# *Simulation of Jointed Rock Mass Behaviour Using Finite Element Method*

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**ABSTRACT**

Non-linear finite element analysis of the jointed rock has been carried out to give better insight in to the mechanical behaviour of discontinuities in rock masses. Intact rock mass is discretized using two dimensional plane strain elements and the joint is explicitly modelled using two-dimensional gap and friction elements. The model is subjected to uniform confining pressure on the two vertical sides and uniform axial stress on the top. Non-linear material behaviour is used in the analysis. The axial load is applied in series of steps or increments. The incremental solution is performed in a step-by-step manner until the full-specified loads are applied. Finite element analysis has been carried out for three different rock masses with single and multiple joints for different confining pressures. The inclination of the discontinuity is varied from 0 to 90 degrees with the major principal stress direction. Results have been presented in the form of stress strain curves, failure stress versus joint inclination angle in the case of single joints and failure stress versus number of joints in the case of multiple joints. The results compare well with the experimental results. The results have also been presented in the form of equivalent plastic strain contours, which give an indication of regions in the model, which are more susceptible to failure. Equivalent plastic strain contours have been presented for both single and multiple joints for

different inclination angles and confining pressures. The major advantage of explicit modelling of discontinuities is that the mode of failure can be traced out and the exact behavior of the discontinuity can be mapped.

*Keywords:* Jointed rocks, non-linear, finite element modelling, discontinuities, Interface element

## 1. INTRODUCTION

Most rock masses encountered in general contain pre-existing discontinuities-joints, clay partings, minor faults and other planar structures, which influence the strength of intact rock. The study of mechanical behaviour of discontinuities/ joints in rock engineering has posed several challenges to the engineering and scientific communities because of the difficulties involved in analysing it. Modelling of these discontinuities in the rock mass to determine their influence on strength and deformation behavior of rock masses is very important for the engineering design of civil structures.

To understand the mechanical behaviour of jointed rocks, three standard tools namely (i) analytical methods (ii) experimental techniques and (iii) numerical methods are available. Analytical methods provide quick close form solutions, but they treat only simple geometries and capture only the idealized structural theory. Using experimental techniques, representative or full-scale models can be tested. A number of experimental studies have been conducted both in-situ and laboratory to understand the behaviour of joints. Experimentation involves more time and is expensive, both in terms of the test facilities and the model instrumentation. Relative to analytical methods, numerical methods require very few restrictive assumptions and can treat complex geometries. They are far more cost effective than experimental techniques. Several numerical techniques are available for the analysis of jointed rock mass. All these numerical methods have their own advantages and disadvantages. One of the most popular numerical tool is the finite element method. In the finite element method the jointed rock mass can be represented as (i) equivalent continuum with equivalent material properties for obtaining the overall response of jointed rock (ii) explicit modelling of joints using joint elements. Finite element modelling of jointed rock using the equivalent continuum approach (Sridevi and Sitharam, 1997, 2000) has been carried out by representing the jointed rock properties by a set of empirical relations. These relations are derived from the statistical analysis of large amount of experimental data. These relationships express the properties of jointed rock mass as a function of intact rock properties and joint factor. The major advantage of equivalent continuum approach is that the most complex joint fabric can be represented by a simple finite element mesh. Alternatively in the present study an

effort has been made to model a few major joints explicitly to study their behavior in detail.

In this paper, nonlinear finite element analysis of the jointed rock has been carried out by representing the joints explicitly to study the mechanical behavior of discontinuities in rock masses. Three different rock masses sandstone, granite and Agra sandstone has been analysed with single and multiple joints for different joint inclination angles and confining pressures. The results have been presented in the form of stress strain curves, equivalent plastic strain contours which give an indication of modes of failure, failure stress versus inclination of joints, failure stress versus number of joints. The major advantage of explicit modelling of joints in a jointed rock mass is that the probable failure pattern can be mapped from the equivalent plastic strain contours. The only limitation of this method is that it is very difficult to model complex joint fabric.

## 2. MODELLING DETAILS

The development of an appropriate model is important and it completely determines the insight into the actual physical problem. The finite element solution will solve only the selected mathematical model and that all assumptions made in the model will be reflected in the predicted response. In the present analysis the jointed rock mass is modelled using rock elements to represent the intact rock and interface elements to represent the joints. The intact rock blocks are modeled using 2-D plane strain isoparametric quadrilateral elements to represent long body and are suitable for structures subjected to in-plane loading. A unit thickness is assumed to this element. The nonlinearity introduced due to change in boundary at the joint is modelled explicitly using 2-D gap and friction element as shown in Fig. 1a. This element is a 2-node nonlinear interface element used to model node-to-node contact between two bodies with or without friction. The element has two degrees of freedom, displacements in X and Y direction at each node. This element consists of a pair of coupled nonlinear orthogonal springs (Fig. 1b and c) in the normal and tangent directions to the interface, which are assumed to be very stiff (with stiffness, $K_n$ and $K_t$) relative to the bodies they are attached to. Small springs (Fig. 1b and c), much softer than the stiff springs ($S_n << K_n$, $S_t << K_t$) may optionally be included to prevent rigid body motion initially, upon break in contact, or during relative sliding motion. These small springs help in soft break in contact thus preventing sudden deformation at the interface. These soft springs provide small stiffness and transmit small forces even if the element is in open status. The effect of these soft springs on the solution accuracy is negligible. The forces transmitted in these soft springs are included in the normal and shear forces of the element. Coulomb law is used for friction. Frictionless contact may be modeled by specifying a zero coefficient of friction. The element may assume open or a closed status depending upon the relative displacement in the normal direction. The closed status may be sticking or sliding depending on

whether the friction limit $\mu|f_n|$ is reached, where $\mu$ is the coefficient of friction and $f_n$ is the normal compressive force in the gap.

Material nonlinearity is simulated using infinitesimal displacements and strains for a small load increase so as to give nonlinear stress-strain relations. Since displacements and strains are infinitesimally small the usual engineering stress and strain relations can be used. The material constitutive relations are dependent on stresses, strains and/or displacements. Elasto-plastic material behavior with Mohr-coulomb yield criteria and perfectly plastic model with no strain softening is used in the analysis. At the joint we have two surfaces which can have open or closed status. This boundary non-linearity arising at the joint due to the presence of two surfaces is represented using interface elements. In the boundary nonlinearity, the material and strain behavior remains linear. The only nonlinear behavior comes from changing of boundary. A particularly difficult nonlinear behavior to analyze is boundary nonlinearity at the joint i.e. contact between two or more bodies as in this case. In this approach the contact forces are updated incrementally until the system is balanced at the contact. This approach is numerically stable but requires more number of iterations and computation time to obtain the solution. Both material and boundary nonlinearity are considered in the present analysis.

## 3. FINITE ELEMENT ANALYSIS

In the present analysis the jointed rock mass is modeled as shown in Fig. 2. Single jointed rock (Fig. 2(a)) has single joint inclined at $\beta$ with the major principal stress direction. Multiple jointed rock (Fig. 2 (c)) has one, two, three, four and five joints inclined at $\beta$ with the major principal stress direction. The intact rock blocks are modeled using 2-D plane strain iso-parametric quadrilateral elements to represent long body and are suitable for structures subjected to in-plane loading (Fig. 2 b and d). The nonlinearity introduced due to change in boundary at the joint is modeled explicitly using 2-D gap and friction elements (Fig. 2 b and d). This element is a 2-node non-linear interface element used to model node-to-node contact between two bodies with or without friction. The model is subjected to uniform confining pressure on the vertical sides and uniform axial stress on the top as shown in Fig. 2.

The nonlinear static analysis is carried out with the assumptions; the material behavior is elastic-perfectly plastic with no strain softening, isotropic & homogeneous material model. The axial load/deviator stress is applied in series of steps or increments while the confining pressure applied on the two sides remain constant throughout the analysis. The incremental solution is performed in a step by step manner until the full-specified loads are applied. In each increment the modified Newton Raphson iterative scheme is performed until the convergence is

achieved, other wise the analysis terminates out once the maximum specified iterations are over. Mohr-Coulomb yield criterion is used in the analysis to determine the major principal stress at failure.

For validation purpose, finite element analysis results has been carried out on the single jointed specimen of sandstone and granite and multiple jointed specimen of Agra sandstone with one, two and three joints. The intact rock properties are given in Table 1 (after Yaji, 1984 and Arora, 1987) and the joint properties are given in Table 2. These properties at the interface are derived from the definition of the interface element and its function. Axial load is applied in series of steps or increments till the model fails. The results are in the form of stress strain curves for different confining pressures (0 MPa to 10MPa) and joint orientation angle ($\beta = 0^o$ to $90^o$). These results are compared with the experimental results of Yaji (1984) and Arora (1987). In almost all the cases the finite element results match fairly well with the experimental results. Some sample stress strain plots are given in Fig. 3 for single jointed specimen of sandstone with joint orientation angle ( $\beta$) of $60^o$ and in Fig. 4 for multiple jointed specimen of Agra sandstone with one, two and three joints with joint orientation angle ($\beta$) of $80^o$. Experimental stress strain curves of Yaji (1984) for single jointed specimen and Arora (1987) for multiple jointed specimens are also plotted in Figs. 3 and 4 for comparison. It can be seen from Figs. 3 and 4 that the FEM results match well with the experimental results thus validating the model.

After the validation of the finite element model the following rocks are analyzed: (i) single jointed specimen of sandstone and granite for joint inclination angle $\beta = 0^o$ to $90^o$ and confining pressures ranging between 0 MPa to 10 MPa (Fig. 2a) (ii) multiple jointed rock specimen of Agra sandstone (Fig. 2c) with one, two, three and four joints with $\beta = 40^o$ to $80^o$. The intact rock properties are given in Table 1 and the joint properties are given in Table 2. The corresponding finite element models along with the boundary conditions are as shown in Fig. 2 (b) for single jointed specimen and Fig. 2(d) for multiple jointed specimens.

Table 1 - Intact rock properties (Yaji, 1984 and Arora, 1987)

| Property | Sandstone | Granite | Agra sandstone |
|---|---|---|---|
| Mass density ($KN/m^3$) | 22.5 | 26.5 | 22.17 |
| Uniaxial compressive strength ($MN/m^2$) | 70 | 123 | 110 |
| Elastic Modulus (GPa) | 5.1 | 10.8 | 20.0 |
| Cohesion($MN/m^2$) | 14.0 | 25.5 | 19.22 |
| Angle of Internal Friction (degrees) | 44.0 | 46.5 | 51.00 |

Table 2 - Properties at the joint/interface (Assigned to the interface elements)

| Property | For all the rock materials at the interface |
|---|---|
| Axial stiffness in normal direction ($K_{n.}$) | 1E+4 times the stiffness of the adjacent element (plane strain element) |
| Tangential stiffness ($K_t$) | 1E-2 times the $K_{n.}$ |
| Small stiffness in normal direction ($S_n$) | 1E-5 times $K_{n.}$ |
| Small stiffness in tangential direction ($S_t$) | 1E-5 times $K_t$ |

The results have been presented in the form of

- Modes of failure for different joint inclinations, confining pressures and number of joints.
- Failure stress versus joint inclination for different confining pressures.
- Failure stress versus coefficient of friction
- Failure stress versus number of joints.

The rock mass is observed to have failed when the yield criterion for the elastic behavior is reached and the rock mass behavior becomes plastic. The rock mass is found to have failed first in the region where the equivalent plastic strain occurs. The equivalent plastic strain contours of the rock mass is represented as modes of failure and stress at which rock mass yields is called as failure stress.

## 4. RESULTS AND DISCUSSION

### 4.1 Single Jointed Rocks

Single jointed specimens of sandstone and Granite with joint planes making $\beta$ = 0°, 30°, 45°, 60° and 90° degrees with the major principal stress direction, subjected to confining pressures ($\sigma_3$) of 0, 1.0, 2.5, 5.0, 10.0 MPa, are analysed. The results have been obtained for different joint orientations and confining pressures. Equivalent plastic strain contours for single jointed specimen of sandstone (modes of failure) for joint orientation $\beta = 0^o$ to $90^o$ with 2.5 MPa confining pressure are presented in Fig. 5. Equivalent plastic strain contours for single jointed specimen of granite with $\beta$ = 45° for different confining pressures (0 to 10 MPa) are given in Fig. 6. Failure stress plots for joint orientation angle, $\beta = 0^o$ to $90^o$ for sandstone and granite are presented in Figs. 7 and 8 respectively for different confining pressures. Failure stress plot for different coefficient of

friction at the joint is presented in Fig. 9. The experimental values of failure stress (Yaji, 1984) are also plotted in the figures for comparison.

*Modes of failure*

- It can be seen from Fig. 5 that the failure occurs in the intact rock when the joint is horizontal and vertical. The failure occurs at the interface for $\beta = 30°$, 45°& 60°. It may be noted that the mode of failure in vertical joint is similar to that of intact rock.
- From Fig. 6 it can be said that results are consistent for different confining pressures. The modes of failure observed in granite are similar to sandstone.

*Failure stress versus joint inclination*

- From the graphs of failure stress versus joint inclination for various confining pressures (Fig. 7 and 8), it can been seen that the failure stress is highest for $\beta = 0°$ & 90° for both sandstone and granite.
- Failure stress is minimum for $\beta = 30°$ for sandstone with a single joint and at $\beta = 45^o$ for granite with a single joint. The failure stress increases with the confining pressure.

*Effect of friction on failure stress*

The study of effect of friction is important, as surface roughness is perhaps the most important factor influencing the friction between joint surfaces, as it controls the movement along the joint planes. When a rock element slides over another, friction is mobilised along the contact surface. The effect of friction at the interface is studied. Failure stress versus effect of friction is plotted in Fig. 9 for sandstone with single joint with $\beta = 60°$, Confining pressure=2.5 MPa.

From above results it is summarized that the mode of failure in the rock mass is influenced by joint orientation and confining pressure. The failure occurs in the intact rock when the joint is horizontal & vertical i.e., when $\beta = 0°$& 90° for all confining pressures, where as the failure occurs at the interface for other joint orientations when $\beta = 30°$, 45° & 60°. The rock specimen has highest strength when $\beta = 0°$& 90° and least at $\beta = 30°$ for sandstone and $\beta = 45°$ for granite. The results match well with the experimental results. From Fig. 9 it can be seen that as the value of coefficient of friction increases the resistance offered to slip increases till a certain value and further increase of friction has no effect on the failure stress. When the joint is horizontal or vertical, coefficient of friction has no effect on the failure stress.

### 4.2 Rocks with Multiple Joints

Multiple jointed specimen of Agra sandstone with one, two, three and four joints with different inclination angles ($\beta$ = 40$^o$, 50$^o$, 70$^o$, 80$^o$), subjected to different confining pressures are analysed. Some typical results of these analysis are presented as equivalent plastic strain contours (modes of failure) for one, two, three and four joints with joint orientation ($\beta$) of 70$^o$ in Fig. 10, and failure stress versus number of joint for joint orientation angle of 40$^o$, 50$^o$, 70$^o$ and 80$^o$ in Fig. 11. The experimental values of failure stress of Arora (1987) are also plotted in Figs. 10 and 11 for comparison.

*Modes of failure*

- It can be seen from Fig. 10 that the failure occurs at the interface for different number of joints.
- Results are consistent for different orientation of the joints and for different confining pressures.

*Failure stress versus number of joints*

- It can been seen from Fig. 11 that the strength of rock mass decreases as the number of joints increases, i.e., the strength comes down as frequency of joints increases in the rock mass.

Multiple jointed specimen of Agra Sandstone fails at the interface for all the joint inclinations and for all the number of joints analysed. Strength of the rock mass decreases as the number of joints increase for all the joint inclinations and for all confining pressures. It is seen from Fig. 11 that the results are fairly close to the experimental results.

## 5. CONCLUSIONS

The results obtained from the finite element analysis match well with the experimental results. It is clear from results and discussions that the failure stress reaches a minimum value for joint inclination of $\beta$ = 30° & 45$^o$ and maximum for $\beta$ = 0° & 90°. The rock mass becomes very weak if the orientation of the joint is between 30° and 45$^o$ with the major principal stress direction. If the orientation of joint (single) is vertical or horizontal with the major principal stress direction the rock mass becomes as strong as intact rock. The joint weakens the rock mass and the failure occurs at the interface when the joint orientation is between 15° to 80°, where as the failure occurs in the intact rock when the joint orientation is horizontal or vertical. The number of joints present in the rock mass effects the

strength and as the frequency of the joints increases the strength of the rock mass comes down. The value of coefficient of friction increases the resistance offered to slip and hence to failure at the interface.

It can be concluded from the explicit modelling of discontinuities that the three important factors, which effect the mechanical behavior of jointed rock mass, are (i) joint inclination (ii) number of joints and (iii) joint strength properties at the interface. The major advantage of explicit modelling of discontinuities is that the mode of failure can be traced out and the behavior of the discontinuity can be mapped. Explicit modelling of joints using interface element is suitable only for rocks having few major joints. This approach is not suitable to model highly discontinuous rocks as explicit modelling of a joint fabric is very cumbersome and the analysis highly complex and time consuming.

## References

Arora, V.K. (1987). Strength and Deformational Behaviour of Jointed Rocks, Ph.D.thesis, Indian Institute of Technology, Delhi, India.

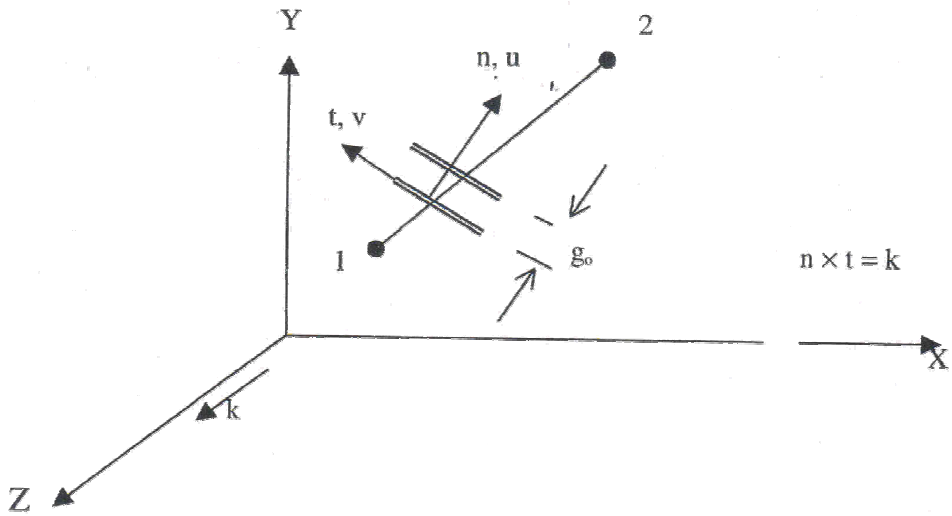
John, H. Curran and Goodluck, I. Ofoegbu. (1994). Modellng of Discontinuities in Numerical Analysis, University of Toronto, Canada, Rock Engineering by Hudson J.A., 1 (18), 443-467.

Klaus-Jurgen, Bathe. (1997). Finite Element Procedures, Prentice-Hall of India, New Delhi.

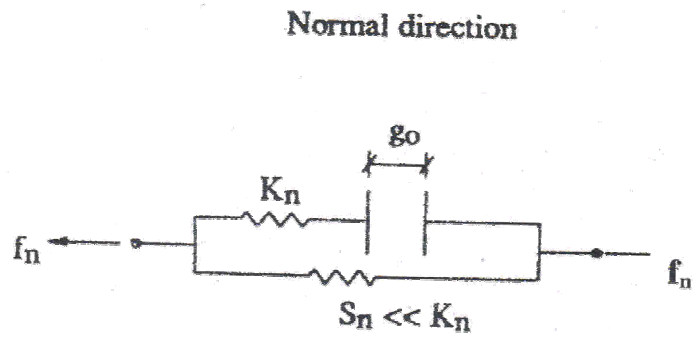
Sridevi, Jade and Sitharam, T.G. (1997). Non-Linear Analysis of Jointed Rock Using Equivalent Continuum Approach, Journal of Rock Mechanics and Tunnelling Technology, Vol.3, No. 2, pp.109-126.

Sridevi, Jade and Sitharam, T.G. (2000). Analysis of Strength and Modulli of Jointed Rocks, International Journal of Geotechnical and Geological Engg., Vol. 18, pp.1-19.

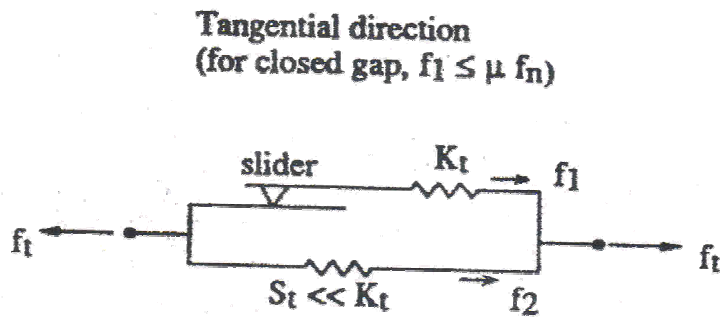
Yaji, R.K (1984). Shear Strength and Deformation of Jointed Rocks, Ph.D. thesis, Indian Institute of Technology, Delhi, India.




(a) Joint element

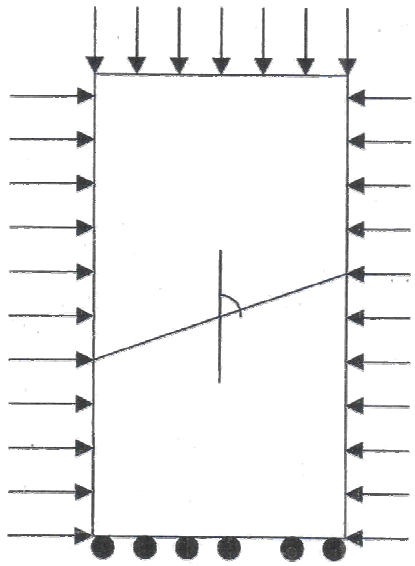



(b) Stiff and soft springs in the normal direction

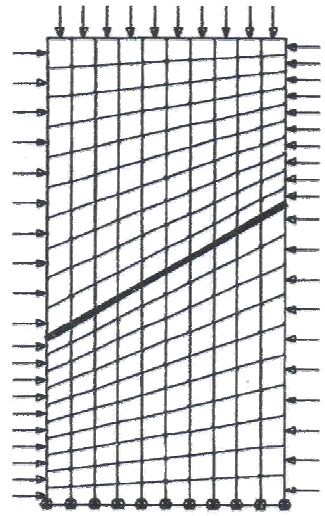



(c) Stiff and soft springs in the tangential direction

Fig. 1– Details of interface element used at the joint

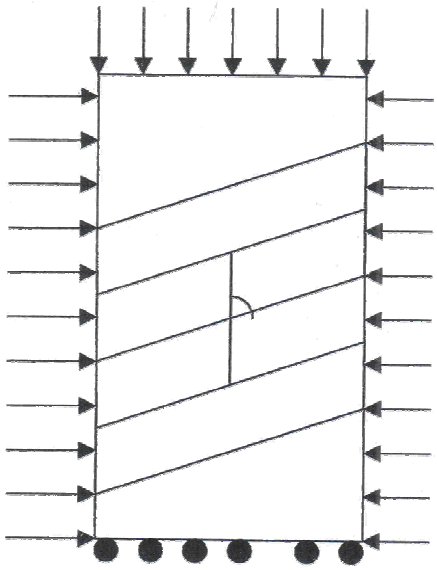


(a) Single jointed specimen

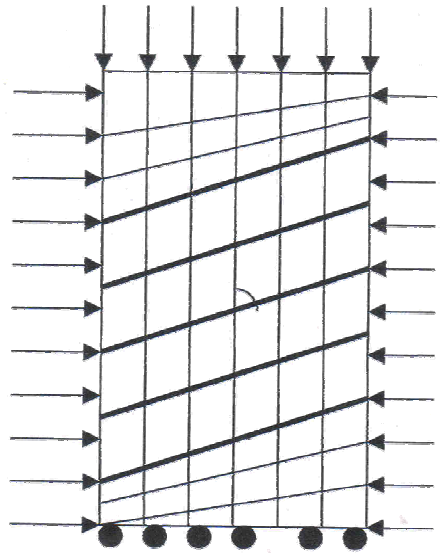


(b) Finite element modeling

Bold line indicates the joint element- 2-D Gap / friction element in finite element model



(c) Multiple jointed specimen



(d) Finite element model

Fig. 2-Jointed specimen and the corresponding finite element model

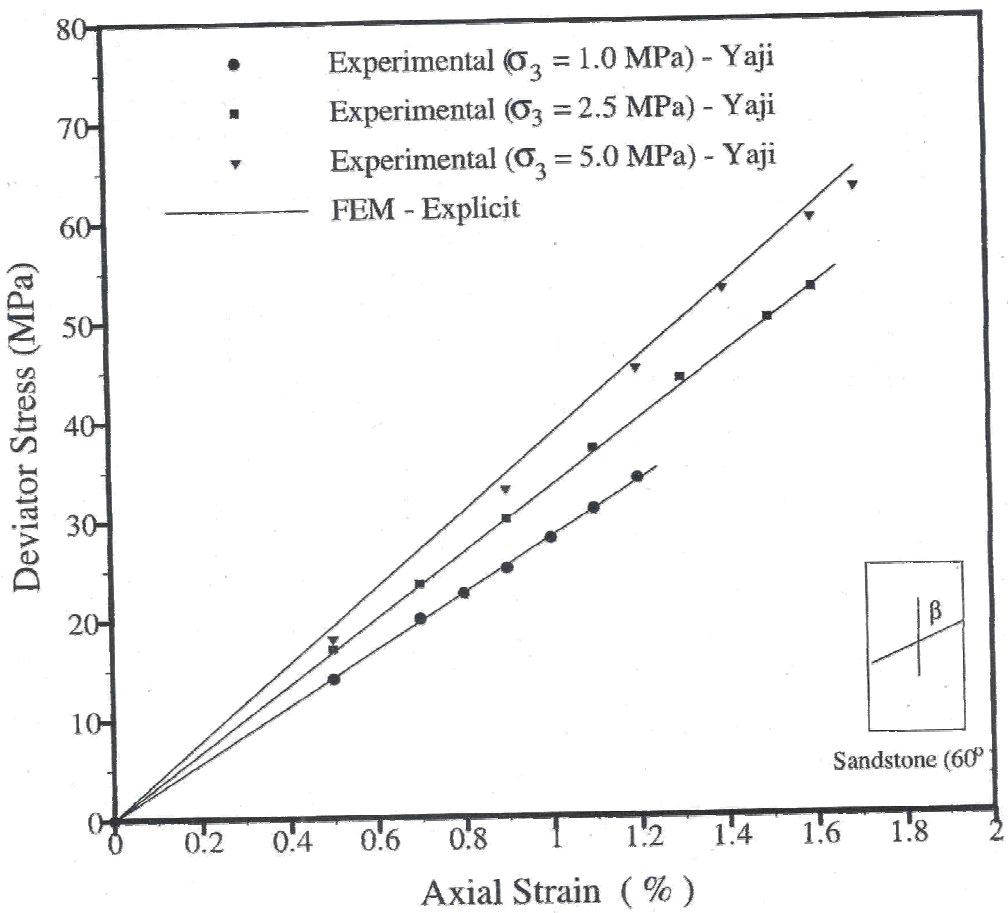



Fig. 3–Stress- Strain curves for sandstone ($\beta$ =60°) along with the experimental results of Yaji (1984)

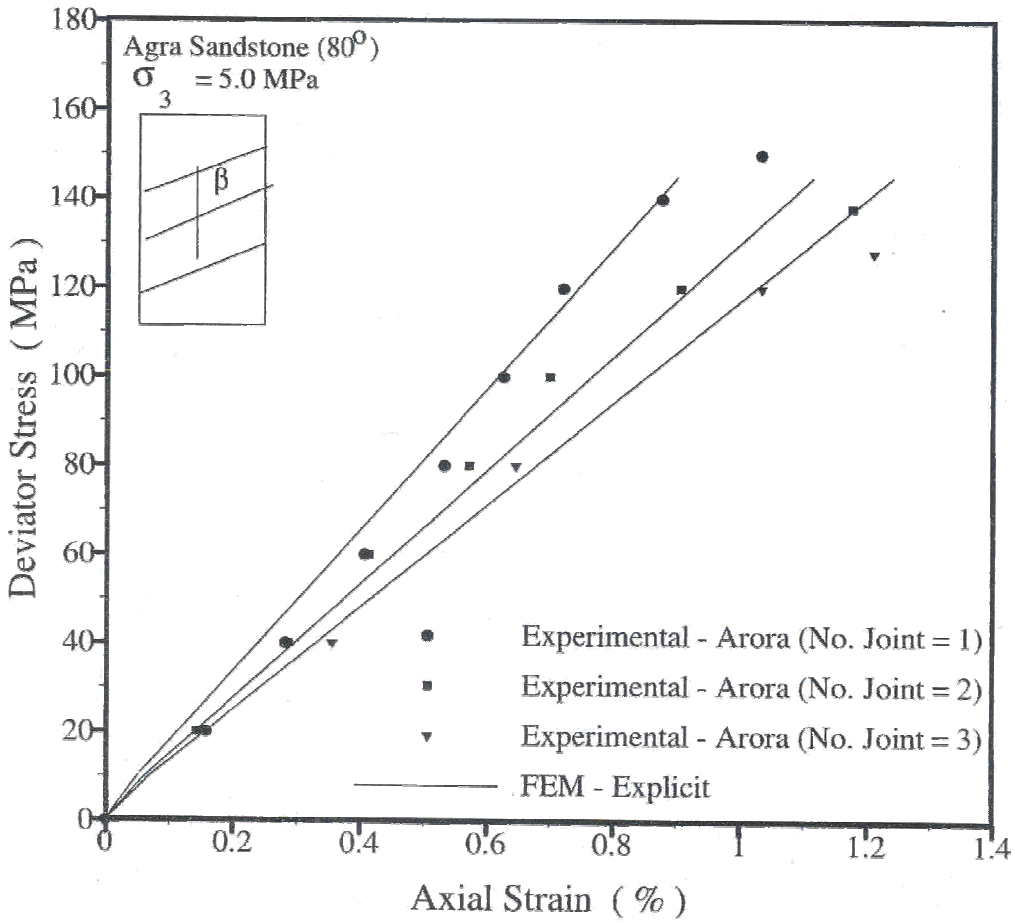



Fig. 4-Stress-Strain curves for Agra sandstone (β=80°) along with the experimental results of Arora (1987)

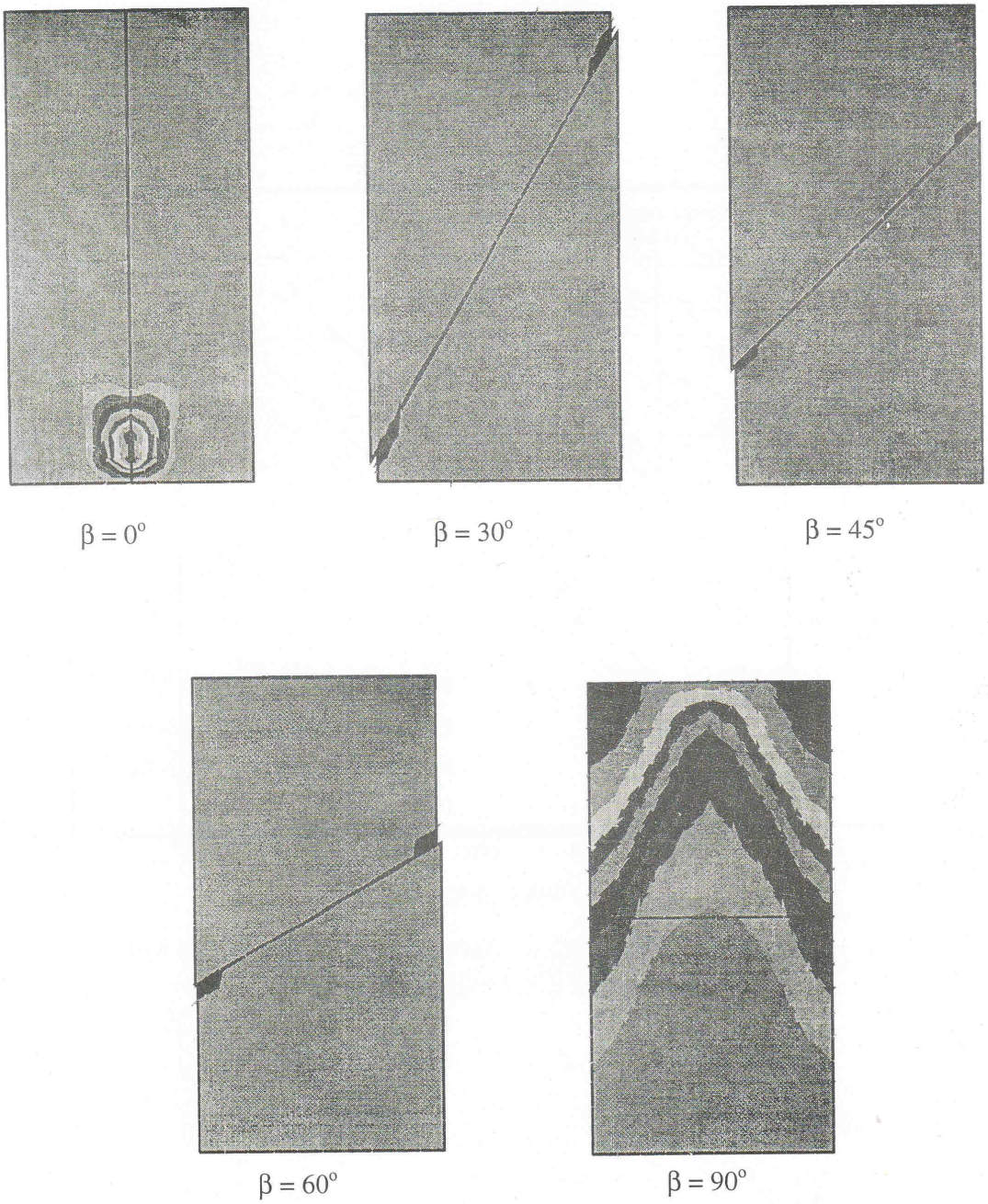
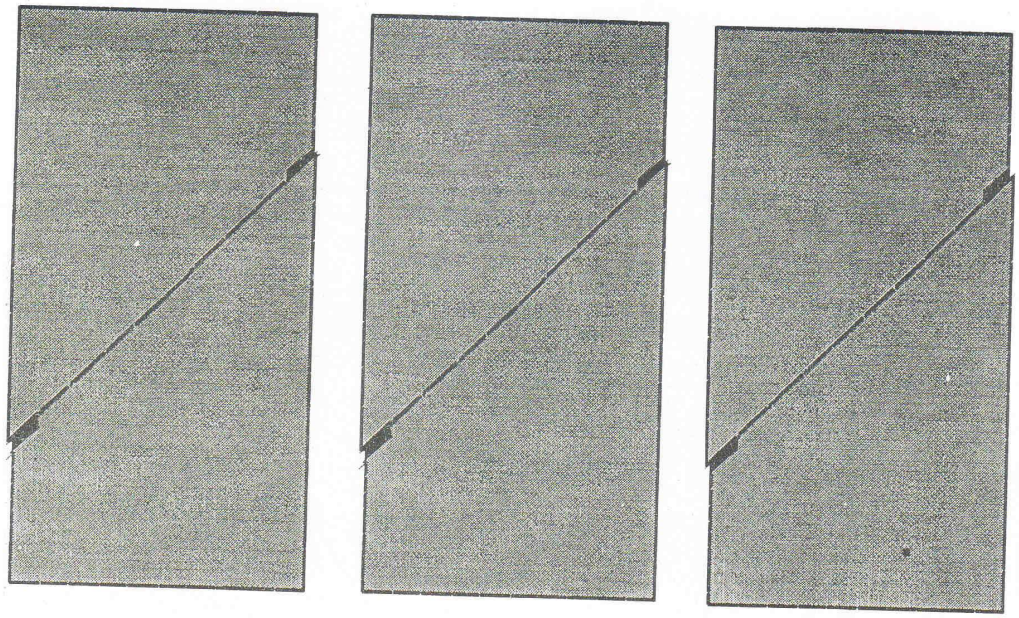
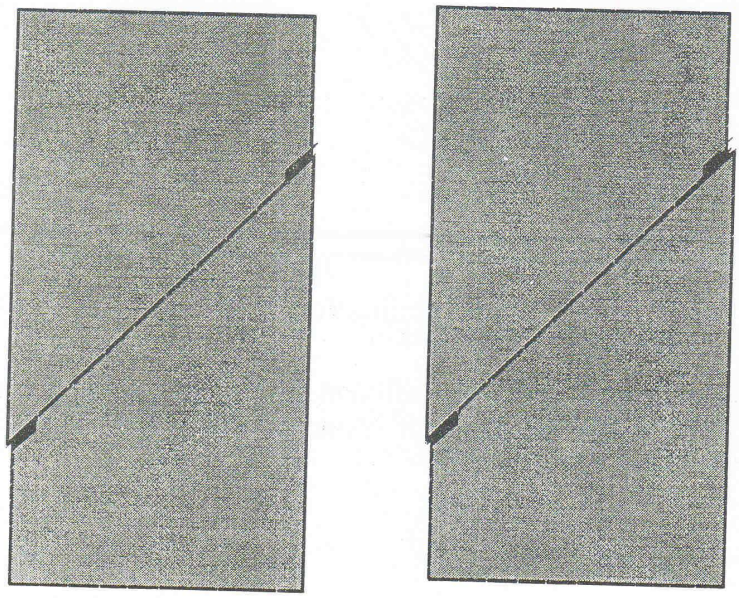



Fig. 5-Modes of failure for single jointed specimen of sandstone for 2.5 Mpa confining pressure



$\sigma_3 = 0.0$ MPa $\sigma_3 = 1.0$ MPa $\sigma_3 = 2.5$ MPa



$\sigma_3 = 5.0$ MPa $\sigma_3 = 10.0$ MPa

Fig. 6-Modes of failure in a single jointed specimen of sandstone for $\beta = 45°$

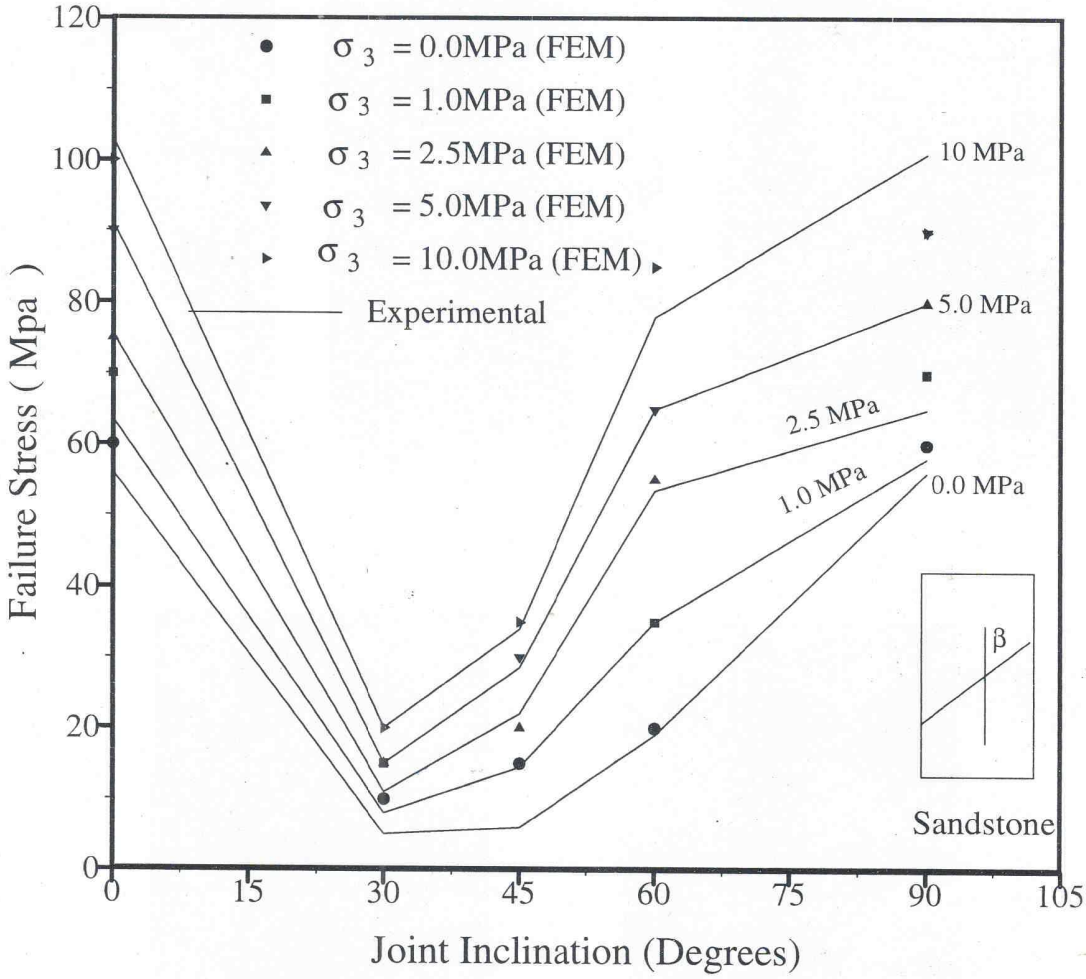



Fig. 7-Failure stress plots for different confining pressure along with the experimental results of Yaji (1984)

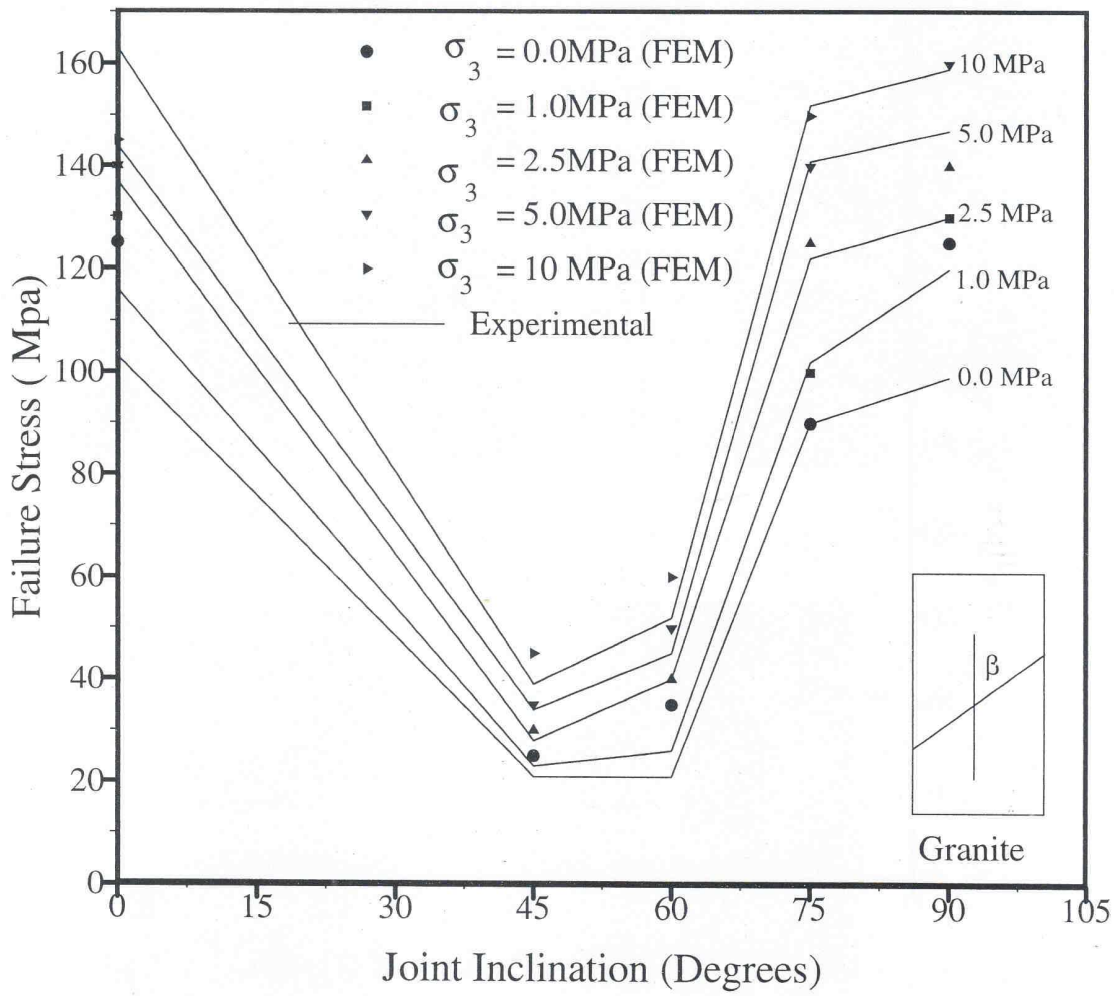



Fig. 8-Failure stress plots for different confining pressure along with the experimental results of Yaji (1984)

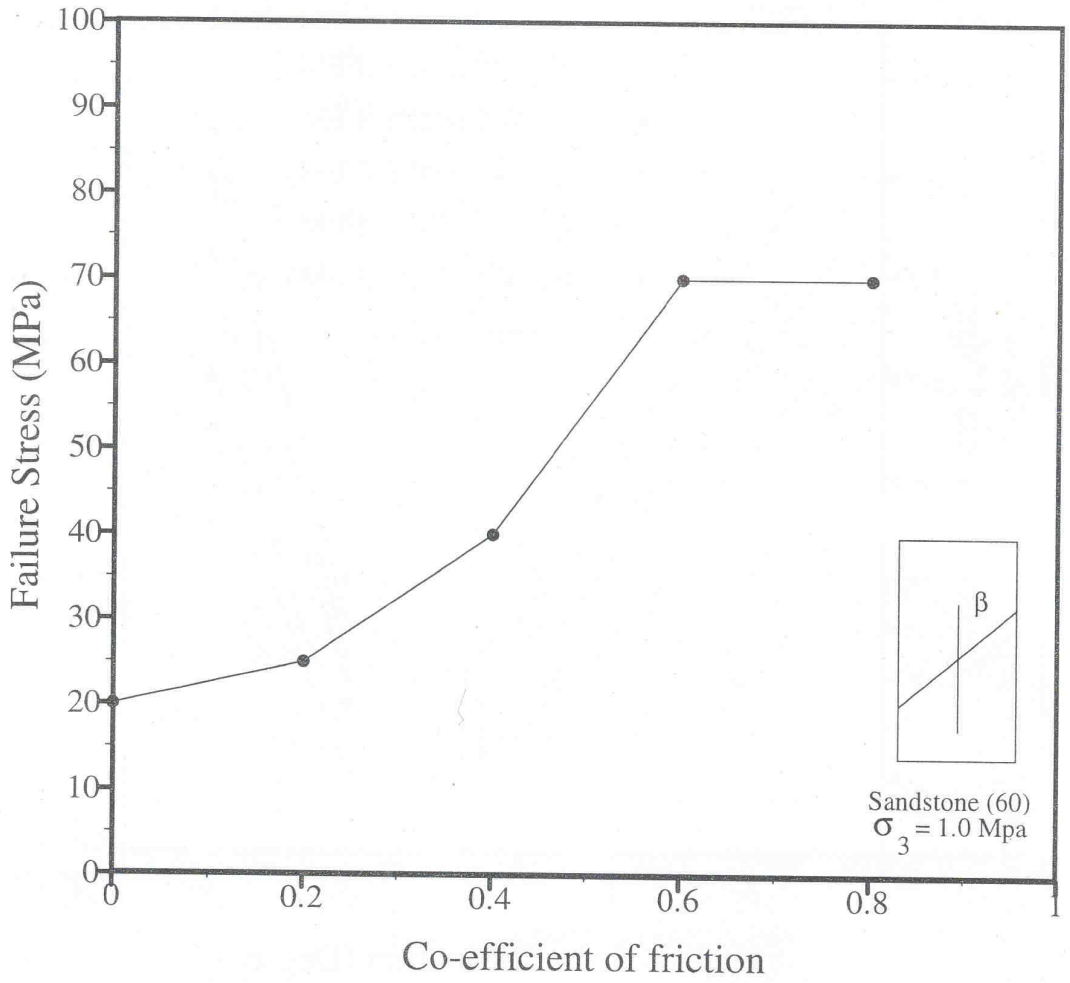



Fig. 9-Variation of failure stress with co-efficient of friction for single jointed specimen of sandstone ($\beta$=60°)

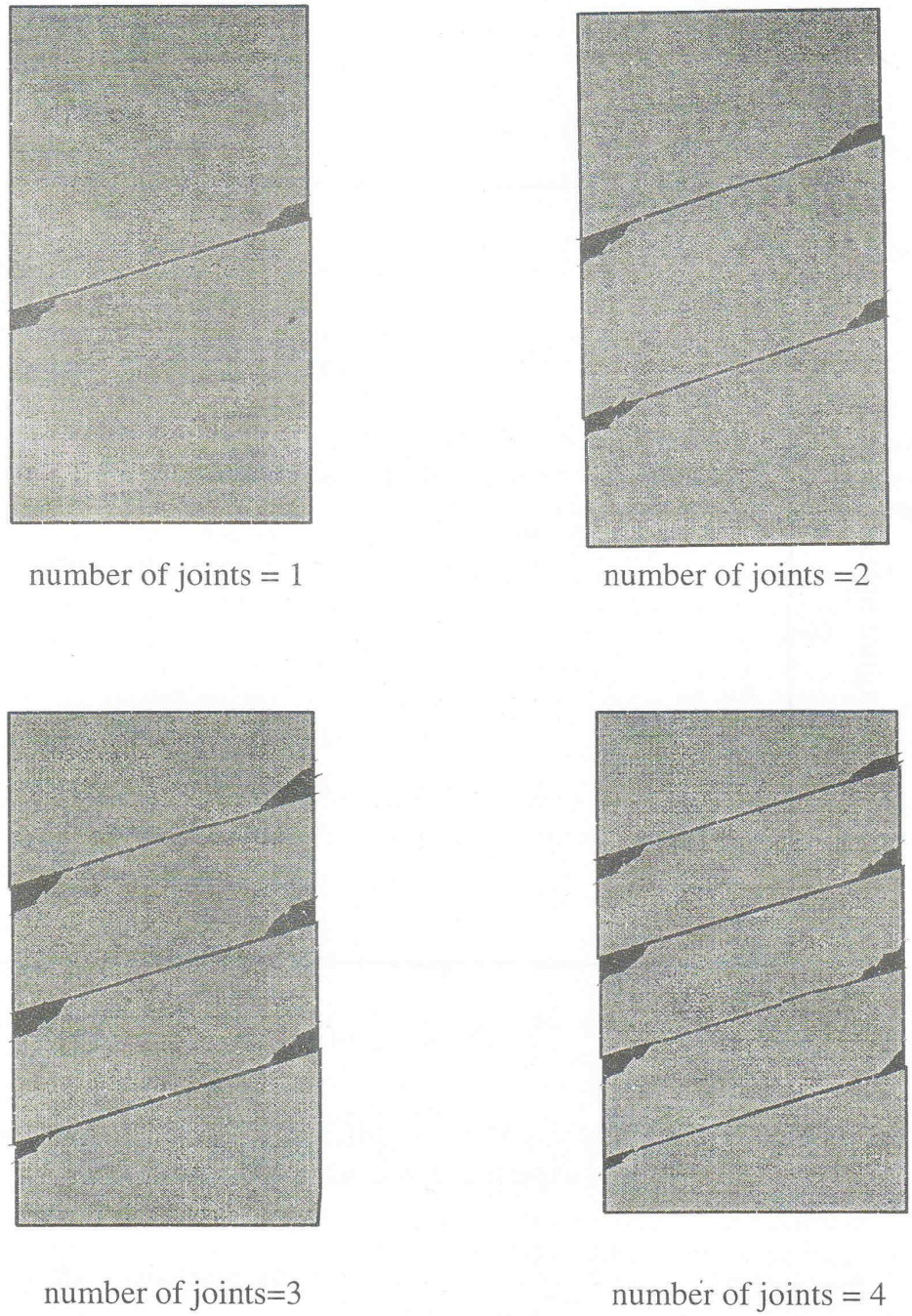



Fig. 10–Modes of failure for multiple jointed specimen of Agra sandstone with 2.5Mpa confining pressure and $\beta = 70^{o}$

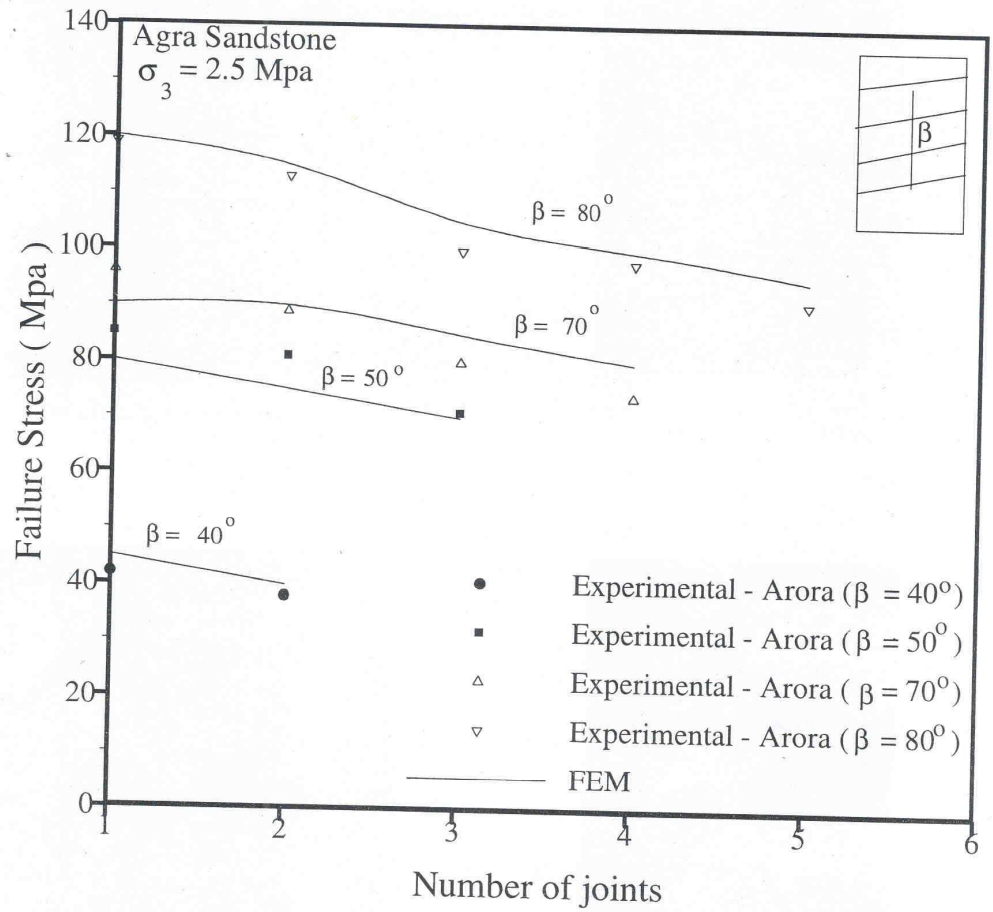



Fig. 11–Failure stress plots for different joint orientation along with the experimental results of Arora (1987)