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Post failure behaviour and a brittle-plastic model of brittle rock

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ABSTRACT: The experimental research of mechanical properties of rock is the basis for establishing reasonable constitutive models.

For brittle rocks, the crux of extablishing such numerical models is how to describe the mechanical properties in the post-failure range.

The complete curves can be obtained from the uniaxial compression test on rock specimens. Traditionally, the complete curves are divided into categories Ⅰ and Ⅱ, the latter representing the brittle rock.

On the basis of the newest test results, the author has proved that the traditional classification model is questionable, and proposed a new classification model of rock. The uniaxial compression tests on brittle rocks show a feature of great importance that the axial stress undergoes a process of abrupt fall after the peak strength, and at last the stresses fall on the envelope of residual strength. An ideal brittle-plastic constitutive model can be used for description of this vertical stress-drop.

It is known from triaxial tests that the confining pressure acting on rock specimens has influence on their post-failure behaviours. Though the stress-drop of brittle rocks at this time may remain steep, but is not vertical. On the basis of the aforementioned ideal brittle-plastic model, a generalized brittle-plastic model is presented, which can deal with the non-vertical stress-drop. The brittle-plastic model has already found its application in several major projects in China and good effects have been obtained.

1 INTRODUCTION

The experimental research of mechanical properties of rocks can provide a solid basis for the establishment of rock constitutive models.

It is know by a survey of constituttive models now in use that the majority of constitutive models are based on plasticity. However, there are a great variety of rocks which have different properties. It is evident that the properties of rocks can hardly be described only in terms of plasticity. For example, the granite and diorite are widely distributed common brittle rocks, which have to be dealt with in a considerable number of projects. Among the numerous mechanical properties the very important and special are the properties when the peak strength is passed, i. e., in the so-called "post-failure behaviour". The uniaxial compression tests on brittle rocks display that after the peak strength the axial stress-drop has an abrupt, even nearly vertical fall, a phenomenon which can not be described in terms of plasticity, and treated with the method for softening problems. The rational description of the post-failure behaviour is the key to the establishment of the constitutive models for brittle rocks. In the opinion of the author, the research in mechanical properties of brittle rocks in the post-failure range can not as yet be regarded as full and deep-going. At present, what is generally adopted in the international academic circles remains the classification model with types Ⅰ and Ⅱ, which are based on rock complete curves obtained from early tests.

The so-called curve of type Ⅱ representing brittle rocks is characterized by the fact that the decrease of the axial compression stress of brittle rock specimens after the peak strength is certainly accompanied by a decrease of the axial compression strain. It should be pointed out that the obtaining of these complete curves is connected with the early test conditions and the early test facilities, which are far from perfect. Now the great progress has been made in test technology and the deeper knowledge has been acquired on the rock mechanical properties, and it is time to have a deep analysis on the above-mentioned classification model and especially on the above-mentioned curve of type Ⅱ.

2 DISCUSSION ON THE EXISTING ROCK CLASSIFICATION MODEL

The whole axial loading displacement curve (or related axial stress strain curve) including the so-called curves of post-failure properties after the peak strength, obtained from the uniaxial compression test on the rock cylindrica specimen, is called the complete curve.

It is well known that for the common test machine it is difficult to obtain the complete curve from the uniaxial test. The obtaining of complete curves for brittle rocks by means of this kind of machines is entirely impossible. The reason is that, after the peak strength, the deformation energy released by the machine is more than needed for the expansion of the fractures in rock. Thus the local cracking will expand abruptly, without control, until the catastrophic failure takes place in the whole rock specimen.

From 1960's many efforts have been made to obtain the complete curves. The improved test technique falls into 3 kinds. The first kind belongs to the comparatively early period; in which the common test machine or rigid test machine are used, around the rock specimen there are placed pilars of large stiffness or auxiliary hydrolic cylinders parallel to the axis of the specimen. When the stress of rock abruptly drops after the peak strength and the cracking violently develops, the deformation energy released by the machine is absorbed by these pilars of large stiffness. In the case of hydraulic cylinders, the failure process in the post-failure range is controlled through the opposite force of compensatory character exerted by these cylinders.

In the second kind of technique, the test machine system of very high stiffness is employed. The stiffness, however, can not be raised without limit. This kind of machines should be specially manufactured, and are of no universality. In the last years there are some people, though not many in number, who are seeking for further improvement. Recently, the advances in this respect has been reported in Russia (Stavrogin & Tarasov 1995).

The third kind is servo-controlled test machine with the closed loop control model. It represents the main trend at present and the use of this test machine system is a great progress in the laboratory test technology of rock mechanics.

Though a great progress has been made in the test technology during the last thirty years, the rock classification model which has been used up to now remains the model presented in 1970.

2.1 *Rock classificaton model with type* Ⅰ *and type* Ⅱ

In 1970, Waversik and Fairhurst, on the basis of their results of uniaxial compression tests performed on six different kinds of rock, proposed that complete curves fall into two basic kinds as shown in Fig. 1. This classification later was further conceptualized and reduced to the rock classification model shown in Fig. 2, with the curve of type Ⅱ representing brittle rocks. The difference between rocks of type Ⅰ and Ⅱ in failure characters is stated by the proposer of the model as follows:

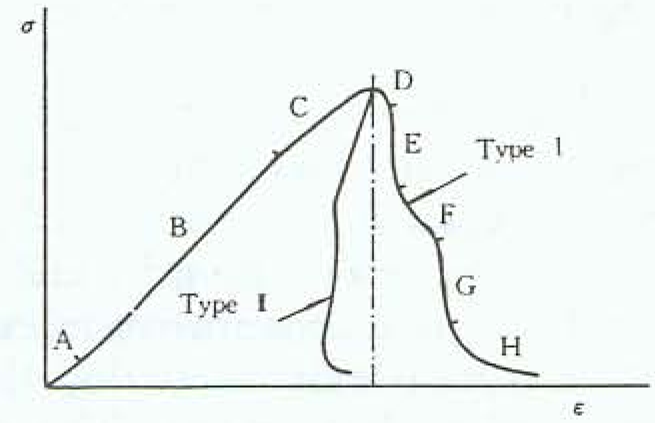

Figure 1. Complete curves of types Ⅰ and Ⅱ

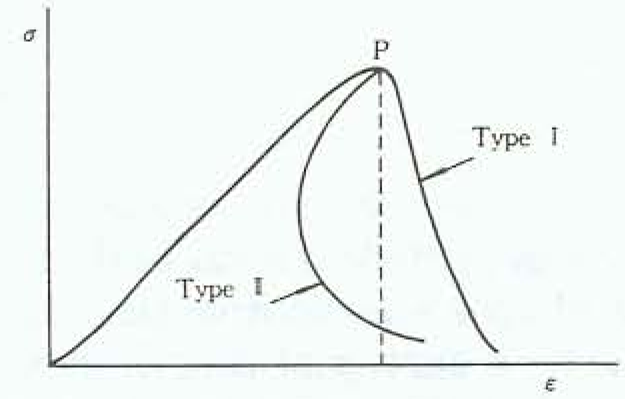

Figure 2. Rock classification model based on the complete curves of types Ⅰ and Ⅱ

In uniaxial quasi-static compression tests the complete stress-strain characteristics of the rocks studied here may be divided into two classes. For class Ⅰ, fracture proposgation is stable, in the sense that work must be done on the sample for each incremental decrease in load-carrying ability. For class Ⅱ, failure is unstable or self-sustaining; to control fracture, elastic energy must be extracted from the material. The fracture of rocks in class Ⅱ cannot be controlled even if a perfect rigid testing machine could be used (Wawersik & Fairhurst 1970).

2.2 *Discussion on the above classification model*

The rock uniaxial compression test, which forms the basis of the above-mentioned classification model is performed on the rigid machine with auxiliary hydraulic cylinders, this test technique belongs to the first kind. When the peak strength is passed, only under the action of the opposite force exerted by three auxiliary hydraulic cylinders placed parallel to the specimen, does the change of the deformation tendency take place that is to say, the axial compression strain diminishes with the decrease of bearing capacity. Such results are obtained in the then imperfect test conditions and their reliability and rationality are questionable.

In the recent years there are a considerable number of reports on the curve of type Ⅱ obtained by means of servo-controlled test machines. But these complete curve are obtained under the control condition that the circumferential deformation rate of the specimen is kept constant: This means that the specimen was kept in a process of repeated loading and unloading in its axial direction when the peak strength is passed. As for the axial deformation, there is also a complicated process of repeated lengthening and shortening. Under such complicated loading conditions, the curves of type Ⅱ can be no means be taken as the reflection of the basic properties for brittle rocks.

Theoretically speaking, since the complete curves describe the relation between the axial load (stress) and the axial deformation (strain) of the rock specimens naturally, it is most rational to take the rate of axial deformation (rate of strain) as control condition, namely $\dot{\varepsilon}_l$ = constant. In this condition, the axial deformation of the rock specimen is constantly in a state of monotonic uniform increase, therefore the curve obtained in this condition is a good reflection of the basic properties, and can be easily expanded for the more complicated conditions.

At present the constant rate of circumferential deformation is commonly adopted as the control condition in the compression tests on servo-controlled machine. The real cause is that the imperfection of automatic control leads to the test being out of control after the peak strength if the rate of axial deformation is taken as the control parameter.

In a standpoint of energy, so long as $\dot{\varepsilon}_l$ = constant is used as the control condition, energy is necessarily needed to put into rock specimen in the post-failure range and should by no means be withdrawn. This is diametrically different from the concept of classification model adopted at present, which holds that the failure process of the specimen of brittle rock (type Ⅱ) can by no means be controlled so long as energy is added in the post-failure range. All that is talked about leads to such a logical deduction: if the post-failure property curves of brittle rock can be obtained under the condition of $\dot{\varepsilon}_l$ = const, that is to say in the post-failure range, a fact that energy is added instead of being removed, and the control of the failure process is succeeded, then the view is proved that the above-mentioned curve Ⅱ is unable to reflect the basic properties of brittle rock, and the unrationality of the classification model with types Ⅰ and Ⅱ is also proved (Ge 1995).

In order that a rational classification model for rock is to be established, and the real post-failure curves of brittle rock is to be obtained, it is clear that the most essential is to obtain the complete curve on the servo-controlled test machine under the condition of $\dot{\varepsilon}_l$ = const. Our efforts in many years have resulted in a successful development of a new type of rock mechanics test machine system and new test results obtained.

3 TEST RESULTS UNDER CONTROL CONDITION OF $\dot{\varepsilon}_l$ = const AND A NEW ROCK CLASSIFICATION MODEL

Although the servo-controlled test machines are used at present in majority of the tests connected with complete curves of rock, a fixed correction link is adopted in most of test machine with closed loop servo-controlled model.

From the viewpoint of the closed loop control system, the test object is a main link in the closed loop, and its mechanical properties is closely related with the characteristics of the system. It is well known that rocks sharply differ from common engineering materials in two respects. First, rocks of different kinds are greatly different in mechanical properties. Second, the mechanical properties before the peak strength has a remarkable difference with those in the post-failure range even for the same rock specimen. Therefore, it is very difficult to obtain the complete curves for brittle rock under $\dot{\varepsilon}_l$ = const by means of the closed loop servo-controlled test machine with the fixed correction link.

3.1 *New rock mechanics test machine of RMT series*

After efforts of many years the new rock mechanics test machine of RMT series was successfully manufactured (Ge & Zhou 1994).

This is a test machine with advanced performance. The advanced self-adaptive control system is used. All the control functions are fulfiled by computer. In the course of the whole test the system identification and automatic search are performed. In other words, no matter how mechanical properties of the rock specimen change in the test, it is ensured that the system is always in the optimal control state. The whole system is highly digitized. On this test machine, many tests can be performed including uniaxial tests, triaxial tests, direct shear tests for weak surfaces (maximal shearing area: $200 \times 300mm^2$). Owing to the high frequency response of the machine, the uniaxial compression fatigue test on the cylindrical rock specimen with a diameter of 50mm can be conducted under cycling loading with multiple waveforms. With $\dot{\varepsilon}_l$ = const taken as control condition the complete curves of brittle rocks can be obtained through RMT test machine.

3.2 *New test results*

Many kinds of rock are used in uniaxial compression tests so that their complete curves can be obtained.

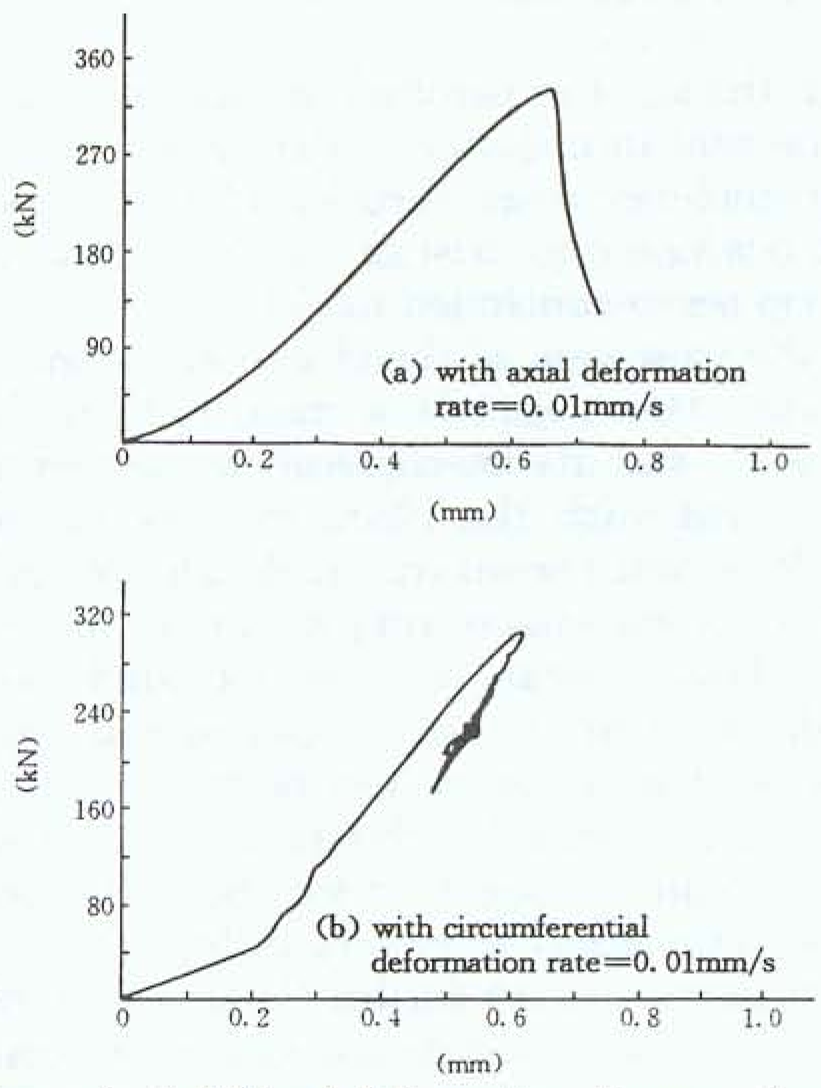

Figure 3. Axial load-deformation diagram of grey granite

For the study on the effect of the control condition on complete curves, two control modes for each rock are adopted for the purpose of comparison, namely: the constant rate of axial

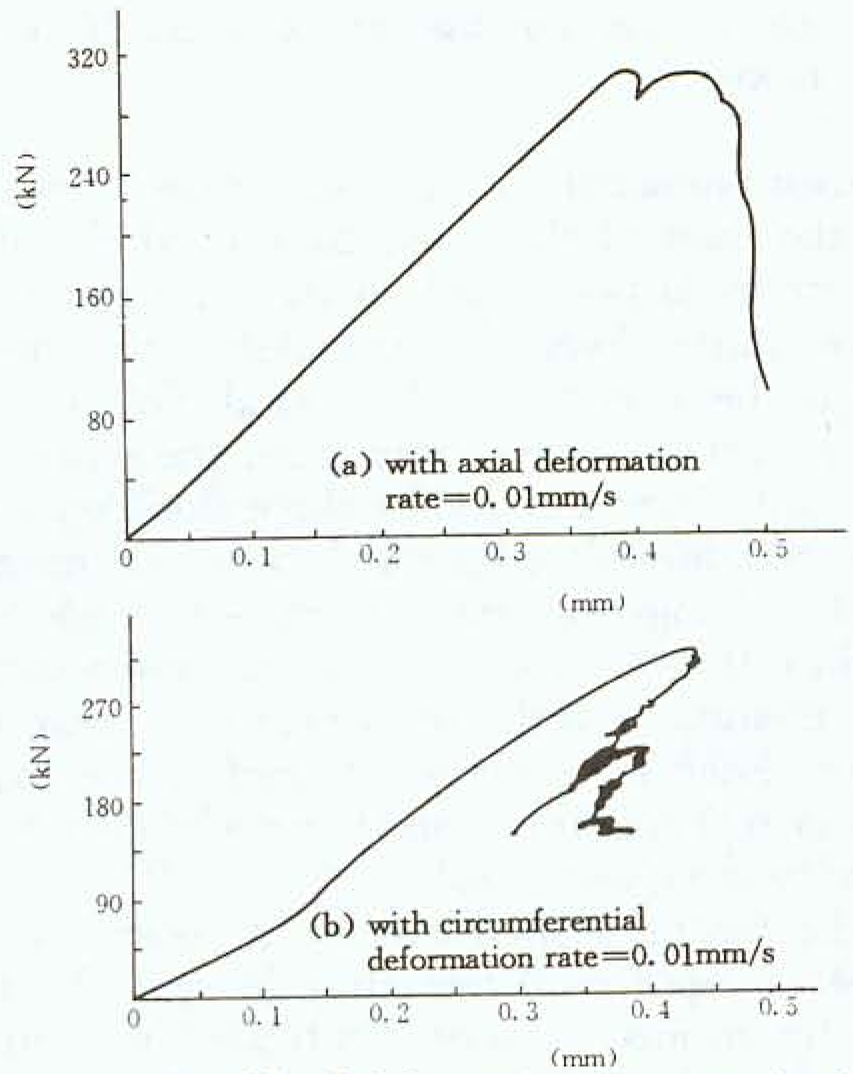

Figure 4. Axial load-deformation diagram of red granite

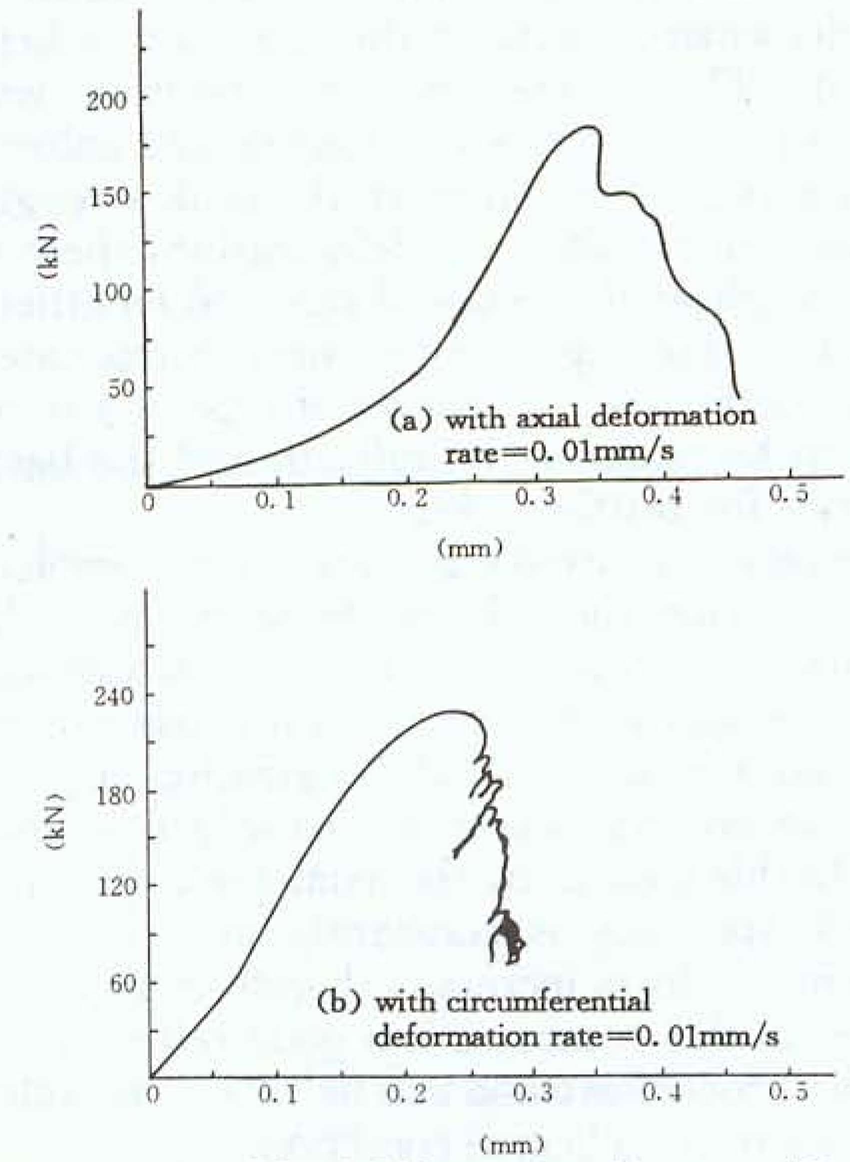

Figure 5. Axial load-deformation diagram of Daye marble

deformation and the constant rate of circumferential deformation. The axial load deformation complete curves of grey granite, red granite, Daye marble and Guixi red sandstone are given in Fig. 3,4,5 and 6, respectively.

3.3 *Discussion on the test results*

The following are dislpayed through the results.

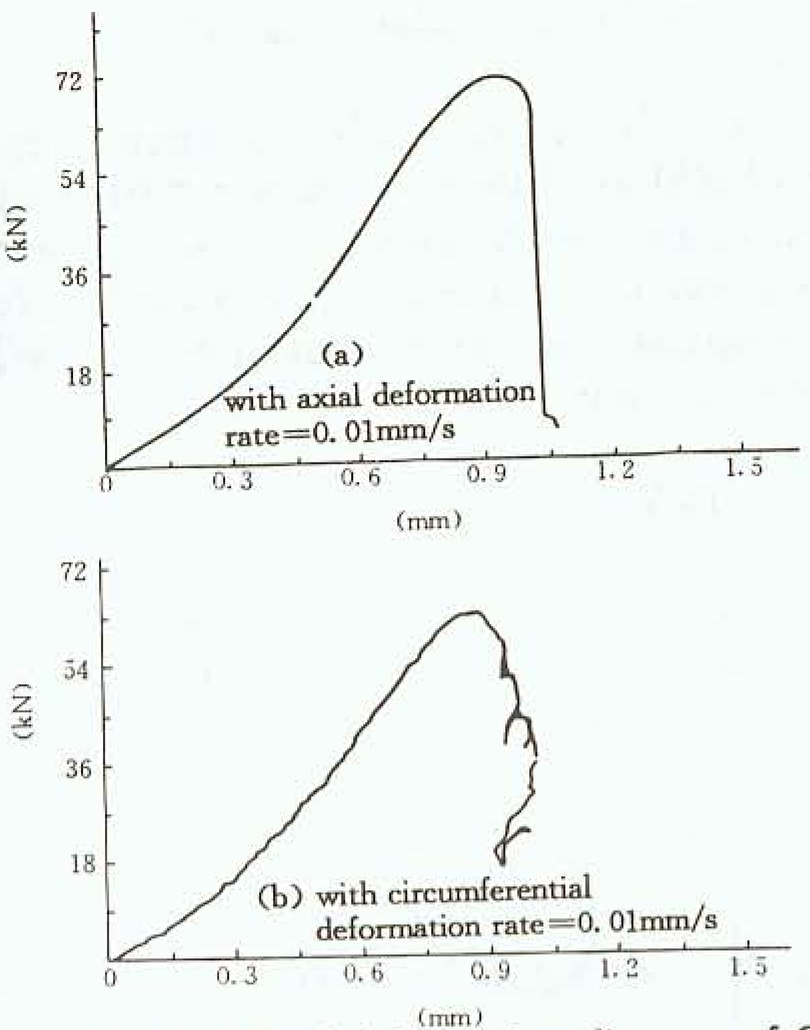

Figure 6. Axial load-deformation diagram of Quixi red sandstone

(1) By means of the RMT test machine with the self-adaptive control system, the uniaxial compression tests can be performed successfully under the condition of $\dot{\varepsilon}_l$ =const, and the complete curves can be obtained, even for typical brittle rocks.

(2) Axial load deformation curves of brittle rocks under the condition of the constant rate of circumferential deformation obtained by the RMT test machine are similar in shape with the curves of type Ⅱ, which are abundantly reported in China and abroad.

(3) Therefore it can be seen that the axial load deformation curves under two different control modes for specimens of the same kind of rock are diametrically different in shape.

(4) As tests have proved, for brittle rocks, the complete curves can be obtained under the condition $\dot{\varepsilon}_l$ = const. This means that in the post-failure range energy should be "put into" specimens even in the case of brittle rocks. The view apparently is not correct that only by withdrawal of energy can the post-failure curves be obtained for brittle rocks.

(5) As mentioned above, since the complete curve of the uniaxial compression test describes the relation between axial load (stress) and axial deformation (strain) from the logical viewpoint, such a control condition is most natural and rational: the rate of one of the above variables, i. e., the rate of axial deformation is kept constant. The condition that the rate of circumferential deformation is to be constant seems very sensitive at the first sight. But it inevitably leads to a complex state of the repeated loading and unloading. Therefore it is not advisable.

3. 4 *A new rock classification model*

On the basis of the above-mentioned experimental and theoretical study, the new rock classification model propsoed in this paper is shown in Fig. 7.

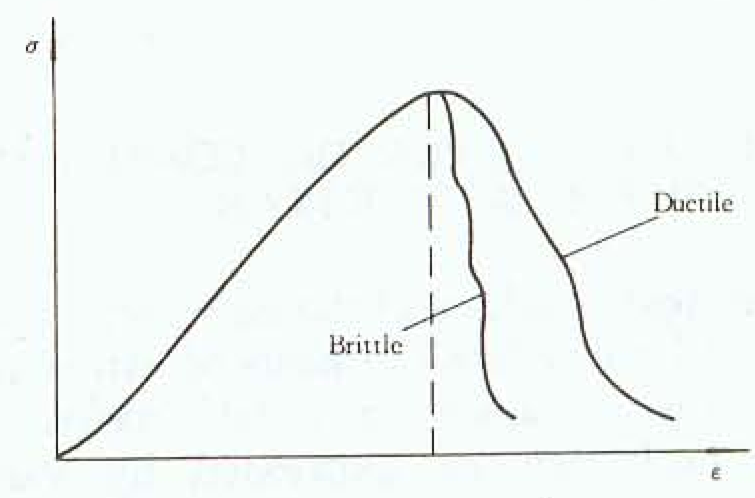

Figure 7. Rock classificaton model

The model displays that the steeper are the post-failure curves of brittlerocks, the more close are they to the vertical passing through the point of peak strength, remaining on the right side of this vertical. The post-failure curves of ductile rocks are more gentle than those of brittle rocks.

As a rule, the post-failure curves after the peak strength are not smooth, and with small benches. This is a manifestation of a complex process of the frequent "resistance" occurring in the course of development from damage to failure, and the continuous development when the "resistance" is overcome.

There is also a view that the development of the cracks inside the brittle rock specimen after the peak strength leads to the release of surface energy, which in turn causes the further expansion of the cracks. Therefore, according to this view, in the condition of no energy supplied, the failure develops abruptly, and out of control, until the failure takes place in the whole specimen. However, attention is to be paid to three non-uniformities. The material properties within the rock specimen can not be completely uniform. The stress states in different parts of the specimen can not be completely uniform, this is particularly true of the specimen made of rock with a variety of mineral components. The strengths in different parts can not also be completely uniform. Owing to these non-uniformities, the local impediment is necessarily caused in the expansion of the cracking, and the surface energy released on the cracked parts is not sufficient for the process developing without control till the complete collapse. The input of an appropriate quantity of energy is needed for the continuation and expansion of the process. A considerable dimension of cracking reached, which means a serious change in the state on a whole, would cause an abrupt collapse in the whole specimen. Owing to these non-uniformities, the post-failure curves under $\dot{\varepsilon}_l$ =const are obtainable in general, but often with some small benches. The more homogenuous the brittle rock is, the steeper and closer to the vertical are its post-failure curves.

4 THE BRITTLE-PLASTIC CONSTITUTIVE MODEL FOR BRITTLE ROCKS

The above test results and the new rock classification model constitute the basis of establishing the constitutive model for brittle rocks. The peak strength can be expressed by various strength criteria, for instance, Coulomb's strength criterion, Mohr's strength envelope and so on. It is the boundary between the pre- and the post-failure properties. Before the peak strength is reached the properties of brittle rocks can sometimes be simplified by elastic mode, or treated by non-linear mode when it is necessary. If the yield is apparent, it is appropriate to regard the pre-failure range as composed of the elastic section and the non-linear section, or as composed of two linear sections.

Although the pre-failure properties are the important problems for establishing the constitutive model for brittle rocks, the modes of treating them are comparatively mature. How to have a rational description of the post-failure properties when the peak strength is passed forms just the key-link of the constitutive model for brittle rocks.

Below are adopted the principles called the brittle-plastic model.

(1) According to the test results, an abrupt change in the stress state of brittle rock should be existent when the peak strength has been reached, and this effect of stress-drop should be simulated by stress transmission.

(2) The position on the residual stress surface (line) where the stress falls is determined when a rational assumption has been made.

(3) A treatment is performed according to the principle of plastic flow when stress-drop has occurred.

(4) If loading or unloading takes place afterward, the deformation modulus will be less than the deformation modulus of rock before the fall. The model in which the abrupt change of stress occurring after the peak strength being reached is treated as a vertical dropping is called the ideal brittle-plastic model.

4.1 *The ideal brittle-plastic model*

Let $F(\sigma)=0$ and $f(\sigma)=0$ be the peak strength surface (PSS) and the residual strength surface (RSS), respectively. Assume that the stress reaches a certain point A on PSS by loading from a certain initial state, as shown in Fig. 8. When the load condition

$$L=\left\{\frac{\partial F}{\partial \sigma}\right\}^T D d\varepsilon > 0 \tag{1}$$

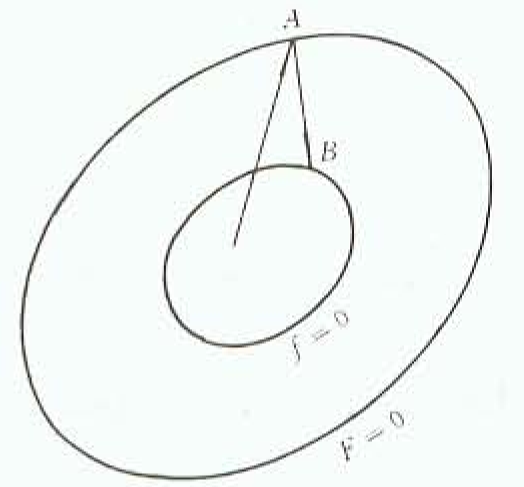

Figure 8. Diagram of stress-drop

is satisfied (Qu & Yin 1981), the stress will have an abrupt change and will fall on a point B on RSS. Three typical methods for determining point B in two-dimensional cases in Mohr's stress space are given in Fig. 9. B_1 corresponds to the assumption of the constant center of circles, namely that of concentric circles, B_2 corresponds to the assumption of the shortest path, and B_3 to the assumption of the constant minor principal stress (compression is possitive). Evidently, different modes of stress-drop will give different answer. The assumption of the constant center of circles based on the concept of relaxation in most suitable. Therefore, the assumption of concentric circles is recommended.

Its result also corresponds to that obtained by the method for determining point B on the basis of potential theory in the 3-D situation.

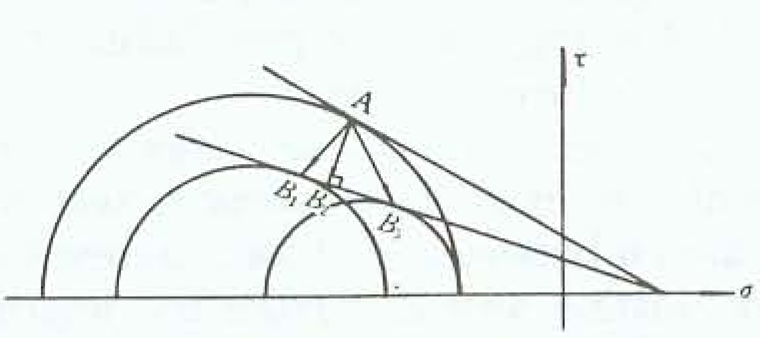

Figure 9. Three typical hypotheses to determine the new stress state

The process of the calculation of stress-drop in terms of potential theory can be summarized as follows (Zheng, Ge & Lee 1997).

The above test results concerning the post-failure properties can prove that brittle-plastic rocks satisfy Ilyushin postulate. Therefore, it is safe to think that the direction of plastic strain increment in the fall conforms to the plastic potential theory. Let the plastic potential function be written as $G(\sigma,\bar{\varepsilon}_p)$. If the associate flow rule is used, then

$$G(\sigma,\bar{\varepsilon}_p) = \begin{cases} F(\sigma), & \text{if } \bar{\varepsilon}_p = 0 \\ f(\sigma), & \text{if } \bar{\varepsilon}_p > 0 \end{cases} \tag{2}$$

The brittleness of rock causes a incontinuous change of $G(\sigma,\bar{\varepsilon}_p)$ in the stress space, which corresponds to a non-differential increment of plastic stran, $\Delta\varepsilon_{ij}^p$.

$$\Delta\varepsilon_{ij}^p = \Delta\lambda\frac{\partial G}{\partial\sigma_{ij}}\bigg|_A = \Delta\lambda\frac{\partial F}{\partial\sigma_{ij}}\bigg|_A \tag{3}$$

where $\Delta\lambda$ is a dropping factor to be determined, for

$$\Delta\varepsilon_{ij} = \Delta\varepsilon_{ij}^e + \Delta\varepsilon_{ij}^p \tag{4}$$

Considering that the major principal strain is constant in the dropping, $\Delta\varepsilon_1 = 0$

Let us assume that

$$\Delta\varepsilon_{ij} = 0 \tag{5}$$

namely,

$$\Delta\varepsilon_{ij}^e = -\Delta\varepsilon_{ij}^p \tag{6}$$

Then, from

$$\Delta\sigma_{ij} = D_{ijkl}\Delta\varepsilon_{kl}^e \tag{7}$$

the stress increment after dropping is obtained:

$$\Delta\sigma_{ij} = \sigma_{ij}^B - \sigma_{ij}^A = -\Delta\lambda D_{ijkl}\frac{\partial F}{\partial\sigma_{kl}}\bigg|_A = -\Delta\lambda\tau_{ij}^A \tag{8}$$

Therefore,

$$\sigma_{ij}^B = \sigma_{ij}^A - \Delta\lambda\tau_{ij}^A \tag{9}$$

in which

$$\tau_{ij}^A \triangleq D_{ijkl}\frac{\partial F}{\partial\sigma_{kl}} \tag{10}$$

$\Delta\lambda$ is determined by

$$f(\sigma_{ij}^B) = f(\sigma_{ij}^A - \Delta\lambda\tau_{ij}^A) = 0 \tag{11}$$

Below we give the calculation method for the dropping factor corresponding to Drucker-Prager criterion. For other criteria, similar principles can be adopted.

Assume that the peak strength surface and the residual strength surface are

$$F(\sigma) = \alpha_o I_1 + \sqrt{J_2} - \kappa_o = 0 \tag{12}$$

and

$$f(\sigma) = \alpha_r I_1 + \sqrt{J_2} - \kappa_r = 0 \tag{13}$$

respectively, where α_0,κ_o and α_r,κ_r are peak strength parameters and residual strength parameters, and $\Delta\lambda$ is a root of the following quadratic equation:

$$a\lambda^2 + b\lambda + c = 0 \tag{14}$$

where

$$a = (9\alpha_o\alpha_r\kappa)^2 - G^2 < 0$$

$$b = 2G\sqrt{J_2(\sigma^A)} - 18\alpha_o\alpha_r K[\alpha_r I_1(\sigma^A) - \kappa_r]$$

$$c = [\alpha_r I_1(\sigma^A) - \kappa_r]^2 - J_2(\sigma^A)$$

Because

$$b^2 - 4ac = [18\alpha_o\alpha_r K\sqrt{J_2(\sigma^A)} - 2G(\alpha_r I_1(\sigma^A) - \kappa_r)]^2 > 0$$

it is certain that the equation has two roots unequal to each other:

$$\lambda_1 = f(\sigma^A)/(9\alpha_o\alpha_r\kappa + G)^2 > 0$$

$$\lambda_2 = (\alpha_r I_1(\sigma^A) - \kappa_r - \sqrt{J_2})/(9\alpha_o\alpha_r\kappa - G)$$

Evidently, $\Delta\lambda$ can be taken as

$$\Delta\lambda = \begin{cases} \min(\lambda_1,\lambda_2) & \text{if } \lambda_2 > 0 \\ \lambda_1 & \text{if } \lambda_2 \leqslant 0 \end{cases} \tag{15}$$

4.2 *The brittle-plastic model considering the non-vertical dropping*

Even the rock to which typical brittle failure occurs in the uniaxial compression condition will exhibit apparent plastic properties, instead of brittle failure taking place, at high confining pressure. This transition from brittleness to ductility is well-known fact. Therefore, when σ_1 is the major compression stress (compression taken as positive) and σ_2 and σ_3 are also compression and of a considerable order of magnitude, sufficient attention should be paid to such a transition. Even if brittle failure takes place under small confining pressure, the stress-drop in general, does not occur in the vertical direction. In the triaxial stress conditions the brittle-plastic model with non-vertical dropping should be, in principle, taken into consideration. There are some kinds of rocks, the stress-drop of which is not vertical even under uniaxial compression conditon. Generally speaking, the non-vertical dropping of the brittle failure can not be treated as material softening. As has been proved, it can be treated as material softening only when the equatin of the yield surface $F(\sigma,w_p) = 0$ meets the follwing conditions (Zheng, Ge & Lee 1997)

$$|A| < \left[\frac{\partial F}{\partial\sigma}\right]^T D\frac{\partial F}{\partial\sigma}, \quad \text{且 } A < 0 \tag{16}$$

where

$$A \triangleq -\frac{\partial F}{\partial w_p}\sigma^T \frac{\partial F}{\partial \sigma} \quad (17)$$

in which w_p is plastic work defined as

$$w_p = \int \sigma^T \mathrm{d}\varepsilon_p \quad (18)$$

The stress-drop of brittle failure does not satisfy the above condition, as a rule.

The more generalized case is non-vertical stress-drop taking place when the peak strength is passed. Evidently, the accurate analysis of this case is difficult.

An approximate method is recommended below.

The governing equation of the non-vertical stress-drop taking place after the peak strength has been reached is:

$$\Delta\varepsilon_{ij} \neq 0 \quad (19)$$

Assume that the total strain increment $\Delta\varepsilon_{ij}$ is proportional to the elastic strain increment

$$\Delta\varepsilon^e_{ij} + \Delta\varepsilon^p_{ij} = \Delta\varepsilon_{ij} = -R\Delta\varepsilon^e_{ij} \quad (20)$$

in which R is a revision coefficient for non-vertical stress-drop and is positive.

In the case of uniaxial compression, R can be calculated by the characteristic parameters of axial deformation shown in Fig. 10.

$$R = \frac{b}{a} \quad (21)$$

where $b = \varepsilon_B - \varepsilon_P, a = \varepsilon_P - \varepsilon_M$.

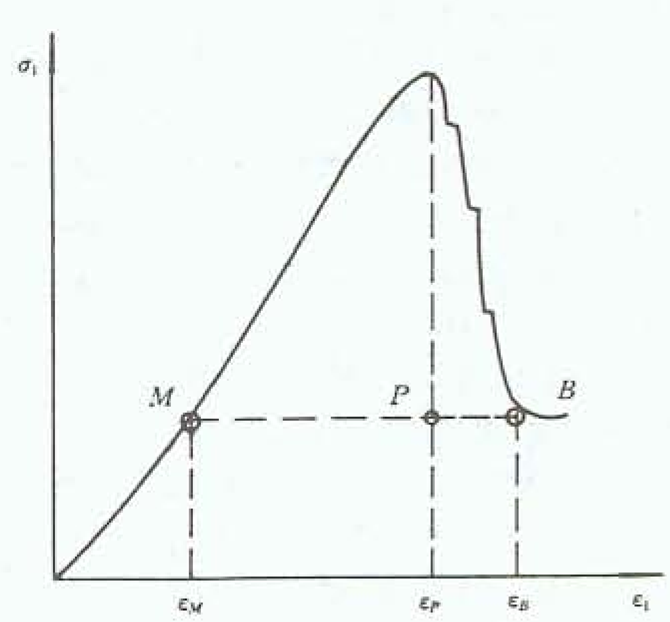

Figure 10. Diagram of non vertical stress dropping

$b = 0$ means the vertical stress-drop, at this time $R = 0$. In general cases, R should be less than 0.8, otherwise the brittle failure can not be considered.

Non-vertical stress-drop is also related with the confining pressure. R can be regarded as the function of $\sigma_c \left(\sigma_c = \frac{\sigma_2 + \sigma_3}{2}\right)$. The curve of $R = f(\sigma_c)$ can be obtained by triaxial testing, and the value of R is taken according to the value of σ_c at the moment when the brittle failure takes place.

The relation between elastic and plastic deformation increments for the non-vertical stress-drop is

$$\Delta\varepsilon^e_{ij} = -\theta\Delta\varepsilon^p_{ij} \quad (22)$$

where

$$\theta = \frac{1}{1+R} \quad (23)$$

For the vertical stress-drop $\theta = 1$ because of $R = 0$, so (5) is special example for (22).

The stress increment for the case of non-vertical dropping is

$$\Delta\sigma_{ij} = -\theta\Delta\lambda\tau^A_{ij} \quad (24)$$

Therefore,

$$\sigma^B_{ij} = \sigma^A_{ij} - \theta\Delta\lambda\tau^A_{ij} \quad (25)$$

$\Delta\lambda$ is determined by

$$f(\sigma^B_{ij}) = f(\sigma^A_{ij} - \theta\Delta\lambda\tau^A_{ij}) = 0 \quad (26)$$

5 ENGINEERING APPLICATION

The brittle-plastic model has found its application in the numerical analyses in a number of major projects in China.

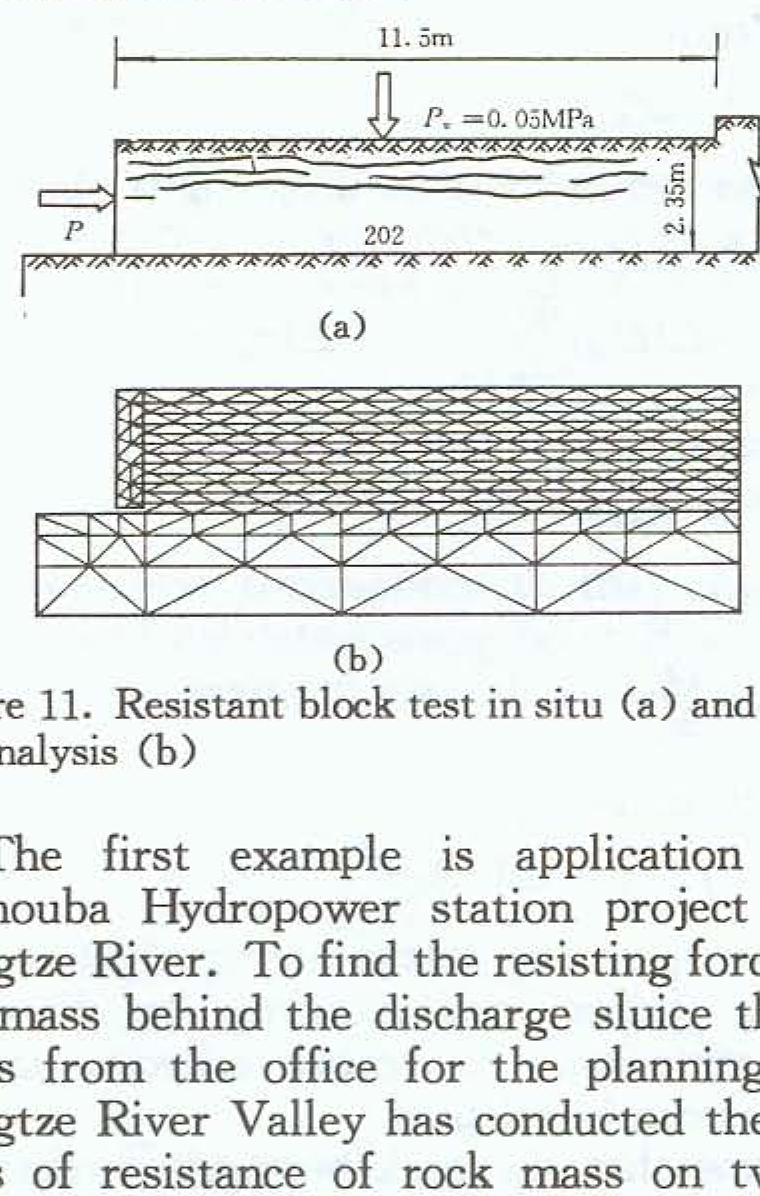

Figure 11. Resistant block test in situ (a) and mesh for FE analysis (b)

The first example is application in the Gezhouba Hydropower station project on the Yangtze River. To find the resisting force of the rockmass behind the discharge sluice the engineers from the office for the planning of the Yangtze River Valley has conducted the in-situ tests of resistance of rock mass on two rock specimens with large size of 11.5×1.7×2.35m³ each. On the top of one of the specimen there is distributed a uniform load (0.05MPa). Fig. 11 (a) is an illustration of the in-situ resistance test, and Fig. 11(b) is a mesh for numerical simulation. There is an argillaceous intercalation No. 202_4 (c = 0. 005MPa, $tg\varphi$ = 0. 2) between the specimen and the bed rock. In addition, there are three horizontal layers (c = 0. 03 MPa, $tg\varphi$ = 0. 88). A 2-D numerical simulation is carried out, a non-linear model is used for layers and intercalations (Ge 1981), and an ideal brittle-plastic model for rock is adopted. Fig. 12 gives the relation between the horizonal thrust obtained by numerical simulation and the damage area. Table 1 is a comparison of in-situ test results with numerical simulation results. A good agreement is displayed (Ge et. al. 1995).

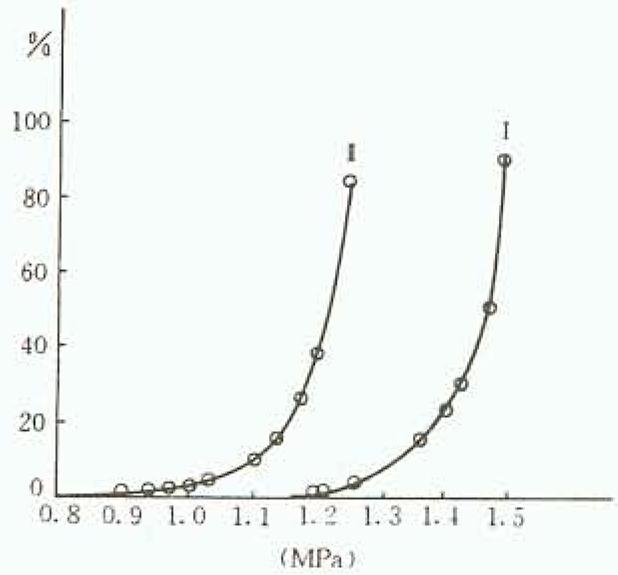

Figure 12. Diagram of damage area (%) versus horizontal thrust P

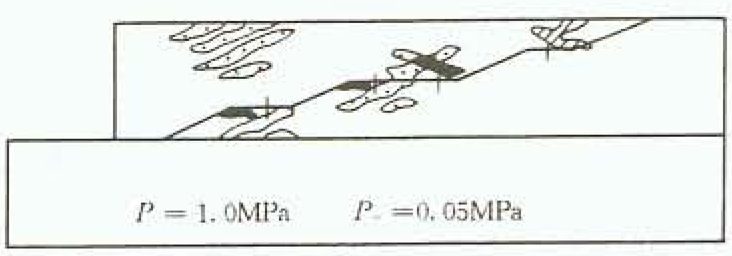

Figure 13. Diagram of damagezone distribution of resistant block with inclined joints

Fig. 13 gives numerical results of the distribution of damage zones in the resistance body #1 at P = 1. 0MPa, when inclined joint surfaces exist in the body.

The other example is the 3-D numerical analysis on the high slope of the permanent shiplock in the key water-conservance project at Three Gorges on Yangtze River.

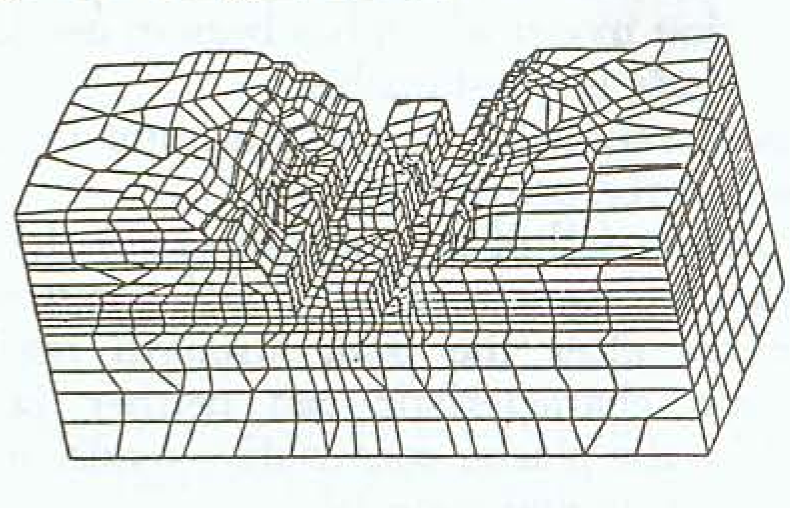

Figure 14. 3D-FE model for the shiplock of Three Gorges Project

Fig. 14 shows the 3-D FE model for some lock chambers and the lock head of this double line ship lock as well as for the slope. The main substance component of this region is a sort of granite, which is a typical brttle rock. In the calculation, 3 large faults and 2 dykes are considered, the loads such as weight, initial geostress, seepage and water load are involved and the excavation process is simulated. There are 7578 nodes, 7409 blocks and joint elements. For the sake of space, only a part of results on an intersection, under the elasto-plastic and the elasto-brittle-plastic conditions are given, as in Fig. 15.

Table 1. Comparison of in-situ test results with numerical simulation results

Resistance body	No. 1		No. 2	
	test	Num. simulation	test	Num. simulation
P at the beginning of dislocation of main intercalation (MPa)	0. 208	0. 10	0. 053	0. 067
P at the full dislocation of 202_4(MPa)	0. 406	0. 50	0. 399	0. 40
P_{max} at the failure of the resistance body (MPa)	1. 534	1. 48	1. 24	1. 26

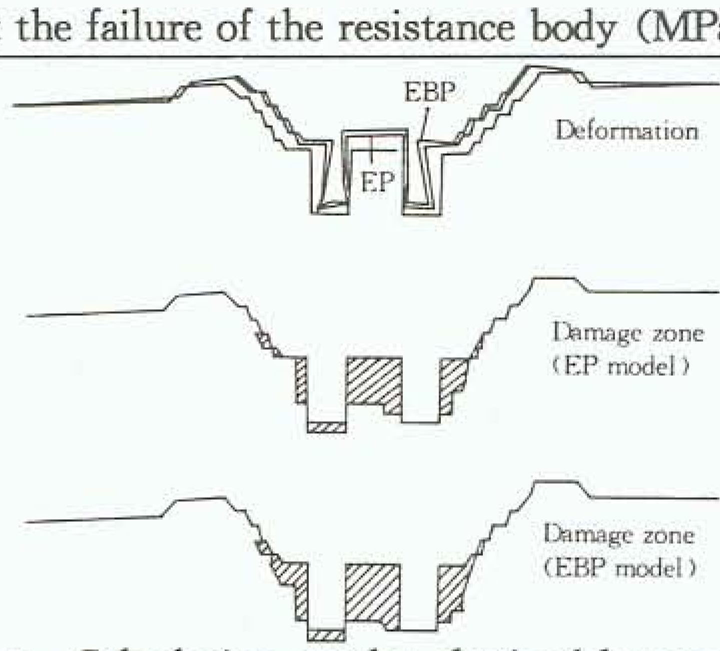

Figure 15. Calculation results obtained by means of elasto-plastic model (EP) and elasto-brittle-plastic model (EBP) for shiplock problem

The elasto-brittle-plastic model used for rock means that the elastic model is used before the peak strength is reached and the ideal brittle-plastic model is used in the post-failure range.

6 CONCLUDING REMARKS

To obtain the relation between the axial load (stress) and the axial deformtion (strain) for rocks under the uniaxial compression, namely, the so-called complete curve, it is most rational to take $\dot{\varepsilon}_l$ =const as the control condition.

For brittle rock, if $\dot{\varepsilon}_l$ =const is taken as the control condition, the post-failure curve remain on the right side of the vertical drawn at the peak point. This displays that energy should not withdraw from the brittle specimen but should put into it, and the failure process is controlable. Therefore the so-called complete curves of type Ⅱ are not rational.

New test results have proved that the rock classification model which has been in use for almost 30 years is questionable.

A more rational rock classification model is proposed in the present paper.

The abrupt fall of the axial stress of the brittle rock specimen in the uniaxial compression test occure after the peak strength reached. This main characteristic can neither be expressed by the plastic constitutive model nor be treated as softening material.

The stress-drop can be approximately regarded as vertical when it is abrupt. An ideal brittle-plastic constitutive model is applicable to this case.

The non-vertical dropping is more common case, for which a more general brittle-plastic model has been proposed in this paper.

The brittle-plastic model has found its application in some important projects in China.

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