LECTURE 5

2.3.2.4 Triaxial Test For Shear Strength Properties

Triaxial testing on the cylindrical rock specimens are conducted by applying confining pressure to a desired value and axially loading the specimen in the compression testing machine till failure. The axial load and the confining pressure are increased simultaneously and in such a way that to maintain a hydrostatic condition until the predetermined test level for the confining pressure is reached. Subsequently the confining pressure has to be maintained in the predetermined level till failure occurs with axial loading. Three minimum test need to be conducted at three different confining pressure levels to get the three different failure loads but testing on five specimens are preferred at 5 different levels of confining pressure and gives more reliable results (IS 13047). Figure 2.13, 2.14 and 2.15 depicts the photographs of the 70 MPa capacity Hoek triaxial cell, hydraulic pressure unit and the 200 tons capacity compression testing machine facility in the Rock mechanics laboratory, IIT Madras.

Figure 2.13: Hoek cell for triaxial testing in rocks

Figure 2.14: Hydraulic pump for confining pressure

Figure 2.15: Triaxial testing of cylindrical rock specimen

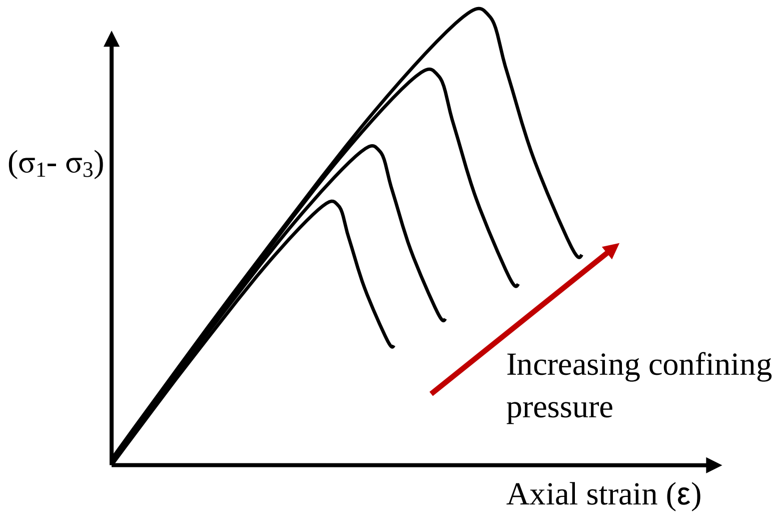


Figure 2.16: Deviatoric stress versus axial strain with increasing confining pressure

The triaxial test carried can be used to determine the shear strength parameters of discontinuity in the rock specimen with effect of confining pressure. The rock specimens for conducting this test are prepared as per IS specifications. Generally by Hoek cell apparatus the triaxial test is carried for rock specimen. The confining pressure is applied by means of hydraulic pump or some other systems and maintained through the oil, filled in the cell. The axial load is applied to the specimen at a constant rate of deformation or loading in a loading machine. This test provides individual points on the failure (peak strength) envelope from several tests. At least three specimen tests under different confining pressures are conducted such that it adequately defines the strength envelope over the required range of confining pressures. From this the value of the internal friction angle ϕ and the apparent cohesion 'c' may be obtained.

Figure 2.16 shows the idealistic stress-strain characteristic of rock with increasing confining pressure. As can be seen, with the increase in confining pressure the peak failure stress also increases and the corresponding Mohr envelops drawn are shown in Figure 2.17. The cohesion and friction values of the rock material is determined from the best fit line as shown in Figure 2.17. The best fit line is basically the Mohr-Coulomb failure envelope.

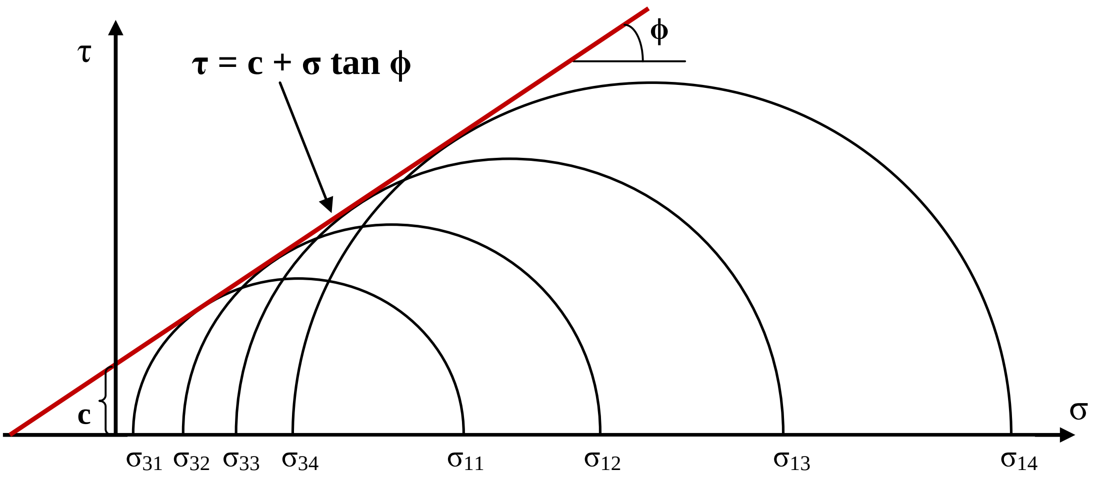


Figure 2.17: Determination of cohesion and friction using Mohr circles

2.3.2.5 Direct Shear Test

The shear resistance of rock is a result of friction and interlocking of particles, and possibly cementation or bonding at particle contacts. Due to interlocking, particulate material may expand or contract in volume as it is subject to shear strains. Due to shearing rock may expands its volume, the density of particles will decrease and the strength will decrease. So the peak strength would be followed by a reduction of shear stress. The stress-strain relationship levels off when the material stops expanding or contracting, and when inter-particle bonds are broken. The theoretical state at which the shear stress and density remain constant while the shear strain increases may be called the critical state, steady state, or residual strength.

The volume change behavior and inter particle friction depend on the density of the particles, the inter granular contact forces, and to a somewhat lesser extent, other factors such as the rate of shearing and the direction of the shear stress. The average normal inter granular contact force per unit area is called the effective stress. If water is not allowed to flow in or out of the rock mass, the stress path is called an *undrained stress path*. On the other hand, if the fluids are allowed to freely drain out of the pores, then the pore pressures will remain constant and the test path is called a *drained stress path*. The rock mass is free to dilate or contract during shear if it is drained.

The shear strength of rock depends on the normal/effective stress, the drainage conditions, the density of the particles, the rate of strain and the direction of the strain. For undrained, constant volume shearing, the Tresca theory may be used to predict the shear strength, but for drained conditions, the Mohr–Coulomb theory may be used.

In this test,

$$\text{Shear stress }(\tau) = \frac{P_s}{A'} = \frac{P_s.\cos\alpha}{A'}$$

$$\text{Normal stress }(\sigma) = \frac{P_n}{A'} = \frac{P_n.\sin\alpha}{A'}$$

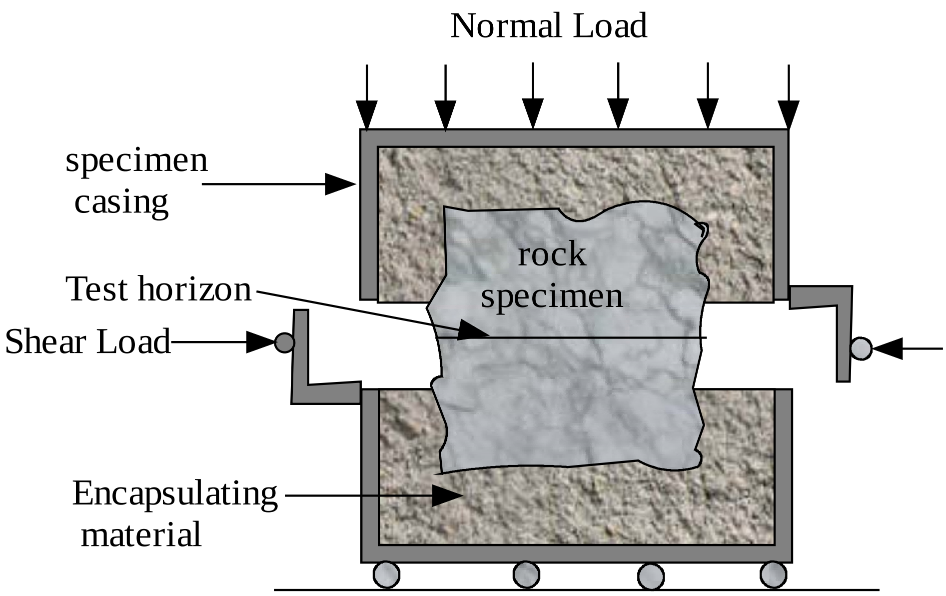


Figure 2.18: Schematic view of rock block with encapsulating material casting in a Direct shear box

The applied shear load by actuators and connecting parts should be designed to ensure that the shear load is uniformly distributed over the discontinuity plane to be tested with the resultant force acting parallel to the shear plane through its centroid. Maintenance of the normal load is also important during shear tests and the apparatus must be devised to maintain the applied force or stiffness within a specified tolerance (±2 %). The cohesion and angle of internal friction then determined from the normal stress vs. shear stress graph as shown Figure 2.22

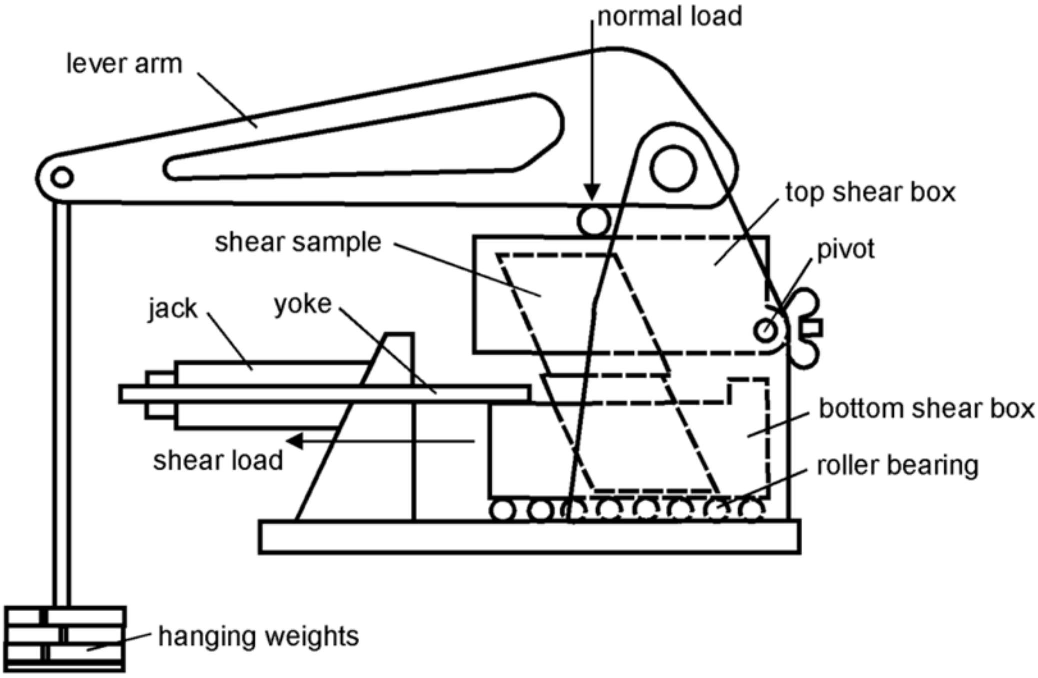


Figure 2.19: Diagrammatic section through shear machine used by Hencher and Richards (1982).

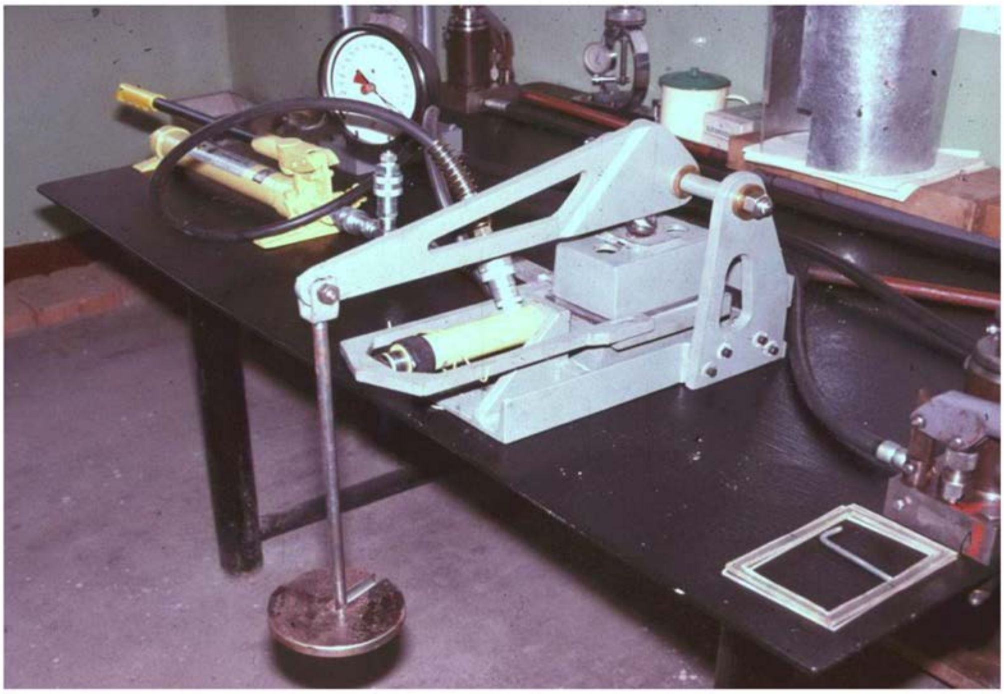


Figure 2.20: Shear machine of the type used by Hencher and Richards (1982) for measurement of the shear strength of sheet joints in Hong Kong granite

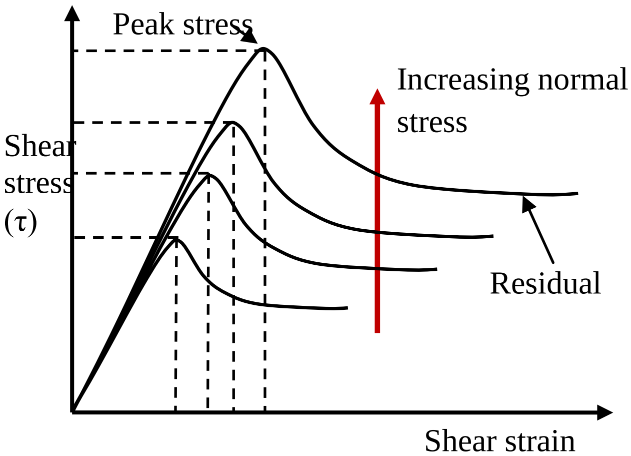
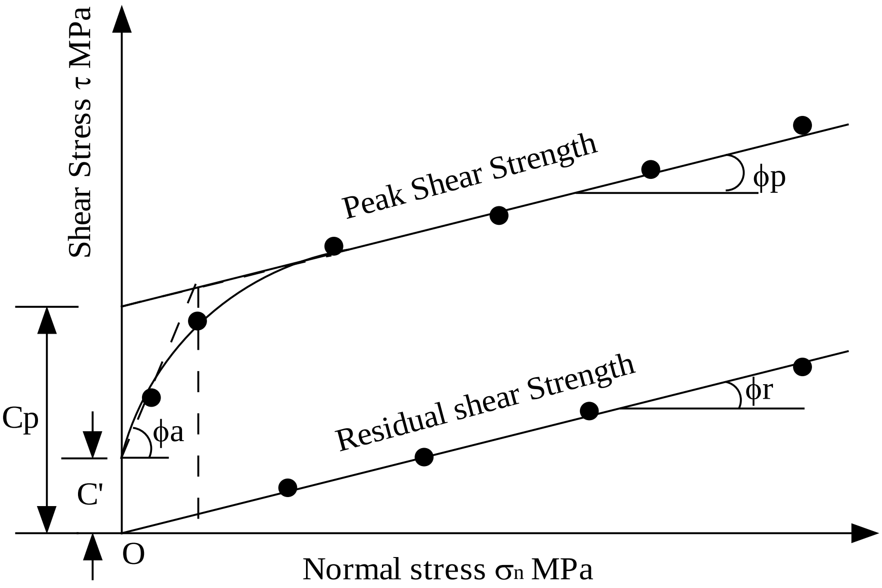


Figure 2.21: Shear vs shear strain plot with increasing normal load



Where, ϕr = residual friction angle, ϕp = peak friction angle
ϕa = apparent friction angle, C' = apparent cohesion
Cp= peak cohesion

Figure 2.22: Shear strength vs. normal stress plot (IS 12634, 1989)

2.5 ROCK STRENGTH AND FAILURE MODES

The failure mode is very significant to decide upon true strength of rocks. Usually, hard brittle rocks fails in longitudinal splitting gives the maximum strength. Rock samples also fail in simple shear or multiple shear which gives relatively lower strength compare to longitudinal splitting. The stress-strain curves for brittle rock material under uni-axial compression could be divided into four phases namely crack closure, linear elasticity, stable crack growth and unstable crack growth. Consequently, the rock fails with fractures developed from the coalescence of several micro cracks. As failure modes of rocks could provide useful information, the examination of failed specimens would be very helpful in design. The relative predominance of the two failure modes depends on the strength, anisotropy, brittleness and grain size of the crystalline aggregates. Common modes of failure in rock sample under compression are shown in Figure 2.23 (Szwedzicki, 2007).

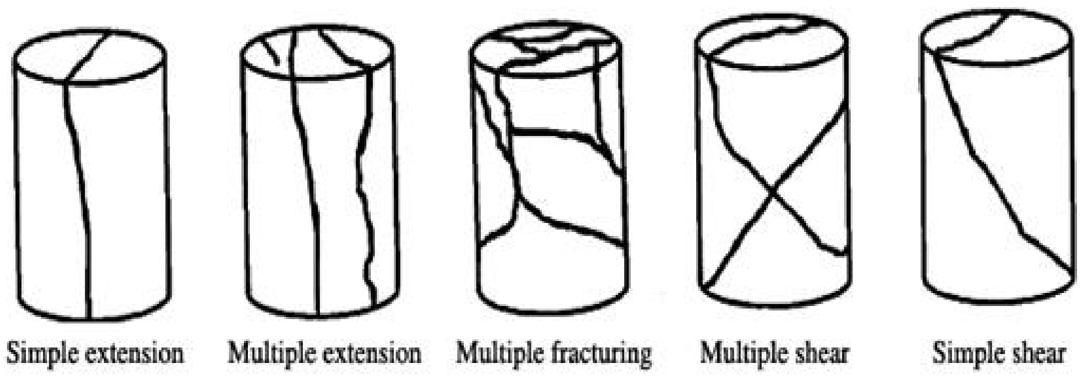


Figure 2.23: Common modes of failure in rock sample under compression (Szwedzicki, 2007)

The failure mode of a brittle rock changes on the application of confining pressure because usually under unconfined compression a rock tends to deform elastically until failure occurs abruptly (Figure 2.24a). With moderate amount of confining pressure, longitudinal fracturing is suppressed and failure occurs along a clearly defined plane of fracture (Figure 2.24b). At very high confining pressure rock becomes fully ductile (Figure 2.24c).

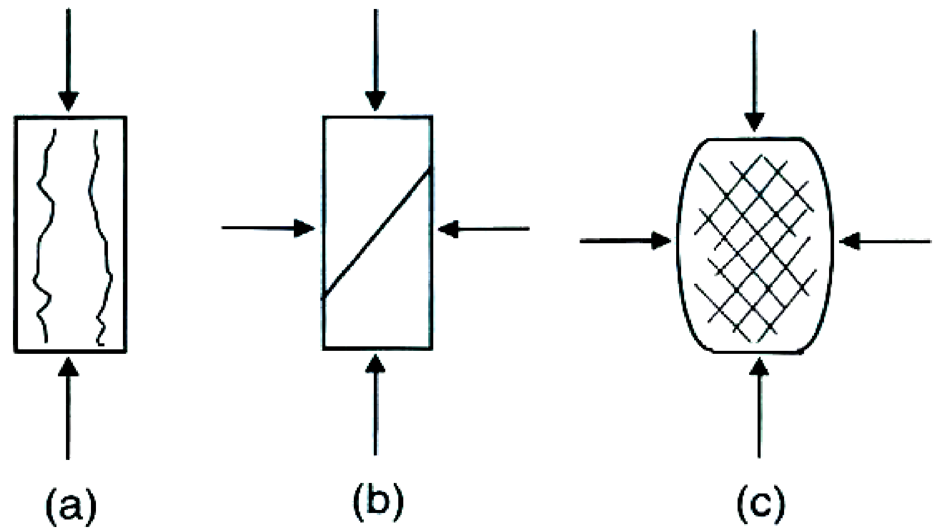


Figure 2.24: Effect of Confining pressure on the failure modes of rock samples Jaeger, Cook and Zimmerman (2007)