Strength Characteristics of Limestone Under High Pressure

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ABSTRACT

With the rapid advancement in construction of large structures, it has become necessary to make detailed study of shear strength of rocks under various conditions of loading. Investigations have been conducted with the objective of observing the behaviour of rocks under high confining pressure. Strength of dry specimens of limestone has been determined in triaxial compression under various confining pressures ranging up to 35.154 MPa. It has been observed that with the increase in confining pressure, axial strain and ultimate load at failure increase. The secant modulus of elasticity increases with increase in confining pressure. Test results have been discussed in the light of Hoek and Brown, Ramamurthy, and Singh and Singh criteria for intact rocks to compare experimental results with proposed criteria.

Keywords: Limestone, shear strength, confining pressure, threshold pressure, ductile, brittle.

1. INTRODUCTION

Shear strength of rocks has been a subject of study. It has been recognised to be an important property as stability of the foundation resting on rock is dependent on its shear strength. It is an important parameter for predicting rock failure and in designing underground openings.

This study has been conducted with the objective of observing the behaviour of rocks under high confining pressure. It is a fact that stresses to which the rock samples have been subjected to in this study may not develop in practice, yet, it is considered necessary to obtain information regarding strength and deformation characteristics of rocks under high pressure. A series of triaxial tests have been conducted on identical dry samples of limestone under confining pressures ranging up to 35.154 MPa. The influence on secant modulus has been investigated and rupture criteria of limestone are examined.

2. MODES OF FAILURE OF ROCKS

There are two modes of failure occurring in rocks depending on the amount of the deformation before failure. Rocks are described as brittle if they fail without large deformation, or ductile if they deform appreciably before failure, i.e. if they deform plastically. True brittle fracture, by definition, is a process which produces no permanent change in the material other than its separation into parts. Two basic processes generally operate together in ductile microscopic behaviour, dissipative processes such as gliding and viscous flow, and processes such as frictional gliding or rotation of grains about one another.

Rocks are usually considered as brittle material. The reaction of the rock to deformation depends upon its structure and upon the magnitude of the confining pressure, temperature, rate of loading and the presence and nature of interstitial solutions.

Terzaghi (1945) classified rock failure into splitting, shear, and pseudo-shear, depending on the inclination of the failure planes. Splitting may be recognized by cracks appearing parallel to the direction of the axial load, which seems to indicate that the bond between grains fail by tension.

Griggs and Handin (1960) described the macroscopic deformation of rocks and minerals deformed at high confining pressures in the laboratory in terms of three principal categories of behaviour - tension fractures, faults, and uniform flow.

Robinson (1959) analysed thin sections of Indiana limestone subjected to high confining pressures and found that crystals twist and then fail by shear fracture.

Serdengecti and Boozer (1961) found that the type of failure of rock in triaxial compression is dependent upon confining pressure, temperature and rate of deformation. Rock failure occurs in a brittle manner at low confining pressures, low temperatures and high rates of deformation. On the other hand, ductile failure is found at high confining pressure, high temperatures, and low deformation rates.

Boozer, Hiller and Serdengecti (1962) further reported that the pore fluid pressure affects the mode of failure. At a confining pressure of 1000lbf/in^2 (7.03MPa), Indiana limestone fails in a brittle manner. As confining pressure is increased, it begins to yield and deform in a ductile manner.

Schwartz (1964) observed failure surface under binocular microscopes and studied micrographs. Transition from brittle to ductile failure occurs when confining pressure is increased from 0 to 10000lbf/in^2 (70.3MPa) for limestone and marble, while no ductile failure is observed for granite and sandstone at confining pressures up to 10000lbf/in^2 (70.3MPa). The mode of failure progresses slowly from tension to pseudo-shear and finally shear.

Ramez (1967) conducted triaxial tests on Darley Dale sandstone under dry conditions and at room temperature. All specimens failed along shear fracture surfaces.

Confirming to non-linear response of strength with confining pressure through trial and error process, Hoek and Brown (1980) suggested the following equation for the intact rocks. The relationship between the principal stresses at failure (σ_1 and σ_3) for a given rock is defined by two constants, the uniaxial compressive strength (σ_{ci}) and a constant m_i.

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left[m_i \frac{\sigma_3}{\sigma_{ci}} + 1\right]^{0.5} \qquad (1)$$

Hoek and Brown (1980) used a range of $0<\sigma_3<0.5\sigma_{ci}$ and in order to be consistent, it is essential that the same range be used in any laboratory triaxial test on intact rock specimens.

Mohr-Coulomb theory was modified by Ramamurthy and co-workers (1985) to represent the non-linear shear strength response of intact rocks in the form:

$$\left(\frac{\sigma_1 - \sigma_3}{\sigma_3}\right) = B_i \left(\frac{\sigma_{ci}}{\sigma_3}\right)^{\alpha} \qquad (2)$$

where α is the slope of the log-log plot between $(\sigma_1 - \sigma_3)/\sigma_3$ and σ_{ci}/σ_3 – for most intact rocks, its mean value is 0.8; and B_i is a material constant, a function of rock type and quality, is equal to $(\sigma_1 - \sigma_3)/\sigma_3$, when $\sigma_{ci}/\sigma_3 = 1$. The value of B_i varies from 1.8 to 3.0 for argillaceous, arenaceous, chemical and igneous rocks.

The values of α and B_i can be estimated by conducting a minimum of two triaxial tests at confining pressures greater than 5% of σ_{ci} for the rock. The above expression is applicable in the ductile region and in most of the brittle region. It underestimates the strength when σ_3 is less than 5% of σ_{ci} and also ignores the tensile strength of the rock.

Singh and Singh (2003) proposed a simple parabolic curve to define the strength criteria for unweathered dry and isotropic rocks. The criterion is derived on the basis of the fact that the friction angle approaches towards zero as the rock passes brittle-ductile transition. The proposed criterion needs estimation of a single parameter 'A', whereas the other non-linear criteria require at least two parameters. The brittle-ductile transition boundary is assumed at a confining pressure equal to the uniaxial compressive strength (σ_{ci}) of the rock material. The equation of a parabola is given as:

$$(\sigma_1 - \sigma_3) = A\,(\sigma_3)^2 + B\,(\sigma_3) + C \qquad (3)$$

$$A \neq 0 \text{ and } 0 < \sigma_3 \leq \sigma_{ci}$$

where A, B, C are the criterion parameters and are computed by putting boundary conditions.

The Eq. 3 may be converted into linear form by putting boundary conditions as:

$$(\sigma_1-\sigma_3) - \sigma_{ci} = A\,(\sigma_3^2 - 2\sigma_{ci}\sigma_3) \qquad (4)$$

3. LABORATORY TESTING

3.1 Preparation of Rock Specimens

The rock selected for the present study is limestone of light grey colour of Shivalik formations collected from Kala Amb near Nahan in Himachal Pardesh. All the specimens were obtained from single slab in order to avoid variation in strength from specimen to specimen. Rock specimens were prepared finally for dimensions of 7.62 cm length and 3.81 cm diameter. The procedure included drilling of rock cores, and their cutting and grinding to final shape.

3.2 Test Procedure

Dry specimens of rock were tested with the help of triaxial equipment under various confining pressures ranging up to 35.154 MPa at a strain rate of 0.0105cm/min. Suitable equipment for the pressure ranges used is not readily available. Therefore, the required equipment was specifically fabricated in the laboratory. The cell is designed for triaxial compression tests for standard 3.81cm diameter specimens having a length of 7.62cm. Details of the cell and other components have been shown in Figs. 1 & 2. The samples were doubly wrapped in rubber tubes of 0.8mm thickness.

The fluid used in the cell is Teresso oil. Pressure in the cell is developed with the help of motorised pump and is indicated by a gauge. A hand-operated ram can do final adjustments. To maintain the constant pressure, a loaded ram is introduced. It is a small hydraulic accumulator in which dead load is applied to the ram.

Friction and leakage are eliminated by using oil as fluid and by rotating the ram continuously with an electric motor. When the pressure in the cell increases due to the friction of the loading ram during the test, the ram is uplifted, allowing the fluid to escape through a release valve to the reservoir and thus maintaining a stipulated constant pressure.

A mechanically operated compression-testing machine having a 50-ton capacity was used (Fig. 3). The machine has been designed to give 12 constant rates of travel of the lead screw ranging from 0.508 cm/min. to 0.0000127 cm/min., with the help of two helical change gears and four sockets. The unit is electrically operated and is intended for use on 440 Volts, 3-phase, 50 cycles A.C supply.

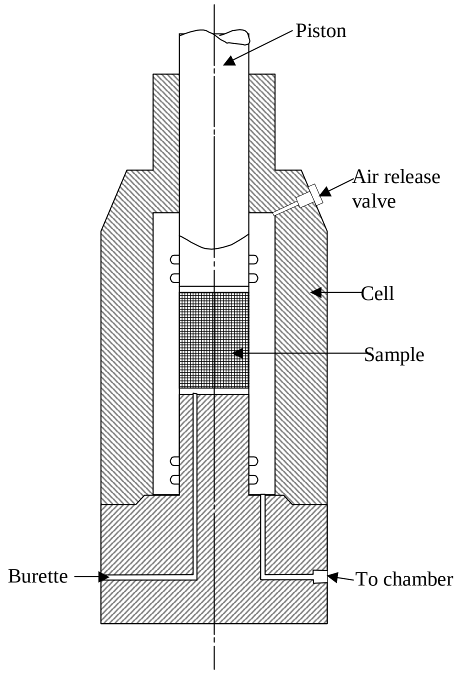

Fig. 1- The triaxial cell for 3.81cm diameter samples under high confining pressure

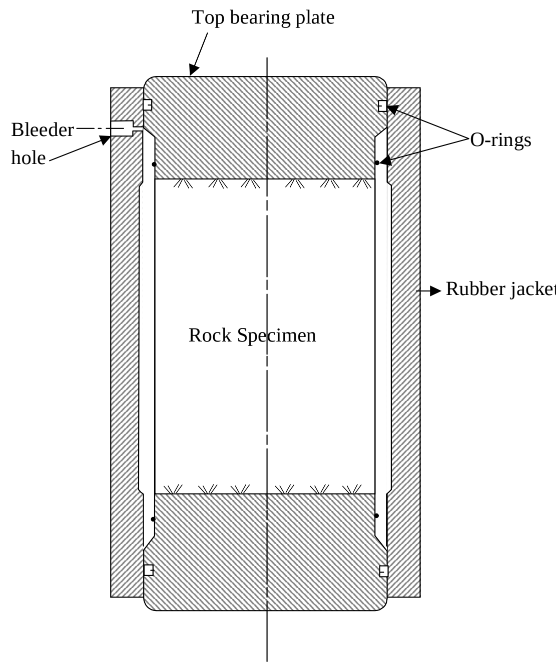

Fig. 2 - Half section of rock specimen pressure jacket and bearing plate assembly

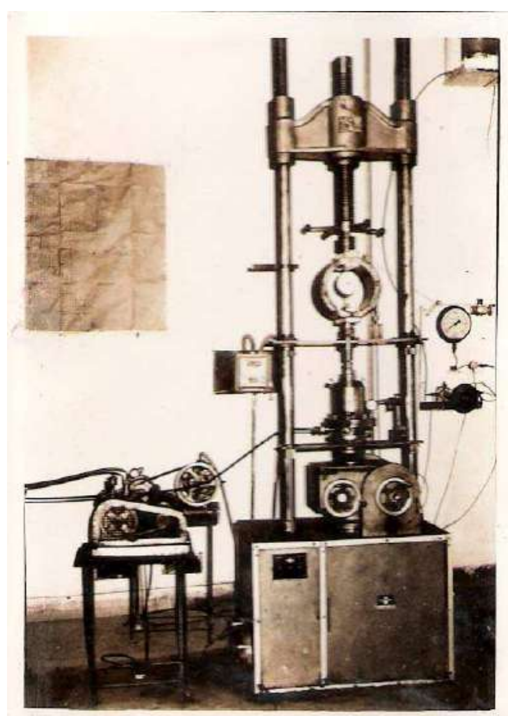

Fig. 3 – 50 ton compression machine with high pressure cell and pressure maintaining system

4. ANALYSIS OF RESULTS

Curves showing variation of deviator stress (σ_1-σ_3) versus axial strain (Fig. 4) for all samples tested show that axial strain at failure increases with the increase in confining pressure (σ_3). The unconfined compressive strength (σ_{ci}) of the sample as determined in laboratory is 47.716 MPa. In triaxial tests, the axial strain at failure ranges between 4.5 to 6.7 percent at low confining pressures, and between 9.5 to 11.8 percent at higher confining pressures. Secant modulus of elasticity under different confining pressures has been calculated and reported in Table 1. The values have been calculated for the linear portion of deviator stress versus axial strain curve. Secant modulus of elasticity also increases with the increase in confining pressure. From Table 1, it is also clear that ultimate load increases with the increase in confining pressure.

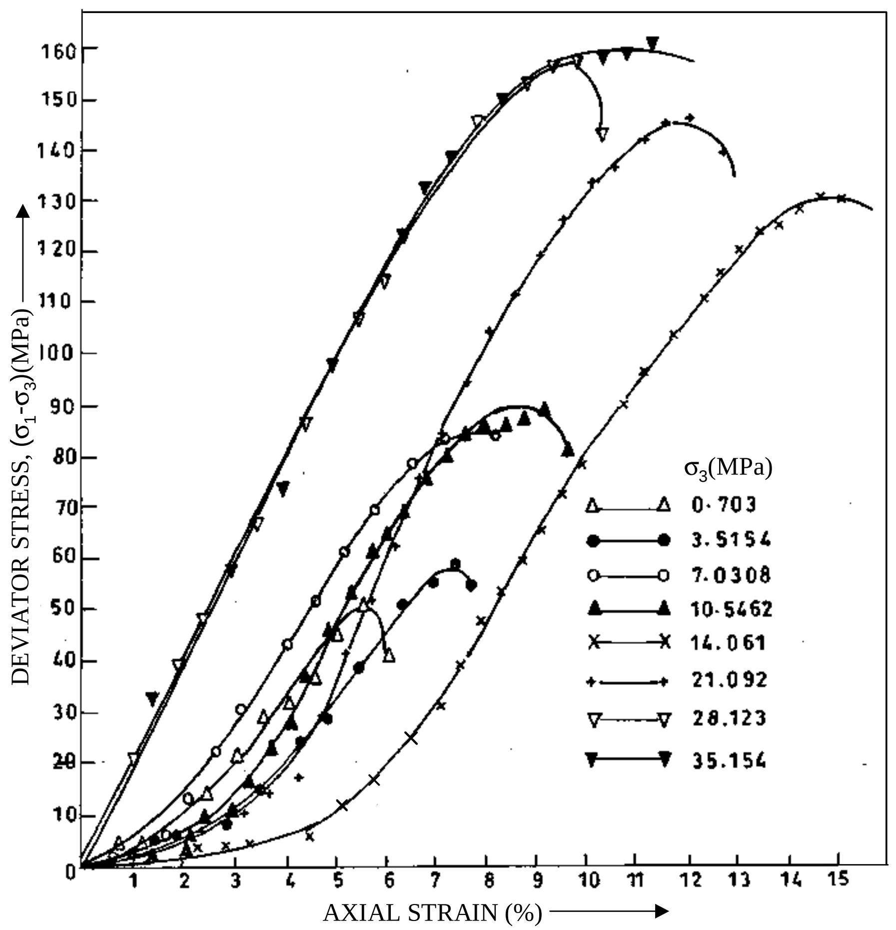

Fig. 4 – Stress-strain curves for limestone at different confining pressures

For the experimental observations, as per Hoek and Brown criterion, the values of m_i have been calculated in Table 1. The values of m_i vary widely from 4.63 to 21.90, whereas the suggested value of m_i for such rocks is 7.

Figure 5 shows a curve between deviator stress (σ_1- σ_3) and confining pressure (σ_3). It approaches to become asymptotic to σ_3-axis. The plot is parabolic as suggested by Singh and Singh (2003), though here, brittle-ductile transition is not reached, as in the present investigation, confining pressure has not been increased to a level so that it becomes equal to unconfined compressive strength. The values of parameter 'A' as suggested by Singh and Singh (2003) have also been calculated at various confining pressures as reported in Table 1. The proposed criterion appears to be more faithful to test data than Hoek and Brown (1980) criterion.

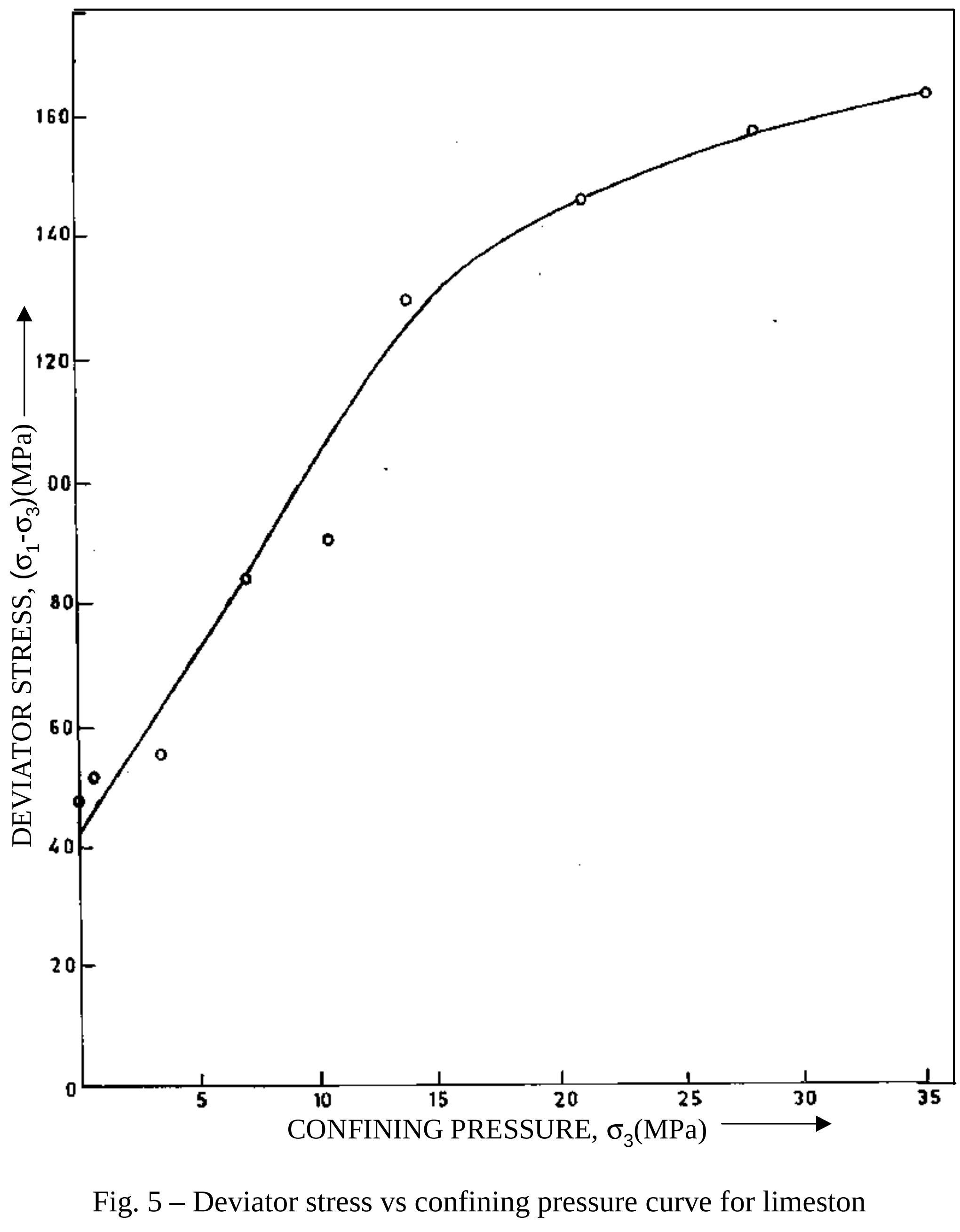

Fig. 5 – Deviator stress vs confining pressure curve for limeston

On the log-log plot between $(\sigma_1 - \sigma_3)/\sigma_3$ and σ_{ci}/σ_3 showing proposed criterion for chemical rocks (Ramamurthy, 1985), corresponding observed values of the present study (as calculated in Table 1) have been plotted (Fig. 6). A good agreement is found between theoretical and experimental results.

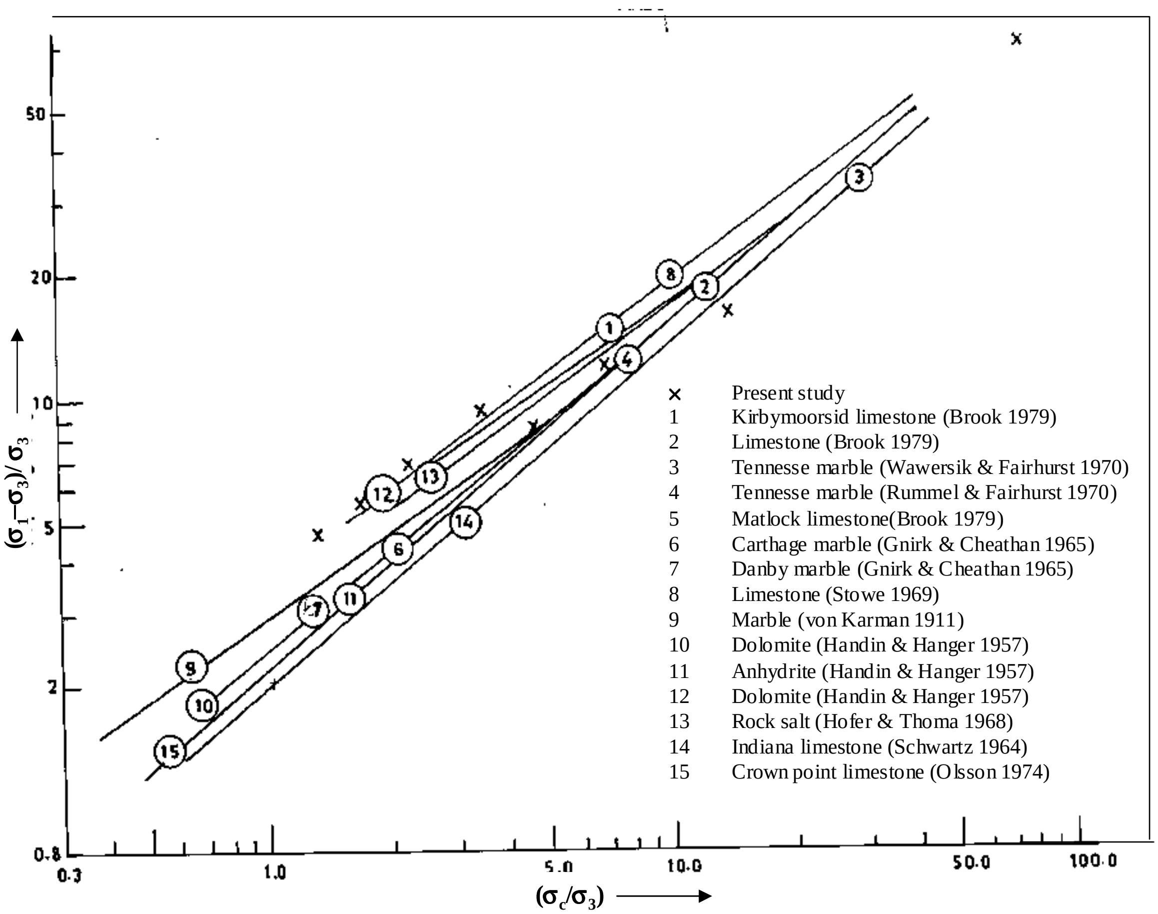

Fig. 6 – Plot of proposed criterion for chemical rocks (Ramamurthy, 1985)

It has been observed that the failure of limestone occurs in a brittle manner at low confining pressures. On the other hand, at higher confining pressures, failure is ductile. Figure 7 shows failure pattern of limestone samples at various confining pressures. Brittle rupture takes place only up to a definite pressure called the 'threshold pressure'. At pressures higher than this threshold, the substance converts from the brittle state into the plastic one. Further pressure increases the plasticity. One of the causes of increased plasticity is that the additional pressure tends to change the stressed state. As a result, the normal stress acting on a sliding plane under a particular confining pressure becomes compressive instead of stretched. The probable formation and development of specimen micro-cracks is thereby suppressed, and the plastic deformation process is promoted. Another cause of increased plasticity is that deformation under high pressure cures the micro-cracks that earlier existed.

Table 1 - Strength and Deformation Characteristics of Limestone at Various Confining Pressures

S.No.	Cell pressure (σ_3) (MPa)	% strain at failure*	Secant modulus E_{sec} in linear zone (MPa)	Deviator stress at failure (σ_1-σ_3) (MPa)	σ_1 at failure (MPa)	Hoek and Brown parameter (m_i)	Ramamurthy Criterion**		Singh and Singh parameter (A)
							$\frac{\sigma_1 - \sigma_3}{\sigma_3}$	$\frac{\sigma_{ci}}{\sigma_3}$	
1	0.703	4.5	1110.1	50.918	51.621	9.41	72.43	67.87	-0.0481
2	3.5154	5.2	1126.3	55.253	58.7684	4.63	15.72	13.57	-0.0233
3	7.0308	5.8	1480.2	84.123	91.1538	14.31	11.96	6.79	-0.0586
4	10.5462	6.7	1502.7	89.8868	100.433	11.52	8.52	4.52	-0.0471
5	14.061	9.9	2161.1	130.157	144.218	21.90	9.26	3.40	-0.0720
6	21.092	9.5	3227.3	145.687	166.779	18.81	6.91	2.26	-0.0620
7	28.123	9.8	3523.0	157.09	185.213	16.72	5.59	1.70	-0.0578
8	35.154	11.8	4355.4	163.678	198.832	14.64	4.66	1.36	-0.0547

(* after applying zero correction in Fig.4; ** comparison plot in Fig. 6)

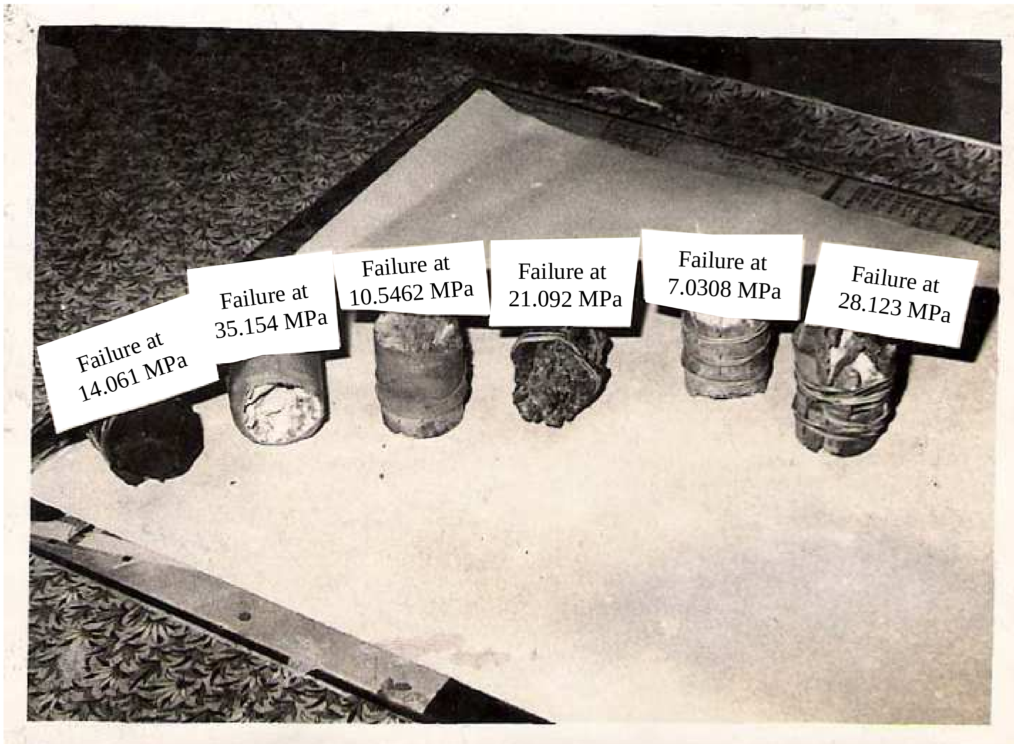

Fig. 7 - Failure under high pressure

5. CONCLUSIONS

With the increase in confining pressure, axial strain at failure increases. The axial strain at failure ranges between 4.5 to 6.7 per cent at low confining pressures, and between 9.5 to 11.8 per cent at higher confining pressures. The secant modulus of elasticity and ultimate load at failure increase with the increase in confining pressure**.** The criteria suggested by Ramamurthy (1985) and Singh and Singh (2003) for strength of intact rocks find better agreement with the test data as compared to Hoek and Brown (1980) criterion.

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