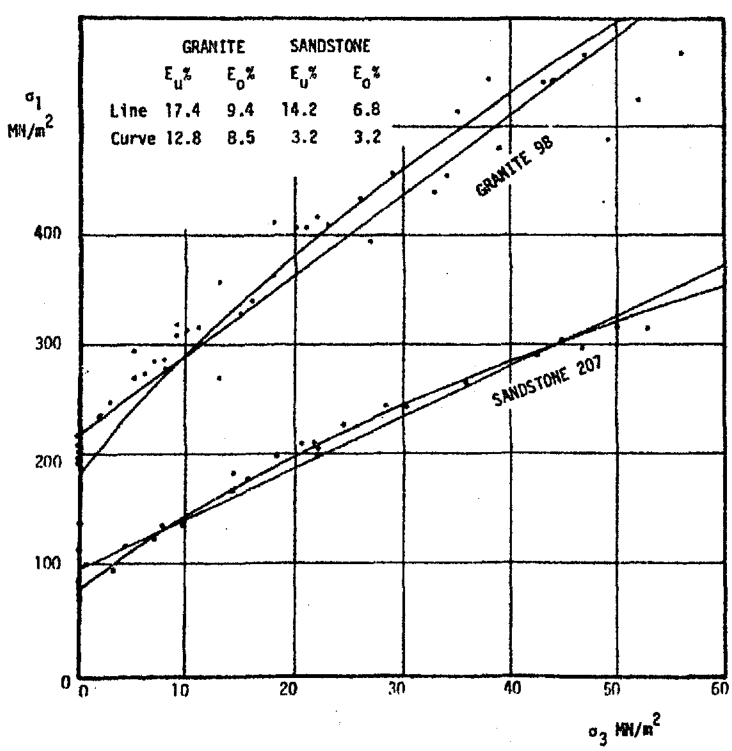


Figure 81b

Principal stress plot of the same data as above, showing fitted curvilinear strength criteria, also fitted linear strength criteria. The resulting strength prediction errors for all data (E_o) and for uniaxial strength data alone (E_u) are also tabulated.

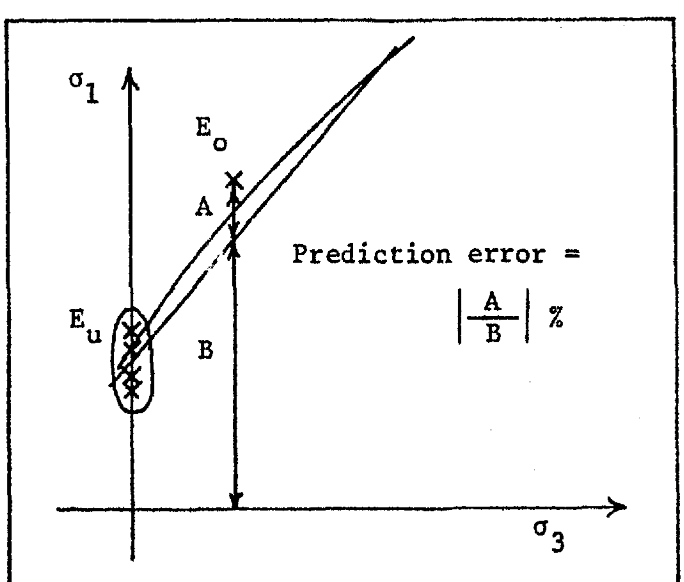


Figure 81c

Method of defining the strength prediction errors E_u and E_o.

observed σ_1 value was used as a first estimate, and the iteration was found to converge giving predicted strength to six significant figures after 100 iterations. Convergence was slower for B approaching 1.0, and in these cases 1000 cycles of iteration were employed.

f) Comparison of linear and curved models

Prediction errors arising from use of one or other model are compared in Figure 82. It must be emphasised that prediction error is not solely due to an inadequate fit of the model to the data, but reflects the scatter due to natural variation between rock specimens and also that due to experimental inaccuracy. However if both linear and curved models are fitted to the same sample of data, then a decrease in the errors E_o and E_u should demonstrate that one model fits better than the other. Use of the curved rather than the linear model resulted in a reduction in uniaxial prediction error from, on average, 35% to 12%, and overall prediction errors were reduced from 10% to 6%. Errors much larger than the average values quoted may arise, and use of a curved rather than a straight line criterion for engineering design appears to be justified, allowing the use of smaller factors of safety.

The straight line criterion may still be preferred for some engineering applications where the stress range is small or the tolerance of error high. It appears to offer little improvement of simplicity over the curved criterion (both incorporate only two strength parameters) although it has certain advantages such as a simpler formulation in terms of Mohr envelope co-ordinates (to be discussed later). The two criteria will be developed side by side to allow a free choice according to particular requirements.

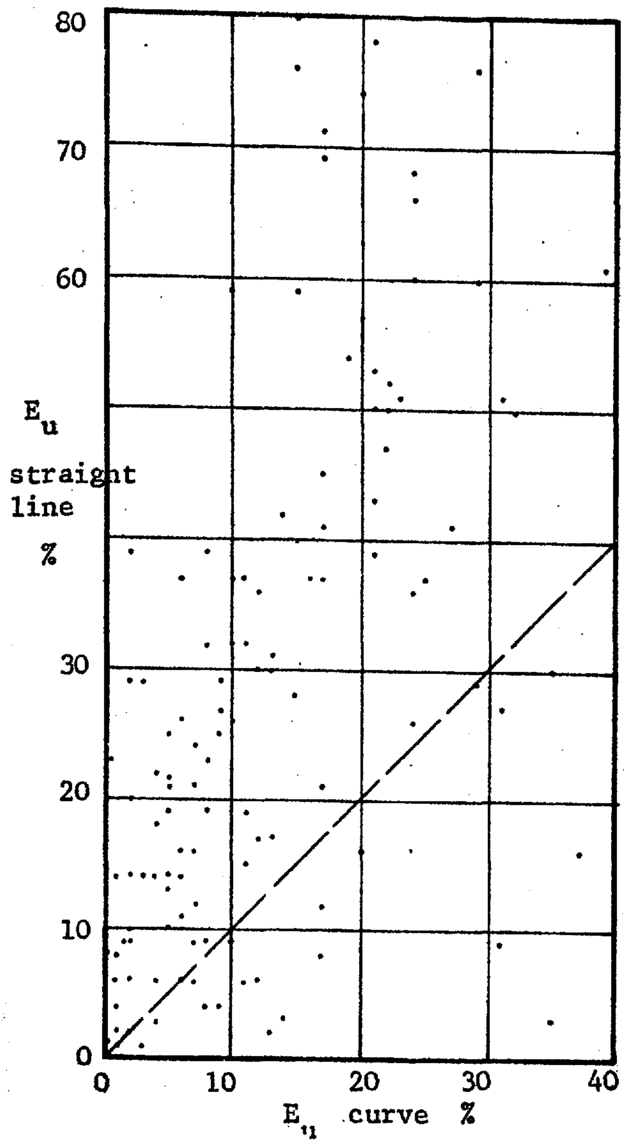
g) Classification of rocks on the basis of their triaxial strength

Rock samples may be classified using parameters of either the linear or the curved criterion. Figure 83 shows classification based on C and $\tan\phi$, and Figure 84 on the curve parameters A and B. Each point plotted represents a sample of data consisting of from four to over forty specimens, and its co-ordinates, the strength parameters, identify a particular strength curve or line.

To assist in the interpretation of these diagrams, uniaxial strength contours may be plotted using the following relationships that will be derived towards the end of this chapter:

$$\tan\phi = \frac{\sigma_c}{4C} - \frac{C}{\sigma_c} \qquad \text{straight line criterion}$$

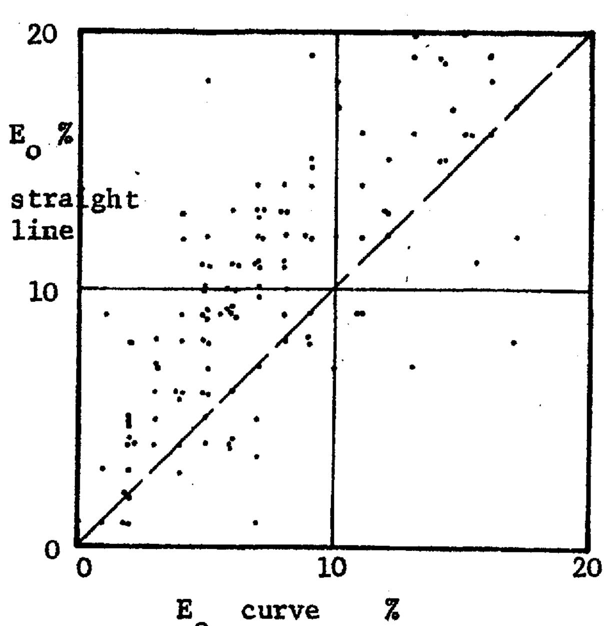
$$A = \sigma_c^{\,1-B} \qquad \text{curved criterion}$$



Errors in predicting uniaxial strength - 156 samples out of 180 showed better uniaxial strength prediction using a curved criterion. Typically a 35% prediction error was reduced to 12% by using a curve rather than a straight line.

Figure 82

Comparison of strength prediction errors using curved and straight line criteria



Errors in predicting strength based on an average of all results - 153 samples out of 180 showed better strength prediction using the curved criterion. Typically a 10% error was reduced to 6% by using a curve rather than a straight line.

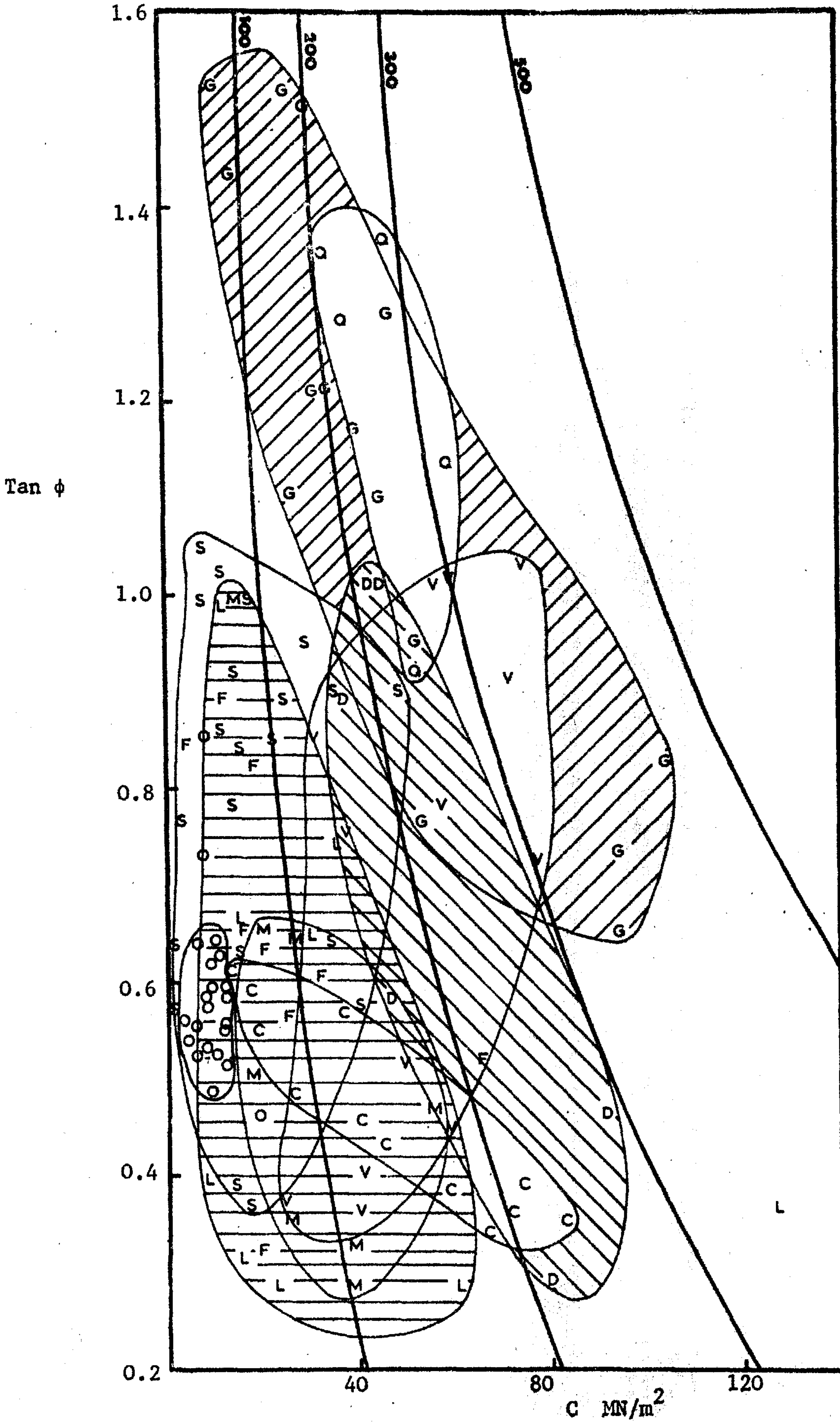


Figure 83 : Classification o. intact rocks on the basis of C and tanϕ, parameters of a Coulomb-Navier strength criterion. A key to rock types appears in Figure 84 Contours are of uniaxial strength

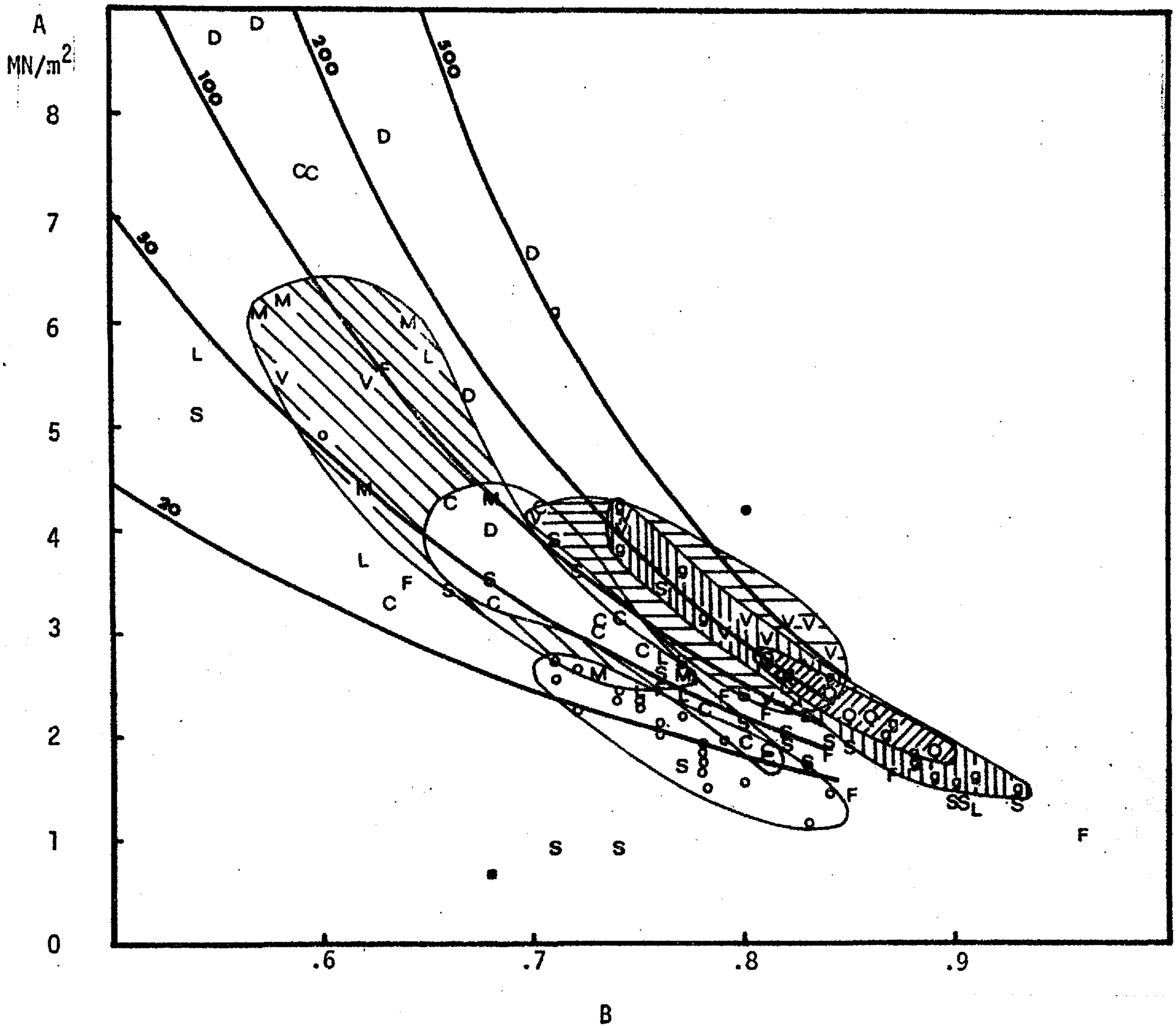


Figure 84 : Classification of intact rocks on the basis of triaxial strength parameters A and B in the criterion $(\sigma_1 - \sigma_3) = A(\sigma_1 + \sigma_3)^B$ Contours are of uniaxial strength (MN/m^2)

M	marble	•	pyrex glass	S	sandstone	g	granite
F	mudstone	•	London clay	o	coal	D	dolomite
L	limestone	Q	quartzite	V	volcanics	C	concrete

Use of these contours may be demonstrated from Figure 84, where coal samples can be seen to plot along a contour of low uniaxial strength, while pyrex glass offers an example of a material with unusually high uniaxial strength.

For the purposes of comparison rocks have been subdivided into 'geological classes' in the traditional manner. Some of the classes cluster tightly, demonstrating that the classification in this case has considerable mechanical as well as geological significance. Other classes are spread widely and overlap each other, demonstrating that for these materials the geological name covers a multitude of materials of widely differing mechanical nature. If such materials were further subdivided on the basis of mechanically important textural differences (porosity, for example), they would very probably form tighter clusters. Such a modified geological classification was indicated in Chapter 4, but insufficient data was available to allow this form of plot to be constructed.

A considerable overlap in parameter values is in fact observed for many of the rock types. If each rock type is considered as a single 'sample' and a curve fitted to the data, then the variation of parameter B from one rock type to the next is often insignificantly small. Rather than demonstrating that the rock types are mechanically identical, this finding illustrates the inadequacy of traditional geological nomenclature when used for mechanical classification. The real differences in mechanical behaviour may readily be observed from the spread of data points in Figures 83 and 84.

The figures may be used to obtain approximate values for triaxial strength parameters for use in design calculations. Precision will be improved if both geological class and uniaxial strength are taken into account in obtaining these values. The data should, however, be used with caution. The coal samples, for example, were largely obtained from British coalfields using results published by the National Coal Board, and coals from other geological environments and of different age might well plot outside the present clusters.

The classification diagrams suggest physical mechanisms that might operate to produce high or low values for one or other parameter, although without further carefully controlled testing these can only be speculative. The different locations of volcanic and granitic clusters, for example, may well be due to a difference in grain size. The rate of strength increase with increasing confining pressure is reflected by the parameters B and $\tan\phi$. High values indicate that a rock gains strength rapidly. Coarser grained rocks probably benefit more from lateral restraint since much of their strength depends on the interlocking of strong but poorly bonded grains.

A complete bibliography of the sources of triaxial strength data used in this study has been presented in an earlier paper (Franklin, 1968) and will not be reproduced in this thesis. Data extracted from the literature was essential to this study and the authors must be given full credit for their experimental work.

h) Mohr transformations of the straight line and curved criteria

The two sections that follow demonstrate how strength criteria, particularly the curved criterion previously discussed, may be re-formulated in different ways to suit particular applications. The re-formulation may also assist in demonstrating certain properties of a given criterion. First we will consider the transformation of a principal stress form of strength criterion into the Mohr co-ordinates σ and τ.

A given expression relating σ_1 and σ_3 at failure may be expressed in terms of σ and τ, the co-ordinates of the corresponding Mohr envelope, using the following transformation equations:

$$\sigma = \frac{\sigma_1 + \sigma_3\sigma_1'}{1 + \sigma_1'} \quad \text{and} \quad \tau = \frac{(\sigma_1 - \sigma_3)\sqrt{\sigma_1'}}{1 + \sigma_1'} \quad \text{where} \quad \sigma_1' = \frac{d\sigma_1}{d\sigma_3}$$

or alternatively

$$\sigma = \sigma_m - \tau_m\tau_m' \quad \text{and} \quad \tau = \tau_m\sqrt{1 - (\tau_m')^2}$$

$$\text{where} \quad \tau_m = \frac{\sigma_1 - \sigma_3}{2}, \quad \sigma_m = \frac{\sigma_1 + \sigma_3}{2}, \quad \tau_m' = \frac{d\tau_m}{d\sigma_m}$$

Examination of the expression for τ indicates that an envelope may be derived provided that σ_1, is positive, equivalent to stating that strength must increase with confining pressure in the triaxial test.

The first pair of transformation equations may be used to derive a Mohr envelope formulation for the straight line criterion. The derivation proceeds as follows

$$\sigma_1 = \sigma_c + G\sigma_3 \quad \text{so that} \quad \sigma_1' = \frac{d\sigma_1}{d\sigma_3} = G$$

$$\text{hence} \quad \tau = \frac{G-1}{2\sqrt{G}}\sigma + \frac{\sigma_c}{2\sqrt{G}}$$

$$\text{compare} \quad \tau = \tan\phi . \sigma + C \quad ; \quad \underline{\tan\phi = \frac{G-1}{2\sqrt{G}}, \quad C = \frac{\sigma_c}{2\sqrt{G}}}$$

Thus if the principal stress criterion is linear, the Mohr envelopes is also linear. The expression for $\tan\phi$ and C may be combined to give the expression relating C, $\tan\phi$ and σ_c that was used to plot uniaxial strength contours on Figure 83. The algebraic expressions for C and $\tan\phi$ offer a superior method of constructing linear Mohr envelopes to triaxial strength data. Data for rock is often scattered, making graphical construction of an envelope subjective and inaccurate

(Balmer, 1949). A better procedure is to use the method of least squares (or a graphical fit by eye) to derive the parameters G and σ_c, and then to estimate the parameters of the Mohr envelope algebraically.

The second pair of transformation equations is more convenient to use in deriving a Mohr envelope formulation for the curved criterion. The criterion is first transformed into the 'maximum shear stress' form (described below), so that

$$(\sigma_1 - \sigma_3) = A(\sigma_1 + \sigma_3)^B \qquad \text{transforms to} \qquad 2\tau_m = A(2\sigma_m)^B$$

$$\text{where } \tau_m = (\sigma_1 - \sigma_3)/2 \text{ , } \sigma_m = (\sigma_1 + \sigma_3)/2$$

so that $\tau_m' = 2^{B-1}AB\sigma_m^{\ B-1}$

substituting values for τ_m and τ_m' in the Mohr envelope transformation equations;

$$\sigma = \tfrac{1}{2}(2\sigma_m - BA^2(2\sigma_m)^{2B-1})$$

$$\tau = \tfrac{1}{2}A(2\sigma_m)^B \sqrt{1 - (AB(2\sigma_m)^{B-1})^2}$$

These are parametric equations for the Mohr envelope corresponding to the curved criterion. Obviously they are less convenient to use than the single equation derived in the linear case, but nevertheless an envelope can be constructed from the data, point by point.

i) Alternative formulations of the curved criterion

The curvilinear criterion proposed in paragraph (e) and subsequently developed may be re-formulated in one of several ways that may prove more convenient in certain applications. The Mohr type of formulation has already been examined. Further types of formulation rely on a relationship that may be demonstrated between the predicted uniaxial strength σ_c and the parameters A and B. The curvilinear criterion as previously defined had the following form:

$$(\sigma_1 - \sigma_3) = A(\sigma_1 + \sigma_3)^B$$

By definition, when $\sigma_3 = 0$ we have $\sigma_1 = \sigma_c$ so that, substituting these values in the above expression,

$$A = \sigma_c^{\ 1-B}$$

This relationship may be used to re-formulate the criterion in terms of σ_c and B rather than A and B. Substituting for A and re-arranging we have

$$\left(\frac{\sigma_1 - \sigma_3}{\sigma_c}\right) = \left(\frac{\sigma_1 + \sigma_3}{\sigma_c}\right)^B$$

This is the *normalised form* of the curved criterion, stresses being expressed as dimensionless ratios of uniaxial strength. It may be observed that B is also dimensionless, whereas A has dimensions of stress.

The normalised criterion may be applied to selection of materials for use in physical models to simulate the strength behaviour of a rock mass. Physical modelling of this sort requires that model and prototype materials should have 'homothetic' failure curves (Fumagalli, 1960). This requirement is equivalent to specifying that model and prototype materials should have identical B values or that the strength curves for the two materials when plotted in 'normalised form' should be identical.

The curved criterion may be transformed into '*maximum shear stress form*' as discussed in paragraph (h) above by substituting

$$(\sigma_1 - \sigma_3) = 2\tau_m$$

$$(\sigma_1 + \sigma_3) = 2\sigma_m$$

so that

$$\tau_m = 2^{B-1} A \sigma_m^{\ B} = \left\{\frac{\sigma_c}{2}\right\}^{1-B} \sigma_m^{\ B}$$

or alternatively, in normalised form,

$$\frac{\tau_m}{\sigma_c} = 2^{B-1}\left\{\frac{\sigma_m}{\sigma_c}\right\}^B$$

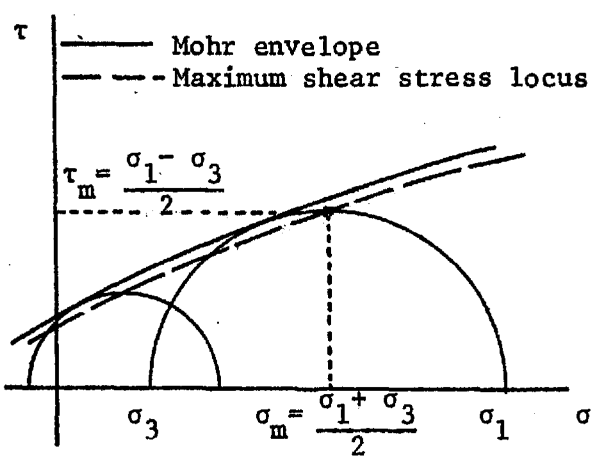


Figure 85 : Maximum shear stress locus.

Strength criteria may be re-formulated in terms of (τ_m, σ_m), the co-ordinates of tops of Mohr stress circles at failure.

Direct comparison of stress with strength for purposes of engineering design is facilitated if the strength criterion is formulated in terms of stress quantities that may readily be determined experimentally. The 'maximum shear stress locus' describes the locus of a point passing through the tops of Mohr circles (Figure 85), this point having co-ordinates τ_m, the maximum shear stress and σ_m, the mean principal stress in the specimen at failure. Isochromatics, determined photoelastically in a model of a rock structure, are contours of equal τ_m. Isopachics, that may be determined using a conducting paper analogue of the rock structure, are contours of mean principal stress. Hoek (1967) describes a procedure for combining the two experimental techniques to outline fracture zones adjacent to a group of tunnels in a rock mass, using a strength criterion in the form given above. The need to isolate values for principal stresses individually ('principal stress separation') is thus eliminated.

An interesting comparison may be made between the curved criterion outlined above and the parabolic criterion of Griffith. Griffith's criterion may be formulated.

$$(\sigma_1 - \sigma_3) = \sigma_c^{\frac{1}{2}}(\sigma_1 + \sigma_3)^{\frac{1}{2}} \quad \text{Griffith's criterion}$$

$$\text{c.f. } (\sigma_1 - \sigma_3) = \sigma_c^{1-B}(\sigma_1 + \sigma_3)^{B} \quad \text{Empirical curved criterion}$$

Hence, if $B=\frac{1}{2}$ the two criteria are identical. It has been demonstrated that in practice B assumes values between 0.6 and 0.9 (Figure 84) so that a parabolic criterion does not adequately fit the data. Rock gains strength more rapidly with increasing confining pressure than Griffith's theory would suggest. His criterion relates to a material with open cracks. The additional strength increase resulting in departure from the Griffith parabolic model may be due to crack closure and to interlocking effects in the cracked rock material.

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