Tensile strength of Sandstone Under Unconfined and Confined Conditions

R. K. Bansal
D. K. Soni
&
Ashwani Jain
Deptt. of Civil Engineering
NIT Kurukshetra-136119, India
Ph.: 01744-238145
E-mail: dksoni@nitkkr.ac.in

ABSTRACT

The effect of rate of deformation on the tensile strength of sandstone under wet and dry conditions has been studied experimentally using the split tensile test technique. It has been observed that the tensile strength of the sandstone is highly dependent on the rate of deformation particularly in dry state. Tensile strength has also been determined under confined conditions. The deformation at failure in confined tests increases slightly with an increase in the confining pressure. There is sharp deviation between the failure envelopes from the confined split tests and the triaxial tests.

Keywords: Split tensile test, sandstone, triaxial test, confined, unconfined, deformation rate

1. INTRODUCTION

An accurate knowledge of the tensile strength of a rock mass is necessary in the design of underground openings, of bolting systems, for blasting and drilling processes, and for many other important engineering applications. Although the design procedures utilize compressive strength of the rock as a the basic property, it is no doubt true that even while a rock fails under a compressive load, tensile cracks first develop and these are often the first failure phenomenon observed.

The most logical method of measuring tensile strength is the straight pull test in which a sample of rock is subjected to a direct pull at its ends. Difficulties in centrally aligning the samples and the effect of grips at the ends lead to severe errors. To minimize the effects of end grips, the central section of the specimen is made of a small cross-section, i.e. the sample is made in the form of a "dog-bone" or a briquette type shape. Alternatively, epoxy resins have been used as adhesives, and suitable metal caps are fixed with these adhesives to the samples and pulled through flexible cables (Fairhurst, 1966).

Indirect estimation of tensile strength has become necessary in view of the many difficulties, which are encountered in conducting direct tensile tests. Diametral compression of solid discs or cylinders has been adopted with considerable success (Hondros, 1959; Hobbs, 1964). Colback (1966) and several others proposed the diametral compression of cylindrical discs with a small hole in the centre. Hobbs (1965) observed that the tensile strength as obtained from this type of 'ring test' is much higher than that obtained with a solid disc or cylinder.

Another method used is to apply an internal radial pressure to a hollow cylinder. From the results presented by Hardy and Jayaraman (1970), the ratio of tensile strength obtained by this method to that of the direct pull test, varies from 1.13 to 1.84 for the different types of rocks tested by them.

Several investigators have used 'point-load' test. The point load tensile strength is determined by applying compressive point loads to the curved surface of a cylindrical core specimen while the axis of the core is horizontal. The point load is applied in a testing machine through a hardened set of steel rollers or conical shaped pointed wedges at right angles to the axis of the specimen. The loading produces tensile stresses perpendicular to the axis of the loading.

McWilliams (1967) presented a method for estimation of tensile strength by subjecting disc of 5cm diameter to stress under two hemispherical indenters acting on the opposite surfaces of the disc at its centre.

2. SPLIT TENSILE TEST

Brazilian test (indirect split tensile test on a solid disc) has by far been adopted in a large number of testing programmes and research projects. This test has been used for determination of elastic properties of concrete by Hondros (1959), the tensile strength of coal by Berenbaun and Bordie (1959) and of rocks by Hobbs (1964). Tensile strengths measured in this manner are reproducible and are in reasonable agreement with values obtained in uniaxial tension. In some cases single diametral fractures are not found to occur but several fractures are seen branching from the diametral plane which may appear as wedges near the contacts. Therefore, some doubts have been raised about the mechanism of failure in this test and it has been suggested that the failure stress starts by shear failure in the region of high compressive stresses near the contacts.

The simplicity of sample preparation and ease in testing is a great advantage of this test. In this test, according to the linear elastic theory, there is a uniform tensile stress of $2P/\pi D$ across the diametral plane under a load P per unit length for a sample of diameter D.

3. CONFINED SPLIT TENSILE TEST

Jaeger (1965) suggested that if an additional confining pressure is applied to the specimen, the transition from tensile to compressive values of the least compressive stress could be studied. He called this as the confined 'Brazilian Test'. The experimental procedure and theoretical analysis was extended further by Jaeger and Hoskins (1966). Their experimental results indicated a small but systematic difference from the triaxial results.

The objective of this paper is to examine the effect of rate of strain on the tensile strength of typical sandstone under dry and wet conditions. The effect of confinement on the tensile strength has also been experimentally observed and compared with the observed strength under the triaxial stress system.

4. EXPERIMENTAL PROCEDURE

Sample of 3.75cm diameter were cored from large blocks of the sandstone. The ends of the cored samples were cut-off with a diamond saw and ground to a length of about 2.5cm. The samples, which were to be tested in dry conditions, were heated in oven for two days to expel moisture. For saturating, the samples were immersed in water for about 10 days.

The samples were placed horizontally between the flat surfaces of the platens of strain-controlled testing machine of a high capacity so that the load is applied across the diameter of the sample (Fig. 1). The deformations were recorded with a dial gauge of high accuracy. The range of deformation rates used varied from 0.00043 cm/min. to 0.118 cm/min.

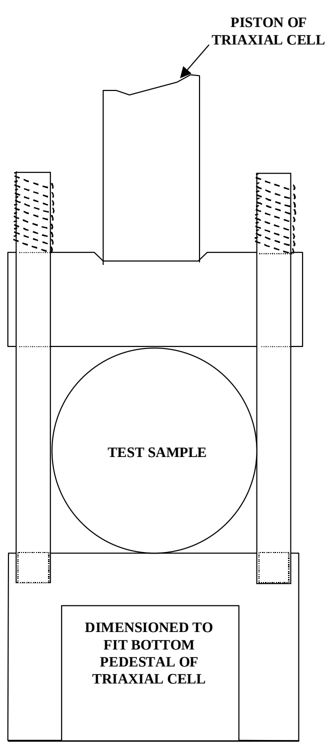

Fig. 1 - Parallel plate equipment for confined Brazilian tests

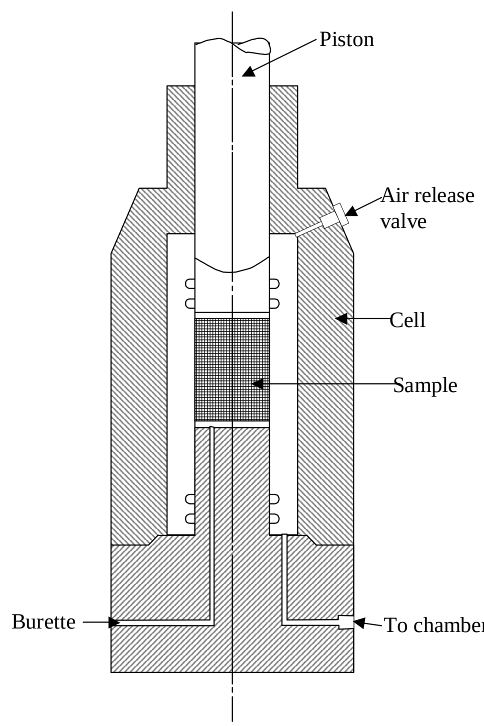

Fig. 2 - The triaxial cell for 1.5in. diameter samples under high confining pressure

For conducting confined split tensile tests, the specimens were jacketed by double rubber membranes sealed along the flat ends with the help of steel caps and O-rings. In order to keep the cylindrical rock samples in position between the two flat plates, a special arrangement was made so that the samples do not get displaced while under test in the triaxial cell. Suitable equipment for the pressure range used is not readily available. Therefore, the required equipment had to be specially fabricated. The triaxial cell is designed for taking fluid pressure up to 1050kg/cm^2 (105MPa). Details of the cell and the other components have been shown in Fig. 2. All the confined split tensile tests were conducted at a strain rate of 0.03048cm/min. The cell pressure was accurately controlled during the test and maintained constant. The range of cell pressure used varied from 35 kg/cm^2 to 280kg/cm^2 (3.5 to 28 MPa).

Samples of sandstone with height to diameter ratio of 2.0 were also tested in triaxial cell by subjecting them to shear stress by applying the deviator stress to their flat ends as per standard procedure. The cell pressure was varied from zero to 350 kg/cm^2 (35MPa). The samples were sheared under a deformation rate of 0.03048 cm/min.

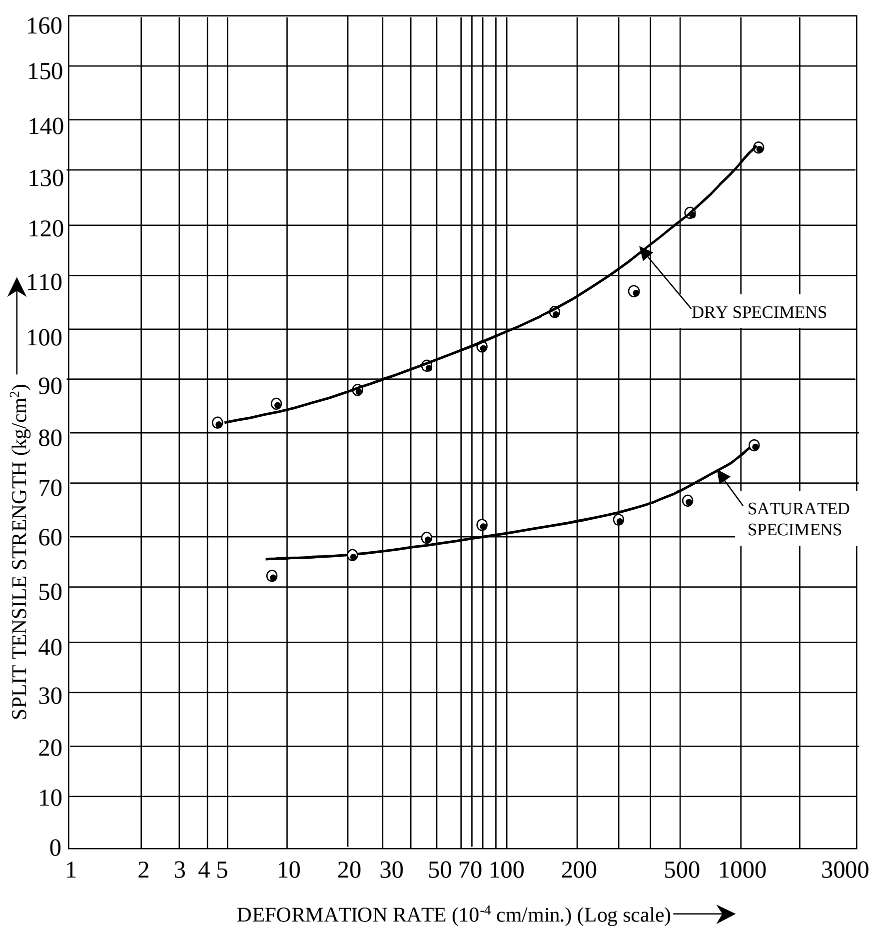

Fig. 3 - Variation of split tensile strength with deformation rate

5. OBSERVATIONS AND DISCUSSION

The influence of variation of the deformation rate on the sample on the split tensile strength is shown in Fig. 3. It has been observed that in the case of sample, which is tested in dry state, the rate of deformation has a greater effect, the higher the deformation rate the higher is the split tensile strength. An increase of as much as 70 percent can occur in the tensile strength of the sandstone when the deformation rate is varied from 0.0004 cm/min. to about 0.2cm/min. For the saturated samples, increase in strength is of about 40 percent for the same range of strain.

The load taken by the sample during the split tensile tests is plotted against the deformation at various stages in the test in Fig. 4. The load deformation paths are independent of the deformation rate.

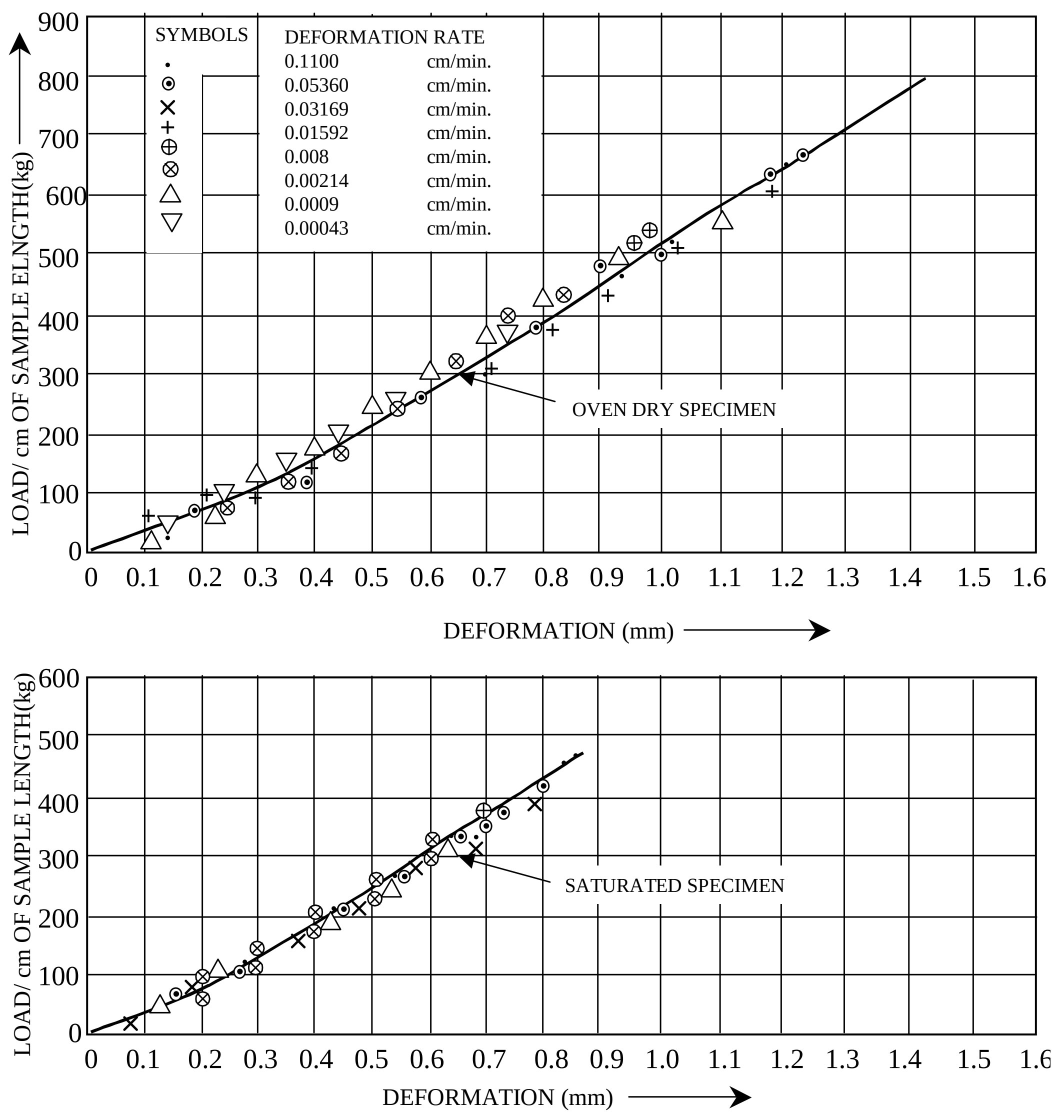

Fig. 4 – Relationship between load taken and deformation (unconfined split tests)

The load taken by the samples when subjected to confined split tensile tests is shown in Fig. 5. It has been observed that the samples fail suddenly after attaining a deformation of about 1.5mm, which appears to be independent of the confining pressure.

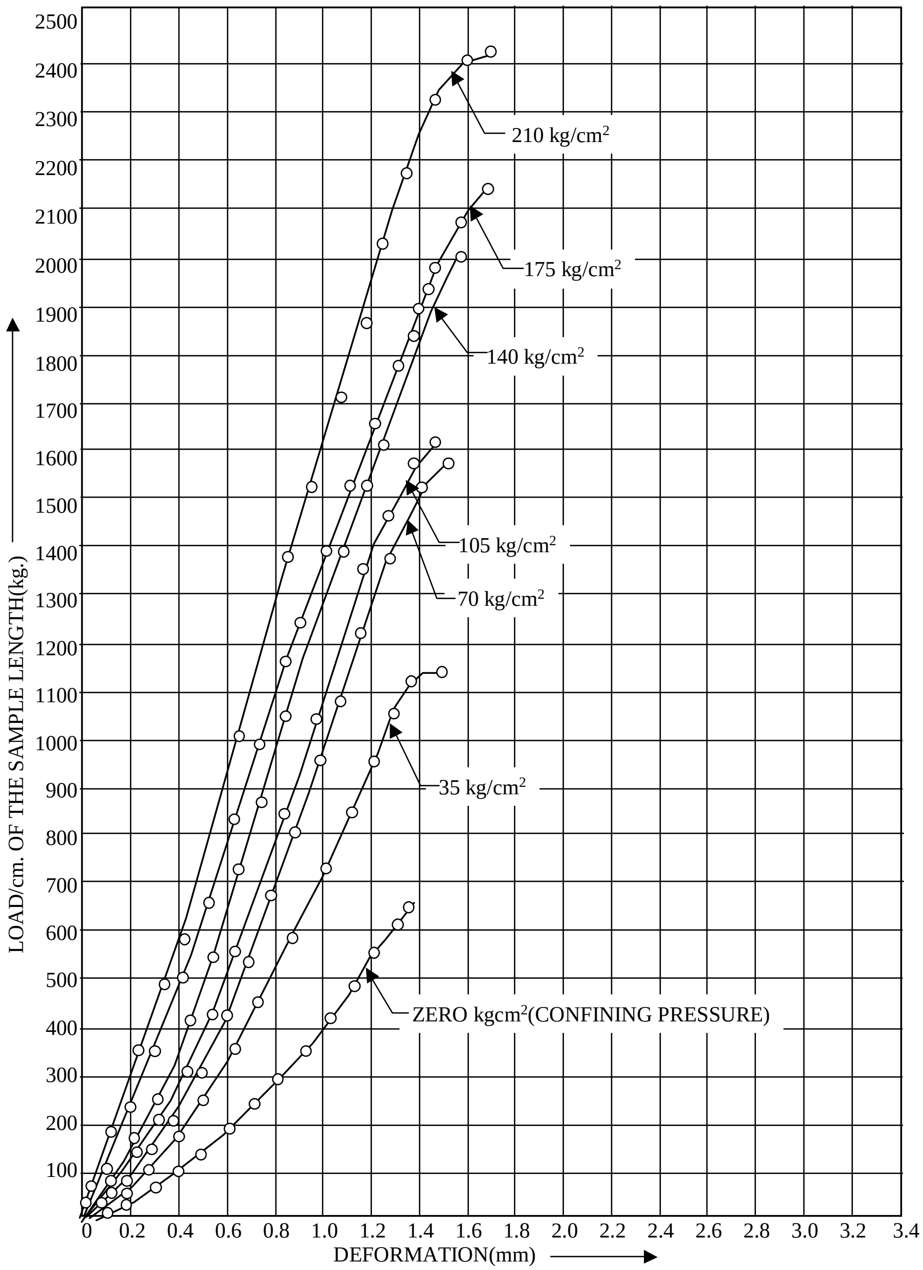

Fig. 5 - Relation between load taken & deformation (confined split tensile tests)

The stress-strain plots of the triaxial tests shown in Fig. 6, indicate that the failure strain is highly dependent on the confining pressure. Samples under unconfined state fail under a strain of about 3.0 percent, whereas when the confining pressure is increased to 280 kg/cm^2 (28MPa), the strain at failure increases to about 7.0 percent.

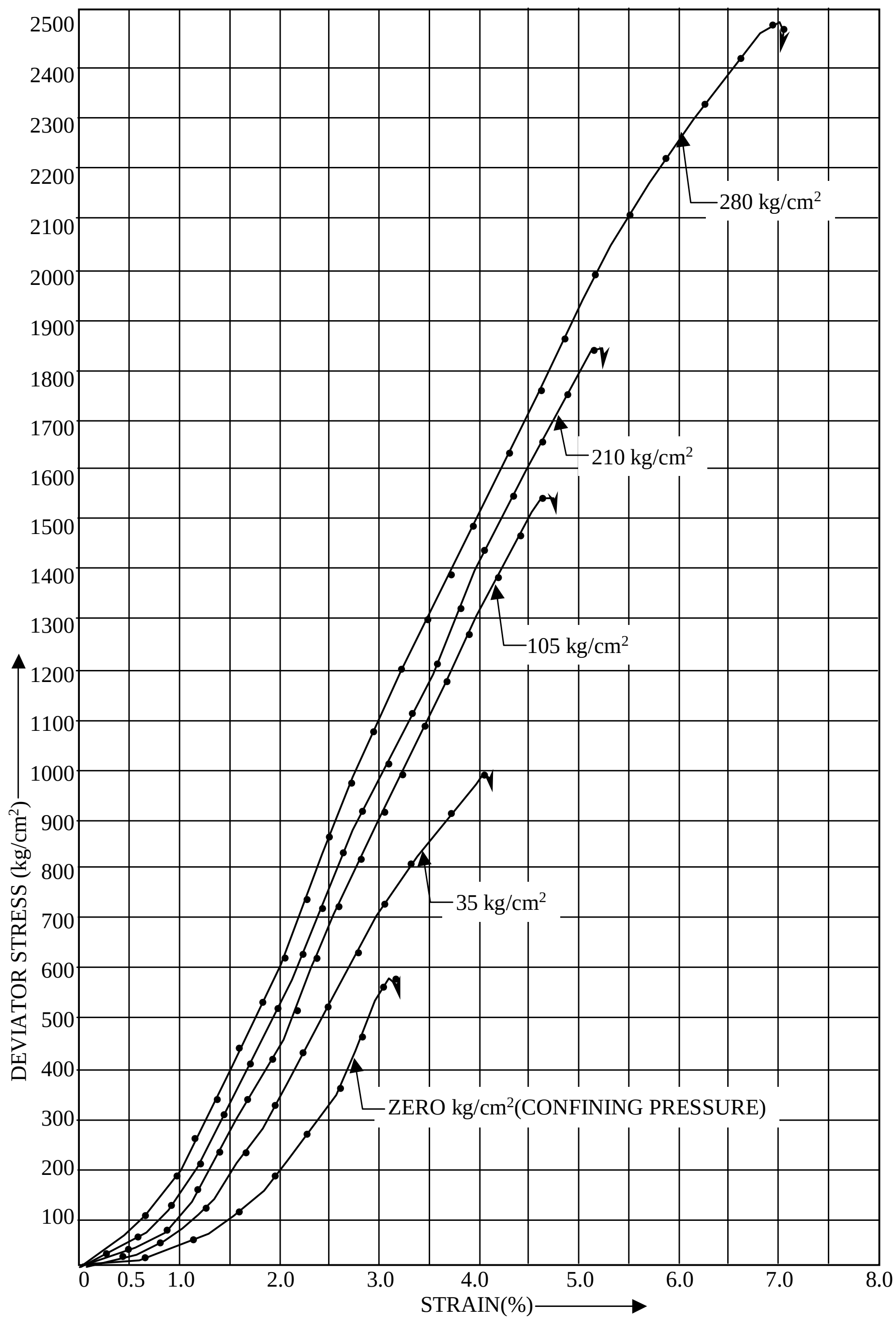

Fig. 6 – Stress-strain plots (Triaxial tests)

The stress system at the centre of the disc in the split tensile test is given by

$$\sigma_1 = \frac{6P}{\pi td}, \tag{1}$$

$$\sigma_3 = \frac{2P}{\pi td} \tag{2}$$

where

P = load at failure taken by the disc,
t = thickness of the disc,
d = diameter of the disc,
σ_1 = major principal stress, and
σ_3 = minor principal stress.

Under a confining pressure 'p' the stress system becomes

$$\sigma_1 = \frac{6P}{\pi td} + p, \tag{3}$$

$$\sigma_2 = p, \tag{4}$$

$$\sigma_3 = p - \frac{2P}{\pi td} \tag{5}$$

The σ_2 is now the intermediate principal stress.

Since load P is known, the magnitudes of $\sigma_1, \sigma_2, \sigma_3$ have been calculated and are tabulated in Table 1. The values of σ_1 against σ_3 can be plotted for various values of 'p'.

Table-1 Calculation of principal stresses

Cell pressure p(kg/cm^2)	Thickness of the specimen t (cm)	Diameter of the specimen d (cm)	Failure load P (kg)	P/t (kg/cm)	$\sigma_1 = p + \frac{6P}{\pi td}$ (kg/cm^2)	$\sigma_3 = p - \frac{2P}{\pi td}$ (kg/cm^2)
280	2.64	3.80	7925	3000	1786	-242
210	2.62	3.75	7287.5	2780	1620	-260
175	2.64	3.77	6350	2410	1393	-231
140	2.58	3.79	5775	2240	1268	-236
140	2.58	3.77	5100	1955	1139	-193
105	2.79	3.78	4900	1755	993	-191
105	2.62	3.80	4450	1700	960	-180
70	2.62	3.79	4400	1680	913	-211
70	2.50	3.78	4162.5	1665	910	-210
35	2.63	3.79	3325	1265	671	-177
35	2.56	3.78	3625	1415	752	-204

5.1 Fracture Patterns

The various types of fracture patterns observed during the unconfined and confined Brazilian tests are shown in Fig. 7. In the unconfined Brazilian tests, the samples failed by splitting along essentially a straight line along the plane of loading. The specimens, which did not show a diametral fracture plane, were rejected in confirmation with the theories of brittle fracture initiation mentioned earlier. In the confined Brazilian tests, the samples had failure planes slightly S-shaped. This is in confirmation with the investigations of Wiseman et al (1966). They attributed this due to variation in σ_2 and σ_3.

Fig. 7 – Failure under unconfined and confined Brazilian tests

The fracture patterns observed in triaxial test specimens are shown in Fig. 8. In the unconfined compression test, longitudinal splitting was observed. In triaxial tests, a single plane fracture was observed inclined at an angle less than 45^o to the direction of the major principal stress.

Fig. 8 – Failure under triaxial tests

5.2 Failure Criterion

A criterion of failure commonly adopted for rocks is the ‘Coulomb Criterion’. This leads to a linear relationship between σ_1 and σ_3. The intercept on the σ_1 axis is the uniaxial compressive strength whereas the intercept on the σ_3 axis is not the uniaxial tensile strength as the physical conditions restrict the criterion to only a portion of the failure line. Paul (1961) suggested a modification to the failure criterion and for low values of σ_1, the failure envelope was considered to be parallel to σ_1-axis. Experimental observations have indicated considerable deviations from the straight-line relationship of σ_1 and σ_3. Somewhat curved envelopes have been recorded for a large number of rocks (Hoek, 1965). Griffith suggested that microscopic cracks, which exist in all solid materials, act as points of stress concentration under load. The stresses developed at the tips of these cracks under certain combinations of applied principal stresses are tensile and when they reach a critical value, crack propagation occurs, which leads to brittle failure. This leads to a somewhat parabolic relationship between σ_1 and σ_3 (Jaeger and Cook, 1968). Under high confining pressure, the cracks are likely to close and the frictional resistance in the cracks is likely to offer additional resistance to failure. The theory has been modified by McClintock and Walsh (1962).

The results at the failure stage in terms of the major principal stress σ_1 and the minor principal stress σ_3 are plotted from the data obtained from confined split tensile test as well as the triaxial tests in Fig. 9. The results from these two types of tests show considerable divergence in the failure envelopes. This is not in agreement to the observations of some research data published earlier.

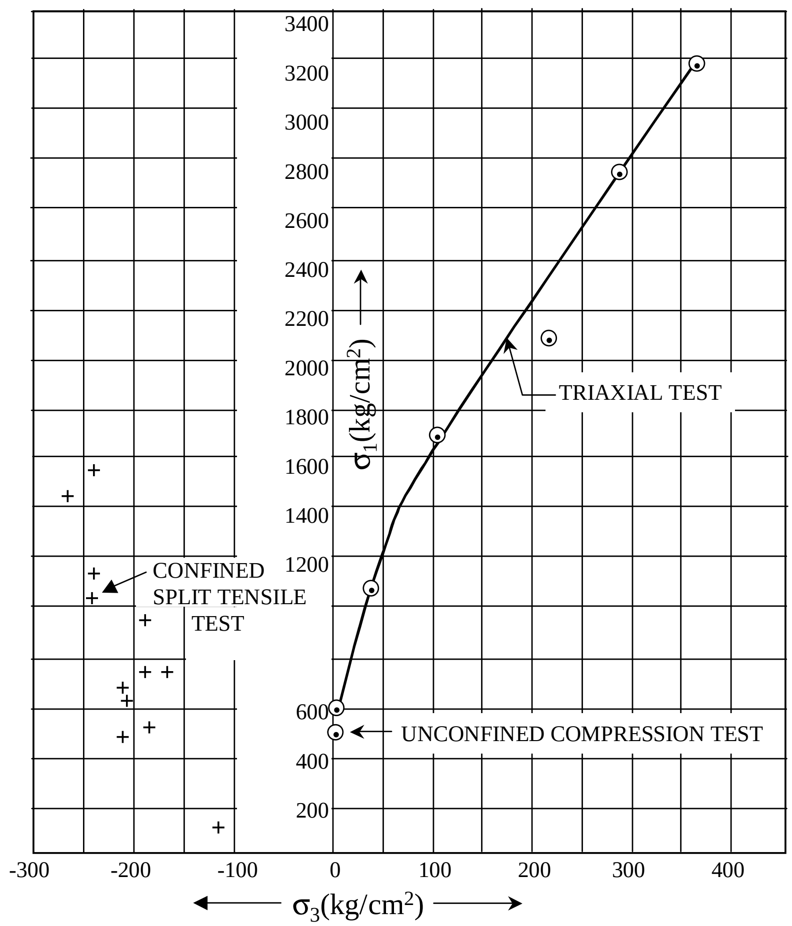

Fig. 9 – Failure envelopes (confined split tests & triaxial tests)

6. CONCLUSIONS

This study indicates that the tensile strength of sandstone is very much dependent on the rate of deformation particularly in dry state, although the load-deformation paths are practically independent of the rate of deformation.

The deformation at failure in the case of confined split tensile test increases slightly with an increase in confining pressure, whereas the strain at failure for samples tested under triaxial system of stresses, is very much dependent on the confining pressure. There is sharp deviation in the observed failure envelope from the expected one. Both these observations lead to the conclusion that the effect of the intermediate principal stress seems to be considerable. It may also be necessary to develop a failure theory in terms of strains rather than maximum stresses so far as tensile failures are concerned.

References

Berenbaun, R. and Bordie, I. (1959). The tensile strength of coal, J. Int. Fuel. Vol. 32, pp. 320-327.

Colback, P.S.B. (1966). An analysis of brittle fracture initiation and propagation in the Brazilian est, Proc. First Congress Int. Society of Rock Mechanics, Lisbon, 1, pp. 385-391.

Fairhurst, C. (1966). Laboratory measurements of some physical properties of rock, Fourth Symp. on Rock Mech., Penn. State University.

Hardy, H.R. and Jayaraman, N.J. (1970). Investigations of methods for the determination of tensile strength of rock, Proc. 2nd Congress of Int. Soc. Rock Mech.

Hobbs, D.W. (1964). The tensile strength of rocks, Int. Journal Rock Mech. Min. Sci., 1 pp. 385-396.

Hobbs, D.W. (1965). An assessment of a technique of determining the tensile strength of rock, Brit. J. Appl. Phys., 16, pp. 259-268.

Hoek, E. (1965). Rock fracture under static stress conditions, Natl. Mech. Eng. Res. Inst. C.S.I.R. Pretoria, C.S.I.R. Report MEG 383.

Hondros, G. (1959). The evaluation of Poisson's ratio and modulus of materials of low tensile resistance by the Brazilian (Indirect Tensile) test with particular reference to concrete, Australian J. of Appl. Sci., 10, No. 3.

Jaeger, J.C. (1965). Fracture of rocks, Tewksbury Symp. On Fracture Engr. Fac. University of Melbourne, pp. 268-283.

Jaeger J.C. and Cook, N.G.W. (1968). Fundamentals of rock mechanics, Methuen & Co. London.

Jaeger J.C. and Hoskins, E.R. (1966). Rock failure under confined Brazilian test, Jour. Geophysical Research, Vol. 71, pp. 2651-2659.

McClintock, F.A. and Walsh, J.B. (1962). Friction on Griffith cracks under pressure, Fourth U.S. Nat. Congress of Appl. Mech, Proc., pp. 1015-1021.

McWilliams, J.R. (1967). Discussion on brittle fracture of rocks-failure and breakage of rocks, Eighth Symp. on Rock Mech., pp. 142-143.

Paul, B. (1961). Modification of the Coulomb-Mohr theory of fracture, J. Appl. Mech., Vol. 28, pp. 259-268.

Wiseman, G., Levy, J. and Aisenstien, B. (1966). Strength studies of chalk specimens under moderate confining pressures, Proc. 1st Congress Int. Society of Rock Mech., Lisbon, Vol.1, pp. 483-487.