C H A P T E R - III

EXPERIMENTAL INVESTIGATIONS FOR JOINTED ROCKS

3.1.0. GENERAL

The behaviour of sliding along joint has been the subject of experimental investigations for almost all the research workers. Various concepts as regards shearing behaviour of a rock is a result of number of investigations carried out using various testing techniques. It can be seen from the litereture that different systems of measuring friction properties along joint have been employed. The most notable among these systems are as follows:

(1) Slider sliding over another surface.

(2) Conventional shear box test.

(3) Double shear test.

(4) Triaxial test.

(5) Rotation of cylinders.

(6) Insitu shear test.

The principles underlying the above methods can be represented as in fig. 3.1. Recognizing the influence of testing techniques on the findings of investigation, attempts have been towards a developement of systems in which various factors affecting the behaviour of jointed rock can be investigated.

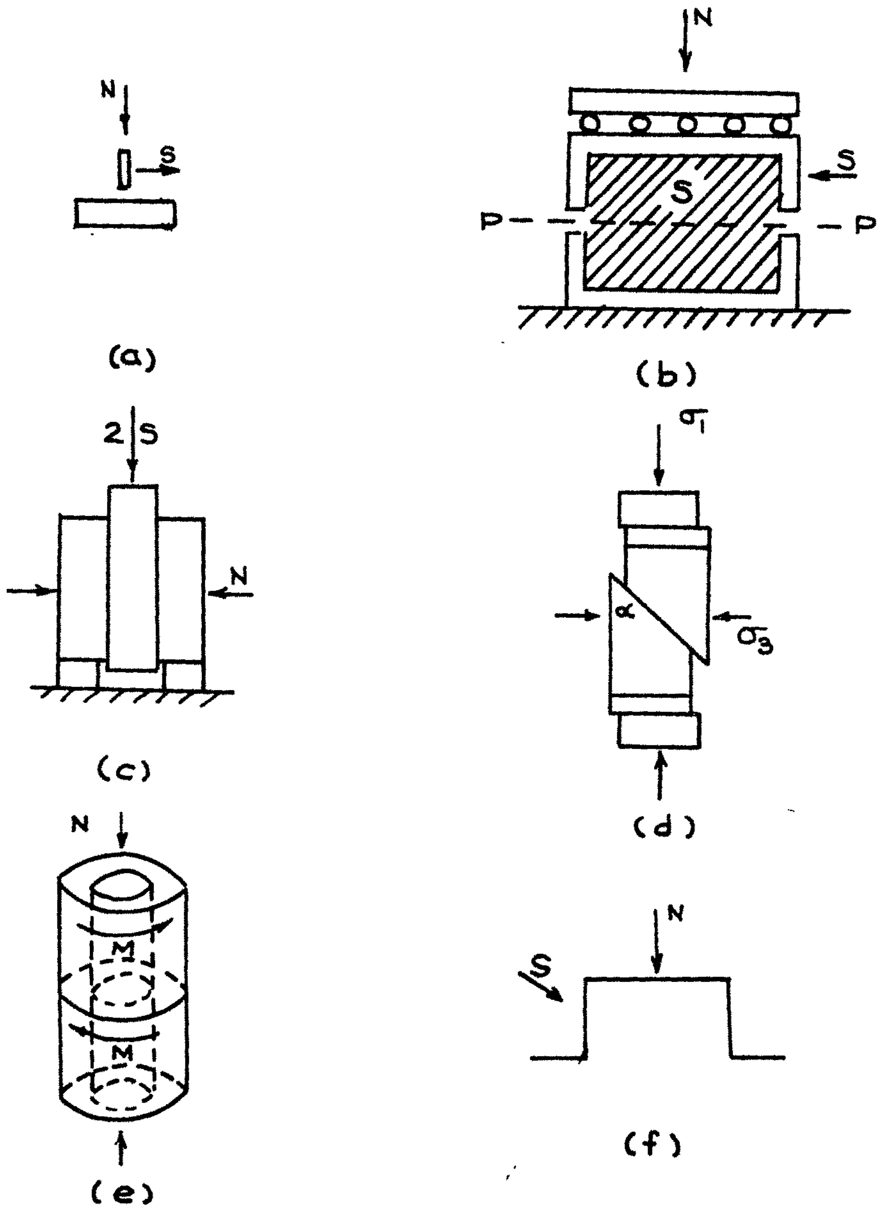


FIG. 3-1 SYSTEMS OF MEASURING FRICTION PROPERTIES ALONG JOINTS

(a) SLIDER SLIDING ON ANOTHER SURFACE
(b) CONVENTIONAL SHEAR BOX TEST ARRANGMENT
(c) DOUBLE SHEAR TEST ARRANGMENT
(d) TRIAXIAL TEST ARRANGMENT
(e) ROTATION OF CYLINDER
(f) IN SITU SHEAR TEST

3.2.0. TESTING TECHNIQUES

3.2.1. Slider sliding over another surface

Earlier investigations for sliding characteristics of minerals and rock Horn and Deere (1962), Byerlee (1967) Jaeger and Cook (1969) adopted a testing set up normally employed in the study of metal friction, According to Bowden and Tabor (1950) there are two varieties of testing setups. In the first case a small slider is made to slide on large surface in which case normal load is limited, (Fig.3.1 a) and the larger surface always remains fresh and same slider surface remains in contact. The method is more suitable to study wear rather than friction and does not represent actual condition occuring in nature. Renger (1971) modified the technique by using two sliding blocks of different sizes and with the provision for larger surfaces. To ensure the uniformity of normal load over the centre surface, Renger developed a loading system in which the normal force is applied by the air pressure rubber bellows. (Fig.3.2). The shear force is applied to a set of two cylinders and the shear force is measured by a load cell. The system could achieve regulation of normal pressure without loss of force, uniform normal load similar to dead weight load independent of the slider position and programmability for various conditions of loading.

3.2.2. Conventional direct shear test

A number of investigations like Yevdokimov and Sapegin (1967), Krsmanovic (1967), Hoek and Pentz (1969), Lama (1974 b) used conventional shear box test to investigate

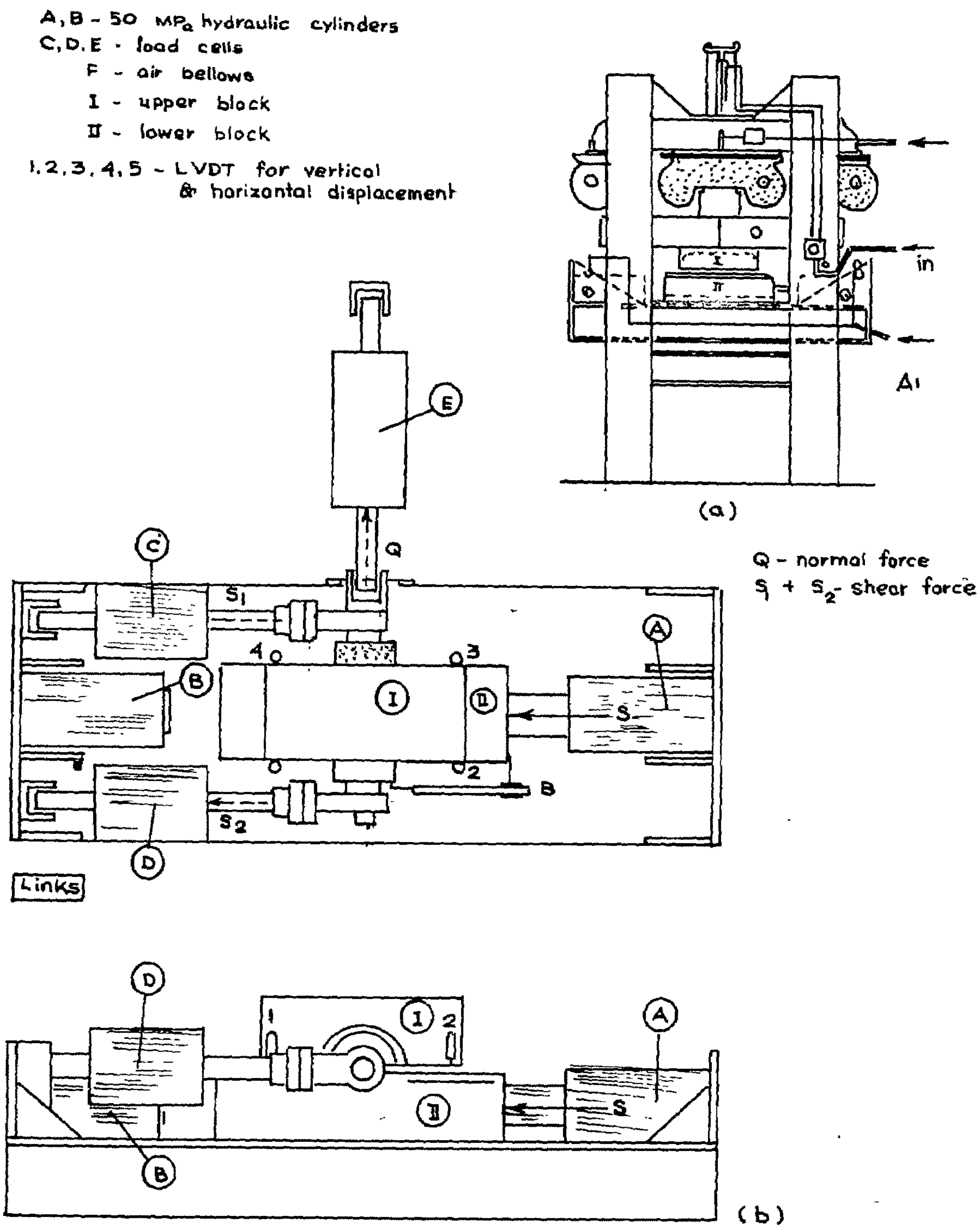


FIG. 3·2 LARGE FRICTION MACHINE

(a) OVERALL VIEW OF MACHINE

(b) LOCATION OF CYLINDERS, LOAD CELLS & L.V.D.T.s

(after RENGERS, 1971)

the behaviour of jointed rocks. The method consists of setting the rock specimen,prismatic or cylindrical or of irregular shape with joint plane at the mid-half of the shear box. Most notable testing setups are that of Locher (1968) Krsmanovic (1967) and Patton (1966 a).

3.2.2.1. Locher setup

Figure 3.3 shows a typical arrangement as used by Locher (1968). The rock specimen is casted in mortar with the joint plane accurately located at the predetermined position in the mould. Two hydraulic jacks exert a normal force and a shear force. The test equipment is capable of giving a movement of 1 cm. Because of the use of proving rings, the value of the normal force gets altered due to the influence of dilation or contraction. Since the normal and shear forces are recorded simultaniously normal-shear force envelope is readily obtained.

3.2.2.2. Krsmanovic setup

Figure 3.4 shows the large size laboratory setup to determine the joint properties. The normal force is applied through two hydraulic cylinders of 0.25 MN each and shear force by four cylinders two on each side of 0.25 MN each. The setting of apparatus is such that the applied shear force makes an angle of 4 degrees with the shear surface so that shear strength even at small normal load can be determined with minimum of disturbances. The specimen diamensions are 40 cm x 40 cmx 20 cm with shearing area of 1600 cm^2. It simulates the loading intensity of large civil engineering foundation structures. The measurement of dilatation is not possible

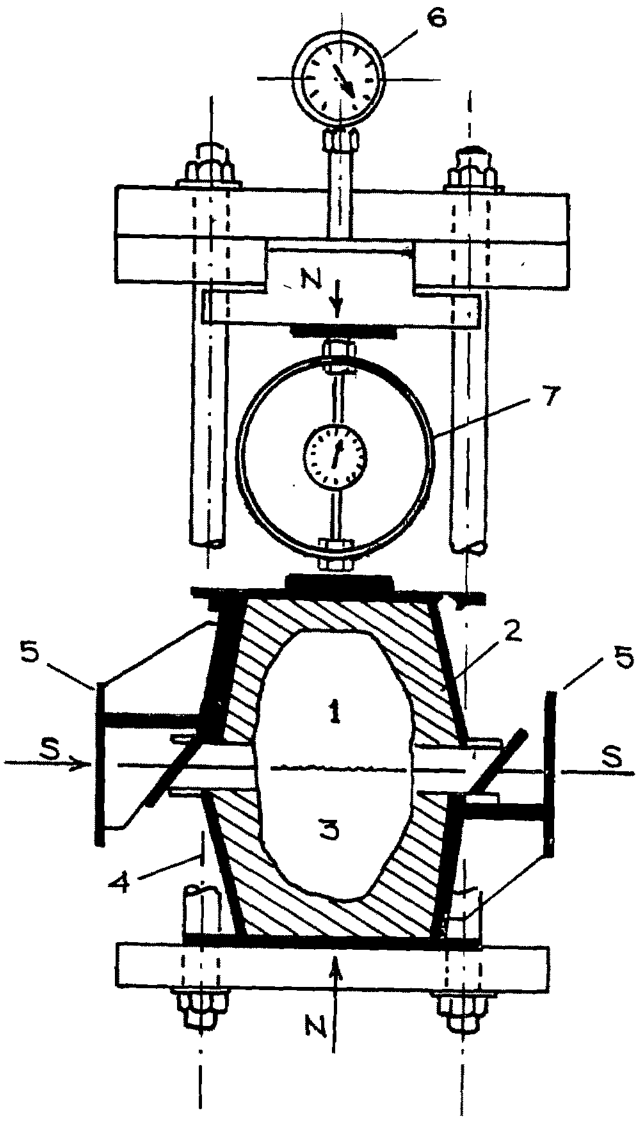


FIG.3·3 SCHEMATIC LAYOUT OF SHEAR TESTING APPARATUS

1. Sample 2 Mortar 3 Discontinuity
4 Double steel form 5 Exchangable shoes transmitting force S 6 Manometer for high loads & control of force N 7 Proving ring for force N (Proving ring for force S not shown)

(after LOCHER, 1968)

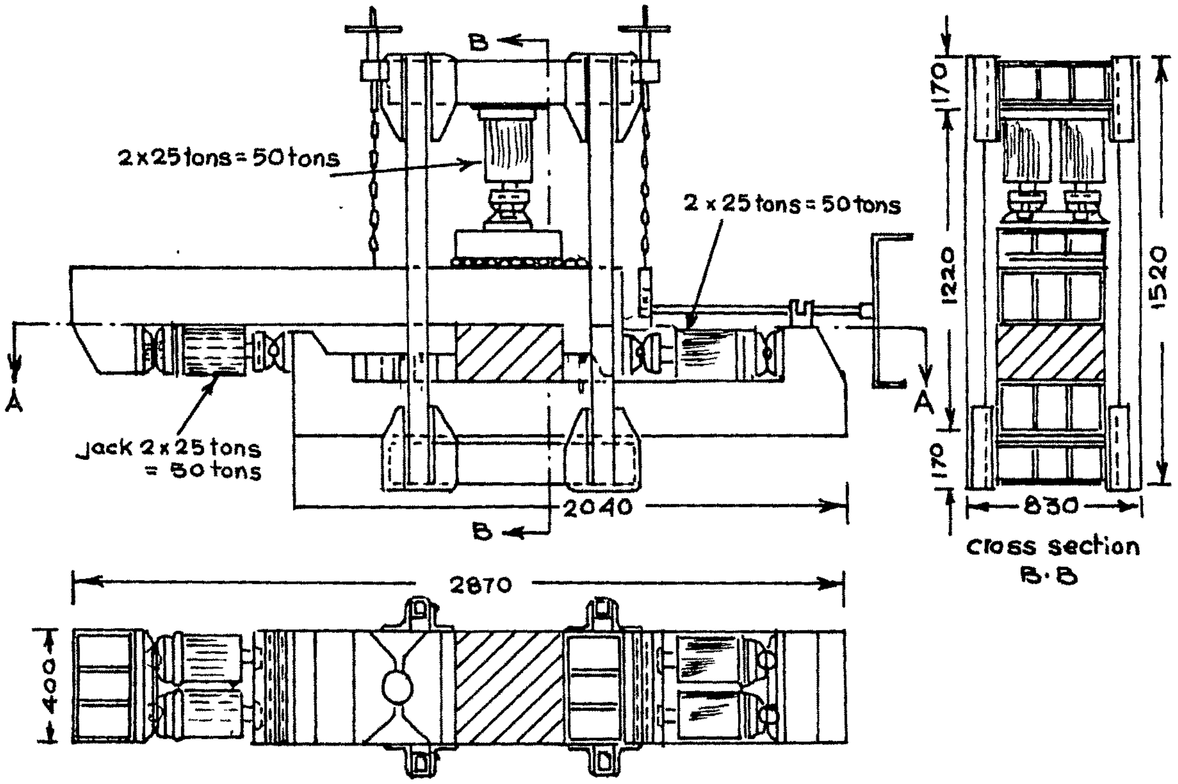


FIG. 3·4 SCHEMATIC DIAGRAM OF THE 0·5/1·0 MN (50/100 tons) SHEARING APPARATUS (after KRSMANOVIC, 1967).

since the vertical load is applied through hydraulic cylinders. Also this system has disadvantage that the vertical constraint of the specimen is uncertain.

3.2.2.3. Patton setup

Figure 3.5 represents the test setup by Patton (1966). It is the modification of a direct shear machine in which normal force is stationary and the bearing friction has greately been eliminated by making the lower block moving on the roller bearings while the upper block remains in a stationary position. The shear force is measured by two load cells while vertical and horizontal displacements are measured by LVDTs. The system has capability to measure the value of dilation at different values of normal pressure.

3.2.2.4. Desai, Drumm and Zaman setup

Figure 3.6 shows a cyclic multi degree freedom shear device by Desai, Drumm and Zaman (1983) which is consisting of a structural loading frame with inside dimension 147 cm x 147 cm and a height of 152 cm. A frame is designed to withstand a vertical or horizontal load upto about 27 KN and frequency of loading can range from monotonic to dynamic upto 5 Hz. The loading mechanism involves an electro hydraulic system of two hydraulic actuators, one in vertical and other in horizontal. The maximum load capacity of each actuator is 62 KN. A 21 MP hydraulic pump with 113.55 LPM capacity supplies a required pressure. The actuators are connected to the load cell to a capacity of $\pm$ 52 KN. The static or slow test is performed by applying the load very slowly. For the cyclic loading the applied forcing function

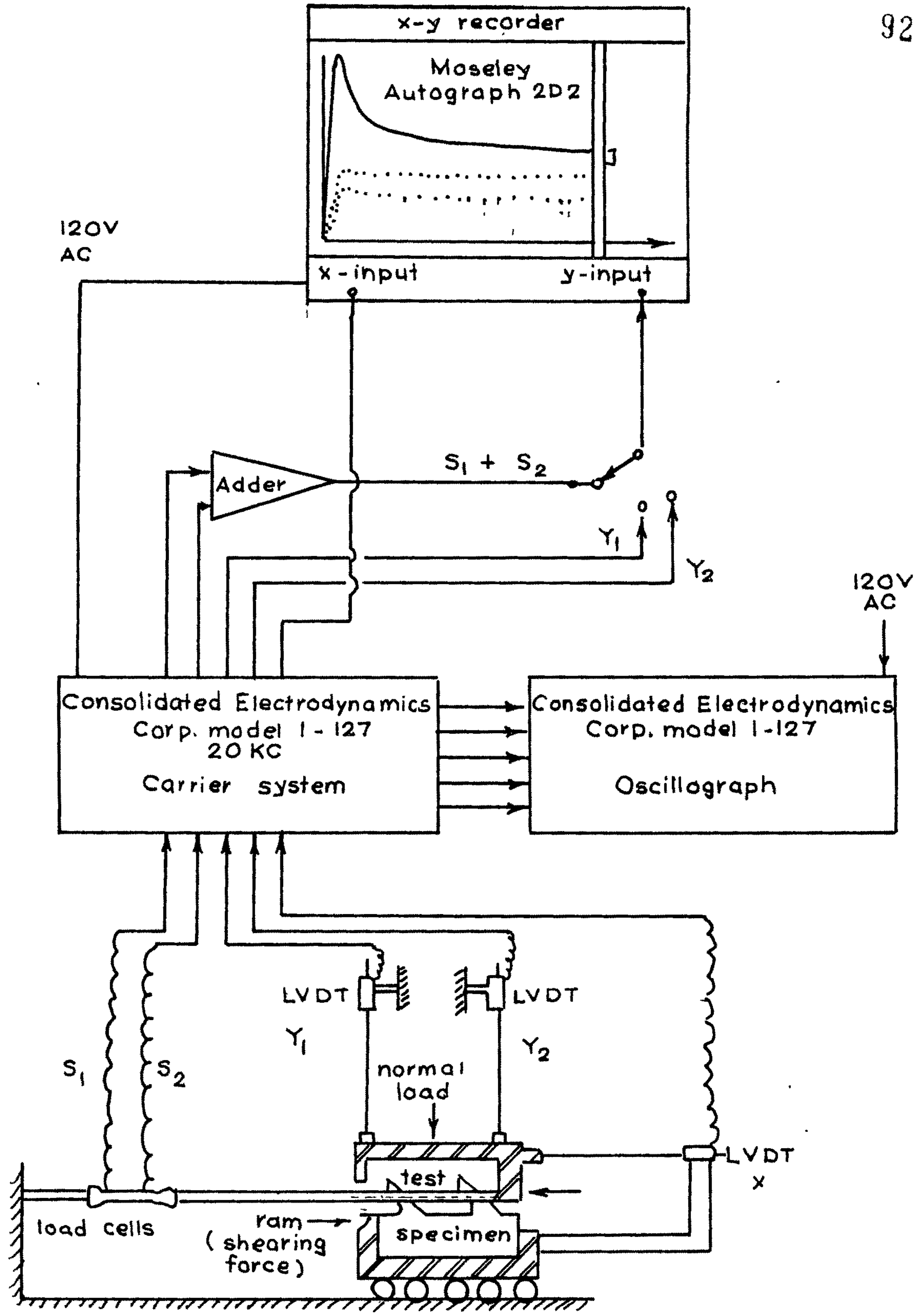
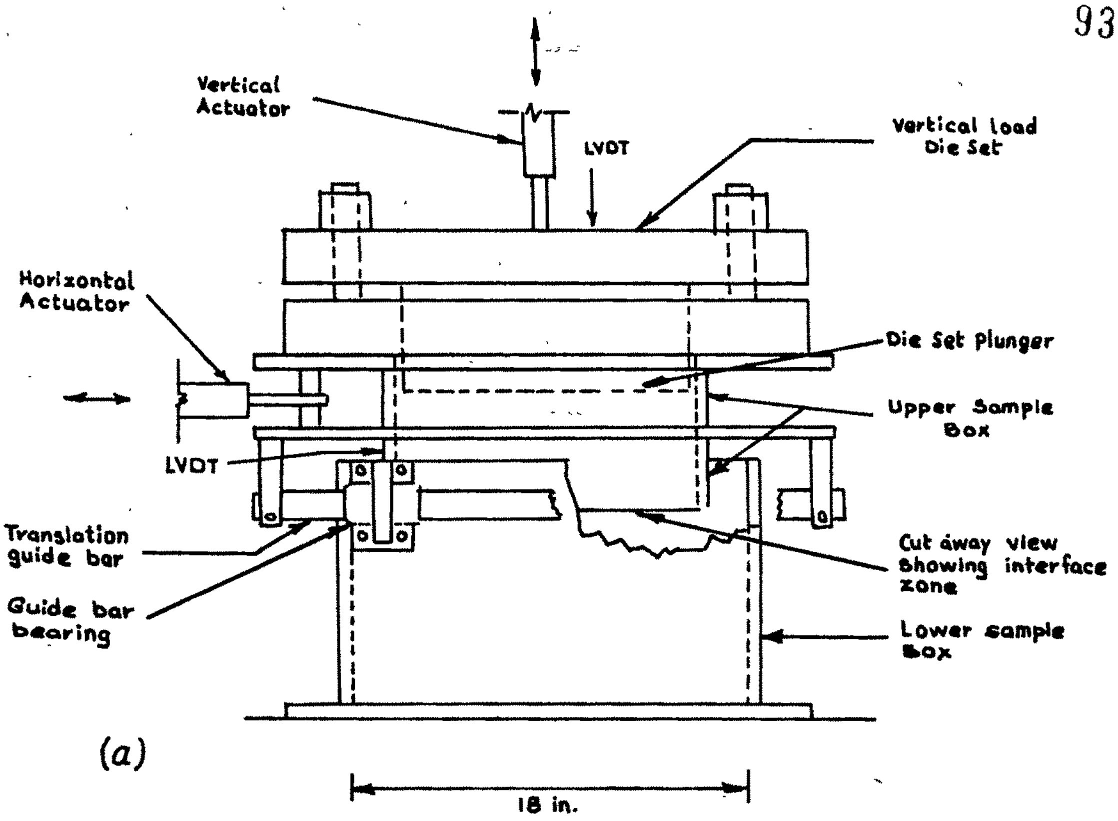
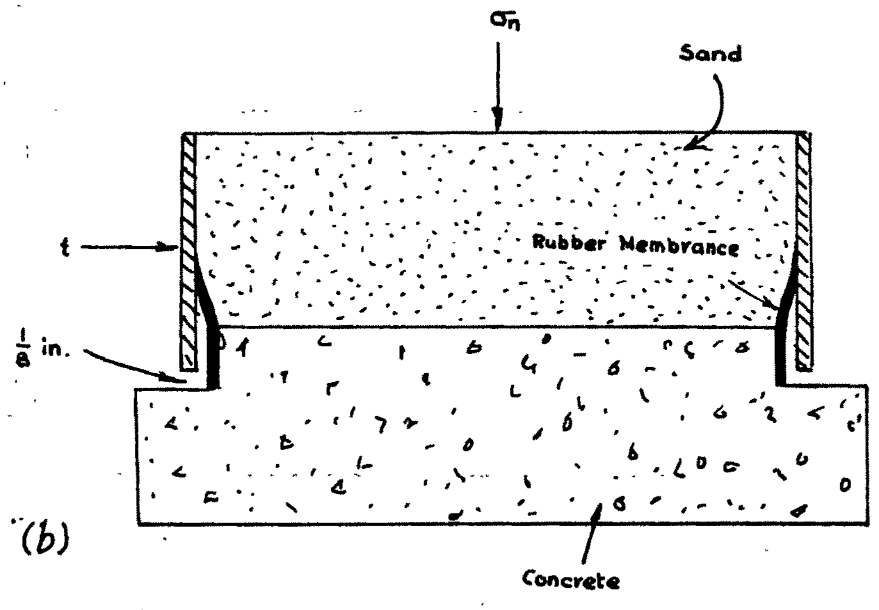


FIG. 3·5 SCHEMATIC DIAGRAM OF THE TESTING AND RECORDING EQUIPMENT.
(after PATTON, 1966 a).



(a)



(b)

FIG. 3·6 DETAILS OF TRANSLATIONAL TEST BOX (a) TEST BOX. (b) SCHEMATIC OF MEMBRANE ARRANGEMENT is sinusoidal and for the static test it is ramp.

3.2.2.5. Double shear test

The modification of single shear test is the double shear test (Fig. 3.1 c). Number of investigations notably by Hoskin, Jaeger and Rosengren (1968), Jaeger and Rosengren (1969), Dieterich (1972) used 30.5 cm square blocks sliding between two other blocks supported at the base, the lateral load being applied through a flat jack. The method is particularly useful for the determination of friction along the artificially prepared surfaces but is not considered suitable for studying the discontinuties.

3.2.3. Triaxial test

3.2.3.1. Conventional triaxial test

Triaxial appartus has been the most extensively used and perhaps the first method of studying the behaviour of rock discontinuities. U.S.B.R. (1954) employed the triaxial test to investigate the bond strength between mortar and aggregate, later on Jaeger (1959) for the study of variety of artifically prepared joints in rock. The method consists in using a cylindrical specimen with a joint plane suitably oriented at an angle inclined to the axis of the specimen subjected to a lateral and axial pressure in a triaxial cell. Because of the simplicity number of investigators viz: Lane and Heck (1964), Murrell (1965), Hobbs(1966,1970), Byerlee and Brace (1968), prefered a triaxial test to study the behaviour of jointed rocks. However the method suffers from certain drawbacks. Firstly, it does not permit independent variation of shear force and the normal force. Secondly, the method is not suitable for study under very low normal stresses. Thirdly, there are certain difficulties due to geometry of the testing appartus as dissuced by Hoskins et al (1968). Most of the investigators used either no spherical seat or one spherical seat at the top. As displacement proceeds the stress system changes so that the results are accurate only for the initiation of the sliding (Fig. 3.7). In the case of no spherical seat there are certain lateral stresses depending upon lateral stiffeness in the machines.

In view of the various studies the best method seems to be to use a pair of hardened steel washers with 'molybond' between the ends of specimen and the platens by which the behaviour can be studied in the range after the failure has been initiated.

3.2.3.2. True triaxial test

Several multiaxial devices have been designed and constructed, overcoming variety of difficulties, particularly the interferences of corner and edges of the prismatic specimens. In these devices the triaxial loading system have been designed such that it provides independent control of the applied pressure in each of the three principal loading directions. Main components of the loading system are diagramatically shown in fig. 3.8 a,b,c,d by Michelis (1985).

The cell designed by Michelis mainly consisting of cylinder in which the body is acting as reaction frame for the application of intermediate and minor pressures, having two cylindrical plates and two pistons of square cross sections with two cylinderical seats, two pairs of opposing

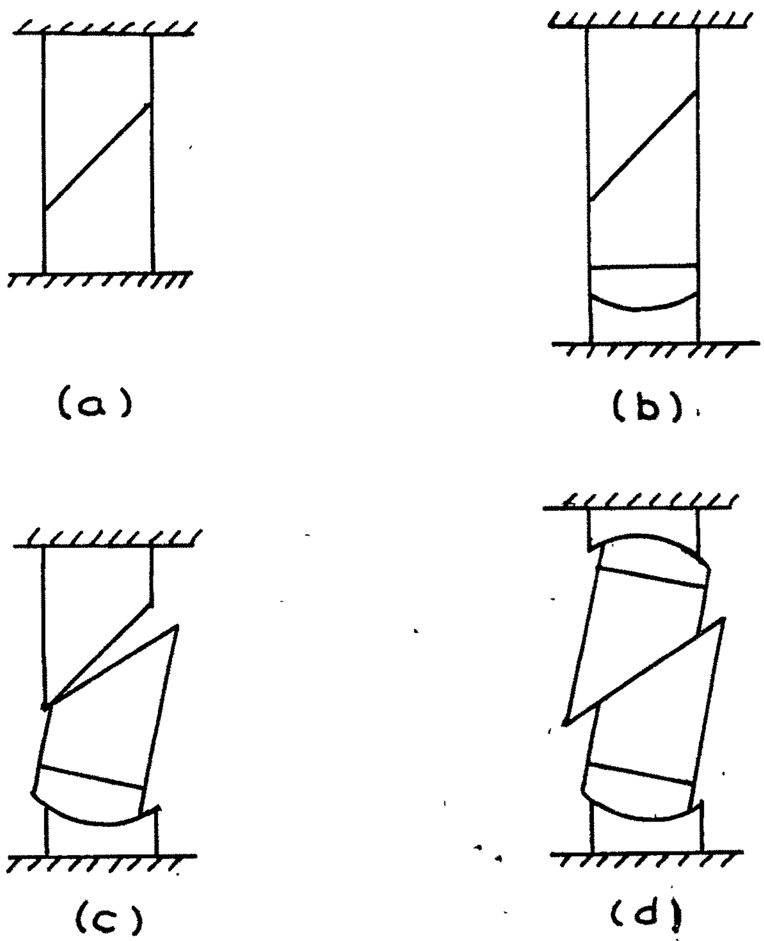


FIG. 3-7 EXPERIMENTAL ARRANGMENTS FOR DETERMINING FRICTIONAL PROPERTIES ALONG JOINTS IN A TRIAXIAL APPRATUS & THE INFLUENCE OF GEOMETRY

(a) NO SPHERICAL SEAT

(b) SINGLE SPHERICAL SEAT - At the start of the movement along the joint

(c) SINGLE SPHERICAL SEAT - After the progress of the movement.

(d) DOUBLE SPHERICAL SEAT - After the movement

(after HOSKINS et al, 1968)

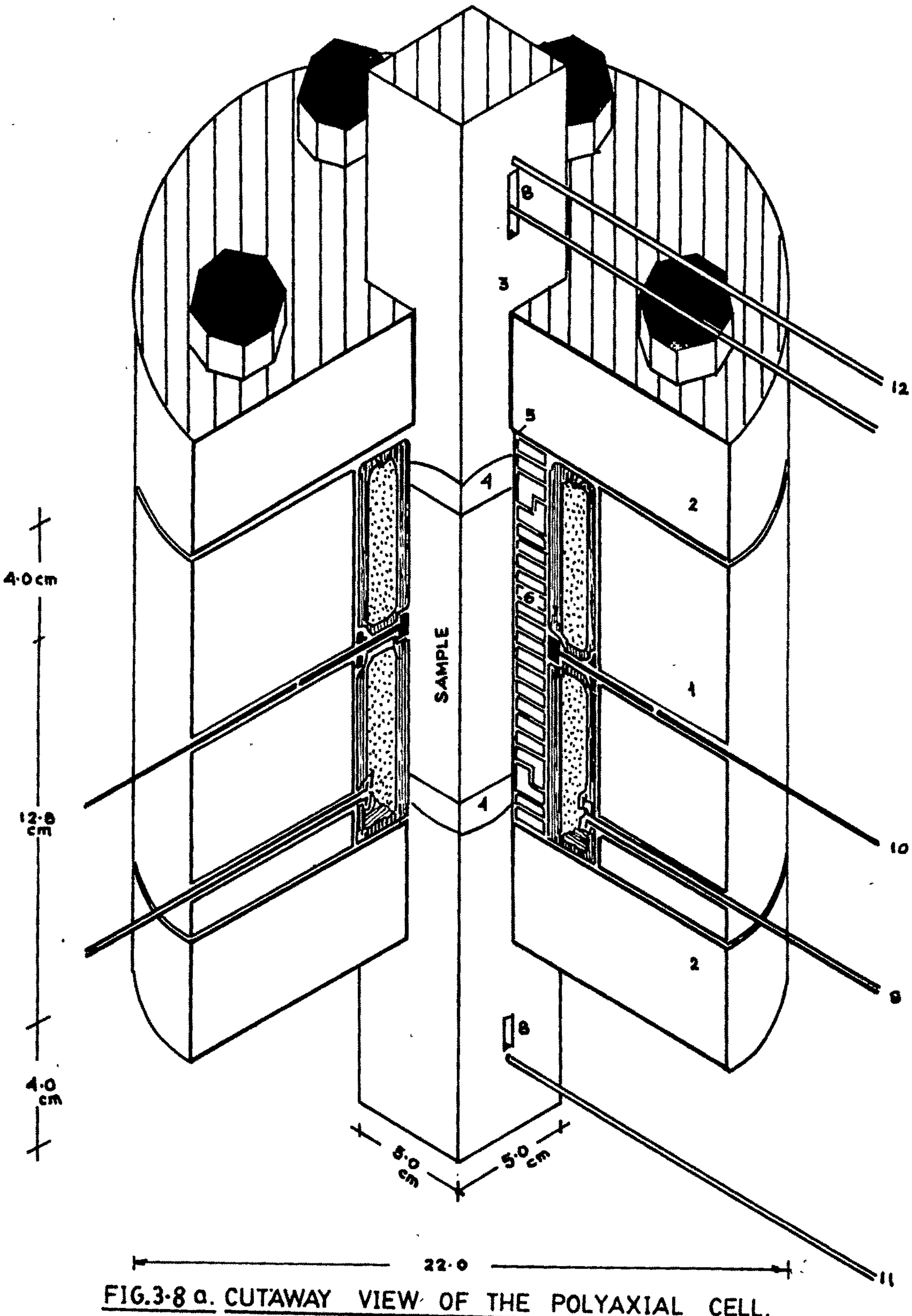


FIG.3·8 a. CUTAWAY VIEW OF THE POLYAXIAL CELL.

1. Cylindrical body; 2. Cylindrical plates; 3. Piston; 4: Spherical seats; 5. Two very thin copper sheets; 6: Steel prisms; 7. PVC bag; 8. (upper): Partial axial strain measuring rod; 8. (lower) exit for pore water & strain gauge cables; 9. High pressure tube for filling & pressurizing the PVC bags with oil; 10, Lateral strain measuring rod; 11, 12: Total axial strain measuring rods.

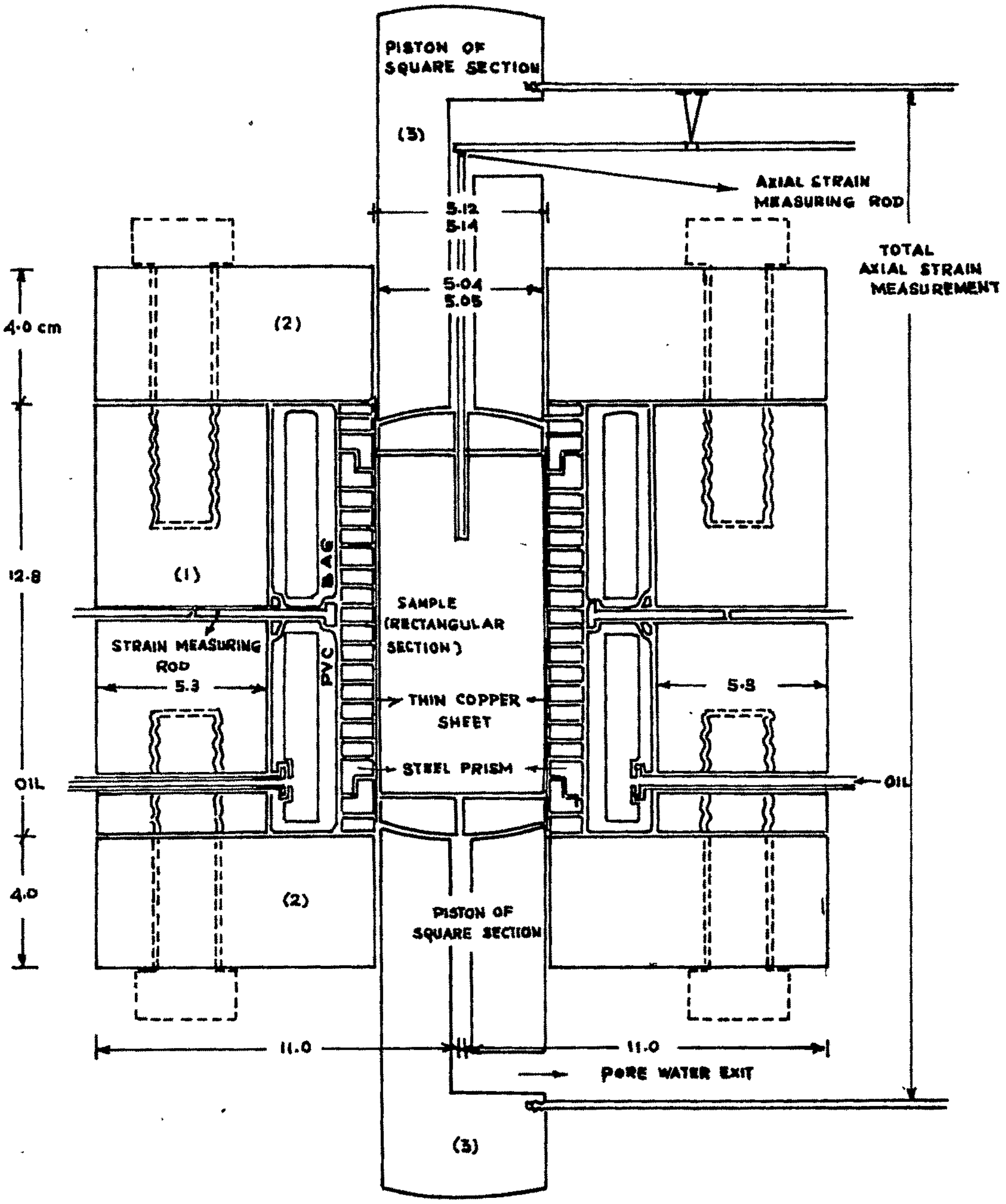


FIG.3-8 b. DESIGN OF POLYAXIAL CELL : Longitudinal section.

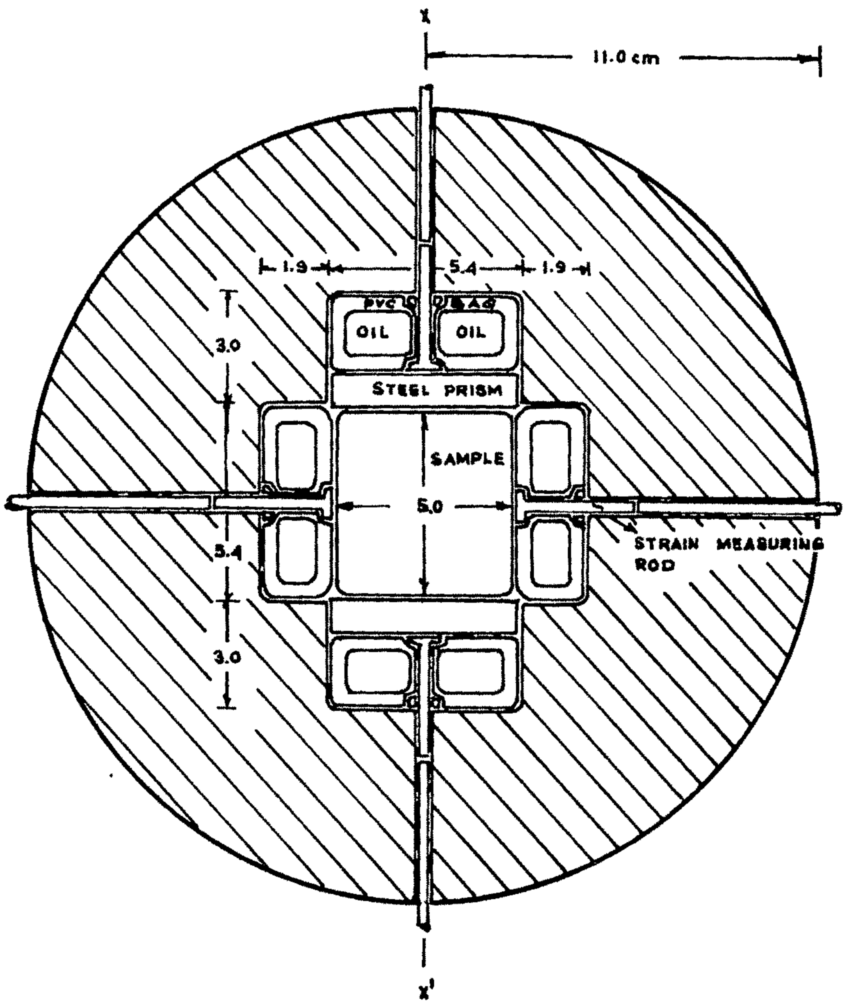


FIG.3·8 c. DESIGN OF POLYAXIAL CELL: Cross-section

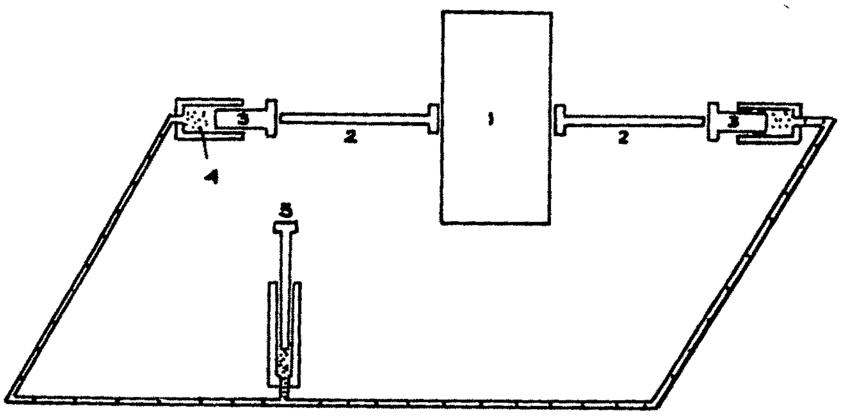


FIG.3·8d. SCHEMATIC PRESENTATION OF LATERAL DEFORMATION ($\varepsilon_2, \varepsilon_3$) MEASURING DEVICE.

prismatic PVC bags filled with the pressurised hydraulic oil acting as fluid cushion around a prismatic specimen 5 cm x 5 cm x 10 cm. The intermediate pressure is applied in the specimen by two series of steel prisms. The cell is symmetrical about longitudinal axis of the specimen along a plane cross-section passing through the centre of the specimen. Displacements are easily and accurately possible to measure at any part of the sample by the use of displacement dial gauges or transducers. Due to plane symmetry of the cell, frictional forces on the central part of the specimen are small. Two pressure vessles filled with hydraulic oil and pressurised by two servocontrolled testing machines are normally required to generate and control the minor and major principal stresses. The set up facilitates to conduct test under varities of stress paths. However, there could be drawbacks arising out of mix loading conditions at the specimen boundaries.

3.2.4. Rotation of cylinders

N.G.W. Cook (1965) devised a setup (Fig. 3.1 e) consisting of placing together two hollow cylinders under axial load and a torque is applied so as to rotate them along their axis. The two cylinders slide over each other at the surfaces of their contacts. The system has the advantage that large amount of sliding can be achived without disturbing the geometry of the system. Further it permits a study of influence of water on friction owing to availability of internal space in the hollow cylinder. The method is applicable to both the artificial as well as natural joints, when it is possible to obtain cores with the joint planes at right angle to core axis. The drawback of the system is a difficulty as regards the measurement of dilatation. A number of investigators notable among them are Kutter (1974), Christensen (1974), conducted laboratory studies on westerly granite whose friction values agreed to those obtained by Byerlee (1968) by triaxial test.

3.2.5. <u>Insitu testing</u>

Insitu tests are conducted to determine the joint properties on large scale particularly to investigate foundation conditions of structure and slope stability analysis. The method in principle is very simple in which a block contained a joint to be investigated is prepared and subjected to normal and shear loads. Fig. 3.9 shows a simple shear arrangment as used by LNEC, Lisbon. It essentially consists of making a block of 70 cm x 70 cm surounded by reinforced concrete or steel frame. The normal and shear loads are applied through cylinders. It is prefered to apply the shear force at an angle in order to limit the amount of excavation required for placing of the jack and to avoid the developement of tensile stresses due to bending. The results of Ruiz et al (1968) indicate the existance of non-uniform stress distribution at the base of the block and also the existance of tensile stresses. The best arrangement, therefore, is to apply the shear force to the plane to be investigated placed at the center of specimen. The large scale field shear test on rock has been reported in second Congress of International Society of Soil Mechanics and Foundation Engineering

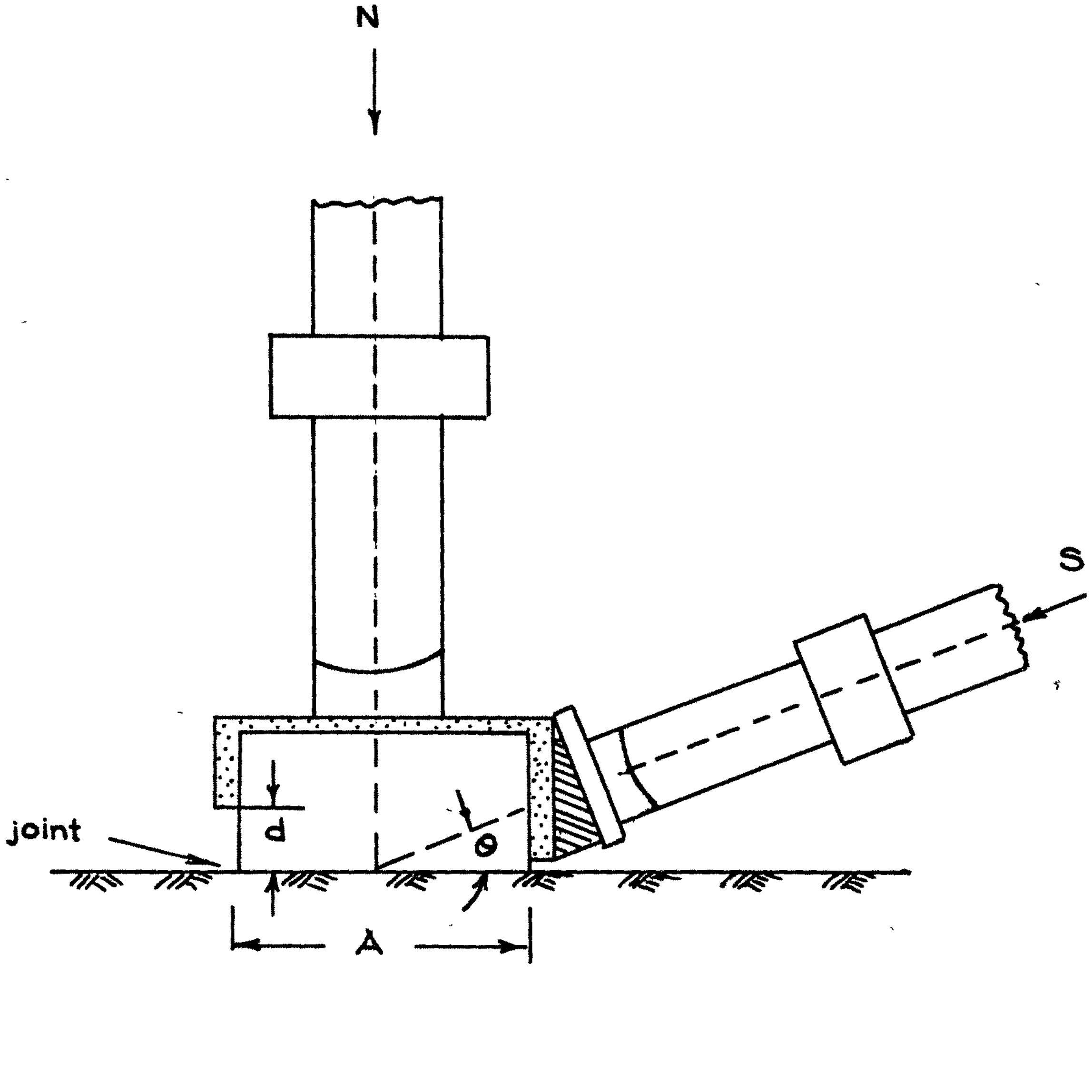


FIG. 3·9 IN SITU TESTING OF A JOINT PLANE.

conducted for Kasponayark concrete gravity dam of 100 m height in USSR. Recently Schurie Wiscold Garner (1985) conducted controlled displacement rate insitu test on predceious shale containing a thin clay seam with pore pressure measurements.

3.3.0. STRENGTH INVESTIGATIONS FOR JOINTED ROCK

3.3.1. Strength of jointed rocks in uniaxial compression test

A number of investigations have been made to study the shearing behaviour of jointed rocks in uniaxial compression. The major findings may be summarized as below:

(i) Boretti-Onyskiewcz (1966) studied the effect of direction of loading with reference to loading planes. In case of sand stone in air dried and water saturated condition it was found that the strength was considerably less if loaded along the bedding plane in comparision to the loading at right angle to the bedding planes.

(ii) Goldstein et al (1966) while testing composite specimens of different cube sizes of plaster of paris and sand mixtures of different strengths observed an asymtotic relationship expressed by equation:

$$\frac{\sigma_{c_m}}{\sigma_{c_e}} = a + b\left[\frac{l}{L}\right]^{\beta} \qquad \ldots.3.3.1$$

Where σ_{c_m} = compressive strength of the model.

σ_{c_e} = compressive strength of the element constituting the block.

l = length of each element.

L = length of the model.

a, b, β = constants where $\beta < 1$ and $b=(1-a)$

(fig. 3.10 and 3.11)

Similar results have been obtained by Cvetkovic (1974) who studied on the basis of ultrasonic longitudinal wave velocity.

(iii) Hayashi (1966) conducted test on the uniaxial strength of jointed specimens prepared out of plaster of paris with joints produced by inserting a wax paper during casting. He found that the uniaxial compression strength increased with the increase in number of joints, even if the total area of the intermittent joint was equal. He formulated a relationship expressed as:

$$\frac{\sigma_{c_n}}{\sigma_{c_1}} = 1 - \nu \sqrt{2 \log n n} + \nu \left[\frac{\log(\log n) + \log 4\pi}{2\sqrt{2 \log n}} \right] \quad \ldots .3.3.2$$

Where σ_{c_n} = uniaxial compressive strength of a specimen with n number of joints.

σ_{c_1} = uniaxial compressive strength of a specimen with one joint.

ν = coefficient of variation.

The results also revealed that the specimen having a joint with an inclination of 30 degree to the loading axis had the lowest strength.(Fig. 3.12 and 3.13)

(iv) Lajtai (1967) conducted tests on discontinuities in the form of interloacking teeth with various angles made with the direction of loading having varying width of teeth and observed a linear relationship between uniaxial compressive strength and the angle of inclination in the range of 30° to 54° with the minimum value at 30°. (Fig.3.14 and 3.15)

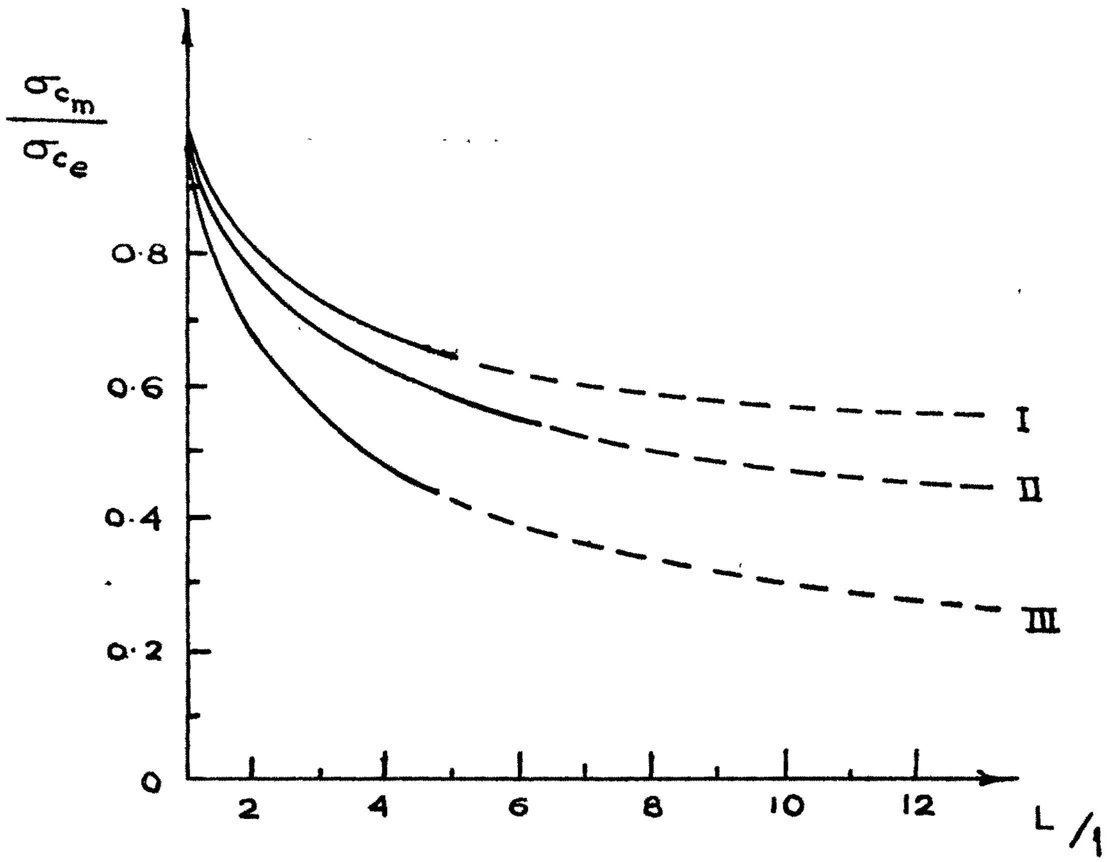


FIG. 3.10 THE RELATIVE STRENGTH OF THE COMPOSITE SPECIMENS.

I σ_{c_e} = 10 kgf/cm^2

II σ_{c_e} = 20 kgf/cm^2

III σ_{c_e} = 30 kgf/cm^2

(after GOLDSTEIN et al, 1966).

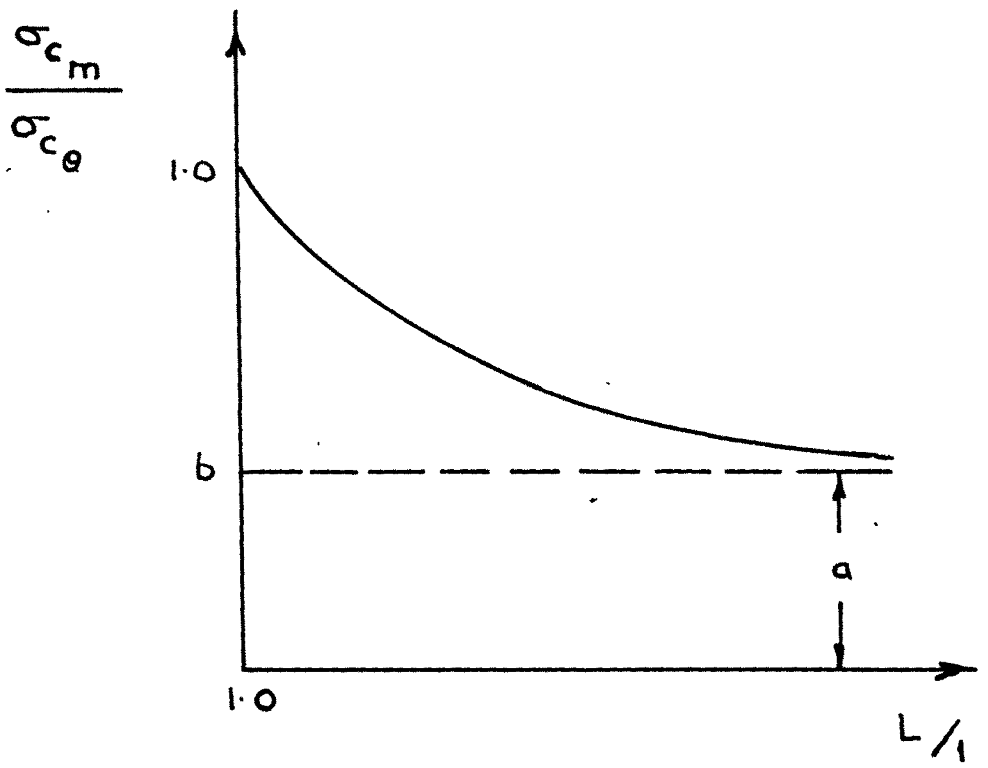


FIG. 3·11 THE RELETIVE STRENGTH OF MASS
(after GOLDSTEIN et al, 1966).

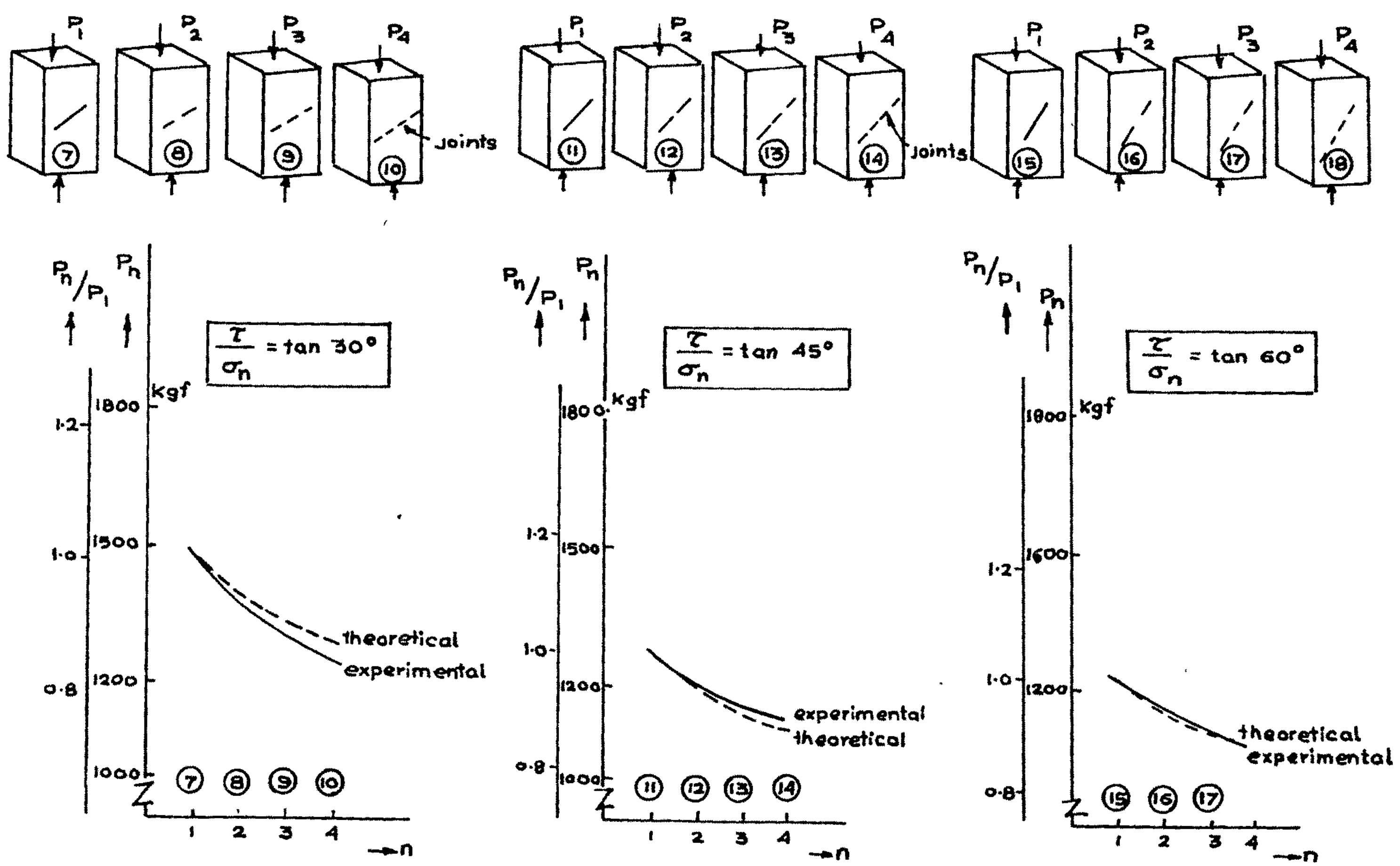


FIG. 3·12 COMPRESSIVE LOAD P_n DECREASES DEPENDING ON THE INTERMITTENT FREQUENCY OF JOINTS n REGARDLESS OF WHETHER THE TOTAL AREA OF JOINTS IS EQUAL. (after HAYASHI, 1966).

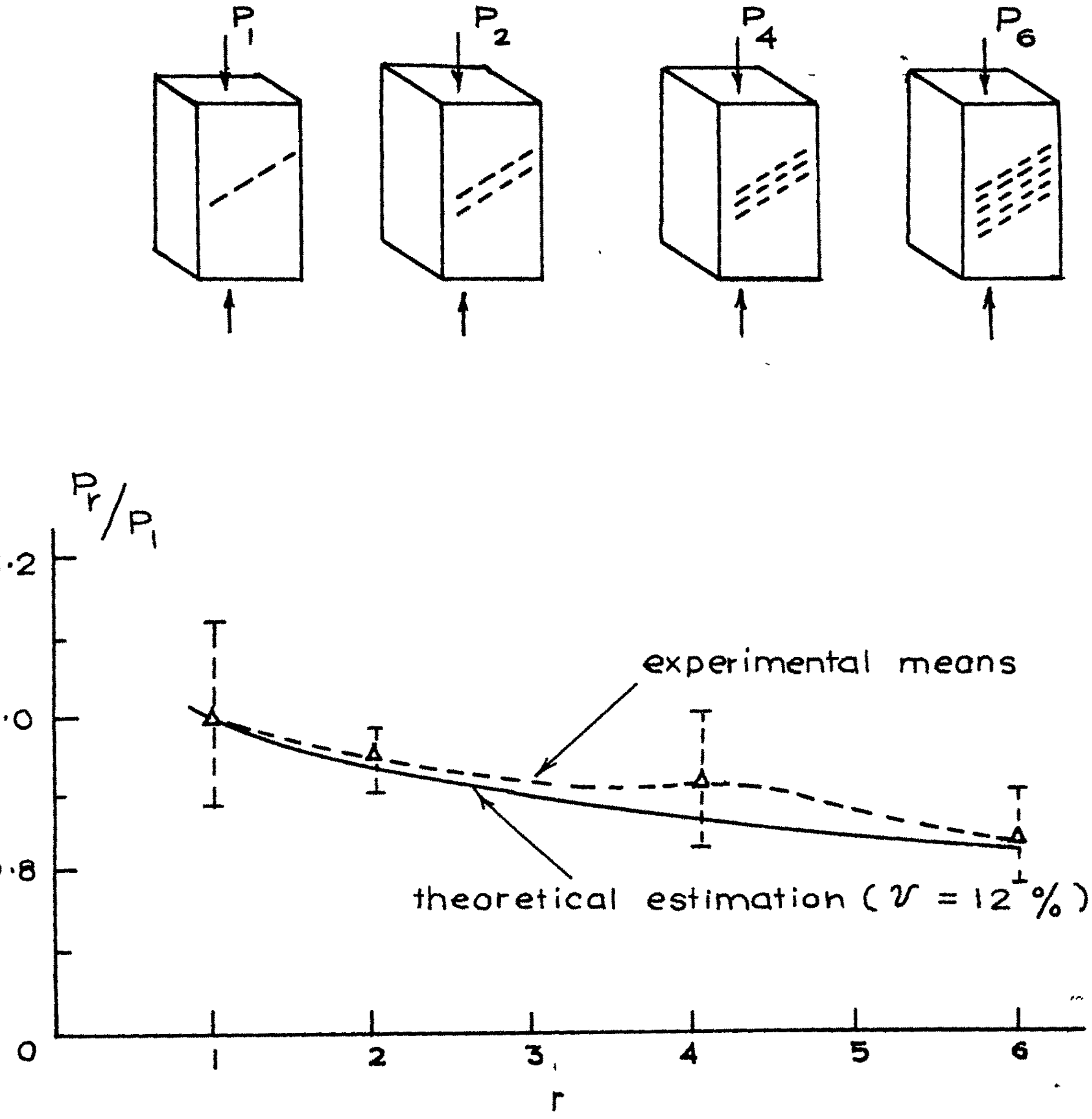


FIG. 3·13 COMPRESSIVE LOAD P_r DECREASES DEPENDING ON THE NUMBER OF PARALLEL ROWS OF JOINTS r
(after HAYASHI, 1966)

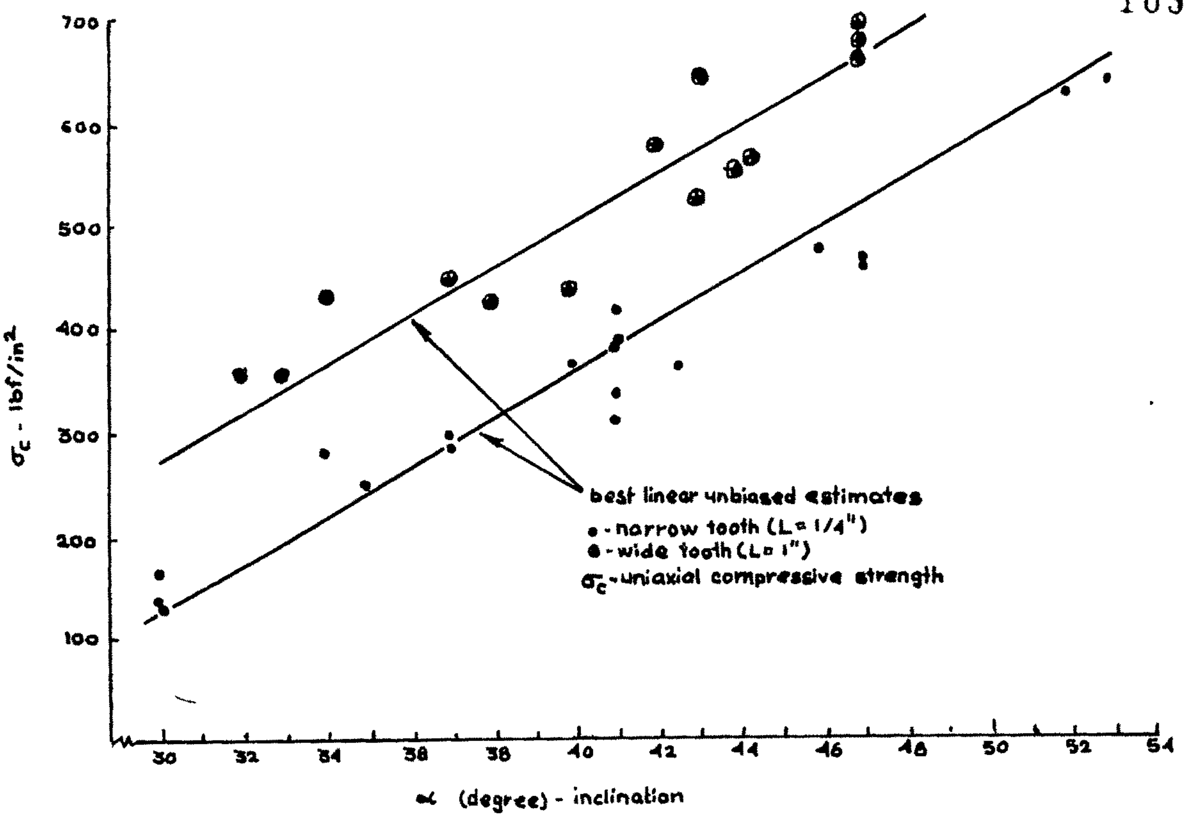


FIG. 3·14 THE RELETIONSHIP BETWEEN UNIAXIAL COMPRESSIVE STRENGTH (σ_F) & DISCONTINUITY INCLINATION (α)
(after LAJTAI, 1967)

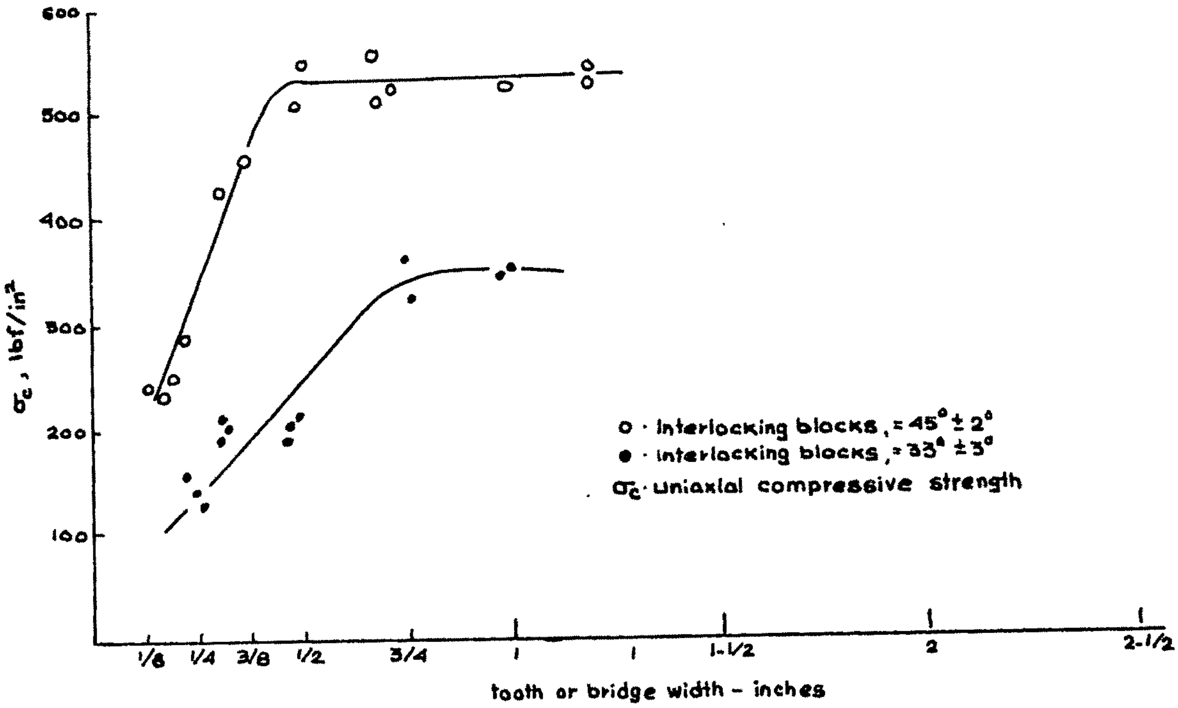


FIG. 3·15 THE DEPENDENCE OF UNIAXIAL COMPRESSIVE STRENGTH (σ_c) ON TOOTH OR BRIDGE WIDTH
(after LAJTAI, 1967)

(v) Horino (1968) determined the effect of angle and spacing of non cohesive planes of weakness on the compressive strength of lime stone, sand stone and granite cores. His studies indicated that the strength decreases rapidly as the angle of plane of weakness increases from 30° to 57° with respect to horizontal. (Fig. 3.16 and 3.17)

(vi) Bamford (1969) reported the effect of bedding planes on the compressive strength and mode of failure of silurian slit stone. For dips of bedding plane greater than 50°, the failure was a shear along these planes while for dips between 32° and 45° the combination of shear failure along bedding and axial cleavage fracturing was observed. Only axial cleavage fracturing (tensile failure) took place for dips flatter than 30°. (Fig. 3.18)

(vii) Akai, Yamamoto and Arioka (1970) performed test on two kinds of schists using cylindrical specimens to determine the change in the compressive strength due to inclination in the lamination. His results show that the compressive strength is maximum for inclination of lamination at 30° between axial stress and plane of laminations. While for 90° there is decrease in strength upto 75 to 90 %. (Fig.3.19)

(viii) Walker (1971) found that the strength decreases with increase in number of slabs from test on slabs of plaster placed parallel to each other under uniaxial compression. When the slabs were placed one above the other, the strength reduced slowly. It was observed that with five joints the value was 40 % of the unjointed rocks. (Fig. 3.20)

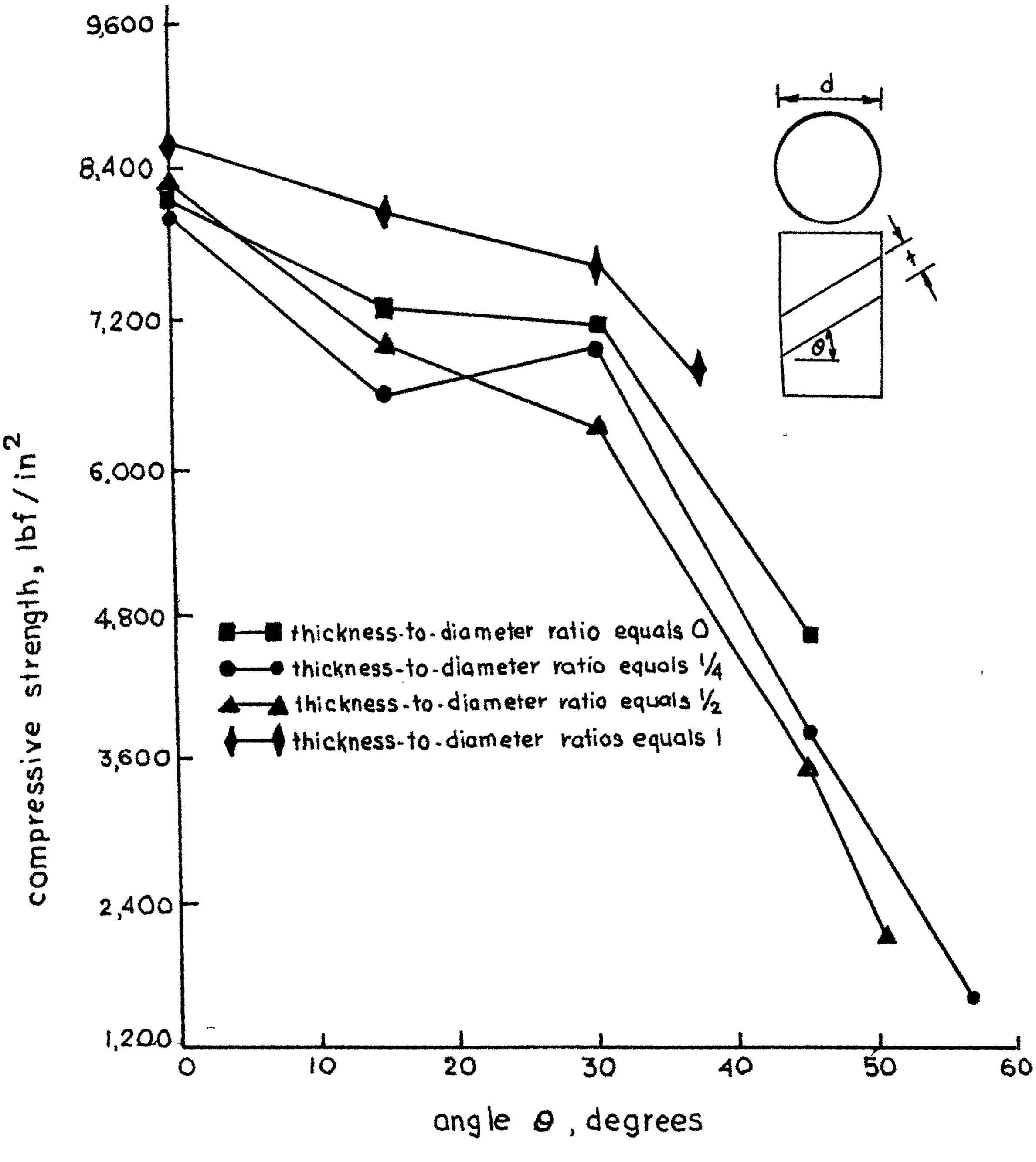


FIG. 3·16 COMPRESSIVE STRENGTH VERSUS INCLINATION OF FRACTURES FOR LIMESTONE

(after HORINO, 1968)

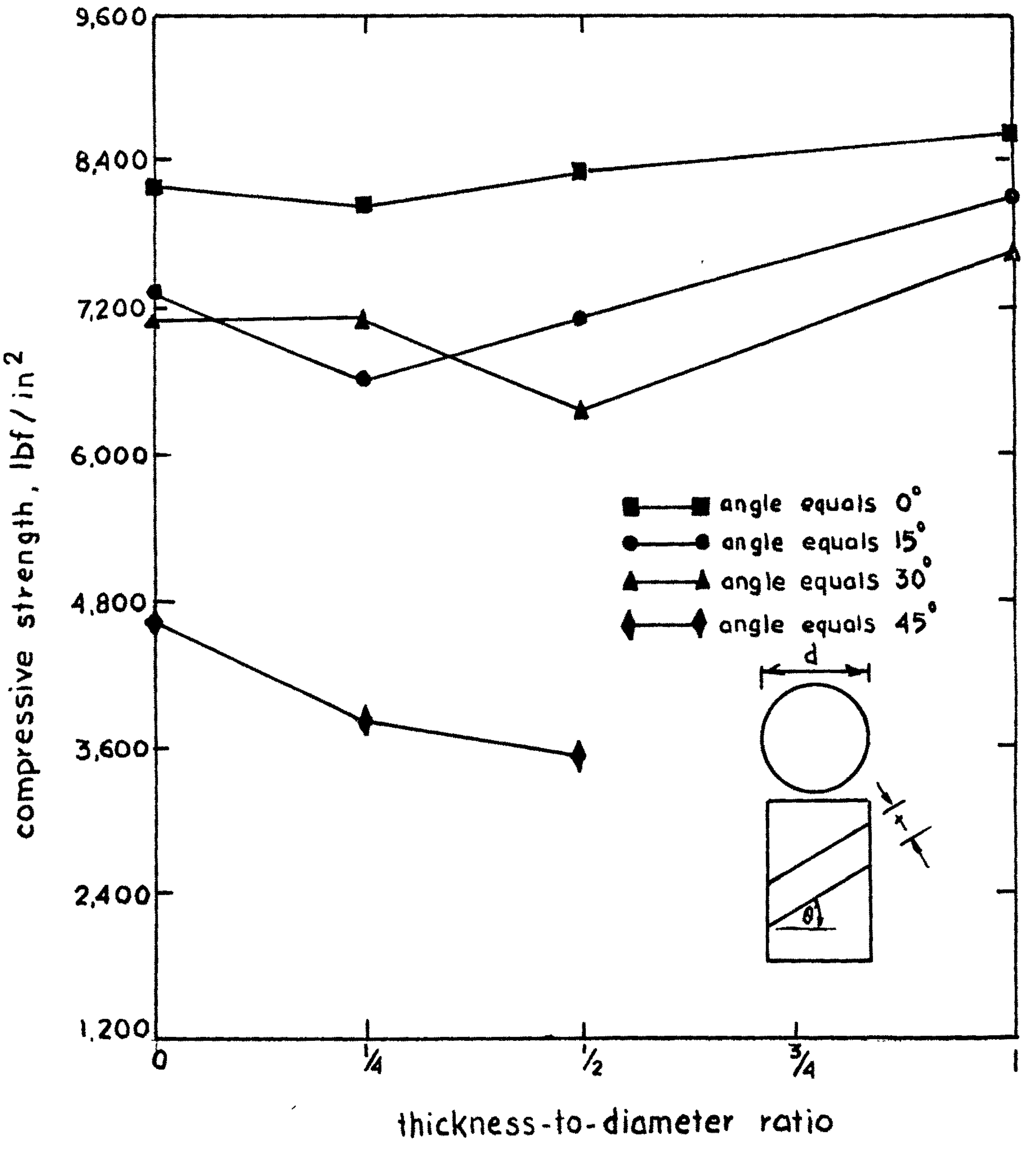


FIG. 3.17 COMPRESSIVE STRENGTH VERSUS THICKNESS-TO-DIAMETER RATIO FOR LIMESTONE (after HORINO, 1968).

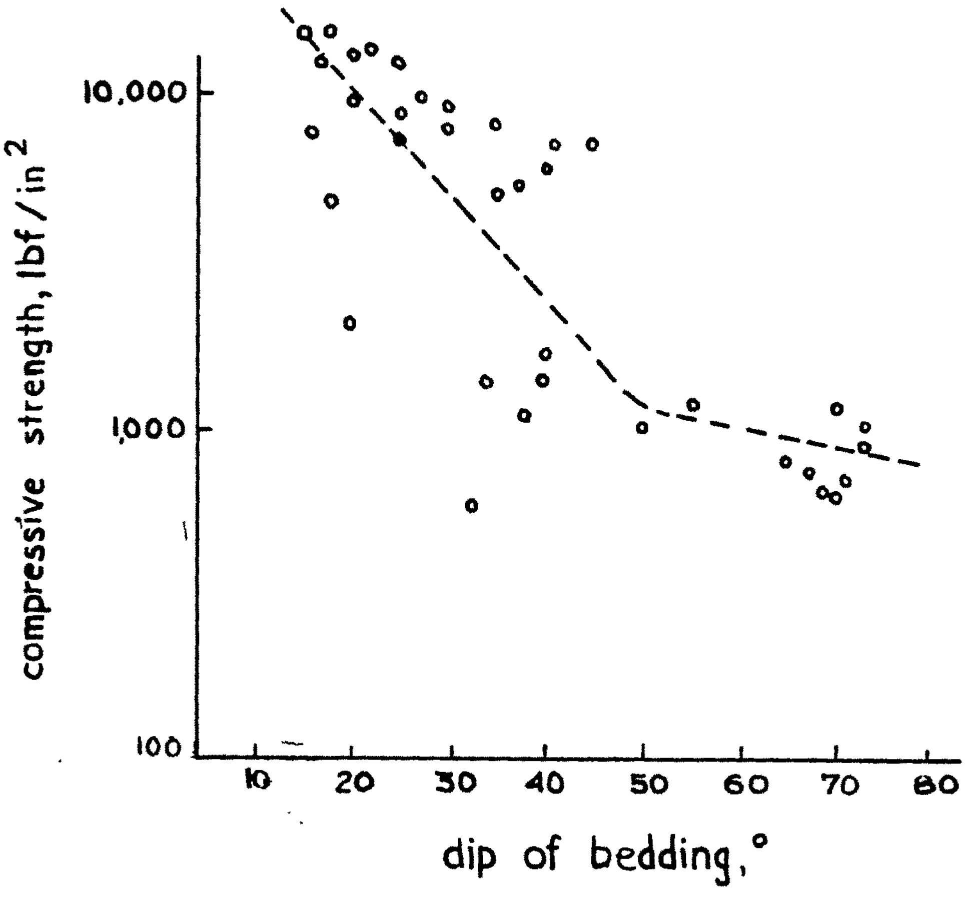


FIG. 3·18 COMPRESSIVE STRENGTH OF SILURIAN SILTSTONE CORE SPECIMENS (h/d = 1) VERSUS BEDDING DIP (after BAMFORD, 1969).

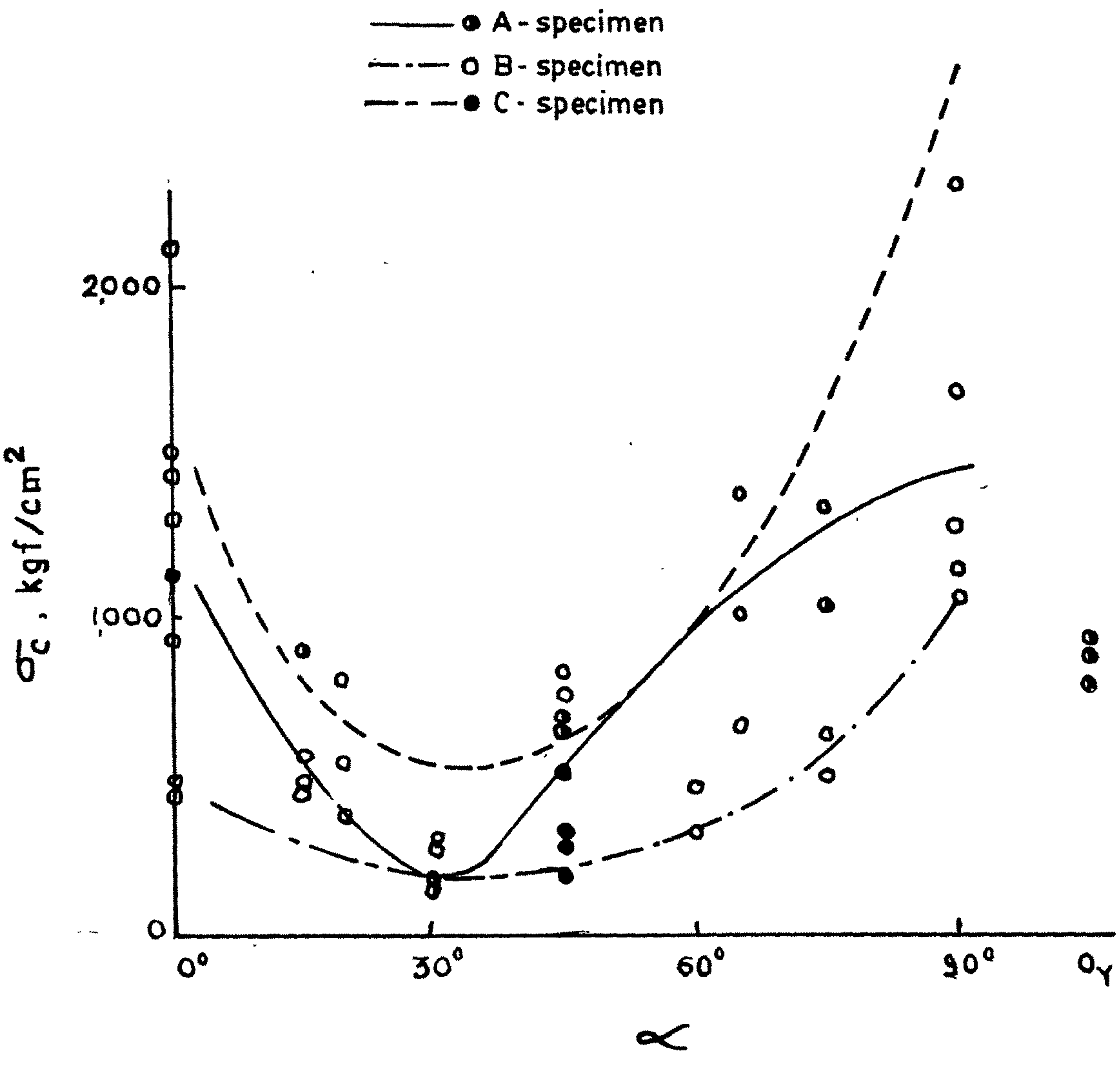


FIG. 3-19 CORRELATION BETWEEN THE INCLINATION ANGLE OF JOINTED PLANE AND THE COMPRESSIVE STRENGTH OF SCHISTS.
(after AKAI, YAMAMOTO and ARIOKA, 1970)

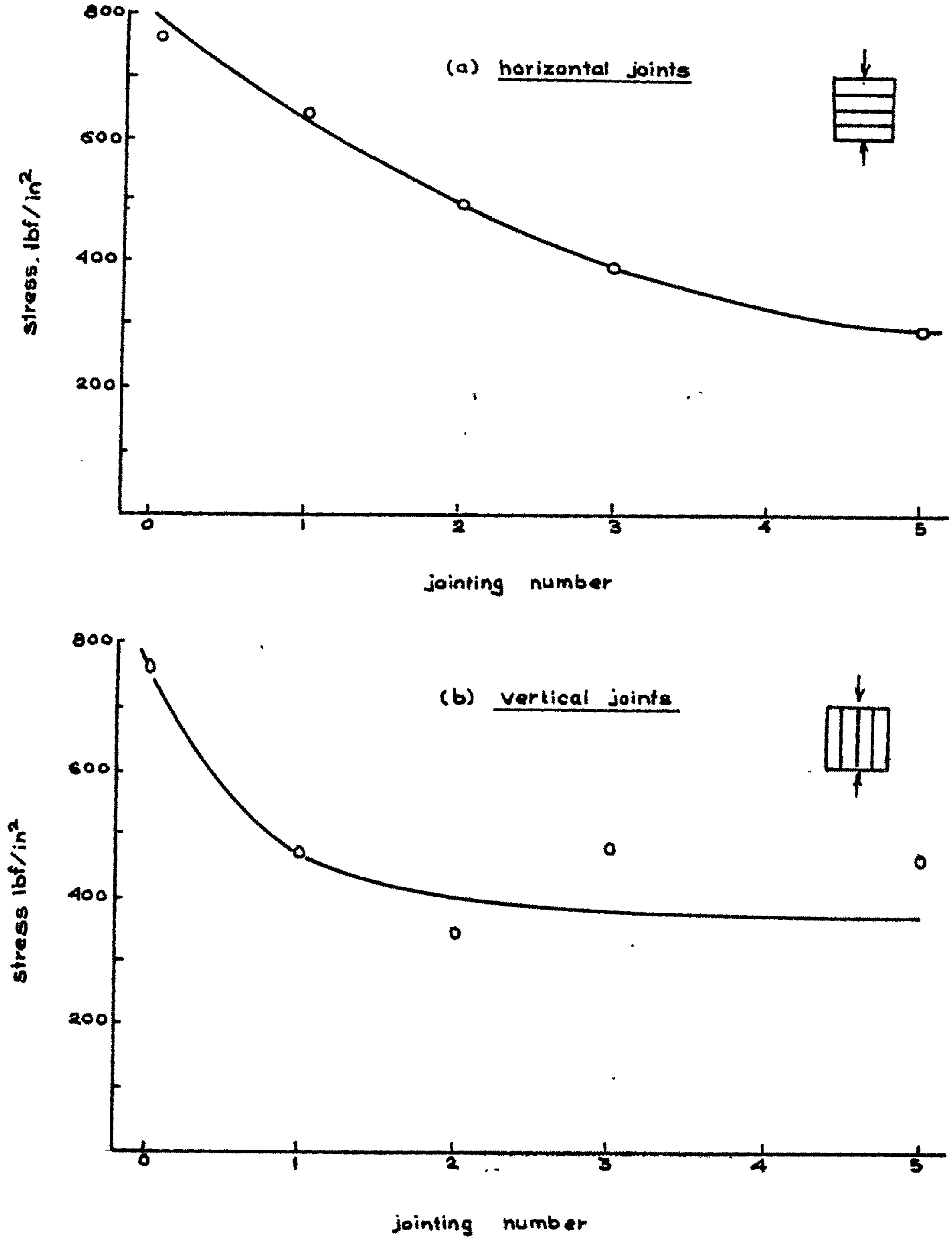


FIG. 3-20 UNIAXIAL COMPRESSIVE STRENGTH VERSUS JOINTING NUMBER.
(after WALKER, 1971)

(ix) Lama (1974 a) working on the effect of density of horizontal, vertical and orthogonal joints on compression of joint and deformation modules indicated that there is decrease in the strength of rock with horizontal or vertical joints upto 30 % while the modulus values are 30 to 40 % for horizontal joints and 60 to 80 % for vertical joints. He further observed that the failure of models with horizontal joints takes place with the developement of cracks at the middle at the joint planes and spreads upwards and downwards, while the models with vertical joints, the columns fail independently. With two different joint systems the mechanism of failure is different. In the case of horizontal joints the stiffness of the joints is important while in case of vertical joints the lateral constraint affects the modulus value. (Fig. 3.21 and 3.22)

(x) Gonano (1974) using orthogonal blocks investigated the mode of failure in composite models. He observed that the failure is progressive, starting with one or two elements and proceeding with more and more number of elements, ultimately fails with the increase in deformation. The mode of failure of oriented orthogonal jointed models is dependent upon joint orientation. He observed dilatation occuring along the joints in stages.

3.3.2. Strength of jointed rocks in tension

The strength of rock in tension has been regarded as one of the most vital parameter to understand the mechanical behaviour of jointed rock. The number of investigations have been carried out and findings therefrom can be summarized

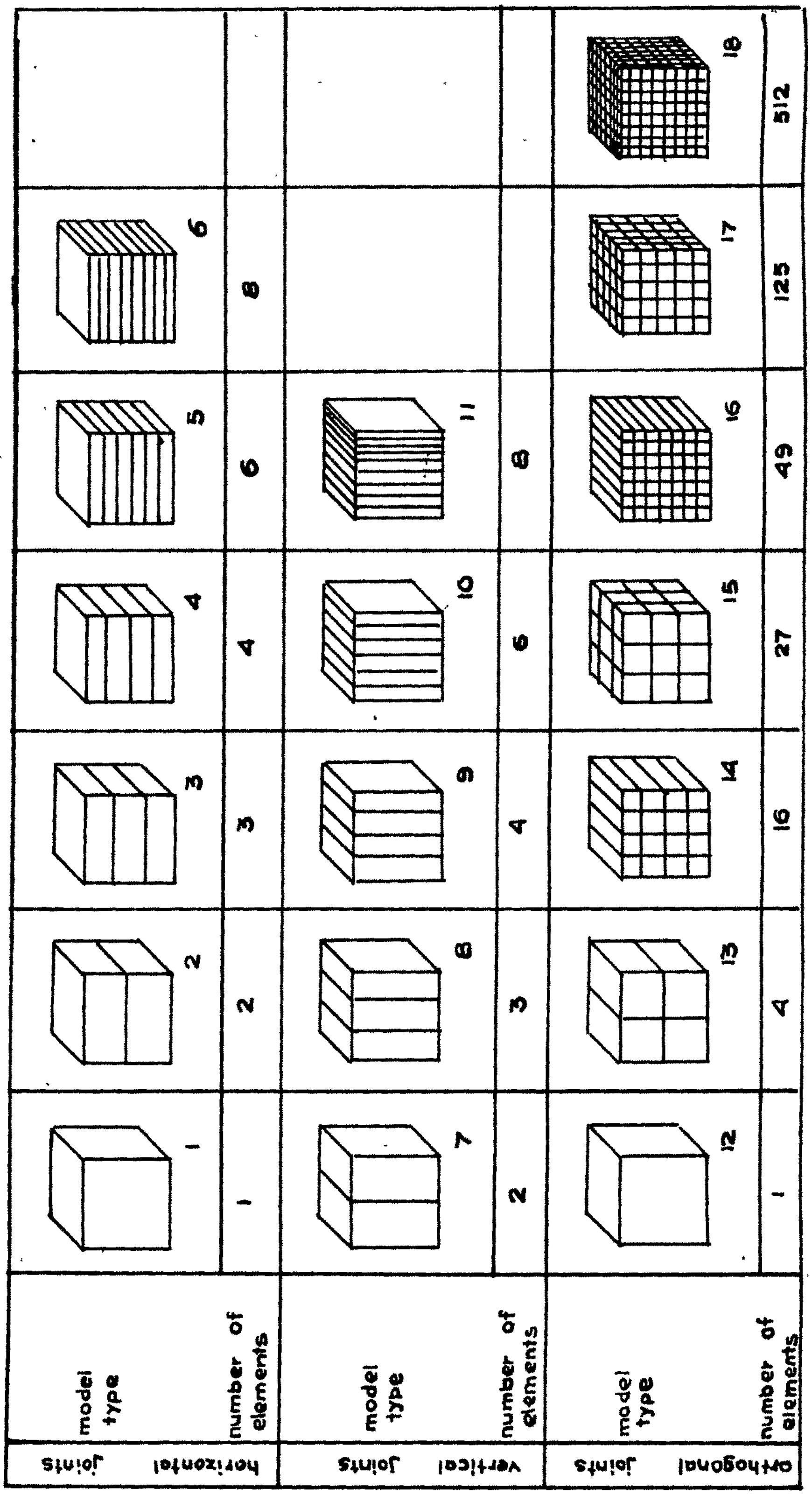


FIG:3·21 DIFFERENT TYPES OF COMPOSITE-MODELS USED IN TESTS.
(after LAMA, 1974 a)

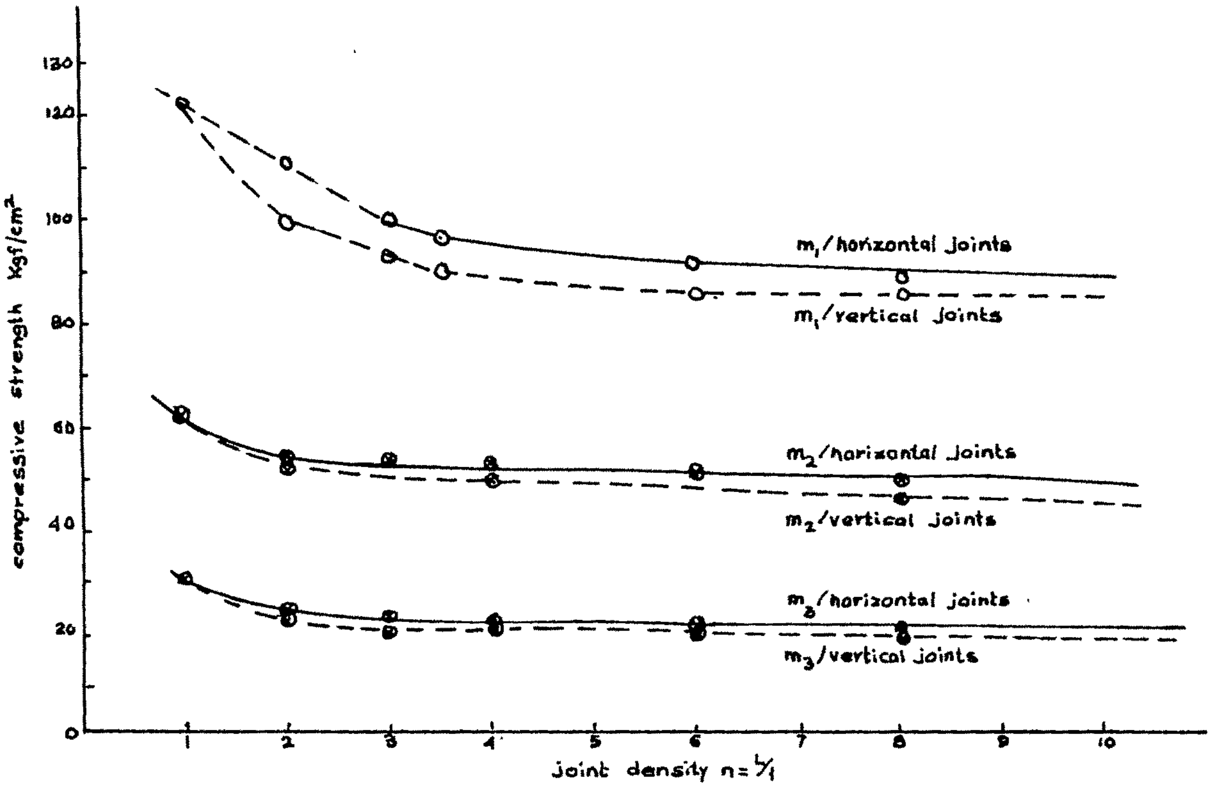


FIG.3·22 INFLUENCE OF HORIZONTAL AND VERTICAL JOINTS ON COMPRESSIVE STRENGTH OF A MODEL
(after LAMA, 1974 a)

as below:

(i) Youash (1966) conducted direct tension test on cores of shale, gneiss and sand stone, whose layers dipping at 0°, 15°, 30°, 45°, 60°, 75° and 90° to the short cylinder axis. Rupture strength increases as the dip increase from 0° to 60° for which the failure occurs along layering. While for 75° and 90° cores, failure occurs along layering and the sharp increase in rupture strength is observed. (Fig. 3.23)

(ii) Dayre (1970) studied the influence of the lineation and the cleavage on the maximum bending moments at failure of slaty shales. The results are shown in fig. 3.24.

(iii) Willard and Mac Williams (1969) developed microstructural techniques to investigate relationship between the mechanical properties. Their results reveal that the frequency of defects tends to be inversely proportional to strength, suggesting that the direction of the weakest tensile strength is approximately normal to the direction of the most defects. (Table 3.1)

(iV) Berenbaum and Brodie (1959) tested core specimen for tensile strength by Brazilian test at various orientations of the planes of weakness. Their results indicate that the strength is greatest at zero degree to the bedding planes or lowest at 90°. (Fig. 3.25)

(v) Dube and Singh (1969) observed the failure of specimens compressed at different angles of bedding planes in brizilian test and found that the strength of the specimen is greatest when the applied load acts 80° to the bedding plane and it decreases as the angle of inclination increases. It is

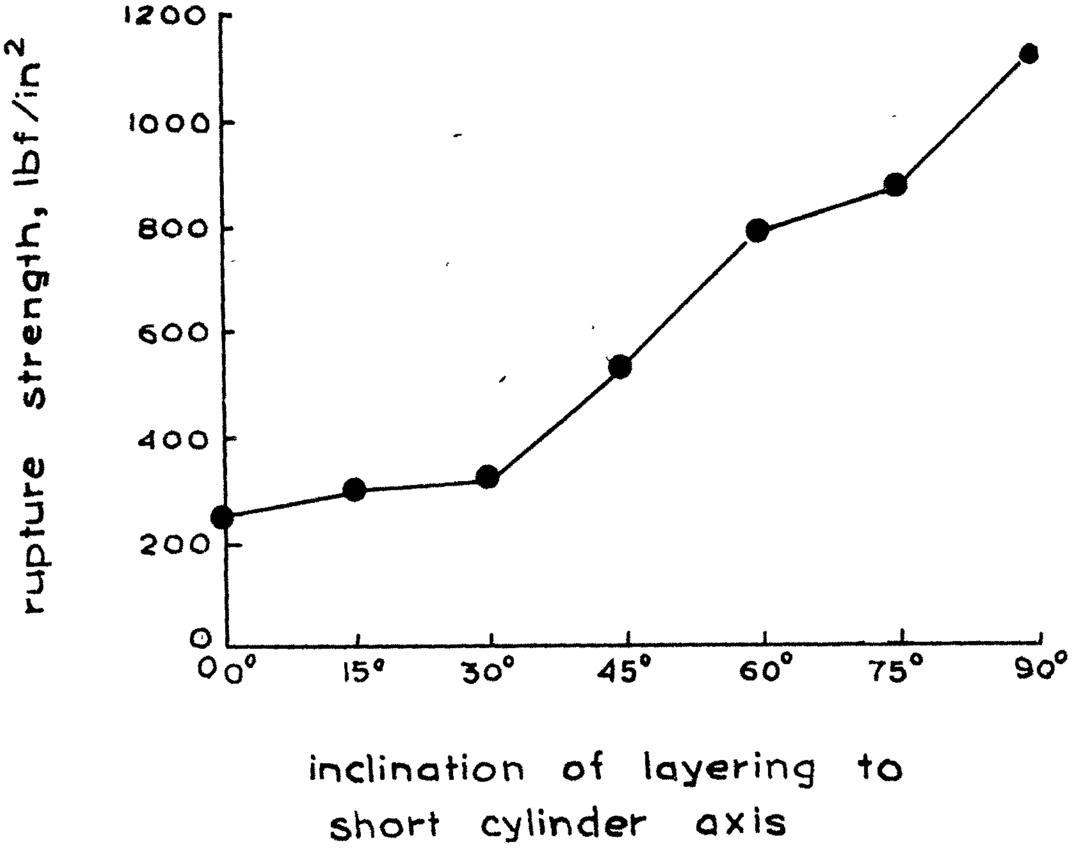


FIG. 3·23 RUPTURE STRENGTH IN TENSION VERSUS INCLINATION OF LAYING FOR SANDSTONE OF LYONS FORMATION.
(after YOUASH, 1966).

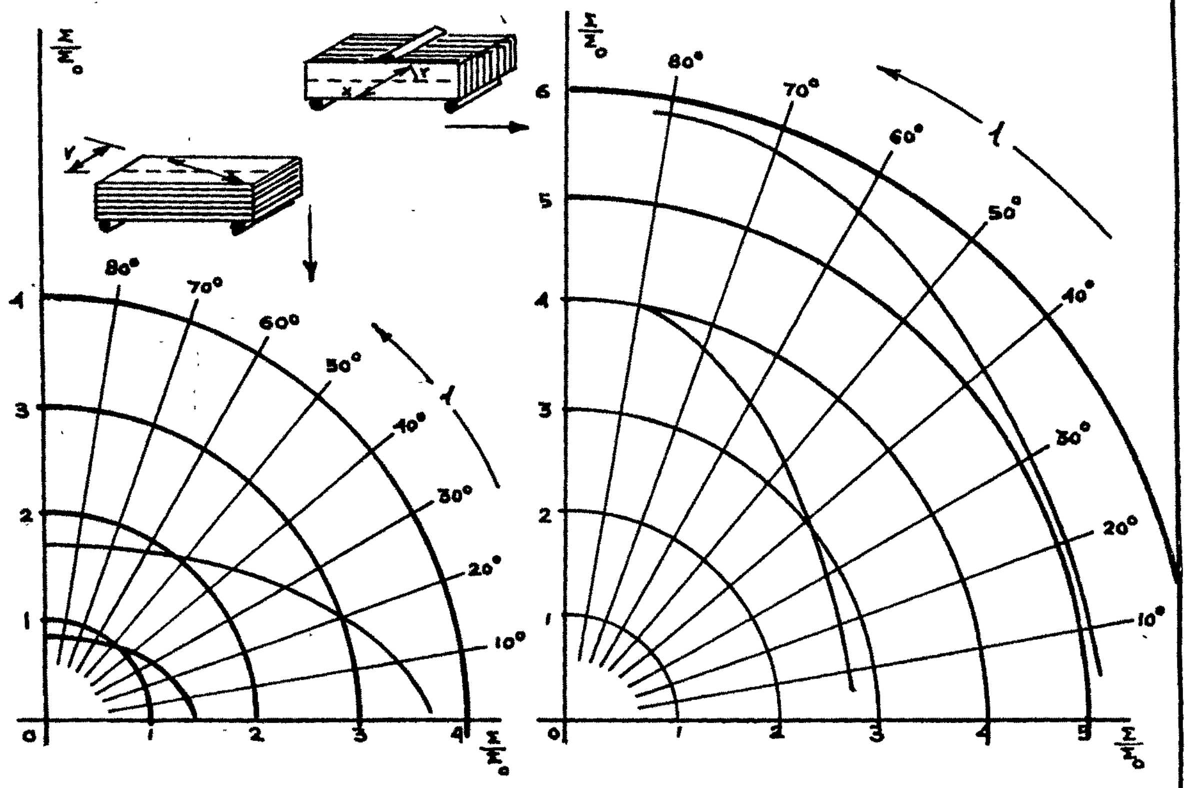


FIG. 3-24 BENDING ANISOTROPY OF THE MAXIMUM BENDING MOMENT DURING FAILURE. Σ = M/I WERE M IS MAXIMUM BENDING MOMENT AT FAILURE, I IS THE MOMENT OF INERTIA.

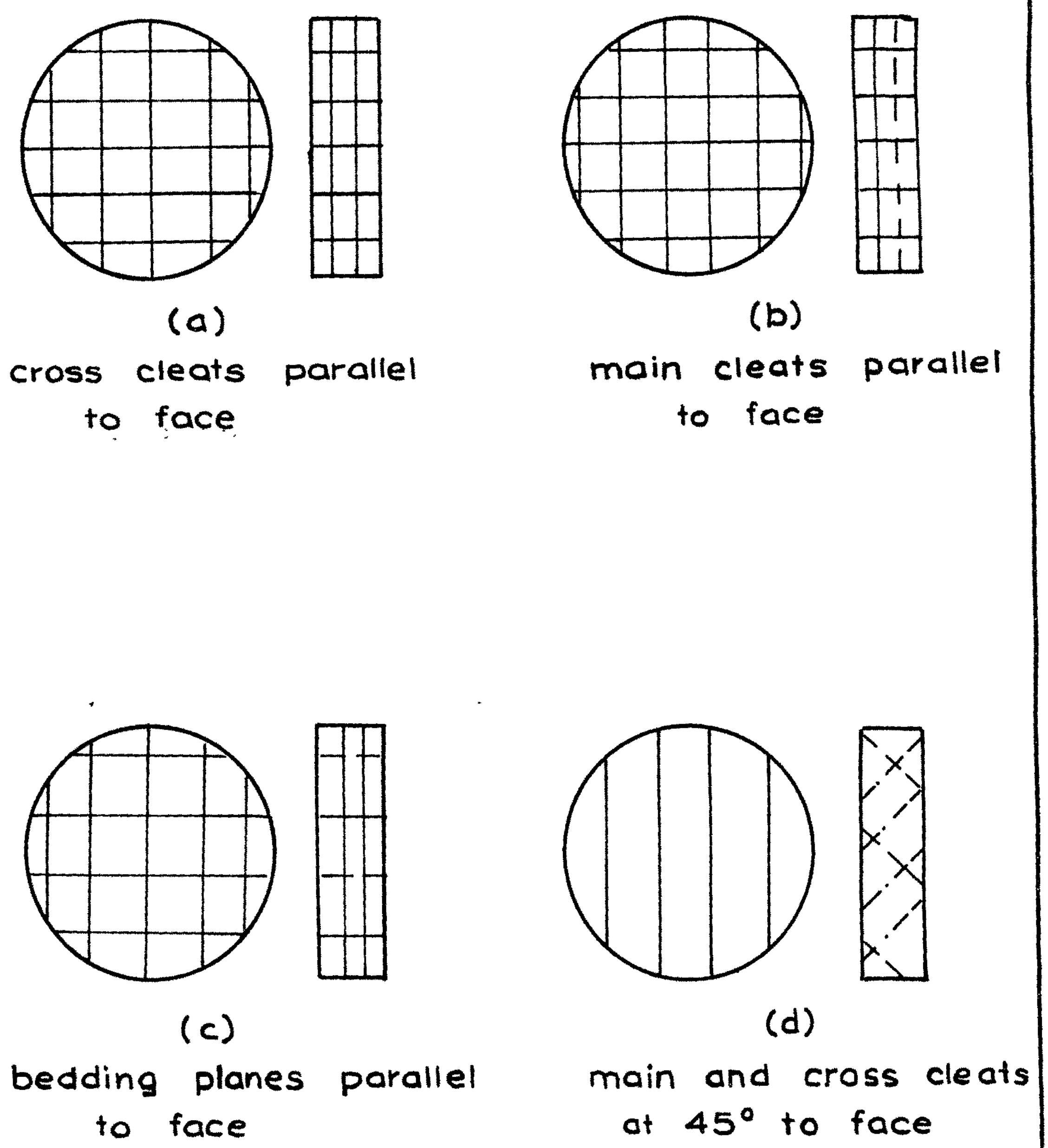


FIG. 3·25 DISC SPECIMENS FOR TEST ON COAL.
(after BERENBAUM and BRODIE, 1959)

lowest when the direction of the bedding plane is parallel to the direction of the applied load. (Fig. 3.26)

TABLE 3.1

Comparison of total defects in six angular intervals with tensile strength in Barre granite (after Willard and McWilliams, 1969)

Angular interval	Defects	Tensile strength MPa	(lbf/in^2)	Line-load angle
0°±15°	213	15.5	(2244)	0°
30°±15°	160	14.8	(2145)	30°
60°±15°	105	9.5	(1385)	60°
90°±15°	76	10.7	(1553)	90°
120°±15°	77	11.7	(1695)	120°
150°±15°	172	13.7	(1981)	150°

(vi) Hobbs (1964) evaluated the effect of orientation of the bedding plane and the tensile strength of rock with the disc of 2.54 cm diameter and thickness of 0.64 cm with a central hole of 0.32 cm diameter. He found that tensile strength is greatest when tensile strength generated at 0° to the lamination. He observed that the majority of discs failed along the loaded diameter irrespective of orientation of the laminations. Subsidiary cracks were seen to occur following the plane of laminations. Parikh and Biyani (1985) also observed this type of pattern of failure on disc and annulies compressed under Brazilian test.

(vii) Berenbaum and Brodie (1959) studied the variation in tensile strength of Barnsley Hard Coal at various angles

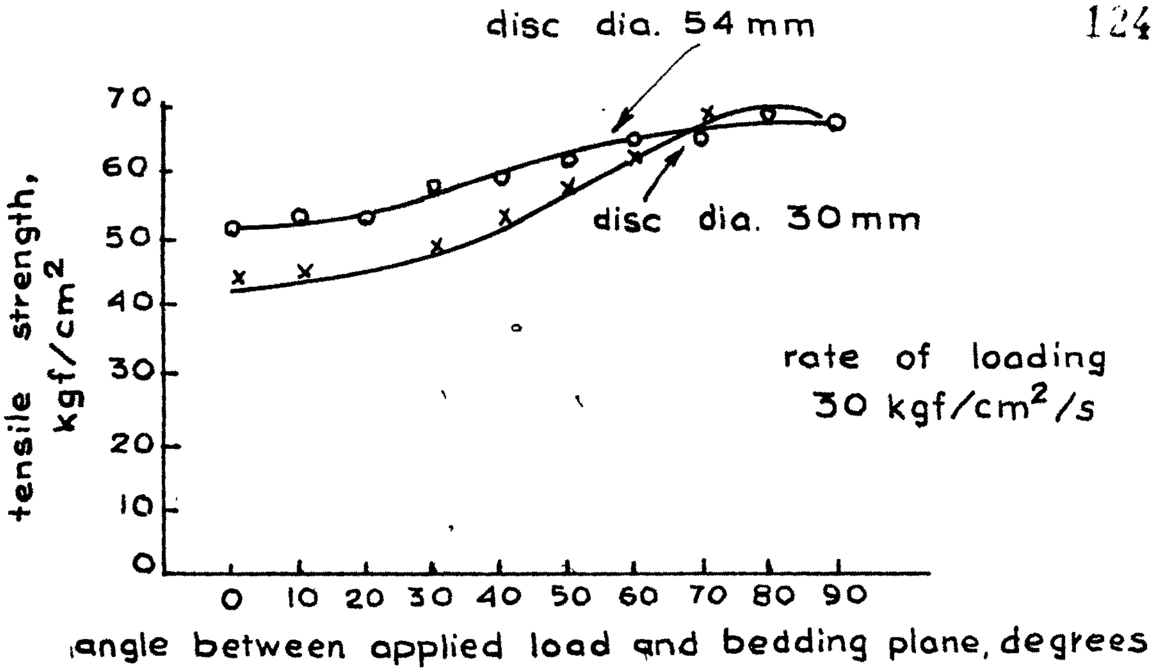


FIG. 3·26 EFFECT OF DIRECTION OF LOADING (with respect to bedding plane) ON TENSILE STRENGTH OF SANDSTONE.
(after DUBE and SINGH, 1969)

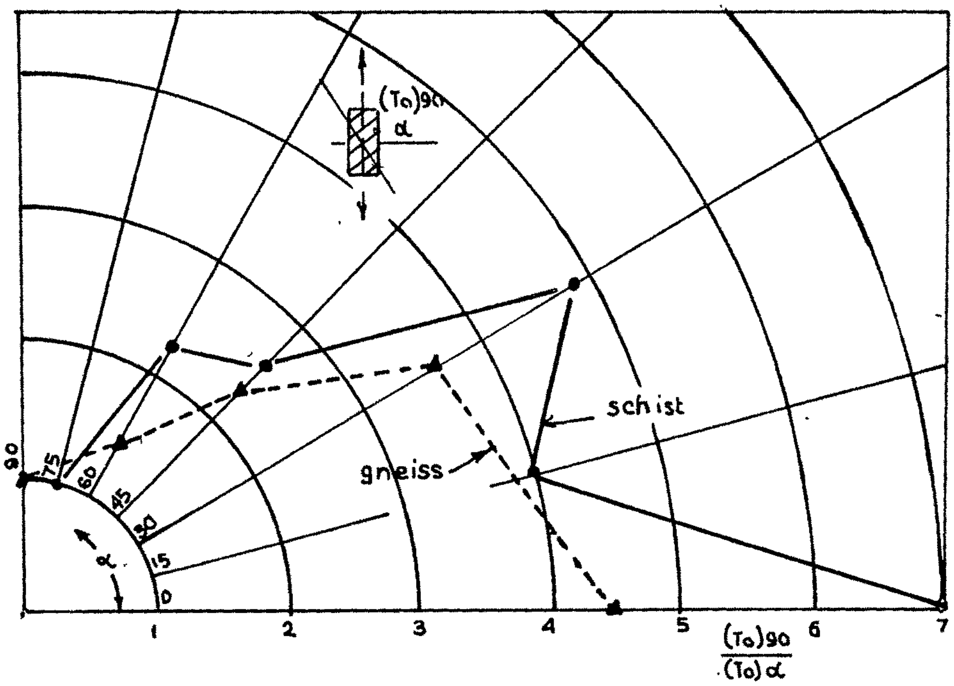


FIG. 3·27 RATIO OF THE TENSILE STRENGTHS $\frac{(T_o)_{90}}{(T_o)_\alpha}$ VERSUS α

to the bedding planes by two methods namely indentation method and Brazilian method (Table 3.2) showing good agreements to each other. Also they compared the direct pull out test on cylindrical specimens of Serpentine, Schist and Gneiss. The maximum value of tensile strength and elastic modulus is when tensile load is applied parallel to the plane of laminations. The minimum value for these rocks are obtained when the load is applied perpendicularly to the bedding plane. The change in elastic modulus with the orientation does not seem to agree with the tensorial transformations for elastic constants which may be due to the inherent anisotropic characteristics. (Fig. 3.27)

TABLE 3.2

Variation of the tensile strength of Barnsley Hard coal with orientation to the bedding planes

(All specimens cut from the same block)

(after Berenbaum and Brodie, 1959)

Method	Tensile strength, MPa(lbf/in^2) Angle between tensile stresses and bedding planes				
	0°	25°	45°	65°	90°
Indentation (0.95 cm(3/8 in) indenter) 15 specimens at each angle	2.8±0.19 (408±27)	2.3±0.28 (329±41)	1.7±0.10 (242±14)	1.6±0.10 (226±14)	1.3±0.12 (191±18)
Brazilian 20 specimens angle	2.8±0.19 (408±27)	2.9±0.27 (415±39)	1.9±0.22 (279±32)	1.6±0.10 (226±14)	1.2±0.10 (168±14)

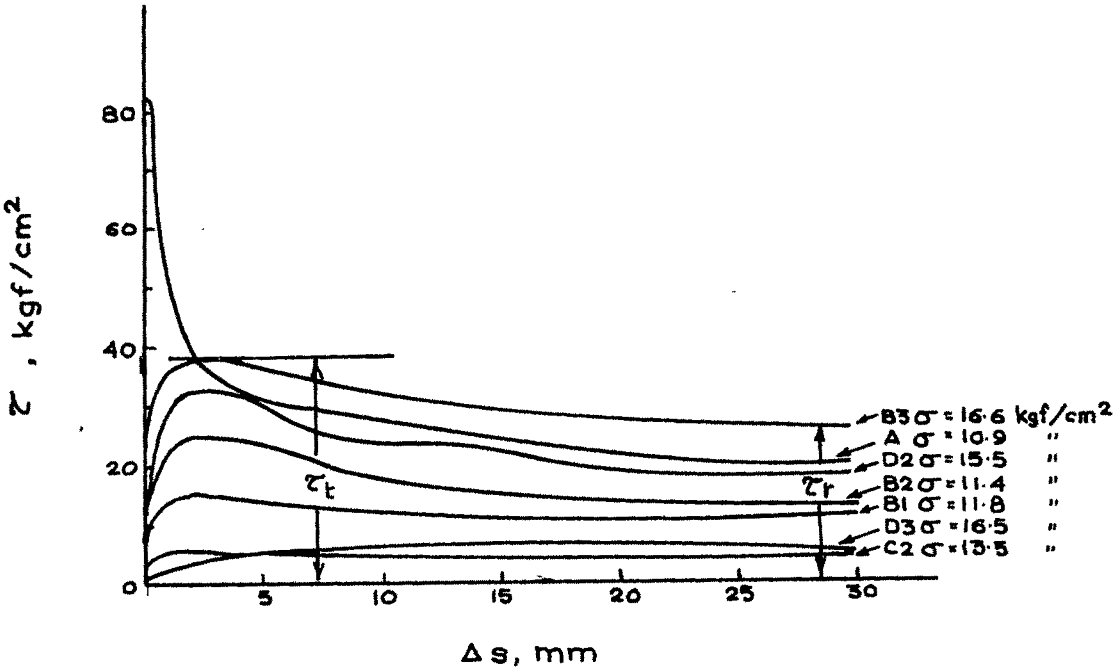
3.3.3. Strength of jointed rock in direct shear

A considerable work has been conducted on the strength of jointed rocks in direct shear under well developed theoretical grounds. The following are the major findings from these investigations.

(i) Krsmanovic and Langof (1964) conducted test on stratified jointed lime stone in direct shear apparatus and the results are categorized into three types. The first category in case of the solid rock possessing very high shear resistance and with slight deformation is necessary to develop maximum shear resistance. In the second catogory in case of bedding joints, to develop shear resistance considerable deformation is necessary. In third category, in case of plastic sedimantation layers, the shear resistance is consequent upon slight or great deformations. (Fig. 3.28)

(ii) Lajtai (1969 a,b) tested several plane, open and closed joints with rock bridges and identified three modes of failure, (a) failure in tension at low normal stresses (b) failure in shear at intermediate normal stresses and (c) Ultimate failure at high normal stresses. According to him the total shear strength of a plane of weakness consisting of discontinuity separated by solid bridges is a sum of cohesional strength, internal friction in solid bridges and the friction along the open joints.

(iii) Hayashi (1966) conducted tests on regularly jointed models of plaster and observed (a) the shear load at failure of specimen with transverse joints decreased with increase in number of joints. (b) the shear load at failure for specimens with positive joint specimens was lower than those



series A - intact limestone

series B - stratification surfaces (bedding joints)

1 - thin calcareous foliated layer;

2 - rough stratification surface;

3 - very rough stratification surface;

series C2 - thin, 2 - 5 cm (0·8 - 2 in) plastic sedimentation layers

series D2 - clean, very rough fissures

D3 - rough fissures with detrital material

FIG.3-28 DIAGRAMS τ-Δs FOR TYPICAL SPECIMENS OF VARIOUS SERIES
(after KRSMANOVIC and LANGOF, 1964)

with negative joint specimen.(Fig. 3.29).(c) the shear load at failure increased with the constraining force, if no dilatation was considered. The scale effect was observed in case of cohesion value but not in friction value.

(iv) Kawamoto (1970) and Hayashi (1966) both working on jointed and layered plaster-sand models found that depending upon the direction of inclination of joint under restraint, there is possibility of joint opening or joint closing due to tensile or compressive stresses developed in the jointed system giving rise to internal buckling resulting in tensile stresses. The typical failure patterns are sliding or shearing rupture, rupture through solid (cleavage and shearing rupture) and combined rupture.(Fig. 3.30)

(v) Uff and Nash (1967) measured the shear strength of shale along the bedding planes in various directions from dip to strike using a shear box. They found strength parameters C and Ø to vary with the orientation relative to the dip. The maximum peak value of friction and cohesion in the bedding plane occur in the direction of strike and dip respectively with minimum value at right angles.(Fig. 3.31 and 3.32)

(vi) Walker (1971) conducted direct shear test on cubic block models oriented at different angles to the direction of normal and shear stresses and observed sliding along joints, rotation of blocks and failure of the block depending upon the direction of orientation of the block irrespective of the applied stress field. (Fig. 3.33)

(vii) Ladanyi and Archambault (1972) observed rotationary failure mode similar to Walker (1971) and noted high dilatation,

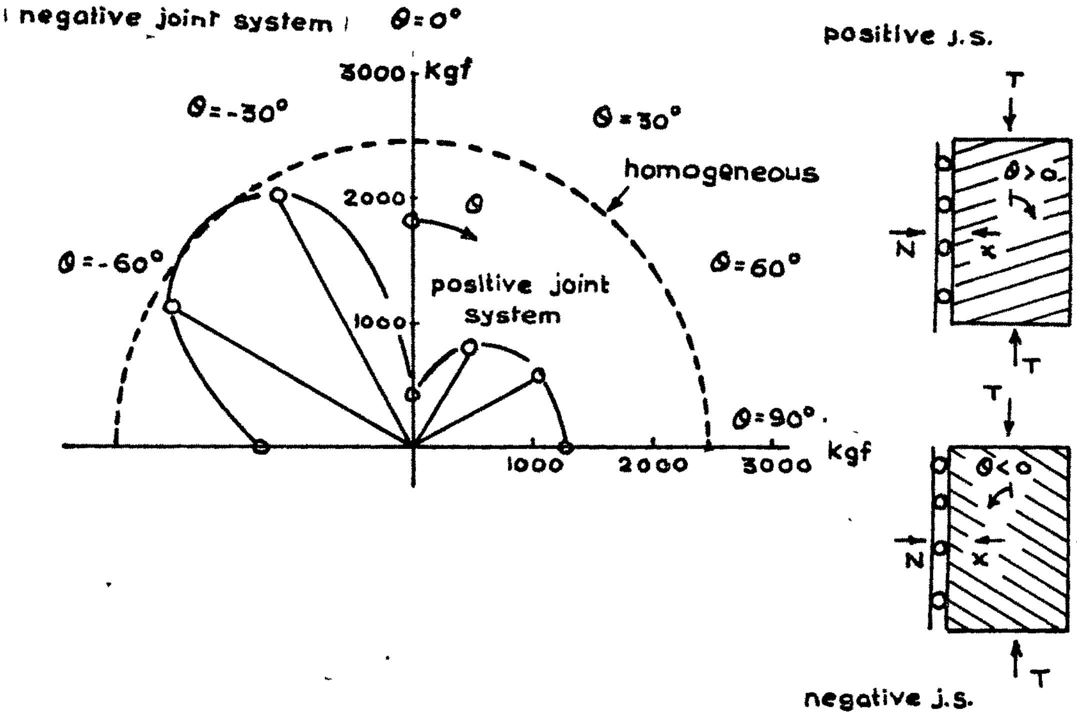


FIG. 3·29 ANISOTROPY OF SHEAR LOAD AT FAILUARE T IN CONFINING THE LATERAL DILATANCY IN DIRECTION x
(after HAYASHI, 1966)

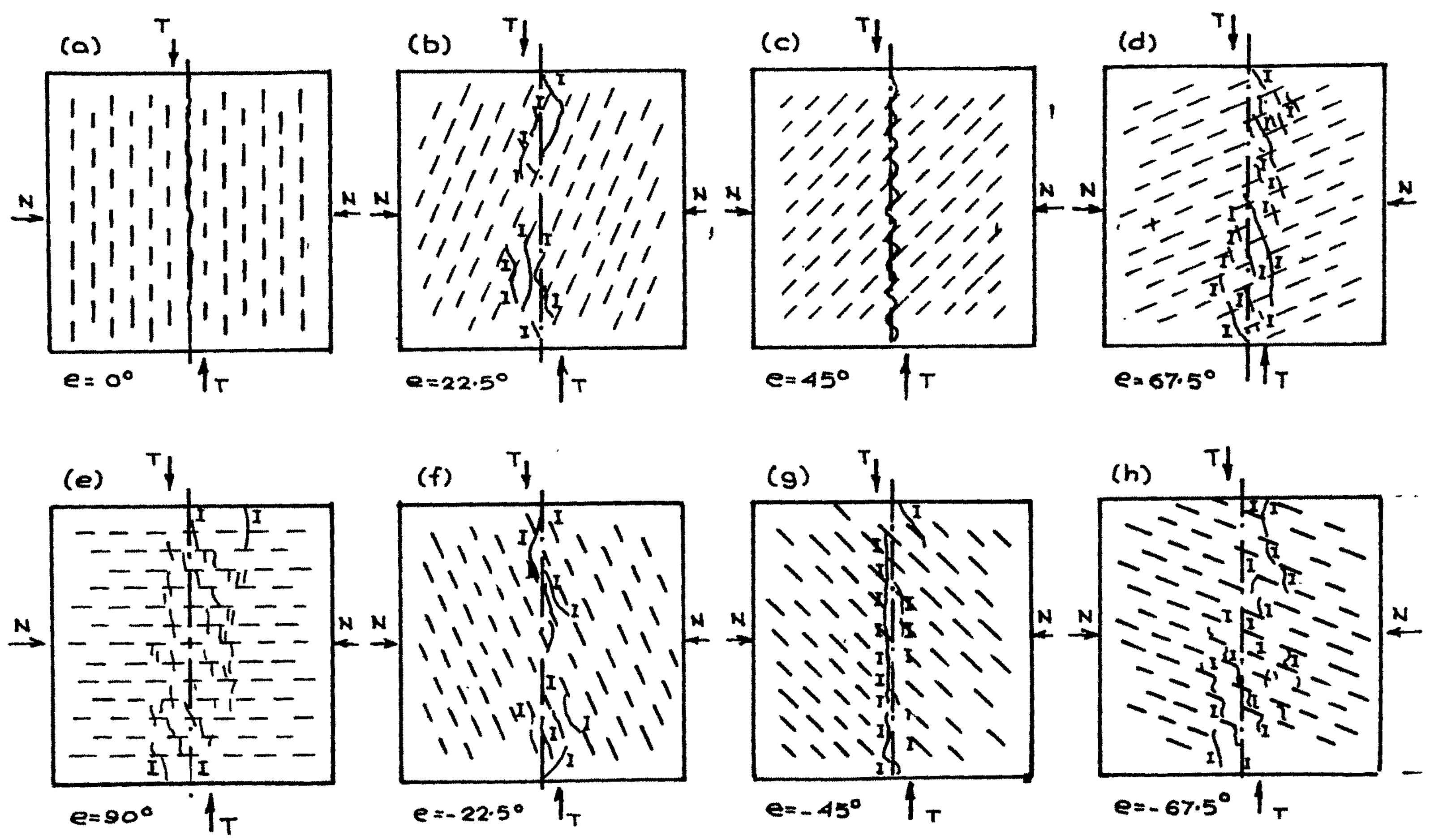


FIG. 3·30 FAILURE PATTERNS OF JOINTED MEDIA BY DIRECT SHEAR TEST USING SHEARING BOX.
(after KAWAMOTO, 1970)

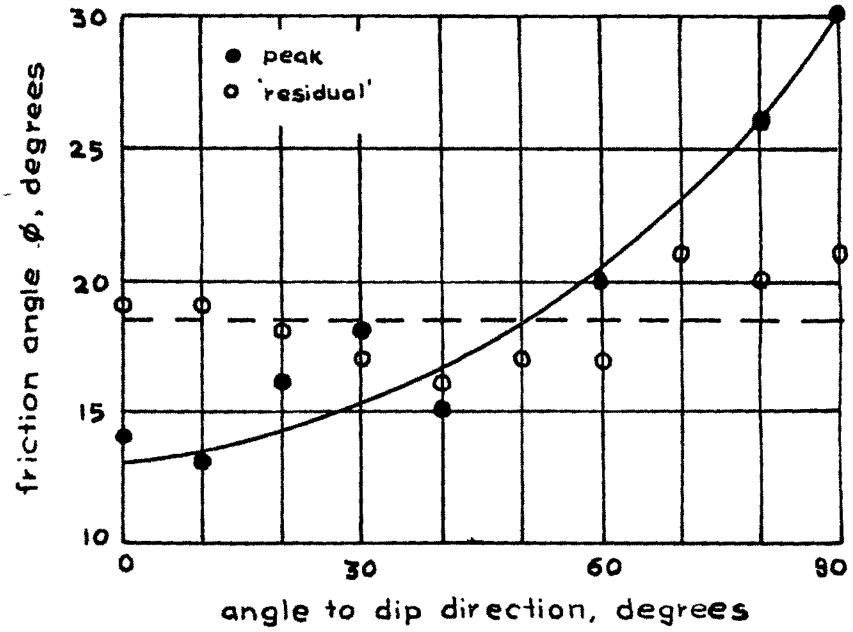


FIG. 3·31 RELETIONSHIP BETWEEN ϕ & ANGLE TO DIP
(after UFF and NASH, 1967).

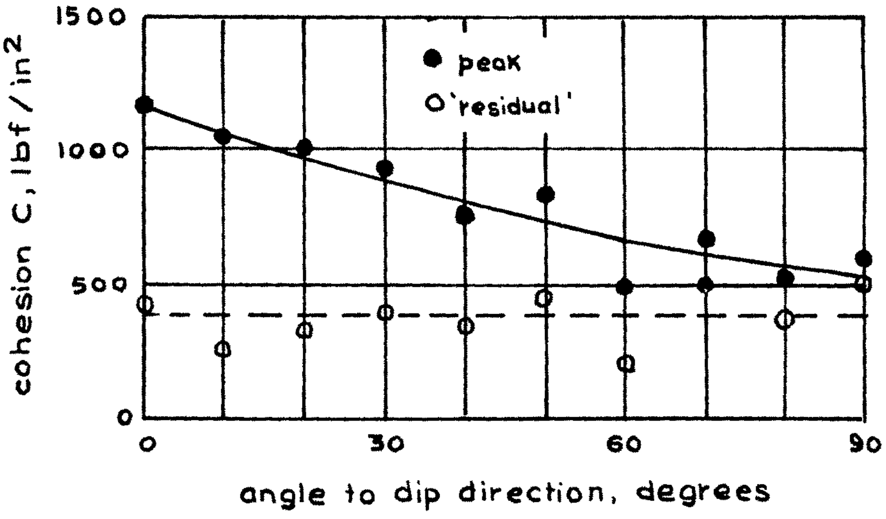


FIG. 3·32 RELETIONSHIP BETWEEN c AND ANGLE TO DIP
(after UFF and NASH, 1967).

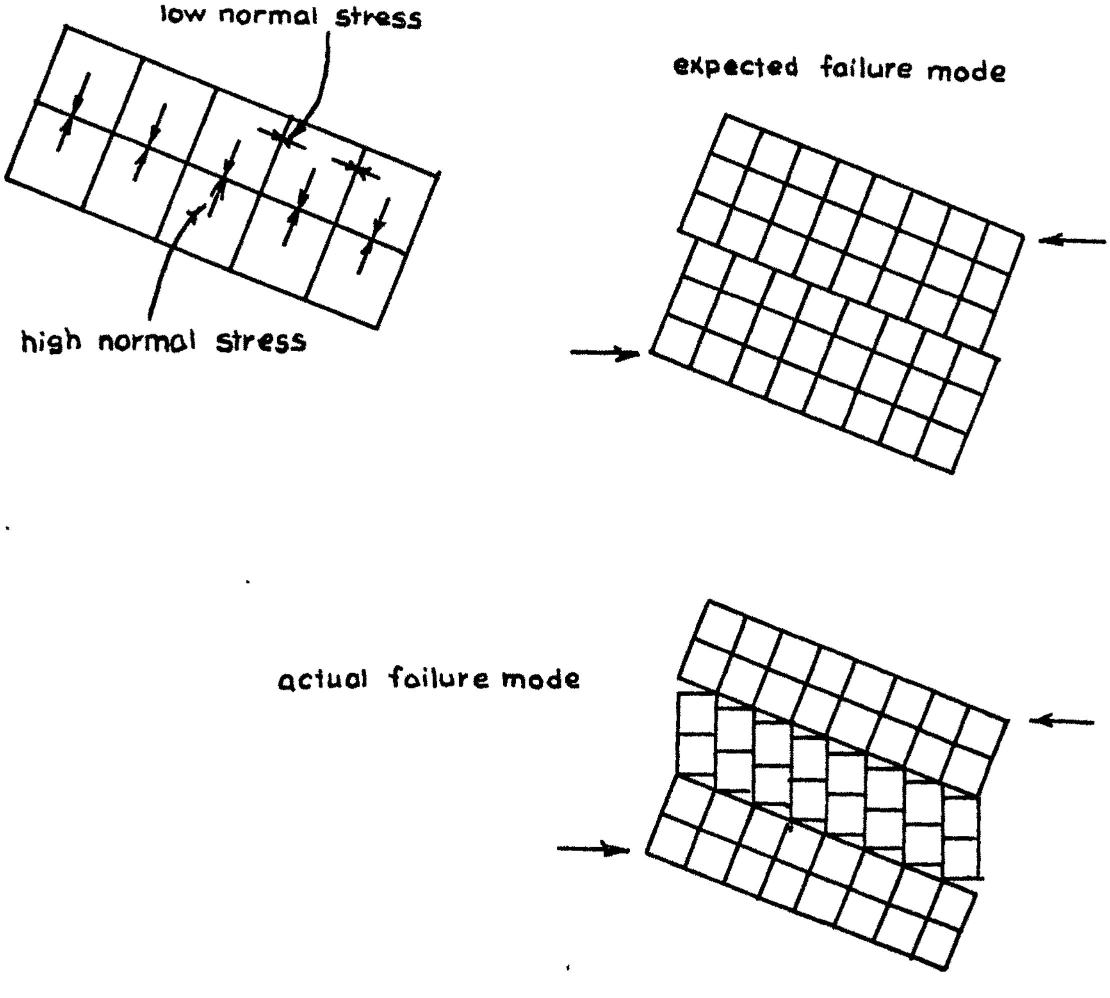


FIG. 3·33 ROTATIONARY FAILURE MODE
(after WALKER, 1971).

crushing and wider shear zones termed as 'KINK BANDS'. He found that when such a failure occurs the strength of the jointed mass is much lower than that predicted by the Mohr-Coulomb concept.

3.3.4. Strength of jointed rock in multiaxial compression

In last few years, series of investigations have been conducted under biaxial and triaxial loading conditions to determine the behaviour of jointed rocks. This has provided insight into various aspects of strength of jointed rocks.

3.3.4.1. Biaxial conditions

(i) Muller and Pacher (1965) undertook test on models of size 70 cm x 70 cm x 30 cm with one set of joints loaded biaxially by a pair of hydraulic jacks. The inclination of plane of the joint set in the direction of the greatest principal stress is varied from 0° to 60° in increments of 15°. He found that the strength falls very rapidly as inclination increase from 0° and this is minimum at 30°. For inclinations greater than 60° the presence of joints have almost no improvement over the strength of unjointed rock. Their results also show that for a single set of joint the mode of failure under biaxial conditions is dependent upon the ratio between the principal stresses and the orientation of joints with respect to maximum stress direction.(Fig.3.34)

(ii) John (1969) conducted biaxial tests on jointed models in both continuous and intermittent joint sets and recognized three modes of failure (a) sliding along joints solely dependent on the orientation and frictional properties of the joint (b) shear through the rock material

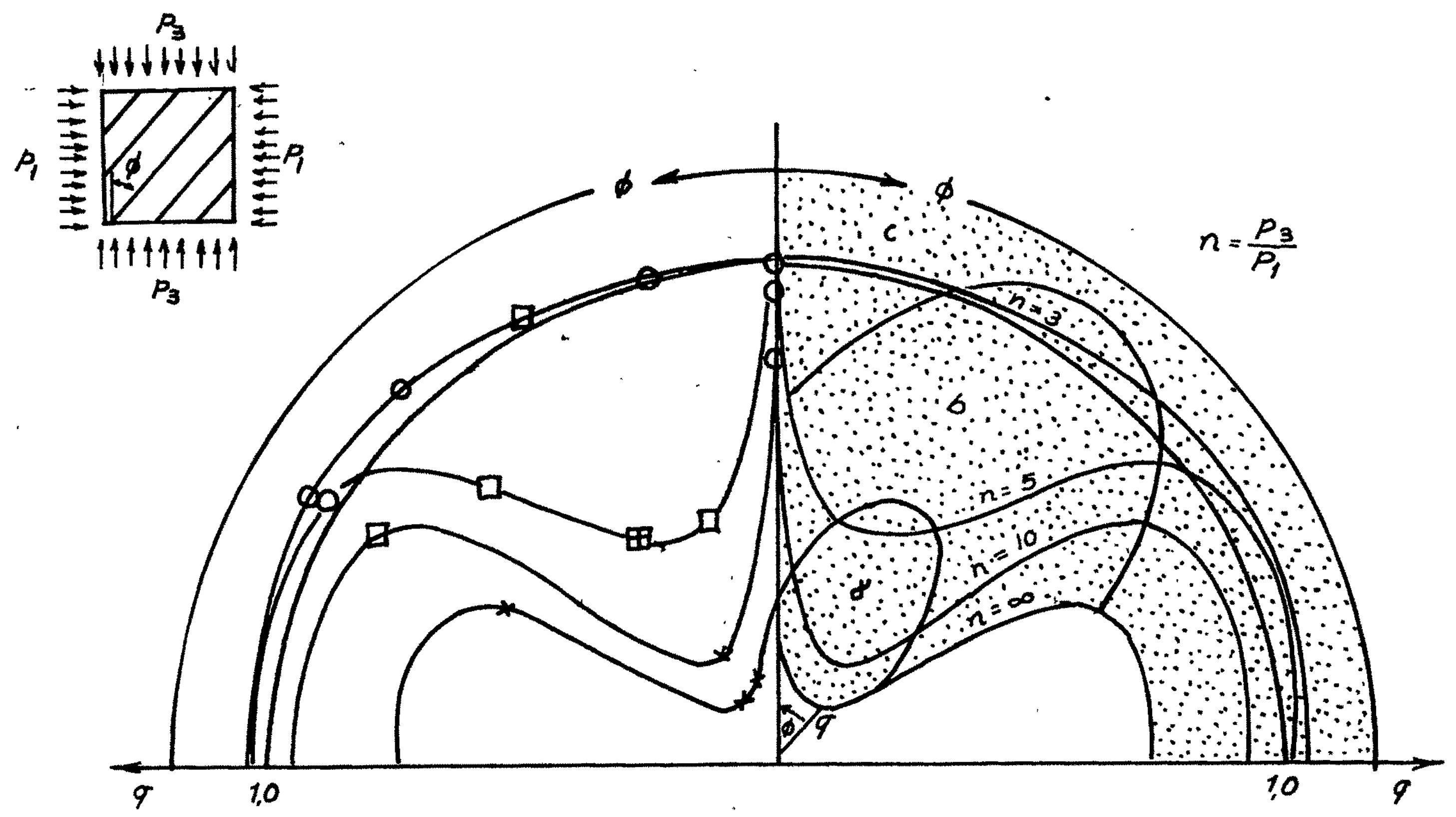


FIG. 3-34 PARAMETERS WHICH DETERMINE THE GROSS STRENGTH OF THE ANISOTROPIC MASS TYPES OF THE FRACTURING AT Xe = 1 (Left): + RUPTURE INVOLVING JOINTS:
□ Combined rupture. ○ Rupture through the solid.
DECISIVE STRENGTH PARAMETERS (Right): (a) FRICTION ON JOINTS. (b) BONDING STRENGTH, (c) STRENGTH OF THE ROCK SUBSTANCE.
(after MULLER and PACHER, 1965)

depending upon the shear strength of the element and the confining pressure (c) shear partially through the rock material and partially through the joint, signifying the combination of above two modes.

(iii) Ladanyi and Archambault (1972) also observed a similar fracture patterns termed as slippage along a shear plane, shear zone including fracturing and shearing along a number of planes and kink bands depending upon the orientation of joint plane with the principal stress direction and horizontal stress,

(iv) Muller, Fecker and Lama using servo controlled biaxial machine found that at higher values of stress ratio between the principal stresses a large scale of dilatation occurs while at low value failure occurs along a limited or a single shear plane.

3.3.4.2. Triaxial conditions

(i) Price (1958) investigated Pennant sand stone by compressing it in two directions and observed rise in strength of specimens with confining pressure and the specimens compressed parallel to bedding plane have more strength than those compressed perpendicular to the bedding plane.

(ii) Donath (1963) reported pronounced effect of confinement both at low and high confinement pressure on the ultimate strength of anisotropic rock like Martinsburg Slate (Fig.3.35). At the high confining pressures strength parallel to foliation is nearly equal to that of 90° orientation.

(iii) Lane and Heck (1964) working on granite found that the inclination of Mohr's envelope is different for different joints and the highest envelope represent the tests

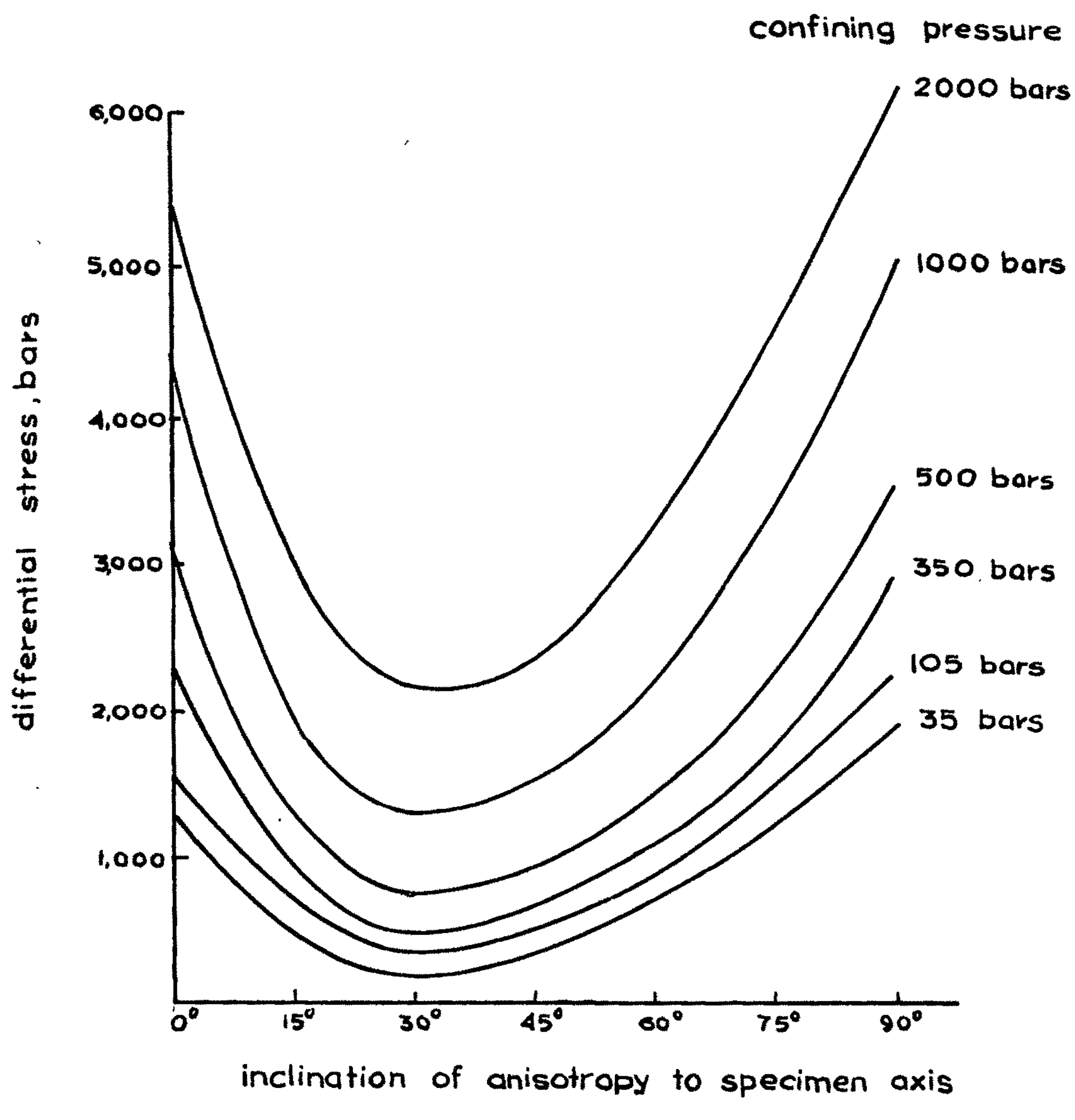


FIG.3·35 ULTIMATE STRENGTH VERSUS INCLINATION OF CLEAVAGE, MARTINSBURG SLATE.
(after DONATH, 1963)

on intact cores while the lowest from tests on open joints (Fig. 3.36 and 3.37)

(iv) Evans and Pomeroy (1966) observed that the strength of coal is dependent upon the direction of loading at small confining pressure but at higher confining pressures (above 13.8 M Pa) the strength is independent of the orientation.

(v) Youash (1966) from the test on core with the joint layers dipping from 0° to 90° to the cylinder axis of shale, gneiss and two sand stones under very low to very high confining pressures found that the maximum stress difference is for 0° to 90° cores and minimum for 45° and 60° core orientation. (Fig. 3.38 and 3.39)

(vi) Einstein et al (1969) investigated the influence of multiple joints on gypsum plastered specimen prepared with different sets of parallel, perpendicular and orthogonal joints and found that for a given normal stress, shear stress resistance with orthogonal joints was the lowest, followed by the horizontal joints and vertical joints. In all the cases shear enevlopes are lower than the unjointed rock though the failure occurs similar to intact material and not by sliding along the joint surfaces. His results reveal that the confining stress at which transition from brittle behaviour to ductile behaviour is the largest for the intact material, smaller for the vertically jointed material and still smaller for the horizontally jointed material and the smallest for the orthogonally jointed material. This transition stress systematically increases with increasing joint spacing for the vertically and horizontally jointed

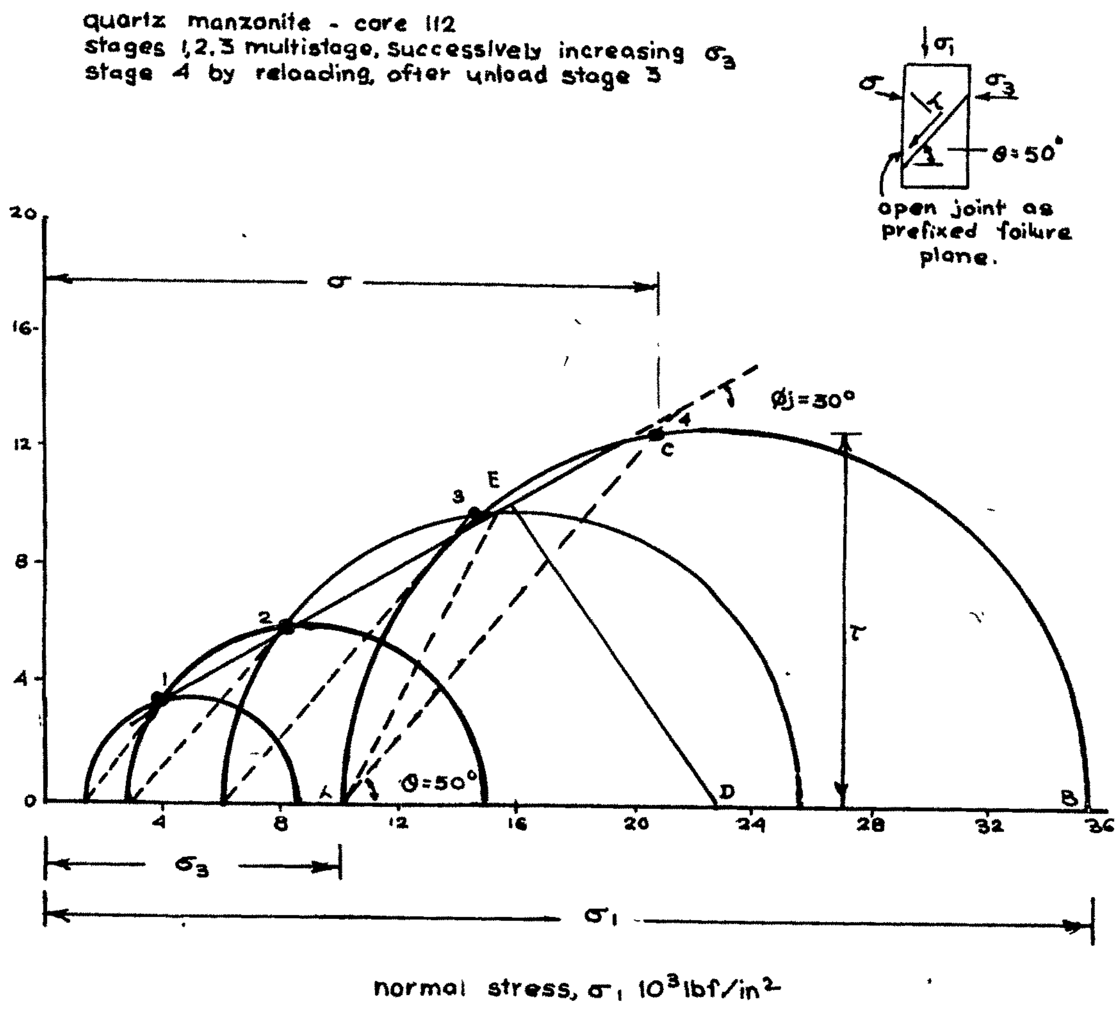


FIG. 3·36 TYPICAL MOHR'S CIRCLES
(after LANE and HECK, 1964)

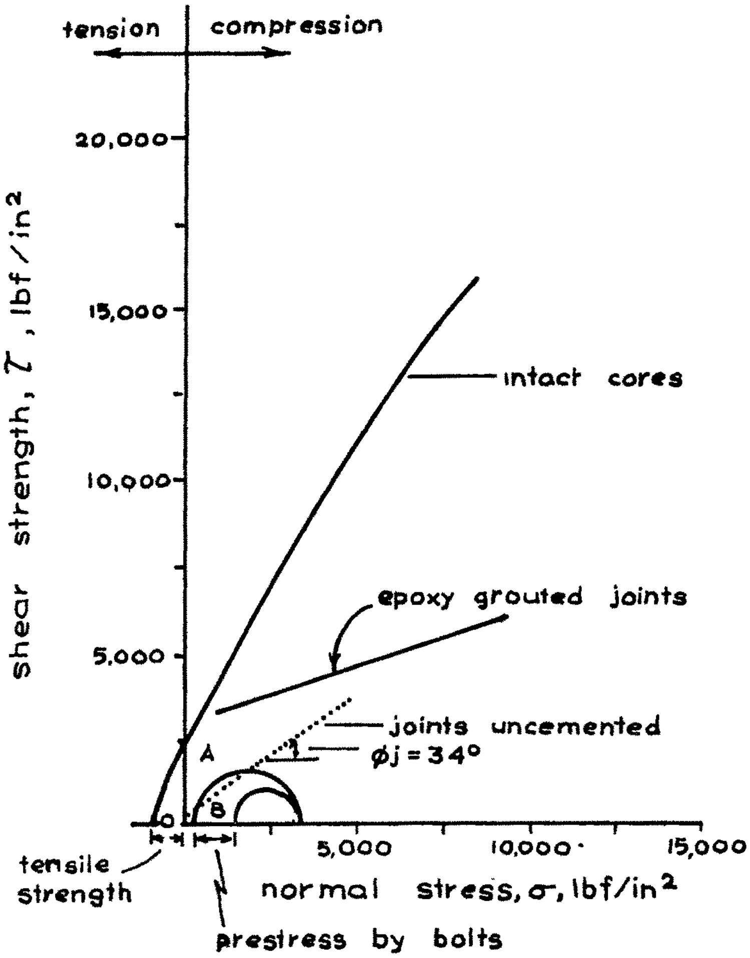


FIG. 3-37 NORAD GRANITE-COARSE GRAINED
(after LANE and HECK, 1964)

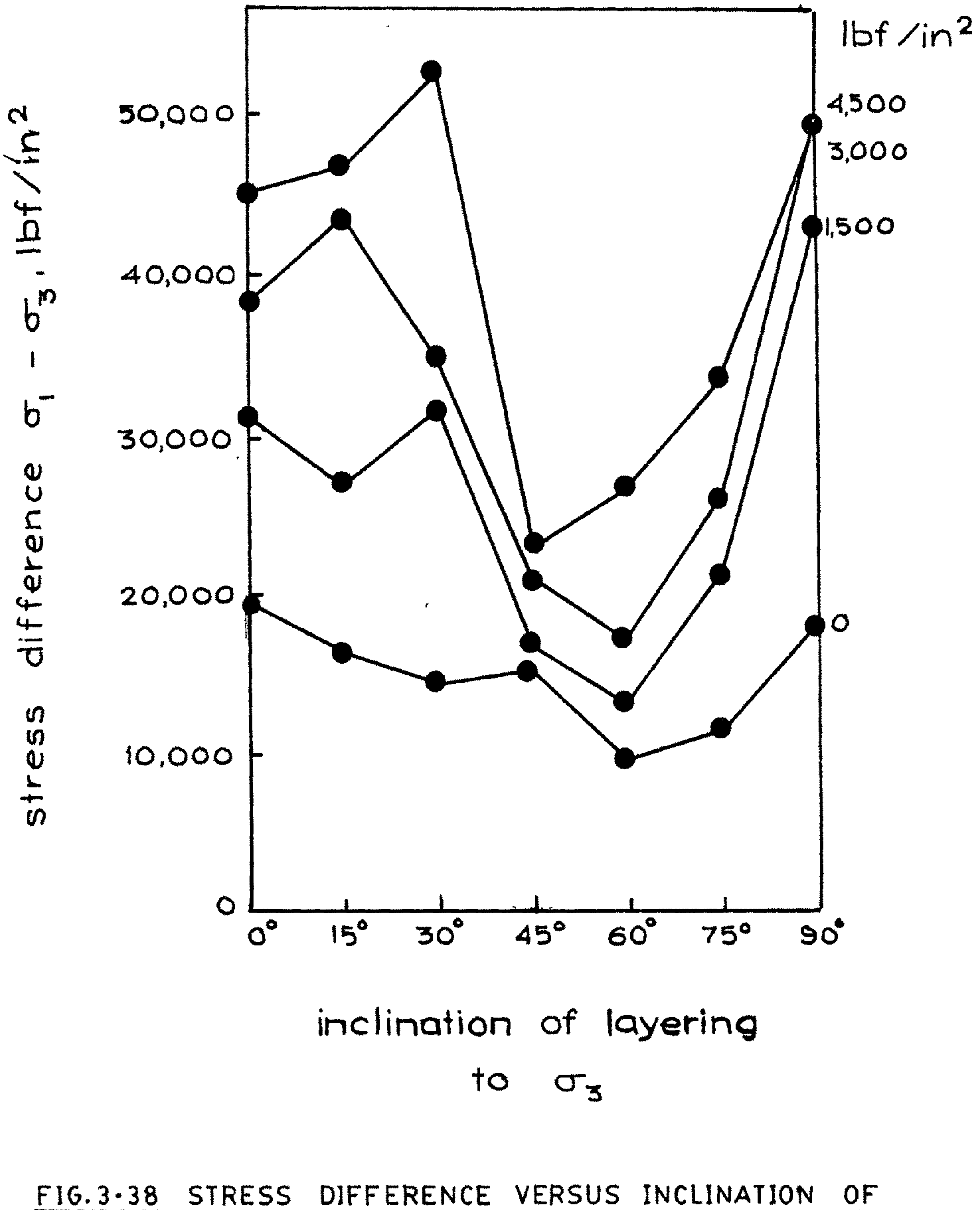


FIG. 3-38 STRESS DIFFERENCE VERSUS INCLINATION OF LAYING TO MINIMUM PRINCIPAL STRESS. σ_3 FOR SANDSTONE OF LYONS FORMATION.
(after YOUASH, 1966)

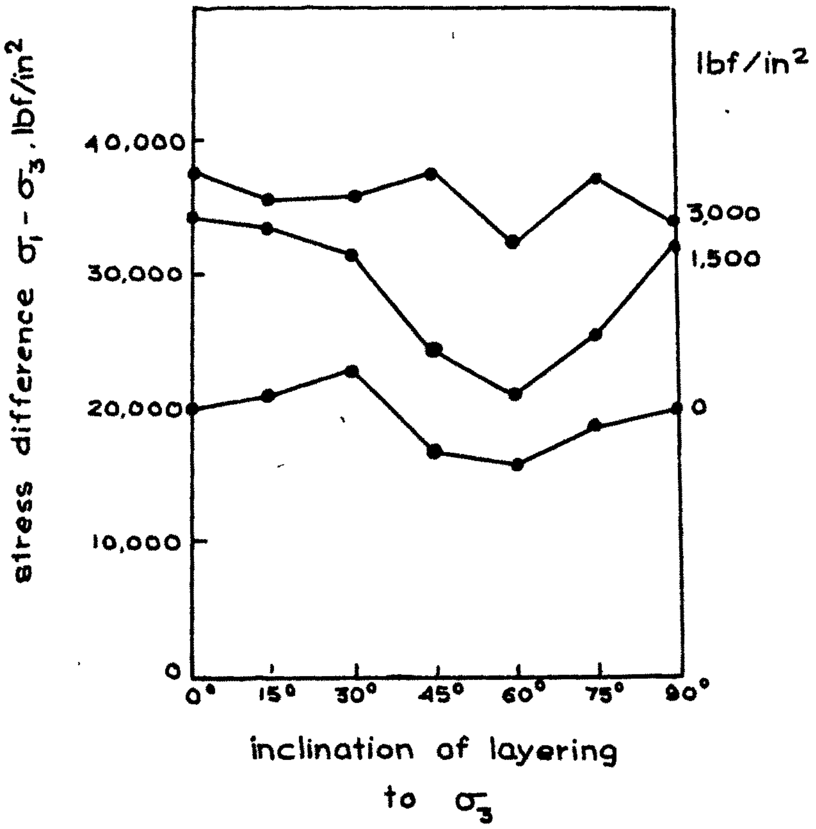


FIG. 3-39 STRESS DIFFERENCE VERSUS INCLINATION OF LAYERING TO MINIMUM PRINCIPAL STRESS FOR SHALE OF GREEN RIVER FORMATION

(after YOUASH, 1966)

specimens. (Fig. 3.40, 3.41 and 3.42 a,b)

(vii) Brown and Trollope (1970) conducted test on cubes of plaster with varying degrees of joint orientation and under different confining pressures and showed that at lower confining pressures the strength of the jointed specimen was lower than those corresponding to unjointed specimens. The lowest values were joint inclination 30°/120° with the major principal stress. (Table 3.3)

(viii) Brown (1970) investigated the influence of non continuied joints using parallelopiped and hexagonal blocks and inferred that the Mohr-Coulomb concept need certain modifications and can be used to determine the strength of the specimens with discontinuities. He also pointed out other types of failure besides those recognized by John (1969). He observed that at low confining pressure axial cleavage spliting the elements, collapse due to block movement and opening of the joints. This type of failure is termed as dilatation failure.

(iv) Pomeroy, Hobbs and Mahmoud (1971) studied the effect of weakness plane orientation on the fracture of Barnsley Hards coal and found that for all orientations there is an increase in fracture stress with increase in confining pressure. The strongest orientations are the bedding planes perpendicular to applied load (90,0, 0) and (75, 15, 0) while the weakest orientations were (30, 60, 0) (Fig. 3.43)

(x) Motoyama and Hirschfeld (1971) conducted tests on models using orthogonal and 45° oriented block with different

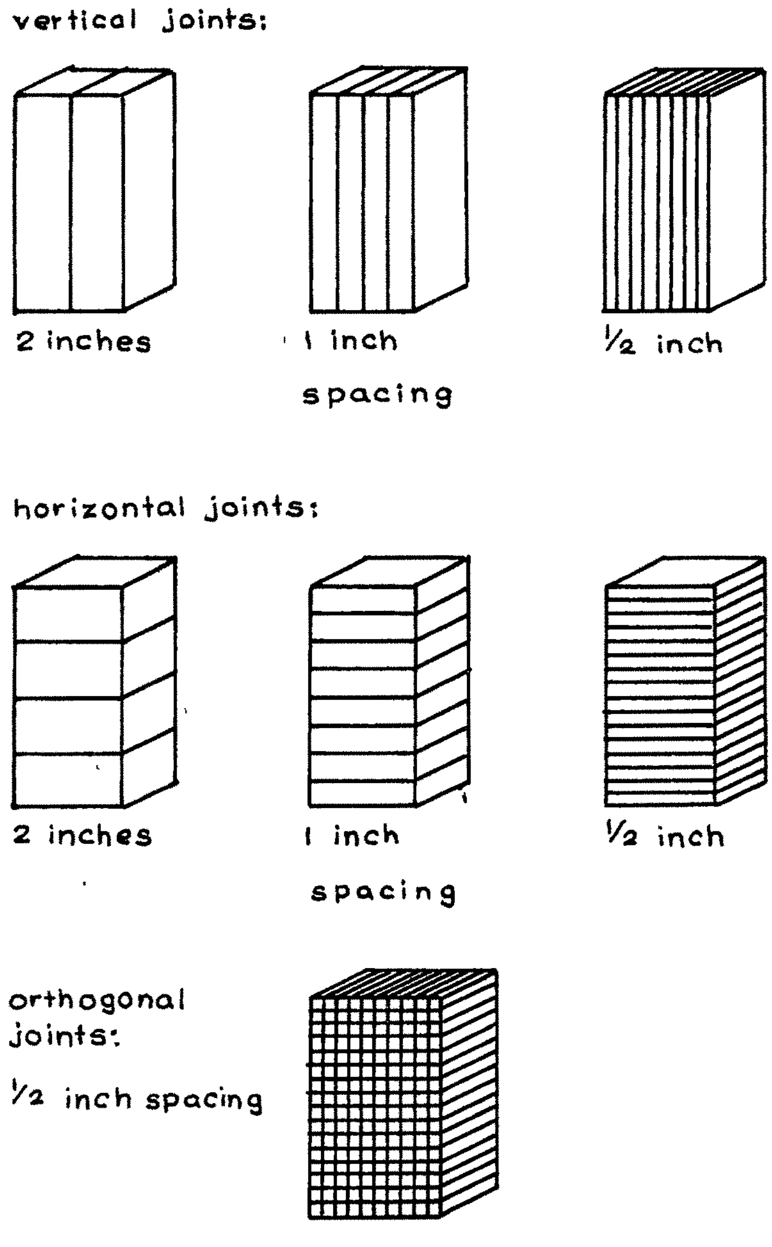


FIG. 3·40 JOINT CONFIGURATIONS.
(after EINSTEIN et al 1969.)

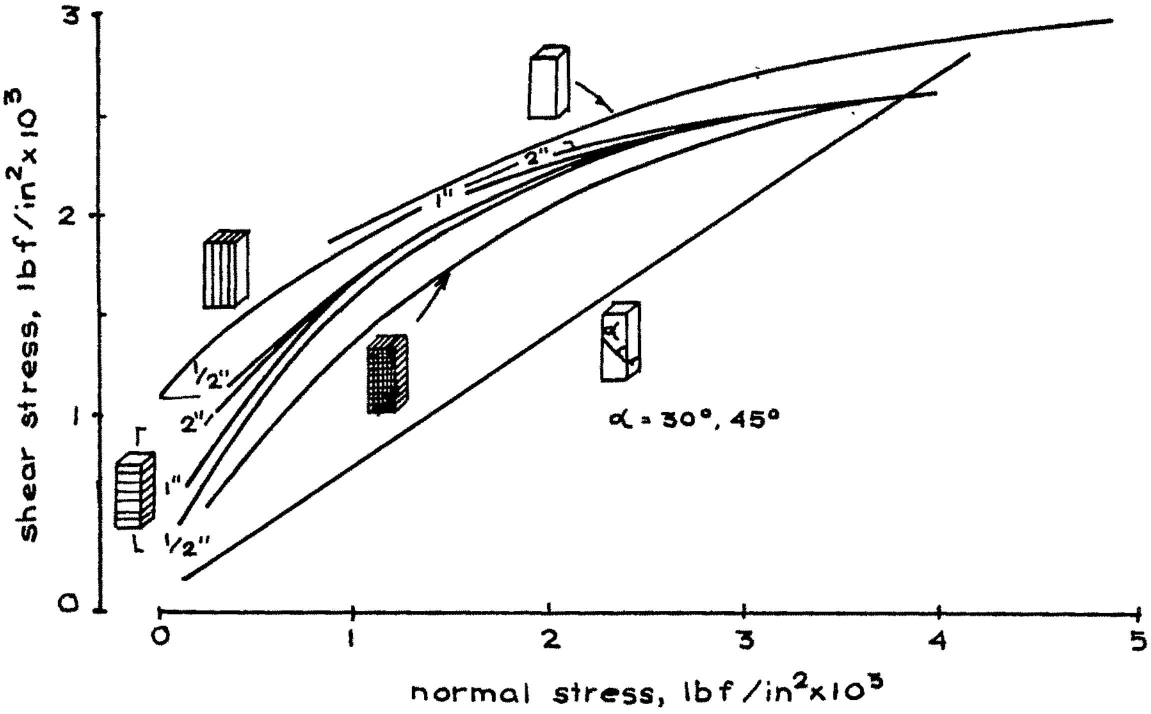


FIG. 3-41 MOHR ENVELOPES FOR INTACT & JOINTED GYPSUM MODELS.
(after EINSTEIN et al, 1969)

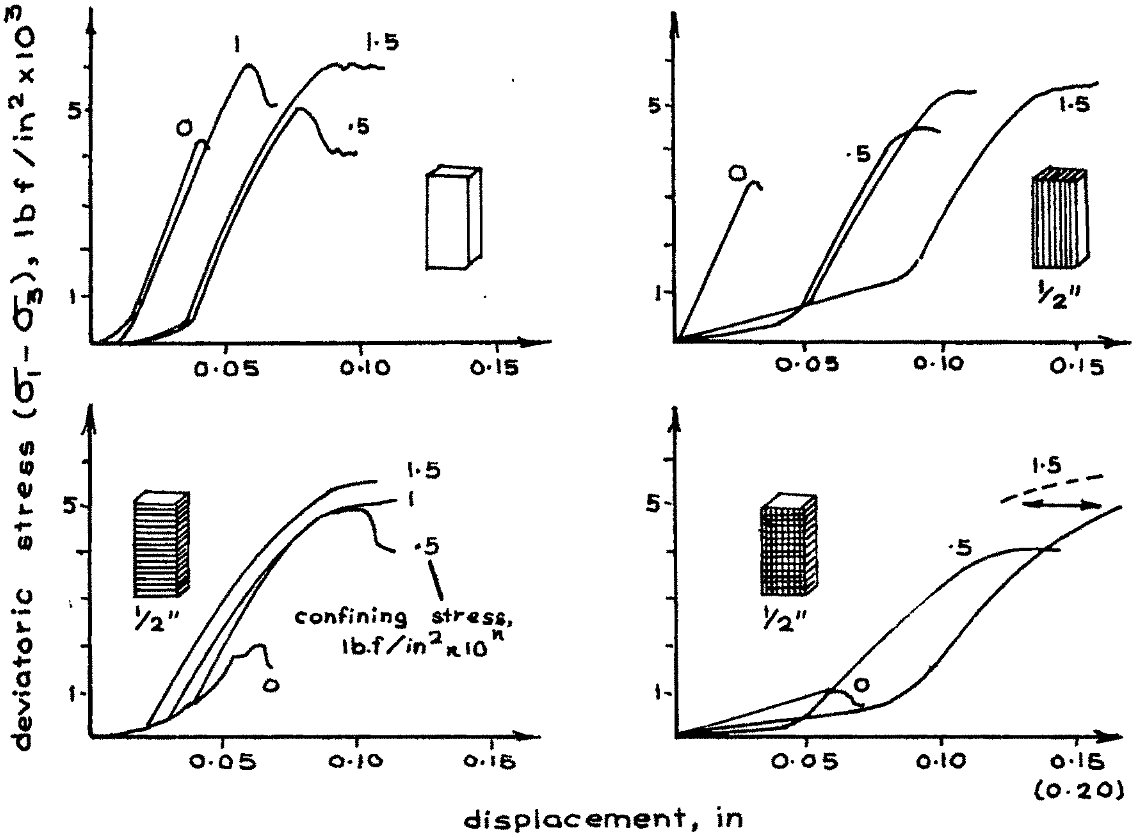


FIG. 3-42 a STRESS-DISPLACEMENT CURVES FOR DIFFERENT JOINT CONFIGURATIONS & CONFINING STRESSES
(after EINSTEN et al, 1969)

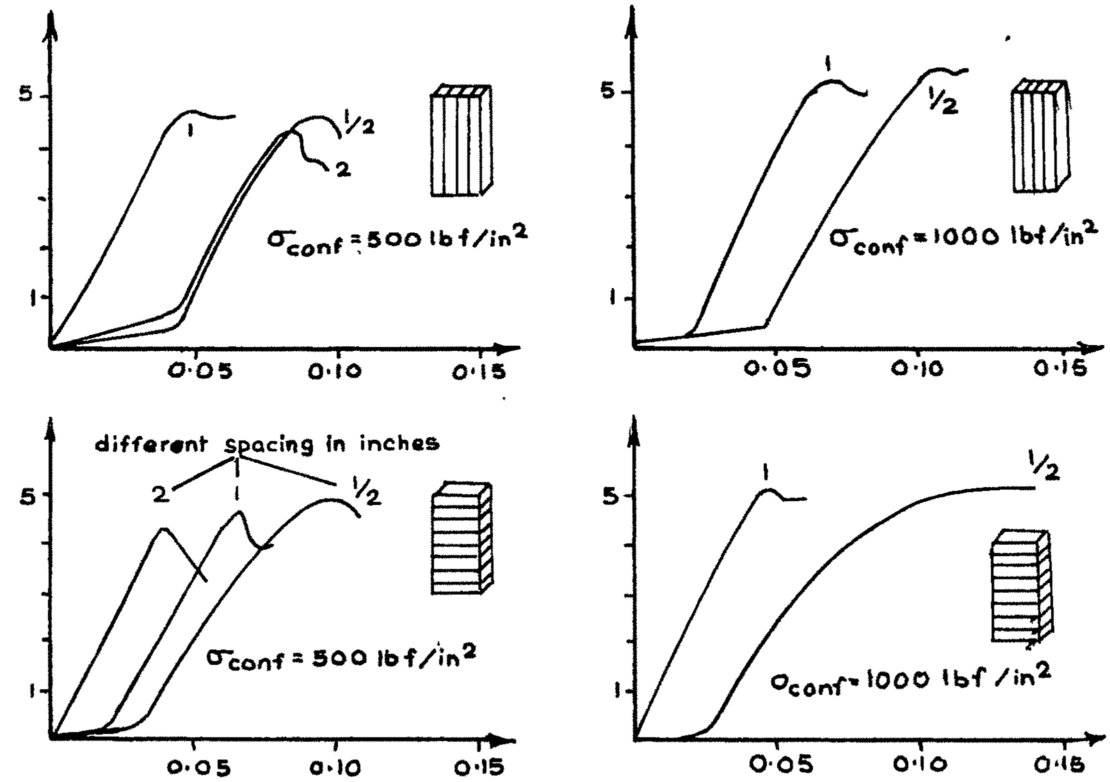


FIG. 3·42b STRESS DISPLACEMENT CURVES FOR DIFFERENT JOINT SPACING AND CONFINING STRESS.
(after EINSTEIN et al, 1969)

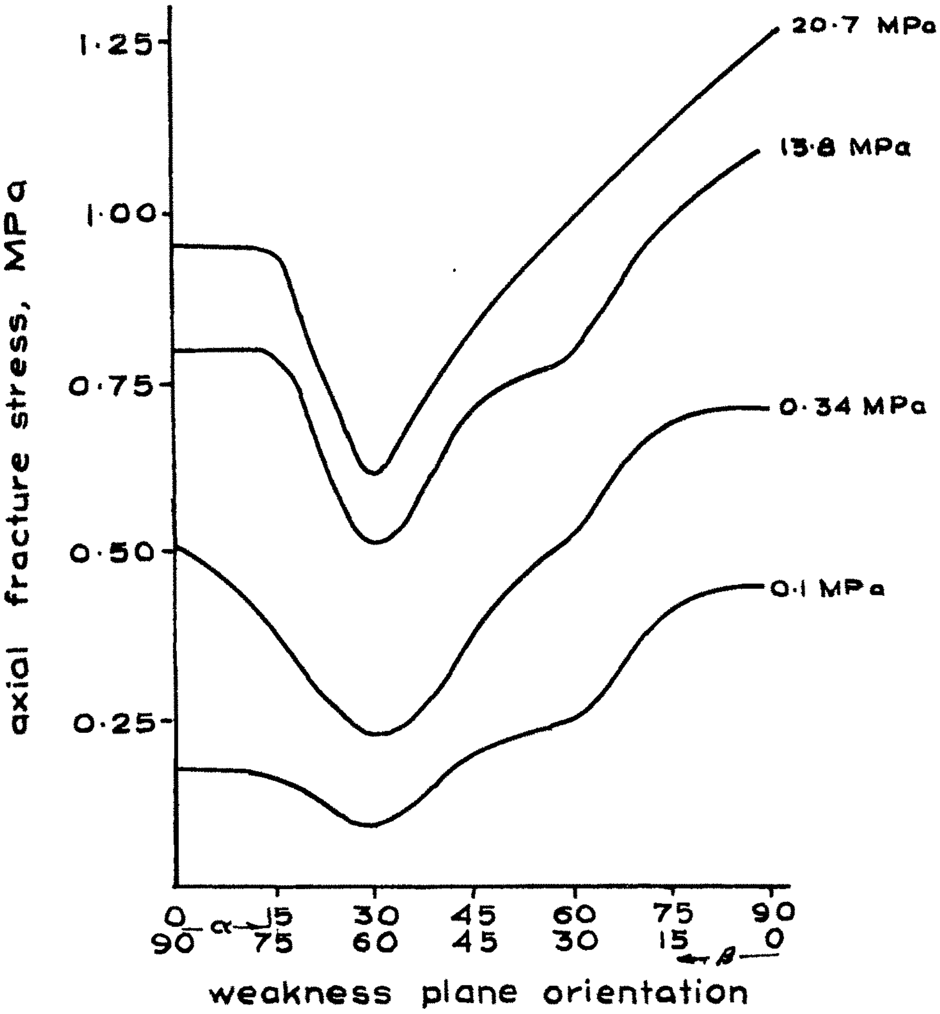


FIG. 3·43 RELATIONSHIP BETWEEN FRACTURE STRESS AND WEAKNESS-PLANE ORIENTATION FOR DIFFERENT CONFINING PRESSURES.
(after POMEROY et al, 1971).

joint density. Their conclusions are very similar to those drawn by other investigators that the strength of jointed rock masses will be much more influenced by the joint system within the rock mass at low confining pressure than at high confining pressure. The confining pressure has greater influence on the strength with two sets of joints than one set of joint. Joint orientation is more important in triaxial condition than the joint density. Deformation modulus decreases in the order for intact, vertical joint, horizontal joint, multiple inclined joint and orthogonal inclined joint at all confining pressures. (Fig. 3.44 and 3.45)

TABLE 3.3

Values of shear strength parameters

(after Brown and Trollope, 1970)

Specimen type	lbf/in^2	Z'	Z	
Unjointed	450	39	0.71	0.50
0°/90°	-	66	0.94	0.47
15°/75°	-	6.3	0.85	0.75
30°/60°	-	0.84	0.84	1.00
45°/45°	-	2.54	0.83	0.86

3.4.0. CONCLUDING REMARKS

From the review it is evident that the capabililies of most experimental setups are deficient to catch the various

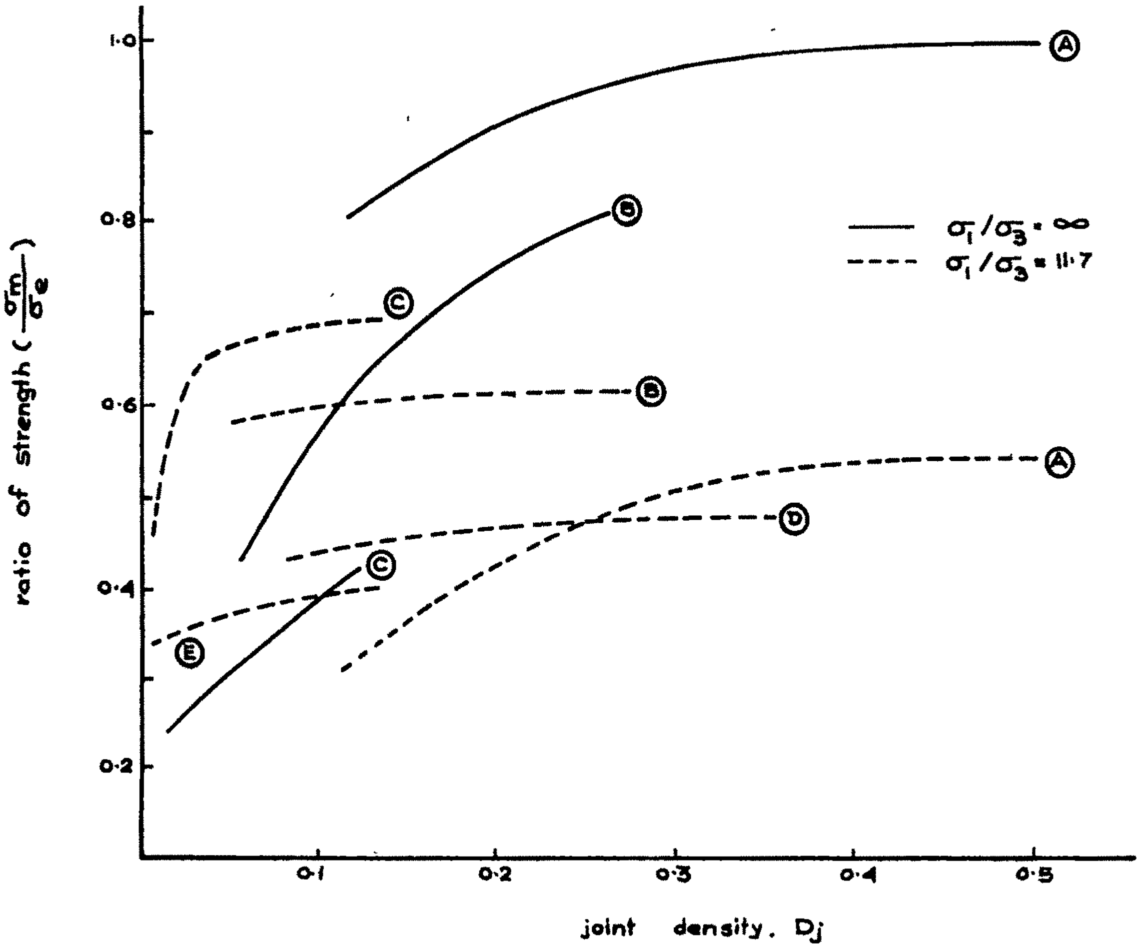


FIG. 3·44 EFFECT OF JOINT CONFIGURATION ON THE STRENGTH OF JOINTED MODELS

(A) VERTICAL JOINTS
(B) HORIZONTAL JOINTS
(C) MULTIPLE ORTHOGONAL JOINTS
(D) MULTIPLE INCLINED JOINTS (45°)
(E) ORTHOGONAL INCLINED JOINTS.

NOTE :- JOINT DENSITY $D_j = \frac{\text{Volume of element}}{\text{Volume of model}}$

(after MOTOYAMA and HIRSCHFELD, 1971)

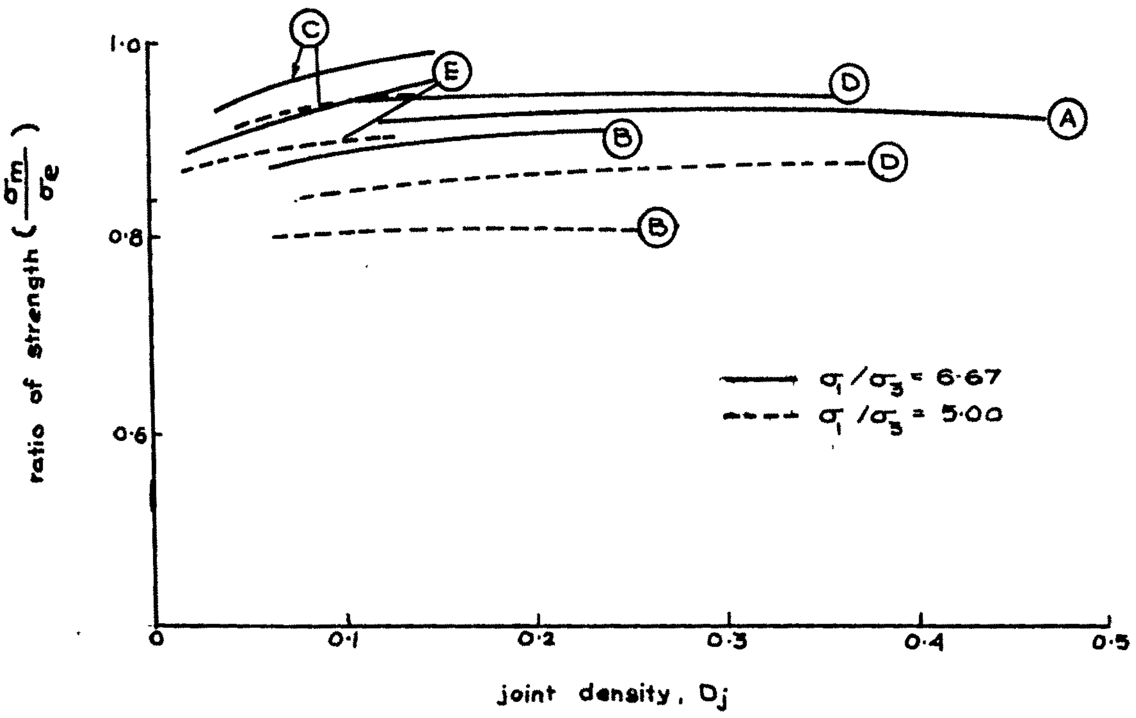


FIG. 3-45 EFFECT OF JOINTS CONFIGURATION ON THE STRENGTH OF JOINTED MODELS

(A) Vertical joints

(B) Horizontal joints

(C) Multiple orthogonal joints

(D) Multiple inclined joints (45°)

(E) Orthogonal inclined joints.

NOTE:- Joint density $D = \dfrac{\text{Volume of element}}{\text{Volume of model}}$

(after MOTOYAMA and HIRSCHFELD, 1971)

aspects of shearing behaviour of jointed rocks. From the experimental investigation also it is evident that there is a need to develop an experimental setup which can provide informations on the various aspects of the shearing, most particularly of dilatation. At present the only experimental setup which is versetile, interpretable is the triaxial cylindrical test. A servo-controlled closed loop test system with a capability to record the volume changes during the process of shearing may be regarded as the most appropriate system for investigating the jointed rocks in laboratory.