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Geotechnics of Weak and Jointed Rock - General Report

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1. INTRODUCTION

In accepting the task of General Reporter for this session on the geotechnics of weak and jointed rock, it was understood by the writer that his role was to review all papers assigned to the session. On the basis of this review, he was then to pose four "key questions" to seed the discussion of the topic during the technical session.

Unfortunately, at the time of preparation of the General Report, only three papers had been received for review. These papers comprised one which was co-authored by the reporter and two more on effectively one subject from another single group of researchers. (In fact it is arguable whether the rocks considered in these latter two papers are weak or jointed). Therefore, it would appear that this review was unlikely to take too long and, since the two sources of the papers involved different topics, there was unlikely to be any major conflicts between these papers. Despite this situation, the reporter will press on with his task of review but, in order to provide fodder for discussion, will have to call upon other sources of data and opinion.

Another aspect of the undertaking which caused the reporter some concern was the meaning of the title of the session. While weak and jointed rock seems to define a specific range of geotechnical materials, the term "geotechnics" is not as well defined. Geotechnics could be interpreted as meaning the whole science of geotechnology implying that this session concerns all aspects of the theory and practice of engineering works on weak and jointed rock. However, since other sessions of this conference are directed at a range of topics such as mining and excavations, slope stability, foundations and geotechnical testing, all of which may include weak and jointed rock, it would appear that geotechnics, in the context of this report, may have a more limited interpretation. This limited interpretation is also reflected in the three papers. Therefore, for the purposes of this General Report, the geotechnics of weak and jointed rock will be taken as the fundamental geotechnical properties of these materials under applied loads; namely elastic modulus and shear strength.

2. TENSILE AND COMPRESSIVE ELASTIC MODULI

The paper by Haberfield and Johnston examines the variation in the unconfined Young's modulus of a soft rock when loaded in compression, bending and tension in the laboratory. The results seem to indicate that the modulus measured under tensile stress conditions is about an order of magnitude greater that the modulus measured under compressive stress conditions. This finding appears to be a material characteristic and not a function of the test techniques employed. Unfortunately, because other investigations examining both compressive and tensile behaviour of soft rocks have been more concerned with strength rather than with stress-strain relationships, there is very little in the literature which supports this conclusion. However, for much stronger materials such as natural hard rocks, there is a reasonable body of evidence to suggest that the tensile modulus can be significantly more than an order of magnitude greater than the compressive modulus (Adler, 1970). For weaker materials such as clays, there appears to be even less evidence, but in an investigation into the deformation characteristics of compacted clays, Ajaz and Parry (1975) found that the tensile modulus could be between about 2.5 and 5 times the compressive modulus.

It would appear then that the results obtained by Haberfield and Johnston have added to the limited body of evidence wherein there is a general trend for geotechnical materials which suggests that the ratio of tensile to compressive modulus is greater than one, with this ratio perhaps increasing significantly as the material becomes harder. It may be of interest to note that the ratio of compressive to tensile strength also increases significantly in a progressive manner as the material considered progresses from stiff clays, through soft rocks to harder rocks (Johnston, 1985a).

3. ELASTIC MODULUS FOR FOUNDATION DESIGN

When foundations are designed for structures, there are a number of criteria which must be satisfied before the design may be considered satisfactory. One of these criteria requires that the estimated settlement experienced by the structure under the design load is acceptable and within tolerable limits. For this estimation, it is usual to employ analytical or numerical methods, both of which would normally involve the use of elastic modulus as an important component of the input data. Because of the difficulty inherent in establishing this modulus under tensile stress conditions, it is normal to find that the modulus is established under compressive stress conditions, often in some form of triaxial test in the laboratory.

When a foundation is loaded, the stresses within the geotechnical zone of influence can be compressive and tensile, with the dilatant characteristics of the stressed material playing an important role with regard to the magnitude and extent of tensile stresses. For clays, even for heavily over-consolidated clays, dilation under normal working stresses may be relatively small. Therefore, it is unlikely that there is any significant development of tensile stresses. It follows that the use of a compressive modulus in the estimation of settlement is a reasonable approach. However, for harder, more brittle materials such as rock, dilation is a very important characteristic and can cause the development of significant tensile stresses. These tensile stresses can have a dominant influence on the behaviour of loaded foundations on rock. This has been demonstrated by model footing and pile tests conducted on confined blocks of soft rock (Johnston and Choi, 1985). These showed that failure was dominated by the progressive development of substantial radial and circumferential tension cracks as shown in Fig. 1, indicating that there must have been significant tensile stress fields associated with application of loading. These mechanisms were also observed in the full scale field tests reported by Williams (1980).

Fig. 1 Development of tension cracks in model footing test on soft rock

It follows that for foundations on rocks, there is reasonable evidence to suggest that significant tensile zones exist. Therefore, it would seem reasonable to suggest that the tensile modulus may have some considerable influence. Since this modulus may be an order of magnitude greater than the compressive modulus, it is likely that the use of the compressive modulus may lead to significant over-estimations of elastic settlement, at least in the case of relatively intact rock. This situation would seem to lead to the development of conservative designs.

For rock which is highly jointed, it could be argued that the joints would not transmit tensile stresses. In this situation it would seem reasonable to suggest that the compressive modulus may be appropriate. However, one must bear in mind that with a high joint content, the compressive modulus obtained from specimens of intact rock is likely to be much greater than the rock mass modulus. Therefore, the use of such a modulus is likely to lead to an under-estimation of settlement.

Based on the above discussions, it would appear that the use of the compressive modulus of a weak rock as determined in the laboratory may lead to a range of inaccuracies when used to determine the settlement of a loaded foundation as shown diagrammatically in Fig. 2. The dashed portion of the curve indicates the region where a laboratory sized sample may contain a representative proportion of the jointing to reflect actual behaviour. Unfortunately, this is also the region where disturbance effects make the taking of such representative samples very difficult indeed. For these latter conditions it is usual to depend more on the application of field test techniques, in particular loading tests, to provide realistic data on mass modulus.

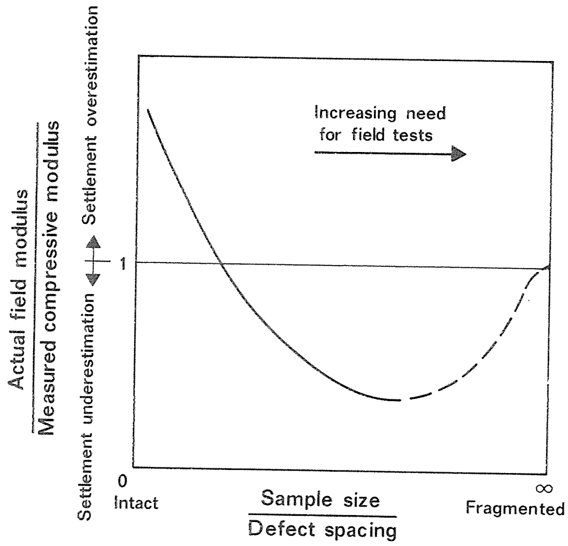

Fig. 2 Effect of representative fissure contact of test sample on prediction of settlement

The first "key question" then relates to the joint characteristics at which laboratory determined compressive moduli become inappropriate for application to foundation design. Under what circumstances can the laboratory compressive modulus be reasonably used and does it have any relevance or application when jointing becomes significant? Although it may not be possible to develop precise guidelines, it may be worthwhile exploring the range of applicability of laboratory determined compressive moduli.

The second "key question" follows on from the above and applies in situations where the laboratory determined compressive modulus is not appropriate to estimations of settlement. What other means of modulus determination do we have at our disposal, particularly for jointed rock masses, and what problems are associated with their interpretation?

It should be noted that although the above discussion and subsequent questions have been framed in terms of structural settlement, the problem of modulus determination applies to many other forms of engineering design. Therefore, it is hoped that discussion can be much broader.

4. STRENGTH CRITERIA FOR ROCK

It is claimed by Ramamurthy, Rao and Rao that of the available strength criteria for rocks, none shows perfect agreement with experimental values for all rock types. Therefore, they have set out to develop a non-linear strength criterion which will show good agreement.

While it would appear that part of the reason for the development of the criterion was for use with anisotropic rocks (the subject of another paper by Ramamurthy and different co-workers, and discussed below), the criterion should also be applicable to intact isotropic rocks. This latter application is examined in the first part of the first paper.

The criterion itself has the following form

$$\frac{\sigma_1 - \sigma_3}{\sigma_3} = B\left(\frac{\sigma_c}{\sigma_3}\right)^\alpha \qquad (1)$$

where σ_1 and σ_3 are the major and minor principal stresses at failure, σ_c is the uniaxial compressive strength, B is a rock material constant which is a function of rock type and quality, and α is the slope of the line between the two stress ratios on a log-log plot. However, before examining the application of the criterion to actual test results, an inspection of its general implications should be made. The first point to consider is what prediction is made for uniaxial strength. By rearranging the terms of the criterion, the following expression is obtained.

$$\sigma_1 = \sigma_3\left[B\left(\frac{\sigma_c}{\sigma_3}\right)^\alpha + 1\right] \qquad (2)$$

For any values of B and α, Eq. (2) should predict that for $\sigma_3 = 0$, $\sigma_1 = \sigma_c$ where σ_c is the uniaxial compressive strength. However, for the values of B and α suggested, it would appear that as $\sigma_3 \rightarrow 0$, $\sigma_1 \rightarrow 0$. Such a prediction must limit the applicability of the suggested criterion, particularly for regions of low confining pressures.

Further, by making $\sigma_1 = 0$, σ_3 becomes the uniaxial tensile strength. It should be possible therefore to make a prediction of the uniaxial tensile strength in terms of the uniaxial compressive strength, B and α as follows:

$$\frac{\sigma_c}{\sigma_t} = \left(-\frac{1}{B}\right)^{\frac{1}{\alpha}} \qquad (3)$$

However, in general, for a value of $\alpha < 1$, Eq. (3) has no real solution.

On the basis of these simple applications, it would appear that the criterion shows certain anomalies which must limit its application, especially in regions of relatively low stress.

The authors then go on to apply the criterion to the results of a series of high pressure triaxial tests on four types of Indian sandstone, as well as to a large number of similar tests conducted by others on a range of different hard rocks. It was found that the value of B for all rock types ranged between 1.8 and 3.0, with sandstone yielding a value of 2.2. The value of α was found to be effectively a constant for all rock types and numerically equal to 0.8. Using these quoted values of B and α for the sandstone, the reporter has used the criterion to predict the strength envelopes of the four sandstones tested and compared the results with the envelopes actually measured. The results of this exercise are shown in Fig. 3 which has been redrawn from the original paper. It will be seen that for the strongest three rocks, the envelope falls below the experimental points, particularly for the Kota sandstone, and particularly for low confining pressures. For the weakest of the rocks tested, the envelope agrees well with the experimental points. This overall comparison seems to be different from the results quoted in the original paper where the agreement between the actual and predicted results is virtually perfect throughout.

The reason for this discrepancy is not clear, but it is suspected that in the original paper, the authors used the specific values of B and α, derived for each rock type rather than the general values of B from their Table I and α of 0.8. If this were the case, then it is not surprising to find perfect agreement.

By way of comparison, Fig. 3 also presents the envelope predictions according to the criteria for intact rock as proposed by Hoek and Brown (1980) and Johnston (1985a). For the Hoek and Brown criterion, the constant m has been taken as 15. It will be seen that both of these criteria are in very good agreement with each other for all of the sandstones but both seem to under-estimate the envelope for the strongest rock and over-estimate for the weakest. It should be noted that the original paper also claims to have plotted the envelopes for the respective rocks according to the Hoek and Brown criterion. However, the reporter is unable to explain why these are significantly different from those Hoek and Brown envelopes shown in Fig. 3. Since virtually no details of testing arrangements, procedures nor results are given in the original paper, the reporter can offer no explanation to account for the differences between the predictions of the three failure criteria considered and the results actually quoted.

What is a little surprsing, however, is that in the paper, the Hoek and Brown envelopes appear to intersect the $\sigma_3 = 0$ axis at values significantly greater than the uniaxial compressive strengths. These envelopes should pass through the uniaxial compressive strength points exactly.

4.2 Anisotropic Rock

In the same paper by Ramamurthy, Rao and Rao, an extension of the strength criterion for intact rock to anisotropic rock is introduced. This extended criterion takes the following form

$$\frac{\sigma_1 - \sigma_3}{\sigma_3} = B_j\left(\frac{\sigma_{cj}}{\sigma_3}\right)^{\alpha_j} \qquad (4)$$

where, in addition to the terms defined previously, σ_{cj} is the uniaxial compressive strength for a given anisotropic orientation, and B_j and α_j are the values of B and α for the same orientation. According to the paper, for most anisotropic rocks, the value of B_j is "very close" to B and α is "around 0.8". Indeed, it is stated that "by using the suggested value of B for intact rock and assuming $\alpha_j = 0.8$ the strength of anisotropic rocks can be predicted reasonably well provided that σ_{cj} is used in the prediction". The authors present a comparison between experimental results obtained with Penrhyn slate and those predicted by the criterion to demonstrate its applicability. However, although introduced in this first paper, it is the second paper by Ramamurthy, Rao and Singh which examines the criterion for anisotropic rocks in some detail.

The second paper shows how the anisotropic criterion of Eq. (4) is applied to four different rocks; namely quartzitic phyllite and carbonaceous phyllite (both these being tested by the authors), and Barnsley hard coal and Penrhyn slate (these test results obtained from others).

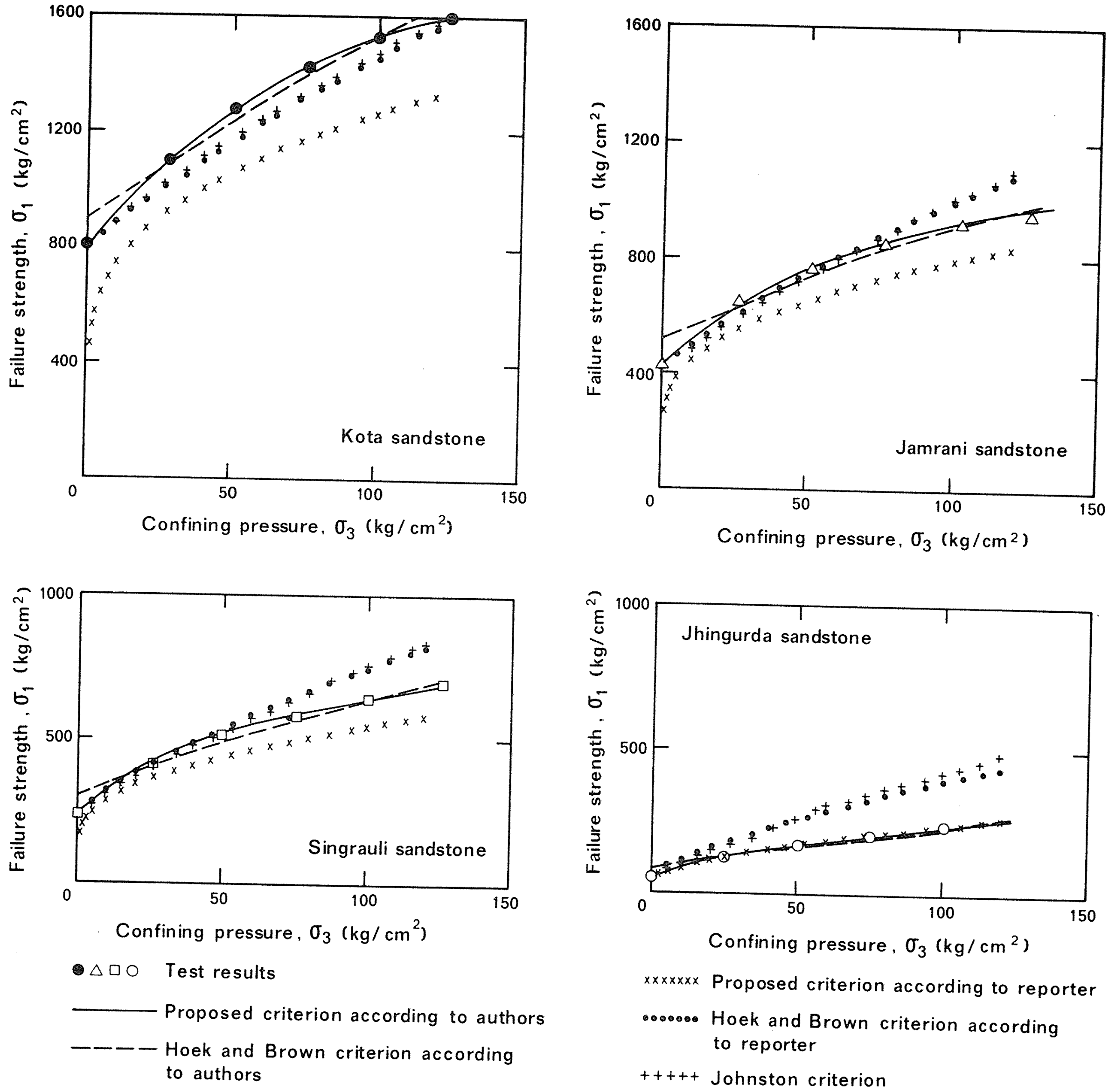

Fig. 3 Comparison of various strength criteria with test results

It would appear that in this second paper, the values of B_j and α_j vary systematically with σ_{cj} as a function of β, which is the angle between the planes of foliation and the major principal stress direction. Relationships are given so that both of the particular values of B_j and α_j can be estimated according to the orientation of the foliations. This would appear to be contrary to statements made in the first paper, where it was implied that both B_j and α_j could be taken as constants.

The reporter has examined the net result in predictions obtained by using the constant and variable B_j and α_j approaches for the quartzitic phyllite confined at 30 MPa. For the constant approach, values of B_j and α_j of 2.6 and 0.8 respectively have been applied to Eq. (4) for the prediction of failure stress, σ_1. For the variable approach, the values of B_j and α_j, given in the authors' Table I, have been used to predict σ_1. A comparison of the result is shown in Fig. 4 and it may be seen that there are only minor differences between the two approaches. This same comment generally appears applicable to all the test results presented in the paper. Therefore, at least as a first approximation, it seems reasonable to suggest that B_j and α_j can each be taken as constants.

The comparison actually provided between observed and predicted test results for the four different rocks at a range of confining pressures indicate that the strength criterion gives reasonable results. However, as has been noted above in connection with the results quoted for intact rock, there may be difficulties in the prediction of anisotropic strength for regions of low confining pressures. It must be emphasised that in order to obtain reasonable predictions of anisotropic performance under elevated confining pressures, it is necessary to obtain meaningful values of uniaxial compressive strength for a range of sample orientations. Although this need is merely stated in the papers, it is a task which requires careful consideration with regard to both the procedures adopted and interpretation applied. The problems involved with these aspects of testing will be discussed further in a later section.

4.3 The Applicability of Strength Criteria

The above discussion has been directed at an empirical failure criterion for isotropic and anisotropic rocks as has been suggested by Ramamurthy and co-workers. It would appear, however, that this criterion has some limitations especially with regard to the prediction of strength at relatively low confining pressures. Mention has also been made of two other strength criteria, both of which are applicable to rocks. The Hoek and Brown criterion, however, although it can be extended to describe the strength of rock masses, seems restricted to relatively hard rocks (Johnston, 1985b). The Johnston criterion, on the other hand has not yet been extended to rock masses, but appears capable of predicting the strength of all intact geotechnical materials from hard rock, through soft or weak rock, down to lightly over-consolidated clays. Therefore, both of these latter criteria have some limitations on the range of applicability.

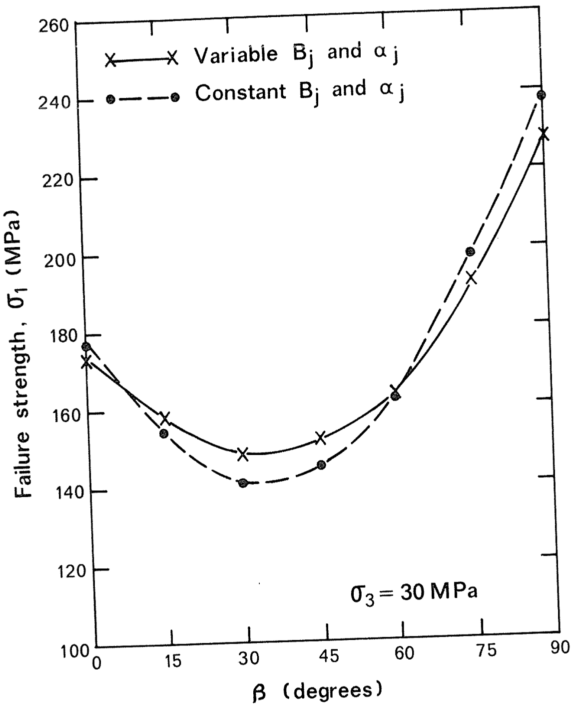

Fig. 4 Predicted strength of quartzitic phyllite using variable and constant values of B_j and α_j for a confining pressure of 30 MPa.

There are of course, many other such criteria to be found in the literature, including the well known Mohr-Coulomb criterion which has proven so popular in geotechnical engineering. These various criteria have a great many forms, limitations and ranges of applicability. However, with so many criteria available, there must be some difficulty in choosing one which is applicable to a given engineering application. Therefore, in order to assess the relative merits of these criteria, the third "key question" is - if it is accepted that the perfect failure criterion will never exist, what characteristics should be shown by one that is at least highly acceptable? While it is appreciated that strength for a given material, stress range, stress direction and defect content must be predicted with acceptable accuracy, to what extent is it desirable that one criterion should cover a wide range of materials (perhaps to include all geotechnical materials ranging from soft soils to hard rocks), for all stress ranges including brittle and ductile behaviour, for tensile and compressive stresses, and for all defect contents ranging from intact to completely fractured?

5. LABORATORY TESTING FOR STRENGTH

5.1 General

In the development of empirical strength criteria for geotechnical materials, it is important that the data used to prove any given criterion is a true reflection of the materials' engineering strength and is not influenced by the test technique employed. The triaxial compression test is the most common form of test used to obtain this strength data. The uniaxial compression strength is also commonly used both as a specific form of triaxial test result and as a means of normalising the principal stresses at peak strength as recorded in triaxial tests. Therefore, it is perhaps appropriate to consider some of the factors which influence the measured triaxial and uniaxial compressive strengths of rocks, particularly the weak variety since they are the main subjects of this session.

5.2 Uniaxial Compressive Strength

When rocks are subjected to uniaxial compressive strength testing, a commonly observed mode of failure consists of axial splitting as shown in Fig. 5(a). The failure mode which should occur under uniaxial compressive conditions is the development of a shear plane inclined to the sample axis as shown in Fig. 5(b). The axial splitting mode may be caused by platen friction on the ends of the sample, and this induces the development of tensile stresses within the test sample (Vutukuri et al., 1974). Because the tensile strength of a rock is significantly smaller than its compressive strength, the result yielded by the test, although apparently conducted under uniaxial compressive conditions, can be significantly less than the true uniaxial compressive strength. This phenomenon was investigated in some detail by Choi (1984), where it was found that failure modes similar to Fig. 5(a) consistently produced results significantly smaller than did failure modes similar to Fig. 5(b). It follows that care should be taken in observing the failure modes of uniaxial compressive tests so that some assessment of the likely true uniaxial strength may be made. Clearly this possible under-estimation of uniaxial compressive strength could significantly influence predictions of strength under elevated confining pressures.

Another point which should be given very careful consideration when planning uniaxial compressive strength testing, particularly for weak rocks, is the rate at which the loading is applied. The ISRM suggested method (Brown, 1981) states "load on the specimen shall be applied continuously at a constant stress rate such that failure will occur within 5-10 min. of loading, alternatively the stress rate shall be within the limits of 0.5 - 1.0 MPa/s". With regard to the alternative rate, while it may be relevant to very strong rocks where generated pore water pressures may be insignificant, when dealing with a weak rock of uniaxial strength of, say, 5 MPa, a test duration of between 5 and 10 seconds is clearly not appropriate.

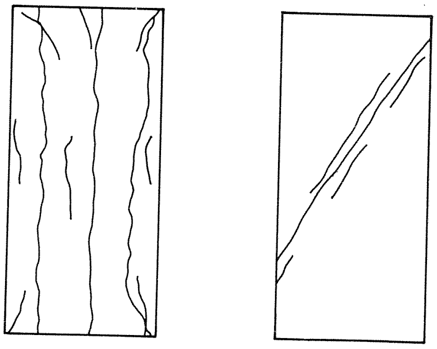

Fig. 5 Failure modes in uniaxial compression tests; (a) axial splitting, (b) shearing

Even for a suggested test duration of 5 to 10 minutes, the pore water pressures developed could be of a similar order to the strength and therefore have a considerable influence on the measured strength. In order to adopt an appropriate rate for this testing, it is suggested that the methods recommended by Bishop and Henkel (1964) and Chiu et al. (1983) are considered.

On the more positive side, it would appear that for weak rocks, ISRM standards for sample preparation may be too stringent. Pells and Ferry (1983) have demonstrated for the Hawkesbury sandstone of Sydney that length to diameter ratios of down to 2.0 are acceptable, that non-parallel ends of up to 2° make no significant difference to results and that adequate end flatness can be achieved by means of a good quality diamond saw without the need for surface grinding.

5.3 Triaxial Strength

With regard to the determination of strength of weak rocks in the triaxial equipment, the principal source of error arises from pore water pressures generated during testing. The ISRM standard for triaxial tests on rocks suggests that failure is achieved in 5 to 15 minutes of loading. Again, while such a rate may be appropriate for hard rocks in which generated pore water pressures may not be significant, for weak rocks, especially those of slow drainage characteristics, such a rate can have a most pronounced effect. For example, Fig. 6 shows the effect of strain rate on the recorded peak deviator stress of a series of similar samples of Melbourne mudstone (uniaxial compressive strength of about 2 MPa), tested under an effective initial confining pressure of about 3.5 MPa with one end of the samples permitted to drain.

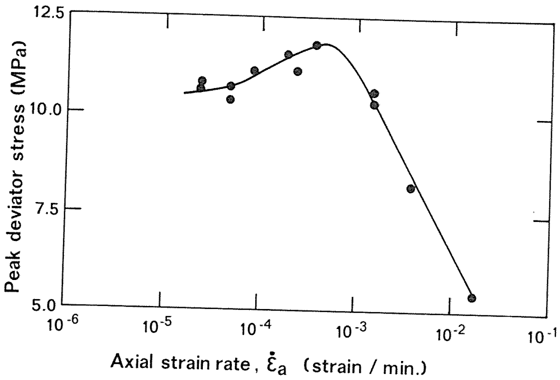

Fig. 6 Effect of axial strain rate on recorded peak deviator stress of a mudstone

This figure shows that the true deviator stress under this confining pressure is a little over 10 MPa and could only be determined if testing took several hours. However, when testing was completed in about 10 minutes, the recorded deviator stress approached 5 MPa. The reason for this apparent dramatic drop in strength was because of the large positive pore water pressures developed during rapid testing. These pressures reduce the effective confining pressures to produce a significant apparent reduction in strength.

Based on the above discussion, it would appear important that if a true indication of strength is to be obtained from triaxial testing, then due consideration of developed pore water pressures must be taken into account. It is suggested that the recommendations of Bishop and Henkel (1964) and Chiu et al. (1983) are again considered when such testing is being planned.

5.3 Future Developments

The above discussion has highlighted a few of the factors which should be taken into consideration when developing a test programme for the laboratory determination of the strength of weak rock. The principal concerns have related to the influence of loading rate on developed pore water pressure which in turn may significantly influence recorded strength. In addition, the effect of tensile splitting on the measured uniaxial compressive strength has also received some attention. However, it would be reasonable to comment that it is only relatively recently that weak rock has received some detailed rational attention and therefore, there still must be much to learn.

This then leads into the fourth and final "key question". Bearing in mind the transitional nature of weak rock as it lies somewhere between the more traditional soft soils and hard rocks, what lessons can we learn from the experiences of strength determinations commonly employed in rock and soil mechanics, and how can these be translated into the very real problem of estimating the strength of weak rock masses?

6. CONCLUDING COMMENTS

Despite the small number of papers assigned to this session on the geotechnics of weak and jointed rock, the reporter has reviewed these papers with the intention of extracting four "key questions" which may act as catalysts for further discussion.

These questions, as they apply to weak and jointed rock, are summarised as follows:

1. When can laboratory derived moduli values be used for predictions of field deformation?

2. What other means of field modulus determination are relevant?

3. What characteristics and range of application should an acceptable strength criterion display?

4. How can the collective experiences of both soil and rock mechanics be applied to the estimation of material strength?

It is hoped that these questions may stimulate some discussion in this session.

However, before finishing this report and opening the floor to general discussion, the writer would like to make some general observations which may assist in placing the problems associated with weak rocks into perspective.

Weak rocks are often considered to fall into a nebulous category of geotechnical materials which to the soil mechanics practitioners is somewhere at or beyond the limit of materials traditionally considered. To rock mechanics engineers, weak rocks tend to be regarded as appearing at the lower extreme of the strength of materials considered. The net result is that little effort has been made to understand their behaviour. Engineering problems for these materials have generally been approached by significant extrapolations from the separate traditional engineering sciences of soil mechanics or rock mechanics. This has led to many misconceptions of behaviour and, understandably, a very conservative approach to design.

It is the writer's considered opinion that weak rocks are not part of a separate category of geotechnical materials but are central to the complete spectrum ranging from clays and sands to hard rocks. It is believed that the principles and physical laws of both soils and rock mechanics apply equally to weak rocks. Therefore, for progress to be made in the understanding of the geotechnics of weak and jointed rock, it is imperative that a rational approach be made to this study from both ends of the geotechnical spectrum with a full appreciation of the principles involved.

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