# CHAPTER TWO

# LITERATURE STUDY ON THE SHEAR STRENGTH OF JOINTS IN ROCK

## 2.1 Introduction

Discontinuities such as bedding planes, joints, shear zones and faults form part of any rock mass. The behaviour of the rock mass at shallow depth, where stresses are relatively low, is largely controlled by sliding of blocks of rock on joints (discontinuities).

There are a number of factors that influence the shear strength of joints in rock and are mainly concerned with the character of the joint surface. These factors include: the roughness of the joint surface, hardness of the joint surface, the presence and pressure of water, the presence of fill material, type of fill material and the thickness of fill material.

## 2.2 Discontinuities in rock

Discontinuities occurring in any rock mass are the result of the formation of the rock mass or movement in the crust of the earth. According to Jennings (1971) two sets of discontinuities are recognized namely (i) major or through going features and (ii) minor or secondary features. Major features include bedding planes, faults, contacts and dykes, all of which can be traced over long distances. These joints are important in the analysis of stability of slopes. Minor features are of limited length such as cross joint in sedimentary rocks

**Types of joints**

Sedimentary rock normally has **bedding planes** as a major discontinuity. The origin of these joints are from the deposition of mechanical or chemical sediments.

**Stress relief joints** form as a result of erosion of weathered rock and soil materials. **Tension joints** are the result of cooling and crystallization of igneous rock.

**Shear joints** is the result of faulting and shear in the rock mass as result tectonic movement.

**Characteristics of joints**

The **orientation** of a joint plane in relation to other joint planes and the direction of disturbing force will determine if parts of a rock mass are free to slide.

Joint **spacing** is a measure of the closeness of joints in a specific set and will effect the shear strength of a rock mass.

**Hardness** is determined by the **wall rock type** or degree of alteration of the joint wall Weathered or filled joints will have lower shear strength than unweathered hard joint surfaces.

**Waviness** is a contributor to higher shear strength over larger areas such as dam foundations.

**Roughness** can be described in terms of (i) asperities (small-scale roughness) (ii) large protrusions (intermediate roughness) and (iii) waviness or undulations (large-scale roughness).

**Filling** of joints with alteration products can have a negative effect on the shear strength of joints.

The **water condition** on the joint surface has a influence on the mechanical behaviour or the joint surface. The presence of water and water under pressure normally reduces shear strength.

## 2.3 The principles of shear

The principle of shear and shear strength is described as follows by Cutnell and Johnson (2001): Experiments have shown that when a body is pressed against a surface, and a force **F** attempts to slide the body along the surface, the resulting frictional force has three properties:

a. If the body does not move, then the static frictional force **fs** and the component of **F** that is parallel to the surface are equal in magnitude, and **fs** is directed opposite that component of **F.**

b. The magnitude of **fs** has a maximum value of **fs max** that is given by:

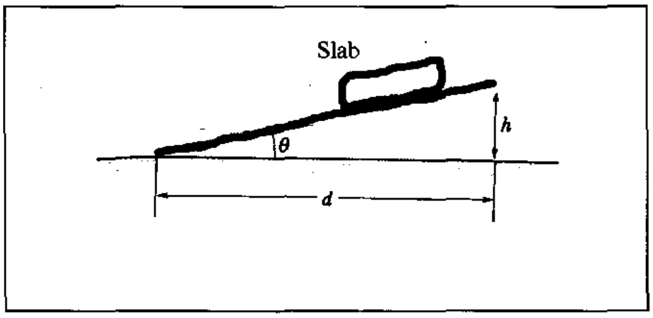
$$\mathbf{fs, max} = \mu_s N \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.1)$$

c. where μs is the coefficient of static friction, and N is the magnitude of the normal force. If the magnitude of the component of **F** that is parallel to the surface exceeds **fs, max** then the body begins to slide along the surface. Just before this point is reached, the factor of safety is equal to one.

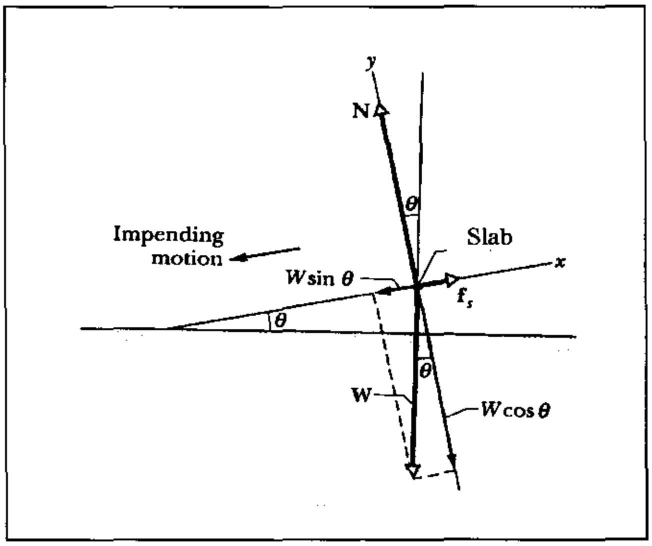
If the body begins to slide along the surface, the magnitude of the frictional force rapidly decreases to a value $\mathbf{f_k}$ given by: $\mathbf{f_k} = \mu_k \mathbf{N}$ ....................................(2.2)

where $\mu_k$ is the coefficient of kinetic friction. Thereafter during sliding, the kinetic frictional force $f_k$ is given by equation 2.2

This principal could be demonstrated by the following example: Figure 2.1 shows a slab of rock resting on an other slab of rock separated by a natural joint tilted at an angle θ with the horizontal. By using a tilt testing apparatus it is found that when θ is increased to 30°, the top slab begins to slide down the joint plane. What is the coefficient of static friction between the bottom and top rock slabs?




**<u>Figure 2.1</u>** **A slab of rock resting on another slab of rock separated by a natural joint tilted at an angle θ with the horizontal. (After Cutnell and Johnson (2001) modified)**




**<u>Figure 2.2</u>** **A free-body diagram of a slab of rock when it is on the verge of sliding. (After Cutnell and Johnson (2001) modified)**

Figure 2.2 is a free-body diagram for the slab when it is on the verge of sliding. The forces on the slab is the normal force N, pushing outward from the plane of the book, the weight W of the slab on top, and the frictional force $F_s$, which points up the plane, because the impending motion is down the plane. Since the slab is in equilibrium, the net force acting on it must be zero. From Newton's second law, we have:

$$\Sigma F = f_s + W + N = 0 \qquad (2.3)$$

For the x components, this vector equation gives us:

$$\Sigma F_x = f_s - W \sin\theta = 0$$

or $$f_s = W \sin\theta \qquad (2.4)$$

For the y components, we have

$$\Sigma F_y = N - W \cos\theta = 0$$

or $$N = W \cos\theta \qquad (2.5)$$

When the slab is on the verge of sliding, the magnitude of the static frictional force acting on it, has its maximum value $\mu_s N$. Substituting this into equation 2.4 and dividing by equation 2.5, we obtain:

$$f_s / N = \mu_s N / N = W \sin\theta / W \cos\theta = \tan\theta$$

or $$\mu_s = \tan\theta \qquad (2.6)$$

The coefficient of static friction for a joint set with a slope of 30° is thus tan 30°.

If a body slides or attempts to slide over a surface, a bonding between the body and the surface resists the motion. The resistance is considered to be a single force called the frictional force or simply **friction**. The force runs parallel to the surface, opposite the direction of the intended motion. The frictional force is a force acting between the surface atoms of one body and those of the other. When two surfaces are placed together, only the high points touch each other. The actual microscopic area of contact is much less than the apparent macroscopic contact area, perhaps by a factor of $10^4$. (after Halliday, D (1993)). Many contact points cold weld together. (If two highly polished and carefully cleaned metal surfaces are brought together in a very good vacuum, they cannot be made to slide over each other, instead they cold-weld together, instantly forming a single piece of metal.) When surfaces move across each other, there is a continuous rupturing and reforming of contact areas or welds. This is illustrated in Figure 2.3

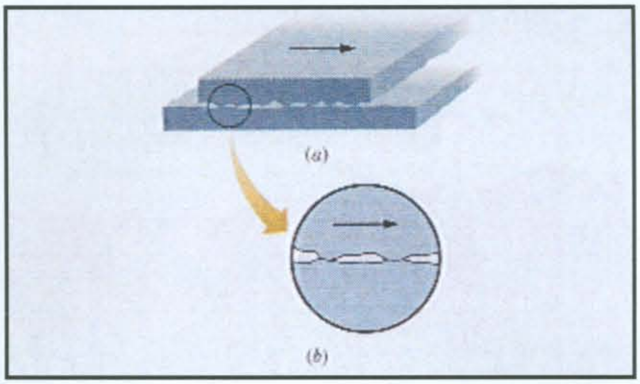



**Figure 2.3** **Continuous rupturing and reforming of contact areas as surfaces move across each other. (After Cutnell and Johnson, 2001)**

## 2.4 Shear strength of planar joint surfaces in rock

A number of samples (at least three) are required for shear testing, ISRM (1974). Each sample contains a through-going plane (discontinuity) that could be used to apply a normal and shear load to, for testing.

The bedding plane is absolutely planar, having no surface irregularities or undulations. As illustrated in Figure 2.4, each specimen is subjected to a stress $\sigma_n$ normal to the bedding plane, and shear stress $\tau$, required to cause a displacement $\delta$.

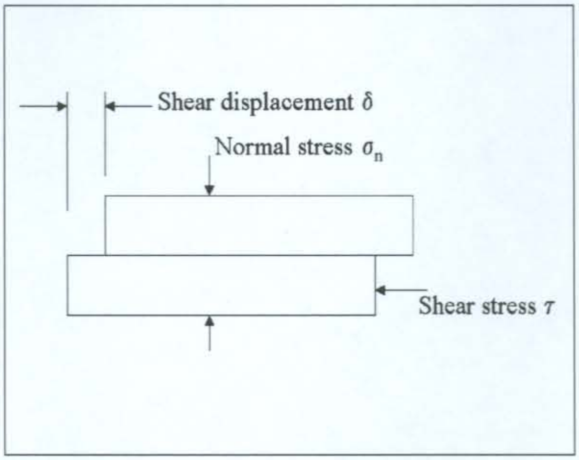



**Figure 2.4** **Schematic representation of displacement and stresses on joint plane.**

The shear stress will increase rapidly until the peak strength is reached. This corresponds to the sum of the strength of the cementing material bonding the two halves of the bedding plane together and the frictional resistance of the matching surfaces. As the displacement continues, the shear stress will fall to some residual value that will then remain constant, even for large shear displacements. This basic test method and theory is described by Hoek, E (2000).

Plotting the peak and residual shear strengths for different normal stresses results in the two lines illustrated in Figure 2.5. For planar discontinuity surfaces the experimental points will generally fall along straight lines. The peak strength line has a slope of $\phi$ and an intercept of c on the shear strength axis. The residual strength line has a slope of $\phi_r$. The relationship between the peak shear strength $\tau_p$ and the normal stress $\sigma_n$ can be represented by the **Mohr-Coulomb** equation:

$$\tau_p = c + \sigma_n \tan \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.7)$$

($\tau_p$ = peak shear strength, $c$ = cohesion intercept, $\phi$ = friction angle of the joint wall (discontinuity wall), $\sigma_n$ = normal stress).

Discontinuities of geological origin that intersect almost all near-surface rock masses are referred to as joints. The most important external factor affecting shear strength is the magnitude of the effective normal stress ($\sigma_n$) acting across the joint. In many rock engineering problems in civil engineering the maximum effective normal stress will lie in the range 0.1 to 2.0 MPa for those joints considered critical for stability (in mining engineering this value can be much bigger). This effective normal stress is about three orders of magnitude lower than those used by tectonophysicists when studying the shear strength faults under stress levels of for example 100 to 2000 MPa. In consequence, the literature contains shear strength data for rock joints spanning a stress range of at least four orders of magnitude. It is partly for this reason that opinions concerning shear strength vary so widely to the results of shear strength investigations on rock joints. If Equation (2.7) is applied to the results of shear tests on rough joints, under both high normal stress and low normal stress, one finds the tectonophysicists recording a cohesion intercept of tens of MPa and a friction angle of perhaps only 20°, while the rock slope engineer finds that he has a friction angle of perhaps 70° and zero cohesion.

Figure 2.5 shows a graph of shear stress vs. shear displacement that illustrates peak and residual shear strength. (Patton, 1966).

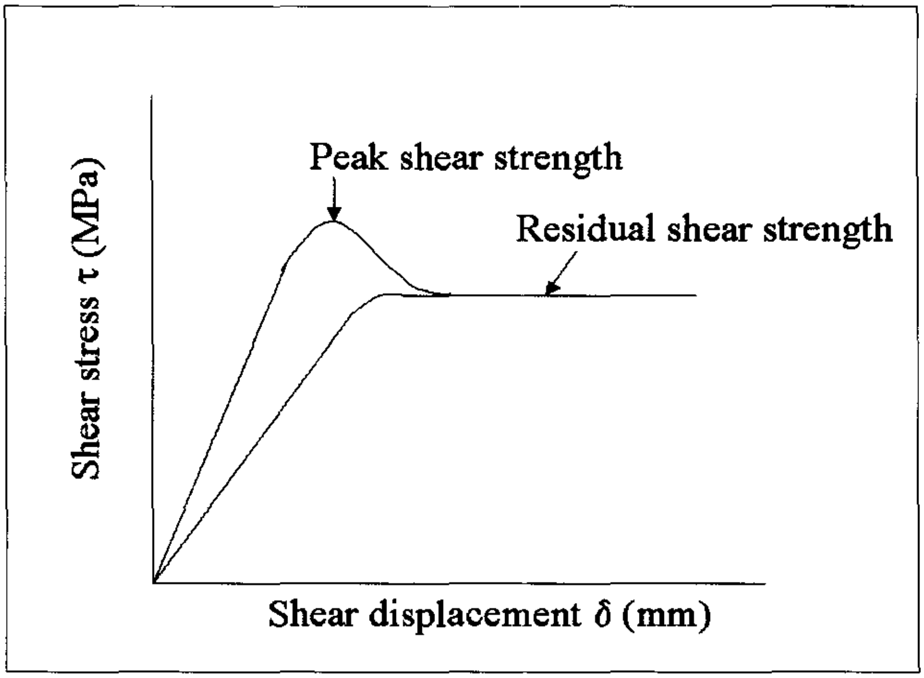



**Figure 2. 5** **Shear stress vs. displacement illustrating peak residual shear strength**

The peak shear strength envelopes for non-planar rock joints are strongly curved. This curved envelope also has the effect that it seems as if there is some cohesion present. This is called the apparent cohesion. (See figure 2.6 where cohesion is indicated by (c)). This fact has been known for many years, however many engineers still describe shear strength as if they were rock properties in terms of Coulomb's constants $\phi$ and c. Both are in fact stress dependent variables. They are also scale dependent.

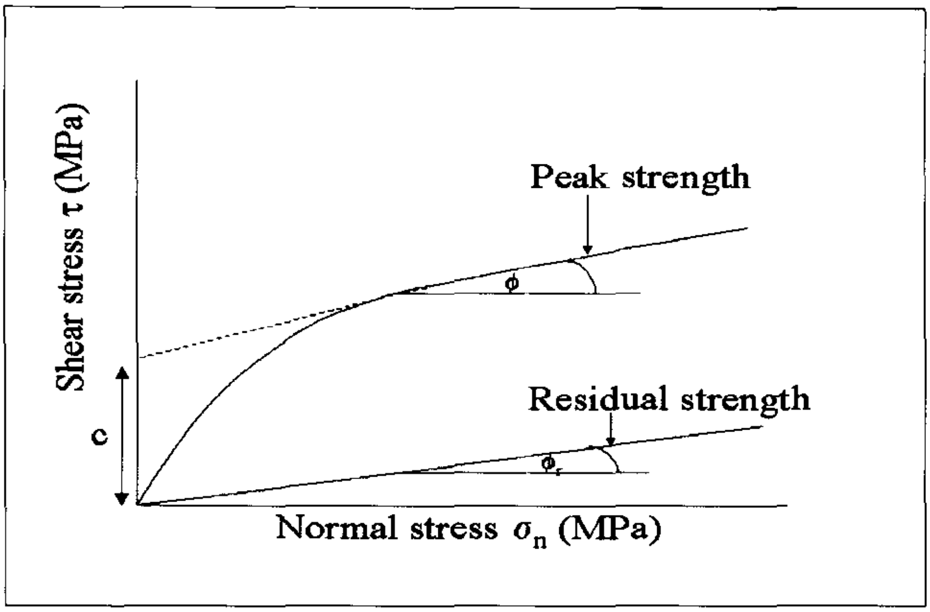



**Figure 2. 6** **Graph of shear strength vs. normal stress illustrating angle of friction and cohesion**

In the case of residual shear strength, the cohesion c is zero and the relationship between $\phi_r$ and $\sigma_n$ is as follows:

$$\tau_r = \sigma_n \tan \phi_r \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.8)$$

where $\phi_r$ = residual friction angle of the joint

In shear tests on soils, the stress levels are generally an order of magnitude lower than those involved in rock testing and the cohesive strength of a soil is a result of the adhesion of the soil particles. In rock mechanics, true cohesion occurs when cemented surfaces are sheared. However, in many practical applications, the term cohesion is used for convenience and it refers to a mathematical quantity related to surface roughness. Cohesion is simply the intercept on the $\tau$ axis at zero normal stress.

The basic friction angle $\phi_b$ is a quantity that is fundamental to the understanding of the shear strength of discontinuity surfaces. This is approximately equal to the residual friction angle $\phi_r$ but it is generally measured by testing sawn or ground rock surfaces. These tests, which can be carried out on surfaces as small as 50 mm diameter, will produce a straight line plot defined by the equation :

$$\tau_r = \sigma_n \tan \phi_b \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.9)$$

where $\phi_b$ = basic friction angle of the joint

The basic friction angle ($\phi_b$) is the friction angle of rock material based on the base strength exhibited by flat unweathered rock surfaces which are prepared by a diamond saw. In some cases these surfaces are sandblasted between tests.

A useful list of basic friction angle values ($\phi_b$) was compiled by Barton and Choubey (1977) from work done by Coulson (1971) and is shown in Table 2.1.

| ROCK TYPE | BASIC FRICTION ANGLE ($\phi_b$) (Degrees) | REFERENCE |
|---|---|---|
| A. Sedimentary Rocks | | |
| Sandstone | 26 - 35 | Patton, 1966 |
| Sandstone | 31 - 33 | Krsmanovic, 1967 |
| Sandstone | 31 - 34 | Coulson, 1972 |
| Siltstone | 31 – 33 | Coulson, 1972 |
| B. Igneous Rocks | | |
| Basalt | 35 - 38 | Coulson, 1972 |
| Granite (Fine) | 31 - 35 | Coulson, 1972 |
| Granite (Coarse) | 31 - 35 | Coulson, 1972 |
| Porphyry | 31 | Barton, 1971 |
| Dolerite | 36 | Richards, 1975 |
| C. Metamorphic Rocks | | |
| Gneiss | 26 - 29 | Coulson, 1972 |
| Slate | 25 – 30 | Barton, 1971 |

**Table 2. 1** **Basic friction angles of various unweathered rocks (Barton and Choubey, 1977 after Coulson, 1971)**

The friction angles obtained are applicable to unweathered joint surfaces and will not be applicable to weathered rock joints unless the level of effective normal stress applied is high enough for the thin layers of weathered rock to be worn away, thereby allowing contact between the fresher underlying rock (Richards, 1975). Under low levels of effective normal stress the thin layers of weathered material, perhaps less than 1 mm in thickness, may continue to control the shear strength past peak strength and even for displacements up to residual strength. Barton and Choubey (1977) also describe the tilt test as a means to determine the base shear strength of joints. The residual tilt test is basically a shear test under very low normal stress. Most specimens slide at a joint surface tilt angle of about 30° that correspond to a normal stress of approximately of 1 to 5 kPa.

## 2.5 Shear strength of rough joint surfaces in rock

A natural discontinuity surface in hard rock is never as smooth as a sawn or ground surface of the type used for determining the basic friction angle. The undulations and asperities on a natural joint surface have a significant influence on its shear behaviour . Generally, this surface roughness increases the shear strength of the surface, and this strength increase is extremely important in terms of the stability of excavations in rock.

Patton (1966) demonstrated this influence by means of an experiment in which he carried out shear tests on 'saw-tooth' specimens such as the one illustrated in Figure 2.7. Shear displacement in these specimens occurs as a result of the surfaces moving up the inclined faces, causing dilation (an increase in volume) of the specimen.

The shear strength of Patton's saw-tooth specimens can be represented by:

$$\tau = \sigma_n \tan(\phi_b + i) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.10)$$

where $\phi_b$ is the basic friction angle of the surface and $i$ is the angle of the saw-tooth face.

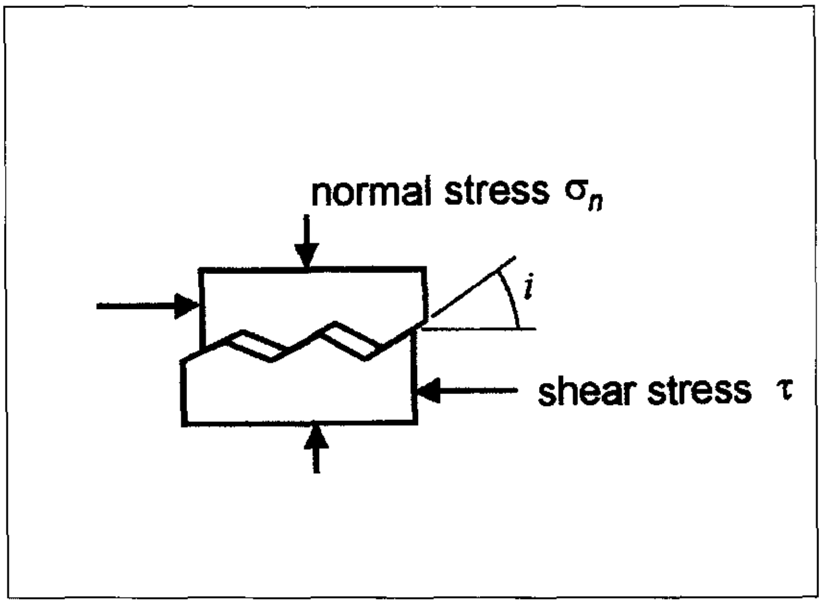



**Figure 2.7** **Saw- tooth asperity roughness by Patton (1966)**

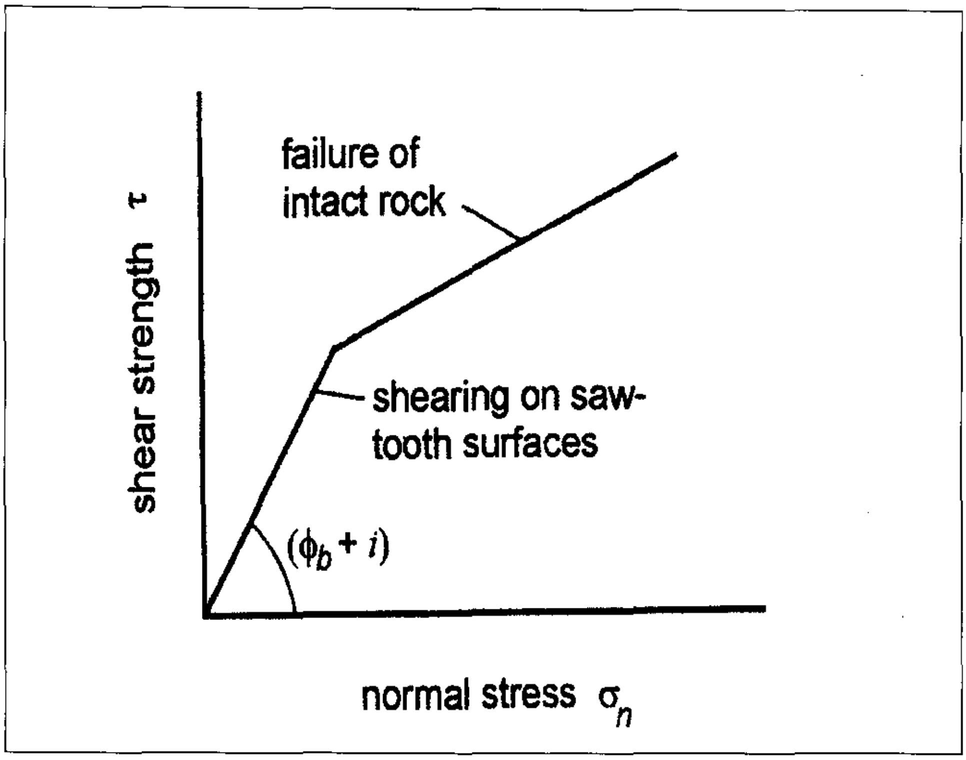
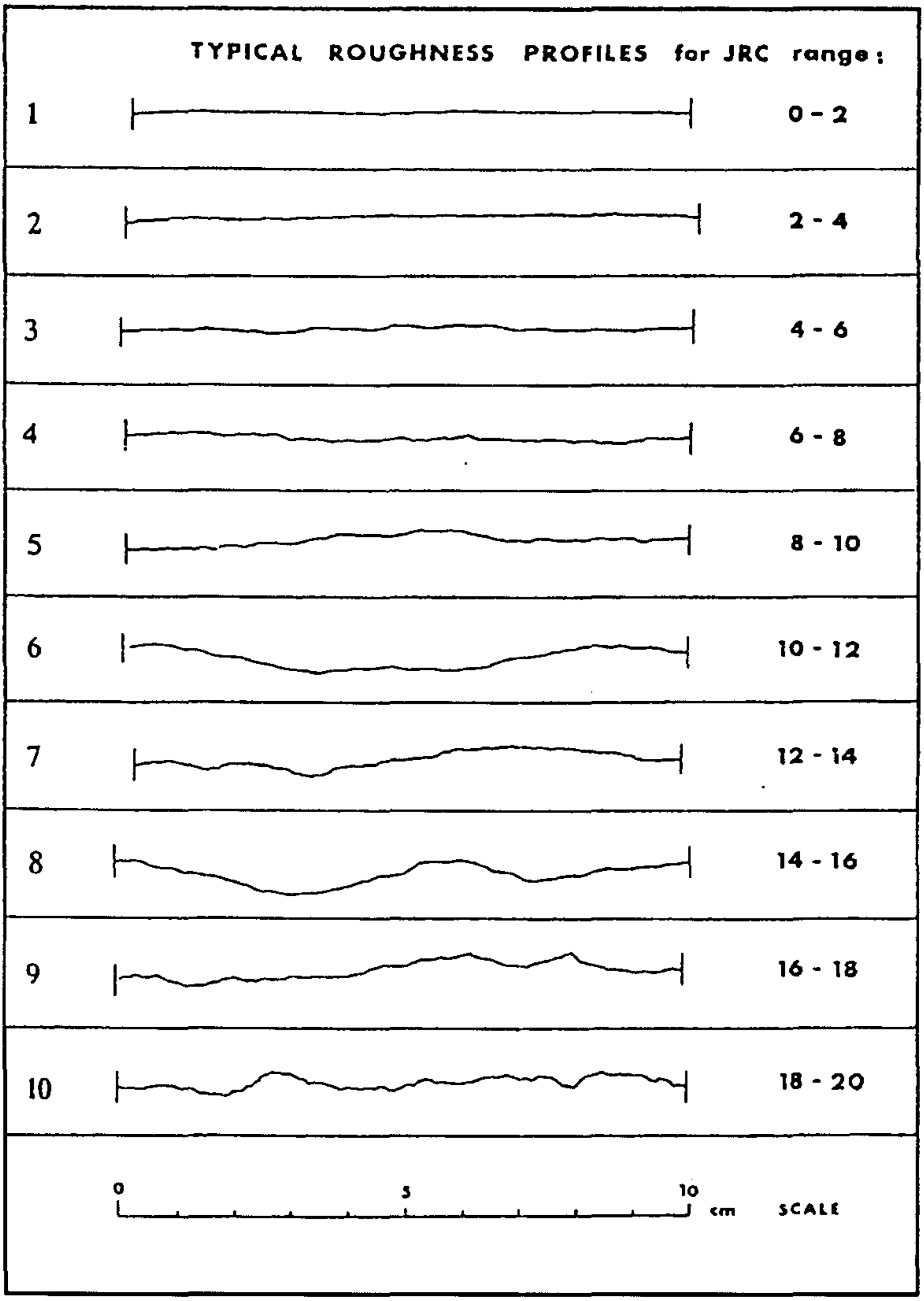



**Figure 2.8** **Shear strength envelope by Patton (1966)**

Many researchers have studied the roughness as important parameter in the determination of shear strength. Initially researchers [Barton and co-workers (1971a, 1971b, 1972, 1974, 1976, 1977, and 1983, Maksimovic (1996), Zhao (1997a and 1997b), Fox et al (1998), Kulatilake (1999), Yong (2000), Lee et al (2001) and Seidel (2002)] studied two dimensional roughness.

Most of these authors studied a two dimensional profile and defined joint roughness. The joint roughness coefficient (JRC) is a number that can be estimated by comparing the appearance of a discontinuity surface with standard profiles published by Barton and others. One of the most useful of these profile sets was published by Barton and Choubey (1977) and is reproduced in Figure 4.2.




(vertical scale = horizontal scale)

**Figure 2. 9** Typical roughness profiles (After Barton and Choubey, 1977)

The surface topography of joints vary widely in any given rock mass according to Bandis, (1993). Individual features can be classified as (i) asperities (small-scale roughness) (ii) large protrusions (intermediate roughness) and (iii) undulations (large-scale roughness). The effects of these broad classes of roughness on shear strength are related to the length of joint under consideration. Asperities with a base length of 1-2 mm will influence the strength of a 10 cm long joint, but will have no effect on a 100 cm long joint. Hence, a distinction is more meaningful if expressed with reference to the joint size. For example, the following base length to joint length percent ratios could be suggested: $< 0.5\%$ for small, 0.5-2% for intermediate and $>2\%$ for large-scale roughness. Many small-scale discontinuities that fit exactly give not unimportant effects.

Roughness measurements can be made by continuous profiling, recordings of peaks and recessions with respect to the mean plane at prescribed intervals, or field measurements of inclinations along selected profile lengths. The instruments used may vary from LVDTs, wire gauges, compass and base plates or any other practical devices to applications of photographic and photogrammetric methods [Maertz (1990)].

Several methods according to Barton and Choubey (1977) are used for the quantitative analysis of roughness, including estimation of:

(i) maximum and median angle $i = \arctan(2a/L)$, where $a$ = amplitude and $L$ = base length of irregularity
(ii) arithmetic mean of peaks and recessions with respect of the mean joint plane
(iii) amplitude index equal to the ratio of the sum of projected asperities over the total length of profile

If a joint profile is analysed geometrically by measuring inclinations $i$ and $D$ the relationship between them will be found as shown in Figure 2.3 (a) and 2.3 (b), also known as Rengers envelope (Rengers, 1971and Fecker and Rengers, 1971)

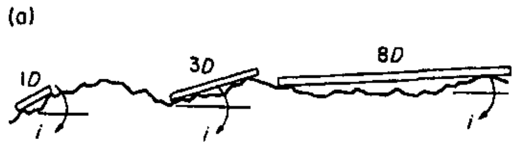



**Figure 2.10 (a) Geometrical scale effects in joint roughness (By Bandis, 1993 after Fecker and Rengers, 1971)**

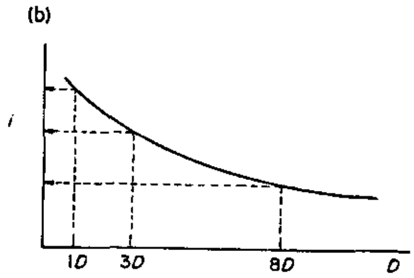



**Figure 2.10(b) Inclination (i) vs. measured distance (D) (By Bandis, 1993 after Fecker and Rengers, 1971)**

In the first experimental demonstration of the surface geometry scale effect, a tilt-tested long joint was found to slide at a smaller inclination angle than smaller samples sectioned from the same surface (Barton and Choubey, 1977). Recent studies of roughness scale effect and fractal dimension also indicate an apparent increase in roughness with decreasing base length (Maertz and Franklin, 1990). The roughness profiles of long joints apparently have been shown not to be fractal objects, confirming a roughness scale effect.

An immediate practical implication of the above concerns the choice of the 'correct' *i* value to be used in equation (Barton, 1974), for cases of shear-overriding behaviour. However, further complications arise when asperity failures are also involved, as will soon be discussed.

No established method is available for recognizing the correct sampling steps for $i$ determinations. A safe assumption may be that large-scale waviness will provide the component $i$ (see Figure 2.6(a). That may be true in some cases of large-scale shear failures or when surface weathering effects smooth the smaller scale features. On the other hand rock masses at or near the surface the blocks resting on a shear plane will probably possess the freedom to move more or less independently thus maintaining contact with all scales of roughness. The freedom for block movement will obviously depend on the stiffness of the rock mass and generally increase as the block size or joint spacing decrease.

Barton and Bandis (1991) confirmed the latter through experimental studies on multiply jointed block assemblies. One conclusion from those studies was that the geometrical scale effect on roughness would probably be limited to the joint lengths corresponding to the average block size as specified by the spacing of the cross-joints. The latter may be envisaged as lines or potential 'hinges' (albeit stiff ones) in the rock mass, above and below a shear plane, which hinder continued scale effects when multiple rock masses are considered. Hence, the average cross-joint spacing may in some cases be an optimum.

Because dams are large structures, individual features like asperities (small scale roughness), large protrusions (intermediate roughness) and undulations (large scale roughness) all contribute to the roughness of discontinuities in the foundation. The effects of these broad classes of roughness on shear strength are related to the length of joint under consideration.

Roughness of joint surfaces is one of the most important factors that determine the shear strength of such surfaces. Barton and Choubey developed the joint roughness coefficient (JRC) as measure of joint roughness.

For many rock-engineering projects, it is necessary to have a good indication of the shear strength of the joints required for design purposes. A method was developed for estimating the JRC by measuring the roughness. Table 2.2 and Figure 2.9 gives a description of the 10 surfaces. (Barton and Choubey, 1977). The descriptions of roughness, i.e. "undulating" and " planar" refer to small and intermediate scale features, respectively.

| Sample | Rock type | Description of joint | JRC |
|---|---|---|---|
| 1 | Slate | smooth, planar: cleavage joints, iron stained | 0.4 |
| 2 | Aplite | smooth, planar: tectonic joints, unweathered | 2.8 |
| 3 | Gneiss (muscovite) | undulating, planar: foliation joints unweathered | 5.8 |
| 4 | Granite rough | planar: tectonic joints, slightly weathered | 6.7 |
| 5 | Granite rough | planar: tectonic joints, slightly weathered | 9.5 |
| 6 | Hornfels (nodular) | rough, undulating: bedding joints, calcite coatings | 10.8 |
| 7 | Aplite | rough, undulating: tectonic joints, slightly weathered | 12.8 |
| 8 | Aplite | rough, undulating: relief joints, partly oxidized | 14.5 |
| 9 | Hornfels (nodular) | rough, irregular: bedding joints, calcite coatings | 16.7 |
| 10 | Soapstone | rough, irregular: artificial tension fractures, fresh surfaces | 18.7 |

**Table 2.2 Descriptive classification of Rock Joints (After Barton and Choubey, 1977)**

The crude estimates of JRC (5, 10 and 20) given by Barton (1971) were proposed as a preliminary guide for those unable to investigate the parameter JRC more closely. Ideally, three profiles are measured on each specimen and the JRC values are grouped in the following ranges 0-2, 2-4 etc. up to 18-20. An attempt is then made to select the most typical profiles of each group. In all cases where the mean joint plane is not within ± 1° of horizontal when placed in the shear box, the shear strengths and corresponding JRC values must be corrected to the horizontal plane.

The appearance of the discontinuity surface is compared visually with the profiles shown and the JRC value corresponding to the profile which most closely matches that of the discontinuity surface is chosen. In the case of small scale laboratory specimens, the scale of the surface roughness will be approximately the same as that of the profiles illustrated. However, in the field the length of the surface of interest may be several metres or even tens of metres and the JRC value must be estimated for the full scale surface.

Recently researchers [Roko et al (1997), Scavia and Re (1999), Geertsema (2000), Gentier et al (2000), Grasselli (2002) and Duzgun et al (2002)] began to study three dimensional characterization of joint surfaces. They attempted to quantify the joint surface and find some relation to the shear strength. Further work in this field has the potential to deliver interesting results.

Another factor contributing to the shear strength over larger areas such as dam foundations is waviness of joint surfaces. Hack et. al. (2002) developed large-scale roughness profiles for slope stability probability classification. This is presented in Figure 2.11. These roughness profiles can be used in the determination of the contribution to friction angle of the waviness

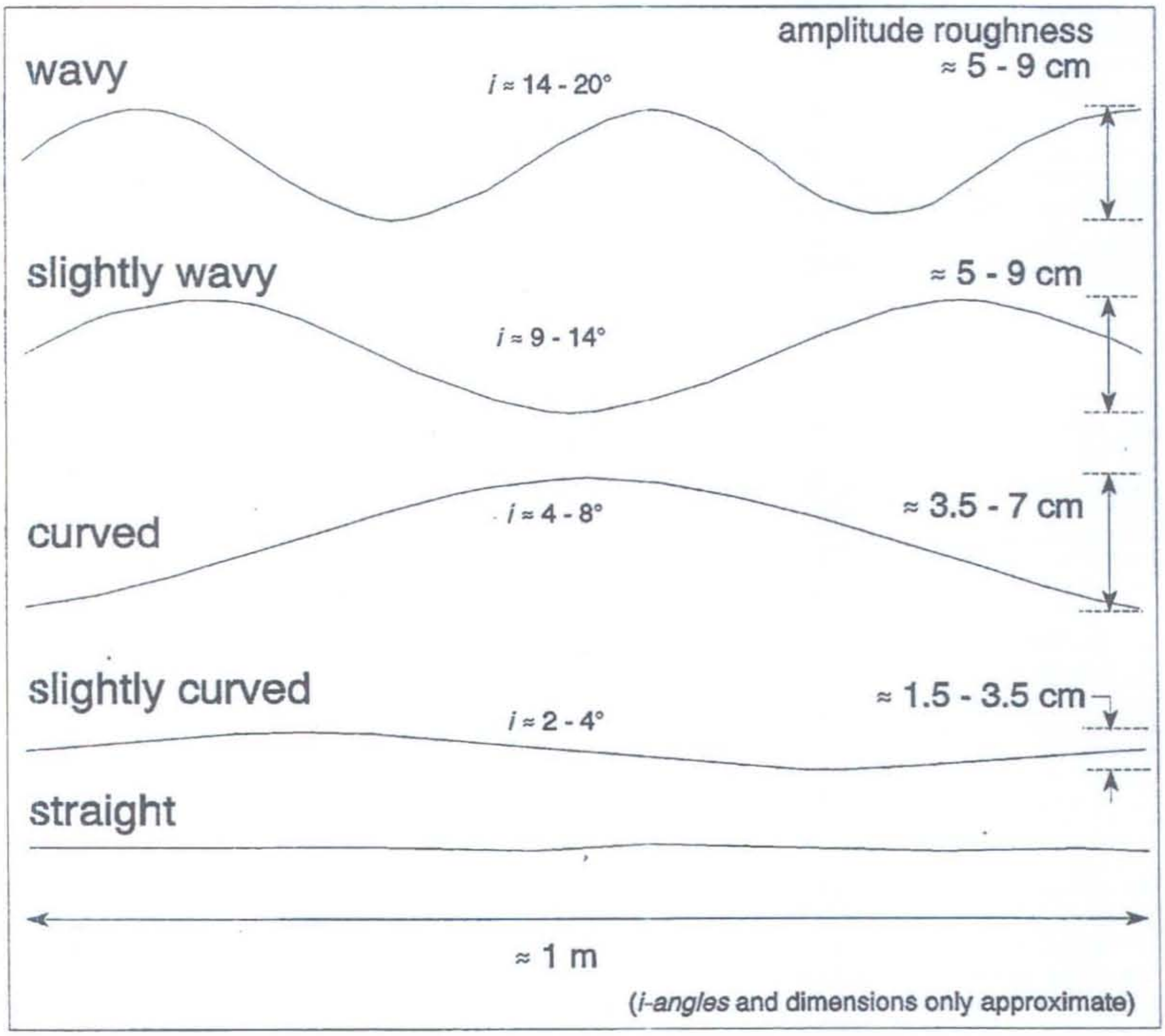



**Figure 2.11 Large-scale roughness profiles used for the slope stability probability classification – After Hack et. al. (2002)**

**2.6 Determination of joint wall hardness.**

Researchers such as Barton and Choubey (1977), Szwedzicki (1998), Katz et al (2000) and van Loon (2003) have studied joint wall hardness.

Barton and Choubey (1977) studied hardness of joint surfaces and stated that the measurement of this parameter is of fundamental importance in rock engineering since it is largely the wall characteristics that control the strength and deformation properties of the rock joints. They describe hardness of joint surfaces as joint wall compressive strength (JCS). The importance of the parameter is accentuated if the joint walls are weathered, since then the *JCS* value may be only a small fraction of the uniaxial compressive strength of the rock material associated with the majority of the rock mass, as typically sampled by borehole core. The depth of penetration of weathering into joint walls depends on the rock type, in particular on its permeability. A permeable rock will tend to be weakened throughout; while impermeable rocks will just develop weakened joint walls leaving relatively unweathered rock in the interior of each block.

Barton and Choubey (1977) propose that the weathering process of a rock mass can be summarized in the following simplified stages:

(i) the formation of the joint is intact in unweathered rock and the JCS value is the same as the uniaxial compressive strength, $\sigma_c$

(ii) slow reduction of joint wall strength occur if joints are water-conducting and the *JCS* becomes less than $\sigma_c$

(iii) common intermediate stage occur with weathered, water conducting joints and impermeable rock blocks between and the JCS becomes some fraction of $\sigma_c$.

(iv) penetration of joint weathering effect into rock blocks with progressive reduction of $\sigma_c$ from the walls of the blocks inwards and the *JCS* continues to reduce slowly.

(v) advanced stage of weathering occur and more uniformly reduced $\sigma_c$ drops to the same level as the *JCS* with the rock mass permeable throughout.

The *JCS* values corresponding to stages (i) and (v) can be obtained by conventional unconfined compression tests on intact cylinders or from point load tests on rock core or irregular lumps though there might be sampling problems in the case of stage (v). Point load testing has been described in detail by Broch and Franklin (1972). In view of the fact that point load tests can be performed on core discs down to a few centimetres in thickness, it might also be possible to use this test for stage (iv) on the core pieces on each side of deeply weathered joints. However, the *JCS* values relevant to stages (ii) and (iii) cannot be evaluated by these standard rock mechanics tests. The thickness of material controlling shear strength may be as little as a fraction of a millimeter (for planar joints) up to perhaps a few millimeters (for rough, weathered joints) with the limits depending on the ratio $JCS / \sigma_c$ that basically controls the amount of asperity damage for a given joint roughness.

Barton and Choubey (1977) state that the Schmidt hammer provides the ideal solution to determine JCS. The Schmidt hammer is a simple device for recording the rebound of a spring loaded plunger after its impact with a surface. The L-hammer used here (L for light, impact energy = 0.075 mkg) is described by the manufacturers as being "suitable for testing small and impact-sensitive parts of concrete or artificial stone". It is suitable for measuring *JCS* values down to about 20 MPa and up to at least 300 MPa. A wide ranging assessment of the suitability of the Schmidt hammer for use in rock mechanics was done by Miller (1965) as reported by Barton and Choubey (1977). He found a reasonable correlation between the rebound number (range 10 to 60) and the unconfined compressive strength ($\sigma_c$) of the rock. However, a better correlation was obtained when he multiplied the rebound number by the dry density of the rock.

$$\text{Log}_{10}(\sigma_c) = 0.00088\ \phi\ R + 1.01 \quad \ldots\ldots\ldots\ldots\ldots\ldots(2.11)$$

where $(\sigma_c)$ = unconfined compressive strength of surface material (MPa)

$\phi$ = dry unit weight of rock (kN/m³), and

R = rebound number

Example:

$$\text{Log}_{10}(\sigma_c) = 0.00088\ \phi\ R + 1.01$$

$$\text{Log}_{10}(\sigma_c) = 0.00088 * 27 * 48 + 1.01$$

$$\sigma_c = 141 \text{ MPa}$$

Deere and Miller (1966) investigated the relationship between Schmidt hardness and the uniaxial compressive strength of rock. Figure 2.12 presents this relationship. Suppose that a vertical downwards held type L-hammer gave a reading of 48 on a rock with a unit weight of 27 kN/m³, the uniaxial compressive strength $\sigma_c$ is given by the graph as 140 ±50 MPa . Note that the hammer should always be perpendicular to the rock surface.

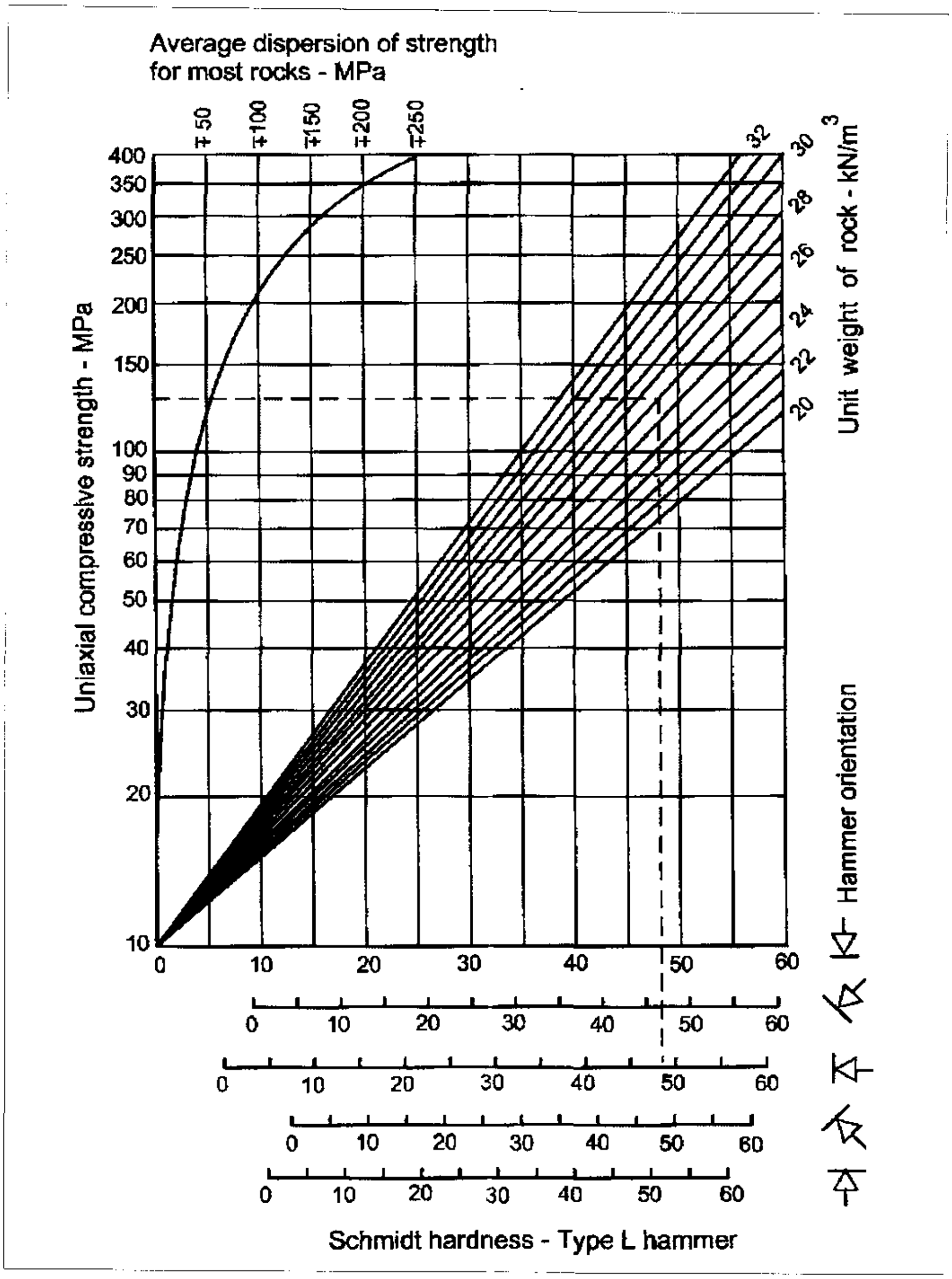



**Figure 2. 12** **The relationship between Schmidt hardness and the uniaxial compressive strength of rock (After Deere and Miller 1966 as reported by Barton and Choubey, 1977)**

## 2.7 Joint matching

Joints in a rock mass are formed at various stages and times, and have been subjected to different methods of alteration which affect the joint surface geometrical and mechanical properties. As a result of these alterations, joints will present different surface profiles on each side and different degrees of matching according to Zhao (1997). It is common for joints to be equally rough but mismatched due to alteration and dislocation. He developed the joint matching coefficient (JMC) based on the percentage of joint surface in contact as an independent joint surface geometrical parameter. The JMC is used with the joint roughness coefficient (JRC) in order to describe the geometrical properties and to assess the hydro mechanical behaviour of joints. It has also been demonstrated that joint matching is an important factor governing the aperture, normal closure, stiffness, shear strength and hydraulic conductivity of joints.

When Barton's JRC-JCS shear strength criterion was used by Zhao (1997) for the interpretation and prediction of shear strength of natural joints, it was found the this model tends to over predict the shear strength for those natural joints with less matched surfaces. To overcome this shortcoming, a new JRC-JMC shear strength criterion is proposed in order to include the effects of both joints surface roughness and joint matching in the form of:

$$\tau = \sigma_n \tan [JRC \ . \ \mathrm{JMC} \ . \ \log_{10} (JCS/\sigma_n) + \phi_r] \qquad (2.12)$$

| | | | |
|---|---|---|---|
| Where $\tau$ | = peak shear strength | $\sigma_n$ | = effective normal stress |
| $JRC$ | = joint roughness coefficient | $JCS$ | = joint wall compressive strength |
| $\phi_r$ | = residual friction angle | JMC | = joint matching coefficient |

The JMC should be set at 0,3 for any measured JMC < 0,3. This is a modification of the existing JRC-JCS criterion.

## 2.8 Infilling of joints

In practice, the assumption that the minimum shear strength of an infilled rock joint is the shear strength of the filler itself is frequently made. De Toledo et al. (1993) tested flat saw-cut and polished surfaces of limestone and basalt in shear boxes using different soils as the infilling material. The ratio of the shear strength of the infilled joint to that of the soil alone varied between 0,61 and 0,95, which means that an infilled joint is normally weaker than the soil that constitutes its filler. The magnitude of the strength reduction seems to be a function of the surface roughness and of the clay minerals present in the soil. Other authors like Amadei, (1990) have also found boundary effects.

When comparing published results in the literature it is important to remember that there are according to de Toledo et. al. (1993), eight basic parameters to be considered in the study of the shear strength of infilled joints. Some parameters relate to the material properties, some to the joint itself and the geological formation to which the joint belongs, and others to the equipment and the particular problem under consideration.

Table 2.3 summarizes the parameters involved in both drained and undrained shear strengths, which should be borne in mind in the testing of infilled joints. Some of these parameters will be discussed in the following paragraphs.

| | |
|---|---|
| a. Material parameters | Infilling properties<br>Infilling thickness<br>Joint stress history<br>Rock properties<br>Joint wall roughness<br>Orthogonal joints |
| b. Equipment parameters | Rate of shear<br>Stiffness of the shearing equipment |

**Table 2. 3** **Parameters controlling the shear strength of infilled discontinuities (After De Toledo et al, 1993)**

Sun et al. (1981) performed shear box tests on joints in concrete blocks filled with clayey sand and sandy clay with variable normal stresses and filler thicknesses. The failure surfaces occurred either at the top or at the bottom concrete contact or as a combination of both surfaces. Pereira (1990), using mainly sand filling between two flat granite blocks, also reported failure along the joint walls due to the rolling of sand grains. Solid walls affect the strength of a joint in two ways namely:

- in clay, fillers sliding occurs along the contact due to the particle alignment
- where as, in sands the rolling of grains seems to be the major factor responsible for the greater weakness of a joint as compared with its filler.

The magnitude of the influence of surface roughness depends on the particle size of the soil. In a simple form, when sand is considered as infilling material, the influence of the rock wall may begin to be felt when its surface is smoother than the roughness of the sand surface defined by its particle size distribution. This is because dilation is reduced.

According to De Toledo et al (1993) the shear strength characteristics of rock discontinuities have been studied for a long time but a complete understanding of the mechanisms and of the parameters controlling the process has never been reached. Infilled joints are likely to be the weakest elements of any rock mass in which they occur and exert a dominant influence on its behavior. The behavior of infilled rock joints in shear has been investigated for decades. Many results of field and laboratory tests had been published since the 1960's and these results were reviewed by Barton (1974). From then on, investigations became more systematic and concentrated mainly on laboratory testing.

Different materials and test procedures were employed in an attempt to study the influence of filler thickness on peak shear strength. Other physical properties were studied less frequently, mainly because of experimental difficulties. Previously published test results lead to contradictory conclusions so far as the influence of filler thickness is concerned.

A laboratory investigation always has to choose between testing real joints and artificially created ones. Natural joints, either filled or unfilled, are virtually impossible to use in systematic research, but the benefits of adopting either a copy of a natural joint or a regularly profiled one are clear. The friction of the discontinuity wall of natural, tension and artificially cut joints will be different because, for example, the cutting process may have changed the discontinuity surface mineral and crystal structure by melting, whereas natural discontinuity walls have been subjected to water percolation weathering, ect. This causes major differences in friction angle and one would expect it to be of major influence on the research and interpretation of the research results. It is a matter of choice between the study of the behavior of the interaction of infill with a real joint shape as opposed to the study of infill within a boundary whose geometry is well constrained but made from real rock material. Idealized profiles allow a better understanding of specific joint properties, yet both methods of testing laboratory-prepared joints may face serious shortcomings, such as inappropriate representation of an infilled joint in the field. In most cases laboratory shear box tests have been adopted for this work with sample sizes of 50 -250 mm and normal stresses of 20-1500 kPa, producing results relevant to conditions of zero normal stiffness, i.e. constant normal stress. Peak shear strength was always investigated and in some cases residual strength was also studied. However, the ability of equipment to study the residual strength of joints infilled with soil is limited by maximum displacements, which may lead to significant errors.

Kutter (1974), using a simplified rotary machine, showed the need to use large displacements in order to define properly the residual strength of an unfilled and rough rock joint. In fact, displacements of up to 100 mm are usually required to achieve the residual strength in soils Lupini, Skinner & Vaughan, (1981) and as much as 200 mm have been observed for some rough unfilled rock joints. It is doubtful whether a shear box apparatus or the ring shear apparatus is capable of producing a correct measure of the residual shear strength characteristics of natural infilled joints.

Research tests are generally conducted on natural joints, and on model infill joint material. Several authors, whose results on the latter can be divided into two categories, have studied the influence of filler thickness. The first category includes the matching toothed joints that have a ratio filler thickness to asperity height t/a of less than three. The second category comprises planar joints using either saw-cut, sandblasted or polished surfaces, where t/a is usually much greater than three.

Ladanyi & Archambault (1977) performed direct shear tests using kaolin clay between two series of juxtaposed concrete blocks. Different asperity heights and different normal stresses were used. The results obtained seem to confirm results obtained by Goodman (1970).

## 2.9 Shear strength equations

The equation, $\tau = \sigma_n \tan(\phi_b + i)$, is valid at low normal stresses where shear displacement is due to sliding along the inclined surfaces. At higher normal stresses, the strength of the intact material will be exceeded and the asperities will tend to break off, resulting in a shear strength behaviour which is more closely related to the intact material strength than to the frictional characteristics of the surfaces.

While Patton' s (1966) approach has the merit of being very simple, it does not reflect the reality that changes in shear strength with increasing normal stress are gradual rather than abrupt. Barton and his co-workers (1973, 1976, 1977, 1990) studied the behaviour of natural rock joints and have proposed that the equation can be re-written as:

$$\tau = \sigma_n \tan\left[JRC \log_{10}(JCS/\sigma_n) + \phi_b\right] \quad \ldots\ldots\ldots\ldots(2.13)$$

Where $\tau$ = peak shear strength, $\sigma_n$ = effective normal stress

$JRC$ = joint roughness coefficient, $JCS$ = joint wall compressive strength

$\phi_b$ = basic friction angle (obtained from residual shear tests on flat unweathered rock surfaces)

Barton and Bandis (1982) and Barton (1991), based on extensive laboratory testing of joints, and joint replicas, and a study of the literature at the time, proposed a scale correction for JRC as presented in equation 2.14:

$$JRC_n = JRC_0 \left(Ln / L_0\right)^{-0.02 JRC_0} \quad \ldots\ldots\ldots\ldots\ldots (2.14)$$

where $JRC_0$ and $L_0$ refer to the JRC and length of 100mm diameter laboratory scale samples and $JRC_n$ and $L_n$ refer to in situ specimen.

Barton and Bandis (1982) argued that the average joint wall compressive strength of large surfaces is likely to have greater possibility of weaknesses and JCS would decres with increasing scale. They proposed equation 2.15 as correction for JCS:

$$JCS_n = JcCS_0\,(Ln / L_0)^{-0.03JCS_0)} \;\ldots\ldots\ldots\ldots\ldots\ (2.15)$$

where $JCS_0$ and $L_0$ refer to the JCS and length of 100mm diameter laboratory scale samples and $JCS_n$ and $L_n$ refer to in situ specimen.