FAILURE CRITERIA OF ROCK
AND ROCK MASSES

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5.1 FAILURE IN ROCKS

The deformation of rock happens because of the stresses from various directions, and can be divided into two parts, deviatoric and non-deviatoric. Non-deviatoric stresses ($\sigma_{\text{mean}}$) are compressions equally applied in all directions, that is, a hydrostatic state of stress. Deviatoric stresses ($\sigma_{\text{dev}}$) are the normal and shear stresses that remain after subtracting a hydrostatic stress, equal to the mean normal stress, from each normal stress component. The non-deviatoric stress is given by $\frac{1}{3}(\sigma_1+2p)$ all around while the deviatoric stress is then what remains: $\sigma_{1,\text{dev}} = \frac{2}{3}(\sigma_1-p)$ and $\sigma_{2,\text{dev}} = \sigma_{3,\text{dev}} = -\frac{1}{3}(\sigma_1-p)$. This deviatoric stress produces distortion and destruction of rocks while non-deviatoric stresses generally do not. In the tri-axial test, the initial pressuring is non-deviatoric, subsequently; both deviatoric and non-deviatoric stresses are raised simultaneously. Normal strains in a tri-axial compression specimen can be measured with surface-bonded electric resistance strain gages. A gage parallel to the specimen axis records the longitudinal strain $\varepsilon_{\text{axial}} = \Delta l/l$, while a strain gage affixed to the rock surface in the circumferential direction yields the lateral strain $\varepsilon_{\text{lateral}} = \Delta d/d$, where ‘d’ is the diameter of the rock and ‘l’ is the length. Assuming that the strain gage readings are zeroed after the confining pressure has been applied, we can write

$$\varepsilon_{\text{lateral}} = -\nu \varepsilon_{\text{axial}} \quad (5.1)$$

In which the constant of proportionality $\nu$ is called Poisson’s ratio. In fact, proportionality is maintained only in the restricted range of loading during which there is no initiation and growth of cracks. For linearly elastic and isotropic rocks, $\nu$ must lie in the range 0 to 0.5 and is often assumed equal to 0.25. Because a rock expands lateral as it shortens axially (Figure 5.1), a negative sign is introduced to define Poisson’s ratio as a positive quantity. For strains of very less value, volume change per unit of volume, $\Delta V/V$, is closely approximated by the algebraic sum of the three normal strains. In the tri-axial compression experiment then,
Volumetric strain produced either by deviatoric or non-deviatoric stresses can be measured indirectly using surface strain ages and applying equation (1) or directly monitoring the flow of oil into or out of the confining vessel as the confining pressure is held constant by a servomechanism.

![Deformations of a cylindrical rock specimen during a compression test](image)

### Figure 5.1 Deformations of a cylindrical rock specimen during a compression test

#### 5.1.1 Hydrostatic compression

Applying a non-deviatoric stress to a rock produces a volume decrease and eventually changes the rock fabric permanently, as pores are crushed. However, it cannot produce a peak load response, that is, the rock can always accept an added increment of load, apparently for as high a pressure as one can generate. Tests have been conducted into the mega bar region (millions of psi) producing phase changes in the solid. The pressure volumetric strain curve is generally concave upward as shown in Figure 5.2, with four distinct regions. In the first, which may be the principal region for many good rocks in civil engineering service, pre-existing fissures are closed and the minerals are slightly compressed. When the load is removed, most of the fissures remain closed and there is a net deformation or “pre-def.” The fissure porosity is related to the pre-def.
After most of the fissures have closed, further compression produces bulk rock compression, consisting of pores deformation and grain compression at an approximately linear rate. The slope of the pressure-volumetric strain curve in this region is called the bulk modulus, $K$. In the porous rocks like sandstone, chalk, and elastic limestone, the pores begin to collapse due to stress concentrations around them: in well-cemented rocks, this may not occur until reaching a pressure of the order of 1 Kbar (100 MPa or 14,500 psi), but in poorly or weakly cemented rocks, pore crushing can occur at much lower pressures. Finally, when all the pores have been closed, the only compressible elements remaining are the grains themselves and the bulk modulus becomes progressively higher. Nonporous rocks do not demonstrate pore “crush up” but show uniformly concave-upward deformation curves to 300 Kbar or higher. Pore crushing is destructive in very porous rocks like chalk and pumice, which are converted to cohesion-less sediments on removal from the test chamber.

![Figure 5.2 Behaviour of rock sample under hydrostatic compression](image)

**Figure 5.2 Behaviour of rock sample under hydrostatic compression**

### 5.1.2 Deviatoric compression

Applying deviatoric stress produces different results as shown in Figure 5.2. With initial application of the deviatoric stress, fissures and some pores begin to close, producing an inelastic, concave-upward stress-strain section. In most rocks, this is followed by linear relationships between axial stress and axial strain and between axial stress and lateral strain. After that, in the stage III, the rate of lateral strain begins to increase relative to the rate of axial strain (Poisson’s ratio increases) as new cracks begin to form inside the most critically stressed portions of the specimen, usually near the sides of the mid-section. In the stage IV, cracks that form propagate to the degree of the specimen and a system of interesting
coalescing cracks is developed, which eventually form a semi-continuous rupture surface termed as a “fault.” Bieniawski (1967) suggested that point C corresponds to the yield point in the axial stress-axial strain curve. The peak load is the usual object of the failure criteria. However, the rock may not fail when the load reaches this point. In a stiff loading system, it is possible to continue to shorten the specimen, as long as stress is reduced simultaneously. If the volumetric strain is plotted against the deviatoric stress as shown in Figure 5.2, it is seen that the attainment of the crack initiation stress is marked by a beginning of an increase in volume associated with sliding and buckling of rock silvers between cracks and opening of new cracks. At a stress level corresponding to stress point C, the specimen may have a bulk volume larger than at the start of the test. This increase in volume associated with cracking is termed dilatancy.

![Figure 5.3 Behaviour of rock sample under deviatoric compression](image-url)
5.1.3 Effect of confining pressure

Most rocks are significantly strengthened by confinement. Sliding along the fissures is possible if the rock is free to displace normal to the average surface of rupture. But under confinement, the normal displacement required to move along such a jagged rupture path requires additional energy input. Thus it is not uncommon for a fissured rock to achieve an increase in strength by 10 times the amount of a small increment in mean stress. This is one reason why rock bolts are so effective in strengthening tunnels in weathered rocks.

As mean pressure is increased, the rapid decline in load carrying capacity after the peak load becomes gradually less striking until, at a value of the mean pressure known as the brittle-to-ductile transition pressure, the rock behaves fully plastically. The brittle-to-ductile transition occurs at pressures far beyond the region of interest in most civil engineering applications. However, in evaporate rocks and soft clay shales, plastic behaviour can be exhibited at engineering service loads.

Without confining pressure, most rocks tested will form one or more fractures parallel to the axis of loading. As the confining pressure is raised, the failed specimen demonstrates faulting, with an inclined surface of rupture traversing the entire specimen. In soft rocks, this may occur even with unconfined specimens. If the specimen is too short, continued deformation past the faulting region will drive the edges of the fault blocks into the testing machine platens, producing complex fracturing in these regions and possibly apparent strain-hardening behaviour. At pressures above the brittle-to-ductile transition, there is no failure, but the deformed specimen is found to contain parallel inclined lines that are the loci of intersection of inclined rupture surfaces and the surfaces of the specimen. The effect of confining pressure is also expressed in changing volumetric strain response as shown for a series of tri-axial compression tests.
Module 5: Failure Criteria of Rock and Rock masses

Figure 5.4 Stress-strain characteristics of typical rocks under increasing confining pressure

Figure 5.5 Hypothetical stress-strain behaviors: a) from intact to highly jointed rock mass and b) with varying confining conditions
5.2 FAILURE MODES IN ROCKS

Rock under natural conditions experiences different stress conditions. Fractures occur in a rock at a certain point when it crosses the threshold stress value. The rock fails with fractures developed from the coalescence of several micro cracks and different failure modes of rock under various stress conditions are possible. They provide useful information for safe and economic design of various structures involving rock. The accurate prediction of failure through better design will significantly reduce the costs involved in construction and increase the safety. It is re-emphasized that 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 tested also observed to be failed 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 examined 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 microcracks. 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](image)

**Figure 5.6 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 5.7a). With moderate amount of confining pressure, longitudinal fracturing is suppressed and failure occurs along a clearly defined plane of fracture (Figure 5.7b). At very high confining pressure rock becomes fully ductile (Figure 5.7c).

Figure 5.7 Effect of Confining pressure on the failure modes of rock samples Jaeger, Cook and Zimmerman (2007)

Figure 5.8 Different failure modes of granulite observed during after uniaxial compression testing
Strength of rock depends on the mode by which it fails. Many failure theories are given to explain the failure mechanism of rock but none of the theories completely captures the brittle fracture behavior of rock. Mohr’s theory is often used in predicting the failure of brittle materials, and is applied to cases of 2D stress. Mohr's theory suggests that failure occurs when Mohr's Circle at a point in the body exceeds the envelope created by the two Mohr's circles for uniaxial tensile strength and uniaxial compression strength. Griffith theory explains the two-dimensional relationship between shear stress and normal stress at the point of failure. The mechanism of failure is based on the formation, propagation, and joining of microscopic ‘Griffith cracks’ whose leading edges concentrate stress. Failure occurs when a critical stress is reached and the cracks propagate fully. Griffith’s failure criterion is used to study the fracture mechanism in the rocks and gives an equation for fracture stress.