PROPOSALS FOR A FAILURE CRITERION APPLICABLE TO DISCONTINUOUS ROCKS

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Abstract The existing failure criteria to evaluate discontinuous rocks are predominatly based on the relations between principal stresses at failure. None of the criteria has a direct reference to the magnitude of deformation as a major parameter in rock. Several reasons are identified indicating that the selection of peak stress as a major parameter, which is obtained through the evaluation of a failure criterion, is not reliable. On the other hand, the experimental investigations indicate that the state of strain is equally important to the state of stress. The proposed failure criterion is to describe the behavior of a jointed rock mass under various loading conditions taking the major principal strain into consideration.

Key Words Criteria, Discontinuity, Failure, Joint, Rock, Stress, Strain

چکیده معیارهای موجود برای توصیف رفتار و ارزیابی سنگ های ناپیوسته و پیش بینی خرابی و شکست آنها بر پایه معادلات و روابط بین تنش های اصلی است. در هیچیک از معیارها، عاملی که اثر مستقیم جابجایی و تغییر شکل را در چگونگی خرابی و گسیختگی سنگ‌های ناپیوسته نشان دهد، وجود ندارد. دلایل متعددی در این مقاله ارائه می‌شوند که نشان می دهند درنظر گرفتن تنش نهایی بعنوان عامل اصلی ارزیابی خرابی و شکست سنگ های درزدار و ناپیوسته، که معیارهای فعلی بر این اساس است، کافی و جامع نمی باشد. تحقیقات آزمایشگاهی نشان می دهند که مقدار تغییر شکل و جابجایی در سنگ های ناپیوسته به همان میزان تنش نهایی در پیش‌بینی رفتاری این نوع توده های سنگی، از اهمیت برخوردار است. در این تحقیق، مشکلات معیارهای موجود مورد بحث قرار می گیرند و بر پایه تحقیقات انجام شده توسط مؤلف، عوامل لازم برای مدل و معیاری که قادر باشد رفتار توده های سنگی درزدار و ناپیوسته را ارزیابی نموده و چگونگی خرابی و شکست آنها را پیش‌بینی نماید، پیشنهاد گردیده و مورد بررسی قرار می گیرند.

INTRODUCTION

For the design of structures in rocks, availability of a criterion capable of assessing the rock mass characteristics has always been a major task in rock mechanics, even when the rock mass has been predominantly a competent rock. The problem has been much more significant, however, when the rock mass contains planes of weakness, or is highly jointed or disintegrated. Consequently, various criteria have been proposed by different authors to evaluate the anisotropic and jointed rocks. Nearly all these criteria are based on a peak strength as the major parameter while deserving attention has not been paid to the magnitude of deformation and control of strain throughout the rock [e.g. the Walsh and Brace (1964) theory, Hoek's (1964) criterion, theory of single plane of weakness and continuously variable shear strength theory, Jaeger (1960), Ashour (1988) and also empirical failure criteria such as Hobbs (1966), Bieniawski (1974), Hoek and Brown (1980), and so on].

PROBLEMS WITH THE EXISTING FAILURE CRITERIA FOR DISCONTINUOUS ROCKS

A considerable number of research has been conducted to establish a versatile and comprehensive failure criterion. Although, they have succeeded in explaining many aspects of rock behaviour, they have failed to explain issues such as long-term behaviour, stress relaxation effects, etc. Jaeger and Cook (1979) justifiably believe that failure criteria are based on the actual mechanism of fracture, which are more sophisticated than the theories offered by Coulomb, Mohr and Griffith's, have yet to be developed.

Results of the tests performed by Fahimifar (1990) reveal that the amount of sliding along the joint surface is a fundamental parameter in jointed rock, and in fact, before obtaining any quantitative peak strength, this parameter would have to be considered and controlled. On the other hand, neither theoretical nor empirical failure criteria have appreciated the effect of some very significant parameters such as time dependency, and stress relaxation associated with a structure where it is established on or in a rock mass.

There are several reasons that the selection of peak stress as the major parameter which is obtained by the evaluation of a failure criterion is not safe and reliable.

a) In rough joints, the magnitude of displacement along joint surface affects the strength of the rock significantly. Figure 1 illustrates the stress-strain plots for a sandstone with natural joints and an orientation angle of 60° for confining pressures of 5, 15 and 30 MPa. The amount of peak strain differs in each curve, particularly the dependency of strength on the strain around the peak is very high, so that with a little increase in strain, the strength decreases about 28.5%, 59% and 44% for 5, 15 and 30 MPa confining pressure, repectively. Increase of strain, for instance, at 15 MPa confining pressure from 0.8% to 0.9% decreases the strength more than 28%, which is quite considerable. Therefore, the use of a failure criterion on the basis of peak stress irrespective of displacement will result in a misleading design approach in such a situation.

b) There are cases in jointed rocks, especially when the surface roughness is relatively high, that more than one peak in the stress-strain curve at different percentages of strains are observed. Figure 2 illustrates the stress-strain plot for a specimen containing a single joint with rough surface and an orientation angle of 45°. Two peaks at different amounts of strain are clearly observed and the second peak shows a higher magnitude in stress. Selecting any of the criteria referred to as the appropriate failure criterion necessitates that the stress at the second peak be chosen as the strength because of its higher magnitude. However, the amount of strain is about 0.7% more than the first peak.

Using a criterion for the design of a structure in such a situation (the maximum accepted level) without determining the upper limit of the strain, one can neither control nor predict the behaviour of the

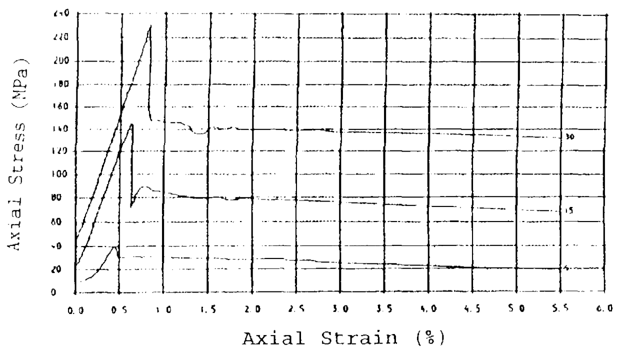

Figure 1. Stress-strain curves for Penrith sandstone specimens with natural joints, joint angle = 60 Deg confining pressures = 5, 15 and 30 MPa.

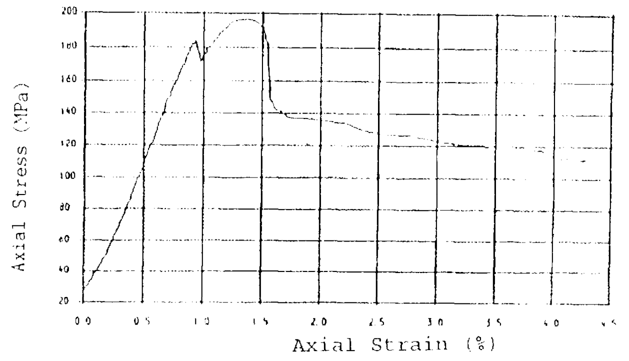

Figure 2. Stress-strain curves for Penrith sandstone specimen with natural joints, joint angle = 45 Deg, confining pressure = 15 MPa.

surrounding rock.

c) The experimental investigation revealed that the effect of time on the stress-strain characteristics of jointed rock is very significant, and therefore, selecting a failure criterion on the basis of peak stress will not meet the necessary requirements. Figure 3 illustrates the stress-strain plots for the speciments containing planes of weakness with rough surfaces and an orientation angle of 45°. The first plot shows the stress-strain curve for a specimen tested in the conventional way (applying a constant strain rate throughout the test). The second plot shows the stress-strain curve for a specimen with the same joint orientation and under the same confining pressure, but with changing strain rates at different points during the test. Comparison of the two plots reveals that the effect of strain rate on the magnitude of peak stress and also on the magnitude of strain is considerable. For instance, the peak stress in the first curve shows a magnitude more than 220 MPa with a strain about 1%. However, in the second curve the peak stress has decreased to about 185 MPa with a strain of about 1.75%.

d) Increase or decrease in stress within a rock mass may affect the stress-strain characteristics of the rock. Figure 4 illustrates the stress-strain plots for the specimens containing rough surface joints with an inclination angle of 45° and the same applied confining pressures. The first plot shows the stress-strain plot for the specimen in the conventional procedure, but the second one shows the stress-strain behaviour for the same specimen in which a cyclic loading (increase and decrease of stress at different points on the stress-strain curve) has been applied on the specimen. The stress and strain at peak for two cases differ significantly.

e) The experiments performed show that the effect of stress relaxation on the stress-strain characteristics of rock is significant. Figure 5 illustrates the stress-strain plots for two jointed specimens with an inclination of 45° for the same confining pressures. Plot 1 shows the stress-strain curve obtained in the conventional way, whereas, Plot 2 shows the stress relaxation at different points

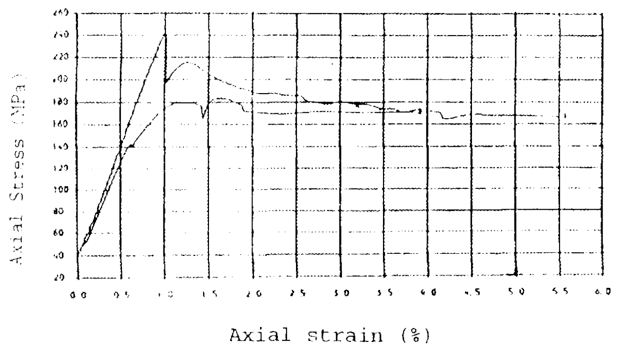

Figure 3. Stress-strain curves for Penrith sandstone specimens with natural joints, joint angle = 45 Deg Plot 1, constant strain rate, Plot 2, changing strain rates, confining pressure = 30 MPa.

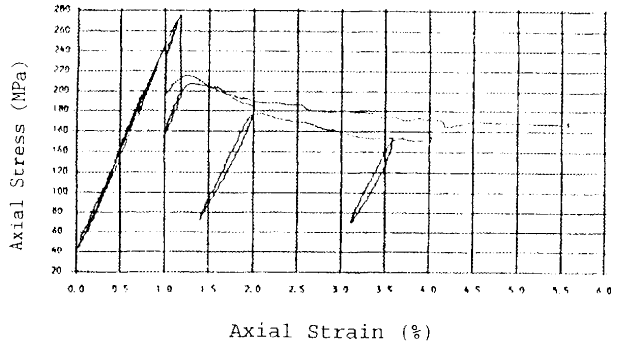

Figure 4. Stress-strain curves for Penrith sandstone specimens with natural joints, joint angle = 45 Deg. Plot 1, constant strain rate, Plot 2, cyclic loading, confining pressure = 30 MPa.

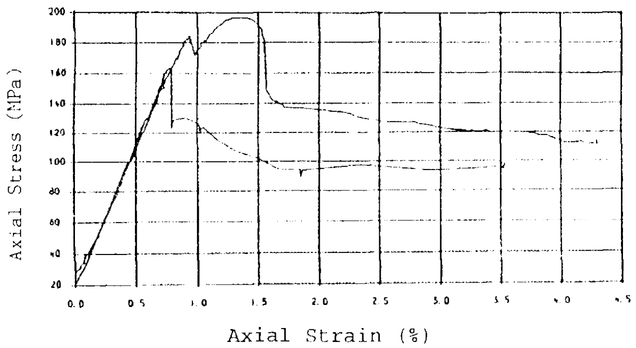

Figure 5. Stress-strain curves for Penrith sandstone specimens with natural joints, joint angle = 45 Deg Plot 1 ordinary test, Plot 2, relaxation test, T, Confining pressure = 15 MPa.

on the stress-strain curve at a constant amount of strain for the relaxation time duration of 5 minutes. The resultant stress-strain curve obtained in this case is entirely different from that of Plot 1 either in the magnitude of peak stress or strain in the two plots.

All of the cases referred to above, prove that selection of the peak stress as the basis of the failure criteria in discontinuous rocks is not adequate and reliable. Therefore, it is essential to develop a criterion on the basis of a parameter in addition to peak stress which would be capable of covering a wider area and would be adequately reliable for design purposes. Figures 6 and 7 illustrate the stress envelopes for jointed specimens with orientation angles of 45° with smooth (saw cut) and rough (natural) joint surfaces. In each figure several curves are observed, each of which belongs to a certain magnitude of strain ranging from 0.25% to 2%. The peak stress envelope has been also plotted in each figure. From the figures the significance of the strain magnitude through structure is clearly understandable. These figures reveal, for instance, that when in a rock selecting a criterion in which the stress at peak is the major controlling parameter, is unreliable. However, when the maximum allowable strain pertinent to the structure is determined, the related stress envelope is obtained.

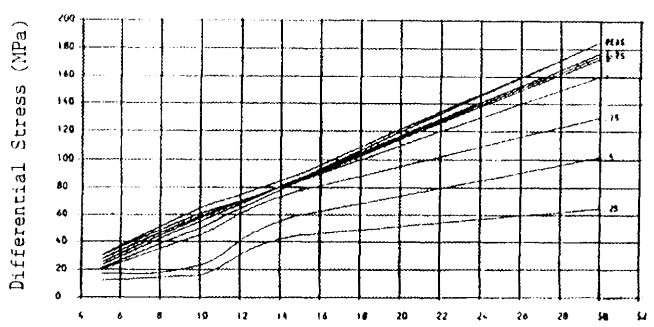

Figure 6. Differential stress-confining pressure envelopes at different axial strains for Penrith sandstone specimens with saw cut joints, joint angle = 45 Deg.

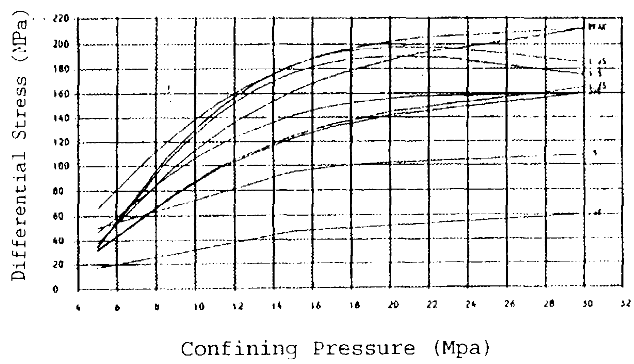

Figure 7. Differential stress-confining pressure envelopes at different axial strains for Penrith sandstone specimens with natural joints, joint angle = 45 Deg.

PROPOSALS FOR A FAILURE CRITERION

The currently available criteria for discontinuous rocks employ the peak stress as the major controlling parameter disregarding the influence of the associated strain. A failure criterion on the basis of the experimental investigations carried out by Fahimifar (1990) is proposed to describe the behaviour of a jointed rock mass under various loading conditions and took into consideration the major principal strain.

This failure criterion is able to accomplish the following two goals.

(a) To Support a Sliding Mode of Failure

It was shown by Fahimifar (1990) that in jointed specimens in the range of the critical joint orientations, i. e. 45°-65°, sliding movement along the joint is the predominant mode of failure. In this range of inclination the effect of the joint on both stress and strain is significant particularly when a great reduction in peak stress is observed and the stability of the jointed rock is controlled by the magnitude of strain. For this reason in the proposed criterion, the maximum axial strain at which sliding is initiated over the joint is taken as the major controlling parameter.

(b) To Take Into Account the Applied Loading Conditions

Fahimifar (1990) also showed that several parameters such as strain rate, stress relaxation and change of strain rate during the loading period affect the stress-strain behaviour of rocks. In order to optimize all these effects, the criterion has been based on the data obtained through different loading conditions investigated by Fahimifar (1990).

Therefore, to start with, the failure criterion should be of the general form:

$$\sigma_1 - \sigma_3 = f(\varepsilon_1) \tag{1}$$

where,

σ_1 is the major principal stress,

σ_3 is the minor principal stress, and

ε_1 is the major principal strain.

For confining pressures ranging between 5-70 MPa, the deviatoric stress and its corresponding axial strain at which sliding commenced over the joint were calculated for the case of saw cut joints with 60° orientation which correspond to the most critical angle. A least square regression analysis was undertaken to fit the best curve on the values for a sandstone giving an R value (coefficient of correlation) of 0.82 which combined with the trend of data.

The relationship obtained from the curve-fitting was of the from:

$$\sigma_1 - \sigma_3 = \frac{\varepsilon_1}{-0.0171\varepsilon_1 + 0.035} \tag{2}$$

The fitted curve to the data is seen in Figure 8 (Plot 60S). In order to achieve a comprehensive solution and to derive a criterion applicable to jointed rock masses with a wide range of joint orientation and joint surface roughness, similar curves were fitted to the data for 45° and 60° orientatios for both smooth (saw cut) and rough (natural) jointed specimens (Figure 8 plots 45S, 45N and 60N).

A curve for intact specimens of Penrith sandstone was also fitted to the data for the range of 0-70 MPa confining pressure (Figure 8 Plot INTACT). As is observed in Figure 8, there is no common trend between the curves, since some are concave upward while others are downward. The curve for 60° orientation for a saw cut joint (Plot 60S) and the first part of the curve 45° orientation with saw cut joint (Plot 45S) are concave upward and the rest of the curves for both intact and jointed specimens are concave downward. This implies the following.

(i) The stress-strain behaviour of jointed rocks is affected by the magnitude of confining pressure. For instance, Plot 45S (Figure 8) at low confinements (below 15 MPa) is concave upward, but at higher confining pressures (more than 15 MPa) the curvature has changed downward.

(ii) The stress-strain behaviour for the joints with low and high degrees of surface roughness is different. The curve for saw cut joints (lower surface roughness) are concave upward; for the natural joints (rough sliding surface), however, they are downward.

(iii) The stress-strain behaviour of jointed rocks differs with that of the intact rock significantly.

Two distinct behaviours are identified in Figure 8. The first corresponds to the intact specimens, split joints, and the upper part of the plot belongs to 45° saw cut joints in which all the plots have the same trends. They are concave downword. The second behaviour corresponds to 60° saw cut joints, and the first part of the plot belonging to 45° saw cut joints in which all the plots are concave upward. In the first group, the stress increment is decreased as strain increased; however, in the second group the stress increment is increased with increasing strain. Comparing the two groups it may be concluded that at a much higher confining pressure the direction of the plot corresponding to 60° saw cut joint will change from upward to downword and it wil become similar to the plot belonging to 45° saw cut joint. Such a level of confining pressure may be termed as "transition pressure" in which the effect of joint through the rock mass is not so critical and becomes

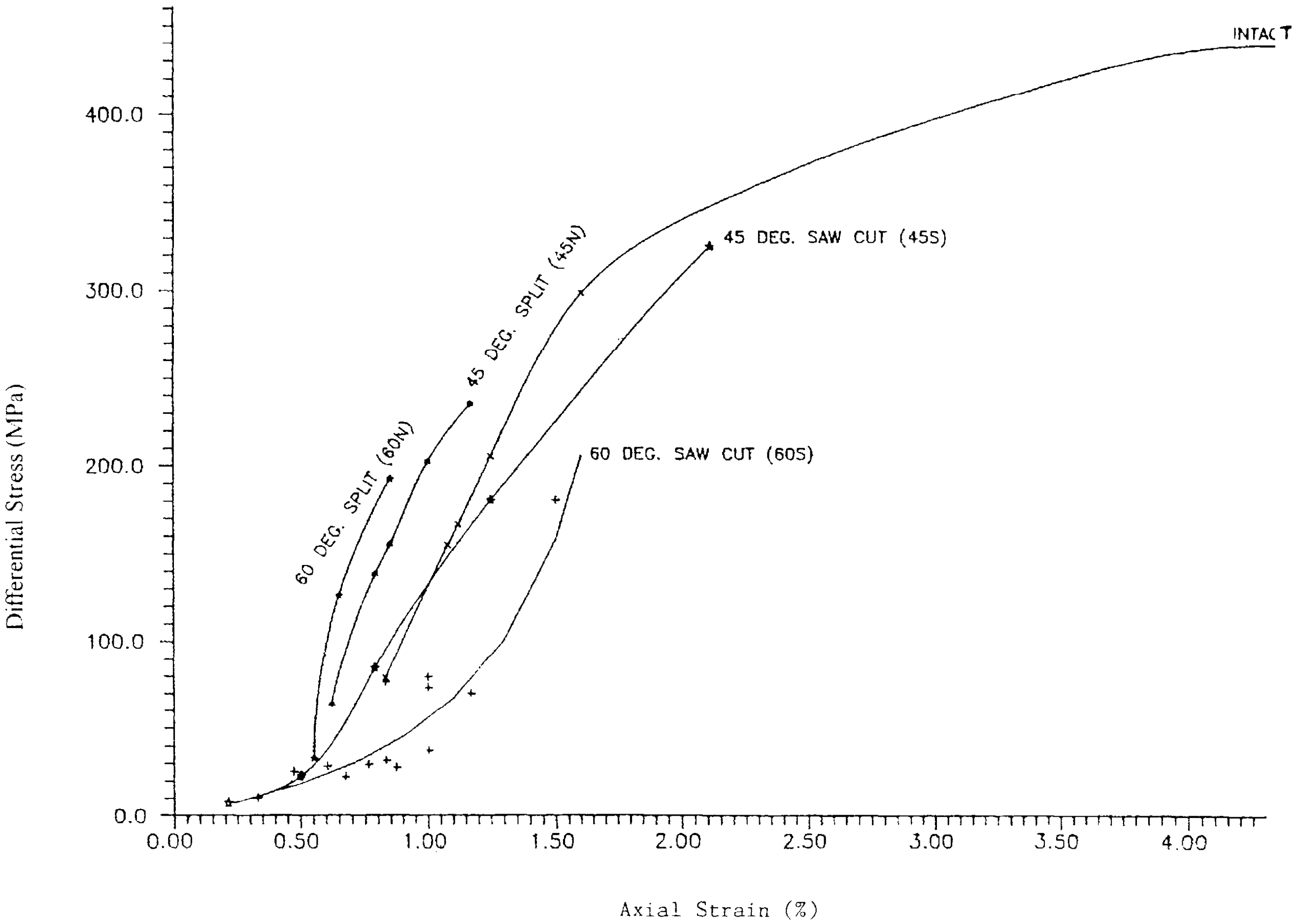

Figure 8. Differential stress-axial strain Plots for Penrith sandstone specimens at various confining pressures.

underground structures in rocks, the transition pressure may be related to a certain depth.

Therefore, the Relation 2 can not be representative of a jointed rock mass for different joint orientations and joint surface roughness. Furthermore, taking into consideration the differences in behaviour between lower and higher confinements, it seems that deriving a comprehensive criterion to assess rock mass behaviour, whether jointed or competent, requires further theoretical and experimental investigations in which several factors such as joint orientation, joint surface roughness, deviatoric stress, and mean normal stress to be incorporated.

CONCLUSIONS

Documentary evidence based on the experimental investigations shows that the failure criteria in which the maximum stress at failure is the major controlling factor is not adequate and reliable for design purposes. The most significant and controlling parameter seems to be the allowable and pre-determined magnitude of displacement (or percentage of strain) throughout the rock surrounding the structure. Consideration of the strain at failure (maximum allowable strain) results in obtaining the associated stress envelope under the real conditions of the rock structure. Then on the basis of the maximum allowable strain introduced through the rock mass the design parameters may be determined.

It seems that there is not a unique relation describing a jointed rock mass for different joint orientations and joint surface roughness.

Furthermore, for a range of low and high confining pressures, to derive a criterion for jointed masses should incorporate not only the deviatoric stress but also the other factors, such as joint orientation, joint surface roughness and mean normal stress.

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