CHAPTER TWO

CHARACTERISTICS OF SOFT ROCKS
2.1 Strength

The unidirectional compressive strength of a rock is usually assessed by testing a small sample to destruction. For soft rocks the compressive strength thus found is only an index and is only an indication of the strength of the rock in the mass. The stress required to cause failure is dependent on the size of the sample tested. Numerous studies of this phenomenon have been made (eg by Evans and Pomeroy [11], Holland [12] and Bieniawski [13]) and the general conclusion has been that the compressive strength varies inversely as the linear dimension raised to some power $\alpha$, where $\alpha$ varies from 0.14 to 0.5. Evans and Pomeroy have explained this by a 'weakest link' theory, ie the chance of finding a weakness which will cause premature failure increases with the size of sample considered. The difference between the strength of a small specimen and the strength of the mass in situ is magnified by the original selection of small pieces of unbroken rock and the rejection of cores which break in preparation.

As an example, if $\alpha = \frac{1}{3}$ is taken as the average value representing coal, then in relating unidirectional compressive strength to the size of sample considered we have

$$\frac{c_1}{c_2} = \left( \frac{L_2}{L_1} \right)^{\frac{1}{3}}$$

This is shown graphically in Figure 1.
\[ \frac{\sigma_1}{\sigma_2} = \left( \frac{L_2}{L_1} \right)^{\frac{1}{3}} \]

**Figure 1. Effect of Specimen Size on Specimen Strength**

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The mass of rock around an excavation has a dimension measurable in metres; the sample tested in the laboratory is measurable in centimetres. It is likely that the ratio \( \frac{L_2}{L_1} \) will be of the order of 100 or more and the strength of the coal in situ may only be about one-fifth of the strength determined from a small sample in the laboratory. This is further discussed in Section 3.5(b).

2.2 Triaxial Properties

When rock is confined, its strength rises dramatically. It is usual in civil engineering and in soil mechanics to represent this by the Mohr diagram shown in Figure 2a, in which \( \tau \) is the shear strength, \( \sigma \) the stress normal to the shear plane, \( \phi \) the angle of internal friction and \( a, b, A, B \) the principal stresses (i.e., the confining pressure and compressive strength) associated with the points, say, \( P_1 \) and \( P_2 \) respectively. This type of presentation is particularly suited to shear box experiments where \( \tau \) and \( \sigma \) are the measured quantities, but is not so convenient when the compressive strength and confining pressure are measured directly, as on a circular core in a triaxial cell.

When the latter type of equipment is used, representation as in Figure 2b is much more suitable. Here \( \sigma_1 \) is the compressive strength, \( \sigma_3 \) the confining pressure, and \( (a,b), (A,B) \) the co-ordinates of the points \( P_1 \) and \( P_2 \). The rate of increase of \( \sigma_1 \) with respect to \( \sigma_3 \) is \( \tan \beta \), and, provided the plots of \( \tau \) against \( \sigma \) and \( \sigma_3 \) against \( \sigma_1 \) can be approximated by straight lines, one can easily establish a trigonometrical relationship between \( \phi \) and \( \beta \).
If $P_1$ and $P_2$ are the corresponding points on the two plots, then from Figure 2a

$$\sin \phi = \frac{b-a}{2b+2a + L} \quad \text{and} \quad \sin \bar{\phi} = \frac{B-A}{2B+2A + L}$$

By eliminating $L$ from these two equations, we obtain

$$\sin \phi = \frac{(B-b) - (A-a)}{B-b + (A-a)}$$

and

$$\frac{1 + \sin \phi}{1 - \sin \phi} = \frac{B-b}{A-a} = \tan \bar{\phi} \text{ in Figure 2b.}$$

Hence $\tan \bar{\phi} = \frac{1 + \sin \phi}{1 - \sin \phi}$ is the reciprocal of the Rankine Constant ($R$).

Figure 3 shows the relationship between compressive strength $\sigma_1$ and confining pressure $\sigma_3$ for a typical Coal Measure rock. The graph of the relationship is usually curved, but, if it can be approximated by a straight line over the range considered, this line will have the equation

$$\sigma_1 = \sigma_0 + k\sigma_3,$$

where $\sigma_0$ is the unconfined compressive strength and $k$ is the slope of the line, $\tan \bar{\phi}$. For future convenience, $k = \tan \bar{\phi} = \frac{1 + \sin \phi}{1 - \sin \phi} = \frac{1}{R}$ is referred to as the 'triaxial stress factor'.

After failure, the rock will be in a fractured condition and $\sigma_0$ will be small. From the work of Hobbs [14] it can be deduced that even after failure, if the movement occurs by sliding along fracture surfaces, the change in the triaxial stress factor $k$ is small.

For the purposes of approximation, $k$ can be considered unchanged. Hence the condition of the rock in the yield zone is represented by the equation
a. Shear strength plotted against normal stress

b. Compressive strength plotted against confining pressure

FIGURE 2. METHOD OF REPRESENTING THE TRIAXIAL PROPERTIES
FIGURE 3. RELATIONSHIP BETWEEN CONFINING PRESSURE AND STRESS AT FAILURE (SILTY MUDSTONE)
\[ \sigma_1 = k \sigma_3, \]

where \( \sigma_1 \) is the maximum principal stress and \( \sigma_3 \) the minimum principal stress. Hence the stress required to cause movement at any point in the yield zone will be \( k \) times the confining pressure at that point. The rock in the yield zone will 'flow' until this condition has been established.

Numerous tests have been carried out on Coal Measure rocks [14] [15] [16] and, as a generality,

- \( k \) for weak mudstone (\( \phi = 20^\circ \) to \( 30^\circ \)) = 2 or 3
- \( k \) for coal and the average coal measure rock (\( \phi = 37^\circ \)) = 4
- \( k \) for sandstone with angular grains (\( \phi = 40^\circ \) to \( 50^\circ \)) = 5 or 6.

As stated, when sliding occurs along broken surfaces there is little change in the value of \( k \). However, subsequent tests [17] [18] have shown that, if the rock is in granular form, there can be a significant reduction in the values of \( k \). All Coal Measure strata tend to have a value of \( k \) of 2 or 3. This phenomenon is still under investigation, but for the present purpose it has been assumed that the harder rocks in the yield zone (sandstone and coal) move along broken surfaces; whether or not very soft rocks (weak mudstone) reach the granular state is academic, there being little change in their \( k \) value.

Because of the curvature of the relationship between stress at failure and confining pressure, the average value of the triaxial stress factor will depend on the range of the confining pressure considered. In strata control the confining pressure can vary from zero to multiples of the overburden load. The average values for \( k \) quoted above refer to a range 0 to 20 MPa.
but, if working in a lower range, say in studying the fracture of rocks by picks, the average value of k will be substantially greater.

The triaxial stress factor is possibly one of the most important parameters in assessing the behaviour of soft rocks around an excavation and it is felt that much more attention should be paid to it.

2.3 Virgin Stress Conditions

It is usually accepted that vertical stress in the unmined or 'virgin' condition is that arising directly from the overburden, ie it can be obtained by multiplying the depth H by the average 'density', or body force which is of the order of 0.025 MN/m$^3$ [19]. Hence:

\[ \sigma_v = \gamma H = 0.025 \times 10^6 \text{ MPa (H in metres)} \]

The value of the horizontal stress in the 'virgin' condition is open to more variation. Actual measurements taken in various parts of the world, mainly in the harder rocks, are summarised in Figure 4 [20], and show that

Horizontal stress \( \sigma_h = 0.5 \text{ to } 4.0 \times \sigma_v \)

The actual cause of this is not definitely known, but a tentative explanation can be put forward. When the rocks were consolidated at great depth, a stress condition \( \sigma_h = \sigma_v \) may have been locked into the rocks. As the rocks were brought nearer to the surface, by uplift and by subsequent erosion, relief in the vertical direction occurred till failure took place in the Rankine 'passive' condition,

\[ \frac{\sigma_h}{\sigma_v} = \frac{1 + \sin \phi}{1 - \sin \phi} = k = 3 \text{ or } 4, \]

this ratio being more prevalent at the shallower depths.
FIGURE 4. VARIATION OF VIRGIN VERTICAL AND HORIZONTAL STRESSES WITH DEPTH (BASED ON OVER 100 RESULTS)
This consideration applies to the harder rocks. In the relatively soft rocks of the Coal Measures it is probable that creep over geological time (a time span measurable in millions of years) will have caused equalisation of horizontal and vertical pressures. It will therefore be accepted that, as a working hypothesis,

\[ \sigma_v = \sigma_h \]

regardless of depth, unless in a given situation there is evidence to the contrary.