

Chapter 7

SHEAR STRENGTH INVESTIGATIONS FOR SURFACE MINING

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ABSTRACT

Simple methods for estimating the shear strength of rock joints and waste rock are reviewed. For the case of rock joints, the methods are based on a quantitative characterization of the joint roughness and the joint wall strength. Size-effects are found to reduce the peak strength of large joint samples to values below the ultimate or so-called "residual" values measured in the laboratory. Tilt tests and surface profiling on natural size blocks within the jointed rock mass are recommended for obtaining scale-free properties. The joint parameters obtained can be used to model complete strength-displacement-dilation behavior if this level of input is required. Large scale tilt tests can be performed with advantage on both rock joints and waste rock. The behavior of these two materials is surprisingly similar. Both are influenced by the size-effects on the compression strength of the rock, and both have similar log-linear relationships between effective normal stress and the peak drained friction angles. The resulting high values of friction near the toe or close to a slope face in either material can be misleading.

INTRODUCTION

It is now known with reasonable certainty that tests on small samples of rock produce artificially high values of strength. In the past, arguments have been put forward to explain size-effects by changed stress distributions, changed machine stiffness, etc. Such arguments cannot be invoked to explain scale effects observed on joints. A simple but convincing demonstration is the tilt test. Tilt angles measured during self-weight gravity sliding tests of a large slab of jointed rock are found to be many degrees less than

the steep tilt angles measured when the same jointed slab is sawn into small samples and these are tilted individually or as an assembled "rock-mass". Tests of this kind performed by Barton and Choubey (1977) and Bandis et al. (1981) indicated tilt angles increased from 59° to 69°, and from 47° to 62° respectively.

Thought of objectively, it is remarkable that rock engineers have entertained the possibility that "laboratory-size" samples could ever represent the characteristics of a failure surface perhaps one thousand times larger. Similar optimism has been displayed in the design of large rockfill dams. The pioneering work of Pratt et al. (1974) in investigating scale effects on rock joints, and of Marachi et al. (1969) in the field of rockfill, have emphasized the importance of large scale tests, and the importance of extra conservatism and adequate safety factors in the absence of such tests.

In their general report at the Denver Rock Mechanics Congress, Hoek and Londe (1974) suggested that when a very large structure such as an arch dam or major pit slope is being designed for long term stability, the design should be based on zero cohesion and residual friction, "which can be determined in small scale laboratory tests".

This philosophy probably results in more than adequate conservatism for these major slopes. However, recent work has shown that the peak strength of a large joint sample may be lower than the "residual" or ultimate strength measured after large displacement of a small sample. It is very difficult to reach the true residual strength of a non-planar joint surface since dilation persists during surprisingly large displacements.

QUANTITATIVE JOINT DESCRIPTION

The requirement for careful characterization of individual joint sets may be elevated to a high priority task, if a preliminary structural analysis indicates the potential for failure of a planned or existing pit slope. Since the mechanics of slope failure are treated elsewhere, this article will be directed towards the estimation of appropriate input data concerning the shear strength, for use in some form of stability analysis. The methods proposed are of a simple, practical nature, but they can be used to produce sophisticated strength-deformation formulations of joint behavior, if this level of input is required.

A simple though quite complete method of characterizing the shear behavior of rock joints was developed some years ago (Barton, 1973). It consists of three components: ϕ_b , JRC and JCS. A basic or residual friction angle (ϕ_b or ϕ_r) for flat non-dilatant surfaces in fresh or weathered rock, respectively, forms the limiting value of shear strength. To this is added a roughness component (i). This is normal stress dependent and varies with the magnitude of the joint wall compressive strength (JCS), and with the joint roughness coefficient

(JRC). The latter varies from about 0 to 20 for smooth to very rough surfaces respectively. The peak drained angle of friction (ϕ') at any given effective normal stress (σ'_n) is expressed as follows:

$$\phi' = \phi_r + i = JRC \log(JCS/\sigma'_n) + \phi_r \quad (1)$$

Example

$$\phi_r = 25^\circ, JRC = 10, JCS = 100 \text{ MPa}, \sigma'_n = 1 \text{ MPa}$$

Equation 1 gives $\phi' = 45^\circ$.

Examples of the strength envelopes generated with JRC values of 5, 10 and 20 are illustrated in Figure 1. The compression strength of the joint walls (JCS) has increased influence on the shear strength as the joint roughness increases. Values of JCS and its variation with weathering are measured with the Schmidt (L) hammer. Experimental details are given by Barton and Choubey (1977).

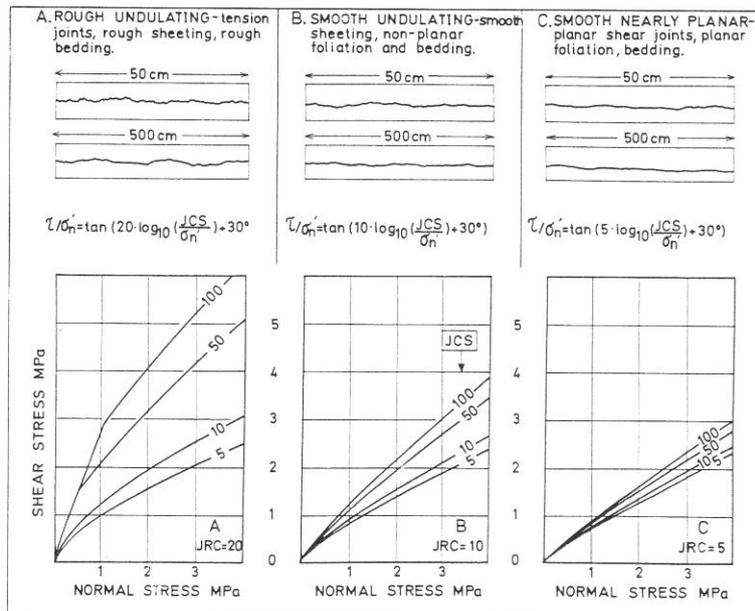


Figure 1. Method of estimating the peak shear strength of rock joints, based on the joint roughness coefficient JRC (20, 10 or 5), and on the joint wall compression strength JCS (100, 50, 10 or 5 MPa), after Barton (1973) (1 MPa = 145 psi).

The residual friction angle (ϕ_r) of weathered joints is very difficult to determine experimentally due to the large displacements required, particularly if only small joint samples are available. A simple empirical approach has been developed as shown below.

$$\phi_r = (\phi_b - 20^\circ) + 20 r_1/r_2 \quad (2)$$

where ϕ_b = basic (minimum) friction angle of flat unweathered rock surfaces (obtained from tilt tests on sawn blocks, or from triple core tilt tests - see Figure 2)

r_1 = Schmidt rebound on saturated, weathered joint walls

r_2 = Schmidt rebound on dry unweathered rock surfaces (i.e., saw cuts, fresh fracture surfaces, etc.)

Example:

$$\phi_b = 30^\circ, r_1 = 30, r_2 = 40$$

Equation 2 gives $\phi_r = 25^\circ$

The joint roughness coefficient (JRC) can be estimated in several different ways. For example, Barton and Choubey (1977) show a set of 10 increasingly rough joint profiles measured on 10 cm long specimens, which can be physically compared with profiles measured on other joints. However, a more reliable method of determining JRC is by conducting tilt tests on jointed core, or on jointed blocks extracted from existing slopes, as illustrated in Figures 2 and 3.

The value of JRC is back-calculated directly from the tilt test by rearrangement of the peak strength equation:

$$JRC = \frac{\alpha^0 - \phi_r}{\log(JCS/\sigma_{no}')} \quad (3)$$

where α^0 = tilt angle when sliding occurs ($\alpha^0 = \arctan \tau/\sigma_{no}' = \phi'$)

σ_{no}' = effective normal stress acting across joint when sliding occurs

Example: $\alpha = 75^\circ$, $\phi_r = 25^\circ$, JCS = 100 MPa, $\sigma_{no}' = 0.001$ MPa

$$JRC = (75^\circ - 25^\circ)/5 = 10$$

The values of JRC, JCS and ϕ_r are used to generate peak shear strength envelopes over the required range of stress. The table of

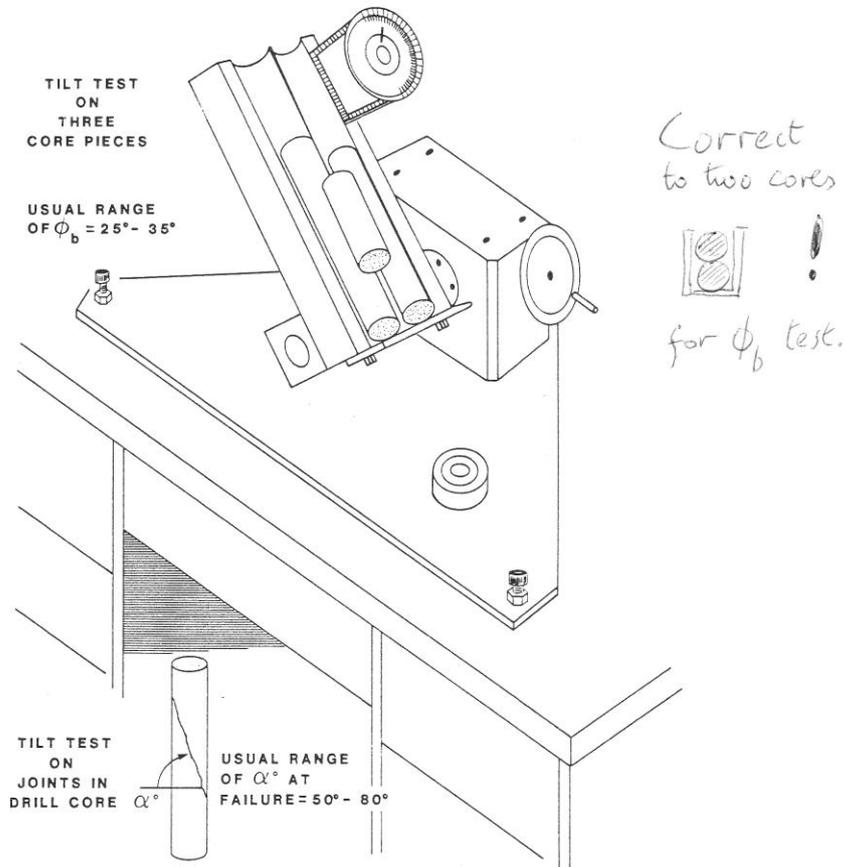


Figure 2. Tilt tests can be used to measure ϕ_b (of flat surfaces) and to measure the friction angle of joints intersecting drill core. These low stress tests are readily extrapolated to design stress levels.

values below indicates how the value of ϕ' varies inversely with the log of effective normal stress. This is a fundamental result for rock joints, rockfill, gravel, etc. (Barton and Kjaernsli, 1981).

Example: JRC = 10, JCS = 100 MPa, $\phi_r = 25^\circ$:

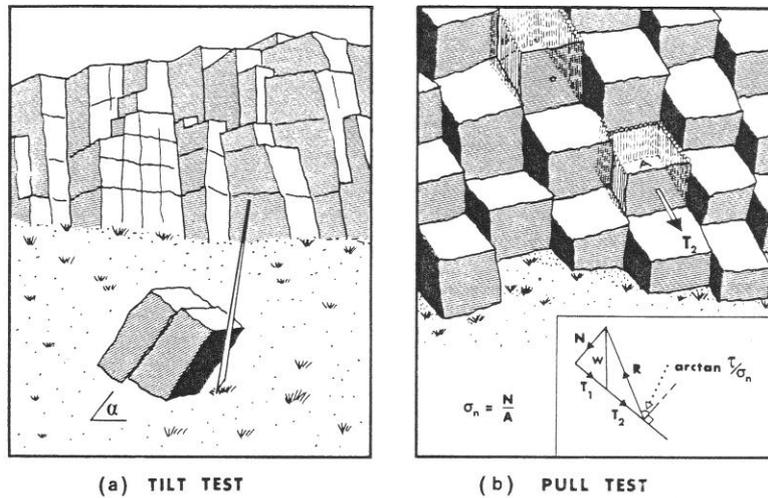


Figure 3. Tilt tests (or pull tests) can be performed relatively inexpensively on large jointed blocks of natural size. Only gravity loading is required, thereby removing the need for heavy jacking equipment. Test results are readily extrapolated to design stress levels.

	σ_n'	ϕ'
Approx. lab tilt test	0.001	75°
Approx. field tilt test	0.01	65°
	0.1	55°
Approx. design loading	1.0	45°
	10.0	35°

SIZE-DEPENDENT JOINT PROPERTIES

Large scale shear tests of joints in quartz diorite (Pratt, et al., 1974) and a comprehensive series of tests performed by Bandis (1980) have indicated that larger shear displacements are required to mobilize peak strength as the length of joint sample is increased. This means that larger but less steeply inclined asperities tend to con-

trough peak strength as the length of sample is increased. Due to the change of significant asperity size and inclination angle, the change of sample size reduces both the dilation component (d_n) and the asperity failure component (S_a), shown in Figure 4. However, the sample size does not apparently affect the magnitude of ϕ_r or ϕ_b , but it does affect the shear displacement needed to reach these values.

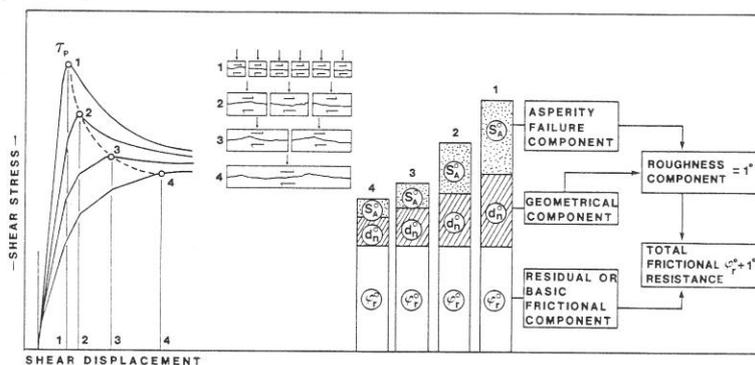


Figure 4. Peak strength, "residual strength", and the shear displacements needed to mobilize them are all dependent on the length of joint tested, after Bandis et al. (1981).

In practice both JRC and JCS are lower when the size of joint sample is increased. A useful method of allowing for this double scale effect has recently been developed as a result of numerous shear tests on different sizes of model joint replicas, as shown in Fig. 5.

Example: Laboratory test, $L_o = 15$ cm, JRC = 10, JCS = 100 MPa,

In situ test, $L_n = 90$ cm, JRC \cong 7, JCS \cong 50 MPa,
(of same joint)ⁿ

Taking our earlier example with $\sigma'_f = 1$ MPa, the laboratory value of ϕ' of 45° would reduce to approximately 37° if measured on a 90 cm long joint sample in situ. The potential effect of sample size on slope stability is too large to be ignored, unless joints are unusually planar (low JRC). Planar joints have many of the characteristics of a residual surface, and sample size appears to have only a minor influence on strength.

Surface Profiling

Shear box tests or tilt tests performed on small samples of a joint, such as those obtained in carefully preserved drillcore (Figure 2) may not produce reliable strength data, even after approx-

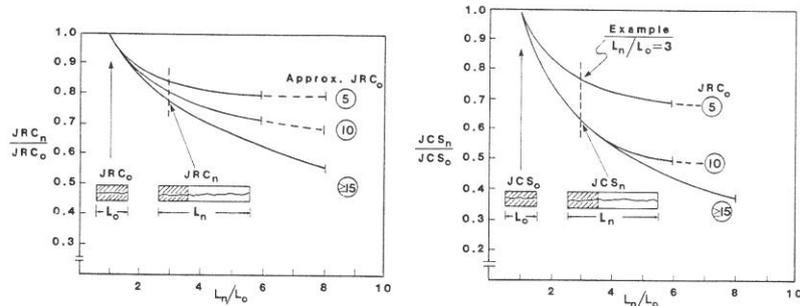


Figure 5. Approximate method for extrapolating the results of small scale laboratory shear tests to in situ scale, after Bandis et al. (1981).

imate correction for the scale effects on JRC and JCS. Joints can have considerable roughness on a small scale, yet be rather planar over a length of meters. Conversely, a joint may be rather smooth on the scale of a drillcore sample, but quite undulating over a length of meters.

Where possible, attempts should therefore be made to sample large scale exposures of the relevant joint set, using a simple straight edge (taut wire) and offset method. Values of maximum amplitude (a) measured over a sample length (L) of 1 meter and up to several meters can be used to obtain a rough estimate of JRC at the appropriate scale, as shown in Figure 6.

The method of estimating (JRC) shown in the figure was developed from the following approximate relationships:

$$\begin{aligned} JRC &\cong 400 \cdot a/L & \text{for } L &= 0.1 \text{ m} \\ JRC &\cong 450 \cdot a/L & \text{for } L &= 1.0 \text{ m} \\ JRC &\cong 500 \cdot a/L & \text{for } L &= 10 \text{ m} \end{aligned}$$

These were obtained from an analysis of some 200 roughness profiles measured on 0.1 m long joint samples (Barton and Choubey, 1977; and Bandis, 1980), and from tests on model replicas of joints of different roughness reported by Bandis (1980).

In practice, it is recommended that as many as possible of the above methods of estimating shear strength are utilized concurrently to improve reliability:

1. Tilt tests on joints sampled in drillcore (Figure 2) (estimate large scale JRC and JCS from Figure 5).
2. Tilt (or pull) tests on blocks of natural size (Figure 3).
3. Roughness profiling at different scales (Figure 6).

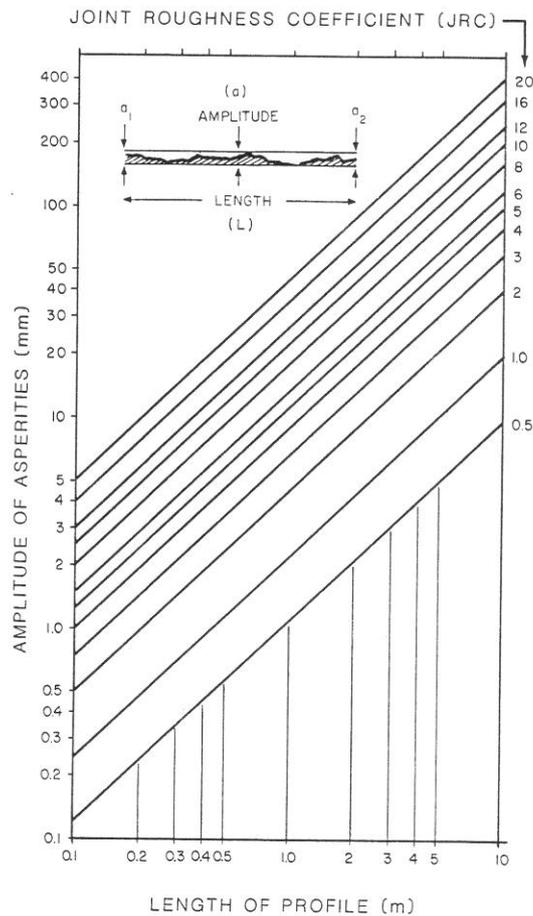


Figure 6. Measurement of joint roughness amplitude on various lengths of joint provides estimates of JRC, and its variation with sample size.

APPROPRIATE TEST SIZE

Up to this point the shear strength scale effect has been discussed without suggesting a specific sample as the "correct" size for tilt testing, profiling or extrapolating towards. Unfortunately, an inherent limitation of direct shear testing of individual jointed blocks is that the response of the surrounding rock mass is absent. A key question is whether the stiffness of the rock mass overlying

and underlying a plane or zone of potential shear failure is "soft" enough to allow the blocks to follow the individual shear paths required to maintain contact with the smaller scales of roughness.

Test results reported by Bandis et al. (1981), shown schematically in Figure 7, indicate that the shear strength of a densely jointed mass (small block size) may be higher than that with a more massive block size. Small blocks have greater capacity to rotate slightly and maintain contact with small-scale features of roughness. In effect, any joints intersecting a potential failure plane are potential hinges, giving the rock mass just the degree of freedom necessary to suffer a similar scale effect to that of individually jointed blocks.

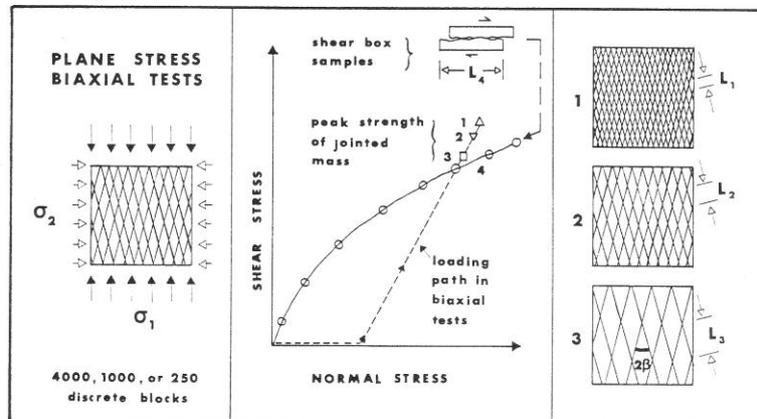


Figure 7. Size effects on individual joint samples also extend to a jointed mass of interlocked blocks.

In the light of these arguments, the appropriate test size would appear to be the natural block size. Simple tests like those depicted in Figure 3 are designed to provide precisely that scale of joint characterization. It is believed that the extrapolation and profiling procedures outlined earlier should also focus on the natural block size. Profiling of longer joint exposures (if available) may help to determine whether a larger scale undulation angle (i) needs to be added to the block-size ϕ' value to account for local changes in joint dip.

NUMERICAL MODELING OF STRENGTH-DISPLACEMENT BEHAVIOR

A limiting equilibrium analysis of a potentially unstable pit slope, using rigid block assumptions, and a convenient stereographic analysis to account for a three dimensional joint structure, gives a

useful estimate of the factor of safety of the slope. However, it does not indicate to mine management whether they should withdraw personnel and valuable plant when the slope monitoring equipment indicates a large displacement in the central part of the slope, after development on a new level at the toe. The alternative, numerical modeling, has now reached the stage where it can assist in evaluating stability of a jointed rock slope. The familiar complaint concerning inadequate input data may no longer be valid.

Mobilization of Shear Strength

A review of some 650 test results on joint surfaces ranging from 40 mm to 12 metres lengths indicates a quite consistent trend for increased displacement to mobilize peak strength as sample size is increased. For convenience results have been grouped into the four surface categories shown in Figure 8, and the following three size ranges: 0.03-0.3 m, 0.3-3.0 m and 3.0-12.0 m. The fifteen data points on earthquake fault slip magnitudes were average values derived from a review paper by Nur (1974).

An analysis of the data (Barton, 1981) indicates that the following equation gives a reasonable approximation to the observed values:

$$\delta = \frac{L}{500} \cdot \left(\frac{JRC}{L} \right)^{0.33} \quad (4)$$

where δ = slip magnitude required to mobilize peak strength, or to remobilize residual strength

L = length of joint or faulted block (meters)

Example 1. Laboratory Specimen. Assume: JRC = 15 (rough), L = 0.1 m

Equation 4 gives $\delta = 1.0$ mm

Example 2. Natural jointed block. Assume: JRC = 7.5, L = 1.0 m

Equation 4 gives $\delta = 3.9$ mm.

Recent developments allow the modelling of not only peak shear strength using equations 1 and 4, but the complete shear stress-displacement history of a shearing event including dilation. This is made possible by the observation that the joint roughness mobilized at any given shear displacement (δ') follows a consistent trend for a great variety of joint surfaces. The dimensionless ratios JRC(mobilized)/JRC(peak) and $\delta'/\delta(\text{peak})$ are used to generate the desired shear stress-displacement data. Figure 9 illustrates how the method produces a realistic simulation of shear test results, spanning a wide range of surface roughness. Changes in normal stress, and changes in scale are readily simulated.

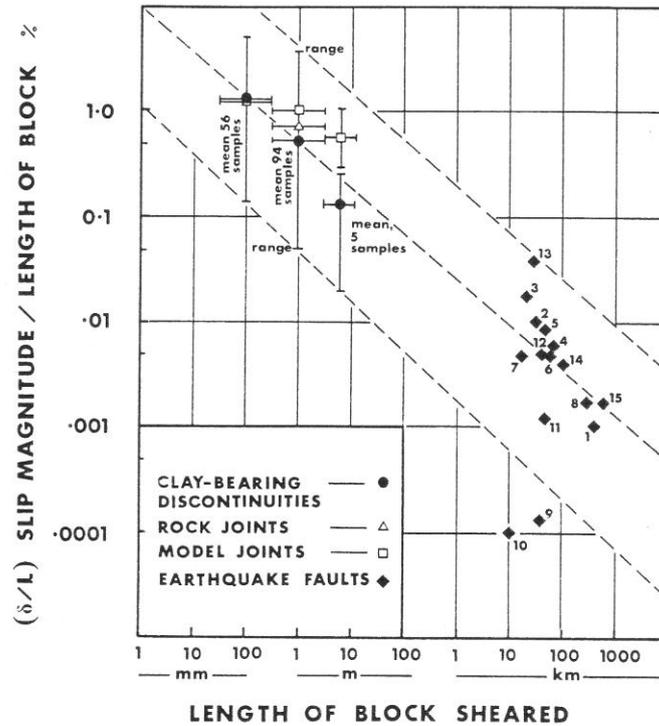


Figure 8. Slip magnitudes depend on the length of surface sheared. Number of samples are relevant to the clay bearing discontinuities. A total of 650 test results are incorporated in the four classes of surface. (After Barton, 1981).

The potential consequences of the scale effect on shear stress-displacement behavior is illustrated in Figure 10. A rough joint in competent rock may show quite artificial behavior at laboratory scale. The key feature to be noted is that the ultimate or so called "residual" shear strength of a laboratory-scale joint sample may be higher than the peak strength of a large sample of the same joint, such as a naturally jointed block. This important result, which was also shown diagrammatically in Figure 4, has been observed in numerous shear tests spanning a variety of surface roughness morphologies (see Bandis et al., 1981).

Design Values of Shear Strength

A numerical analysis of pit slope stability that incorporates stress-displacement modelling of the joints as illustrated in Figures

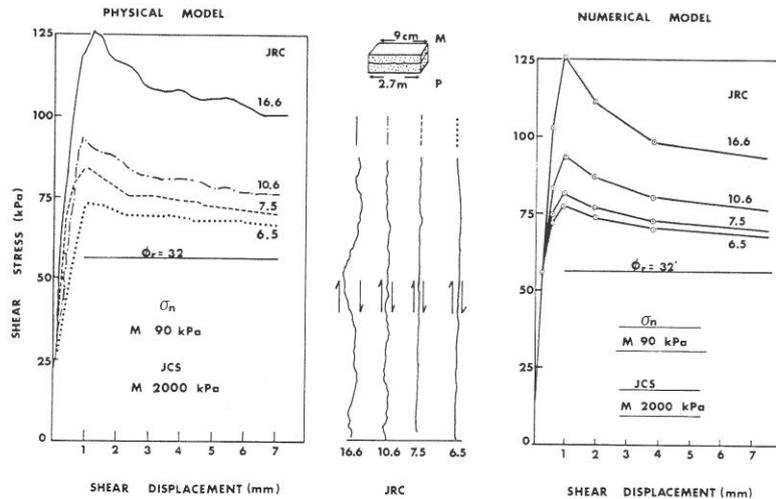


Figure 9. Numerical models of shear tests on a variety of surfaces illustrate the potential for generating shear stress-displacement data for use in slope design studies. The tests on physical models of rock joints are reported by Bandis et al. (1981).

9 and 10, will automatically mobilize pre-peak, peak, and possibly post-peak shear strengths in different parts of the slope, as appropriate.

However, in a rigid block equilibrium analysis, a whole failure surface is artificially allocated an appropriate value (or values) of strength. The design engineer who adopts true residual strength (ϕ_r) for his whole failure surface is simultaneously implying that the slope would be equally stable were it excavated to heights of 10, 100 or 1000 meters. This level of conservatism is probably correct when applied to a persistent clay-filled discontinuity.

However, for the case of rock joints that are substantially free of clay fillings, it appears appropriate to consider using the full-scale peak strength for at least that part of the failure surface where stresses do not exceed the available strength (Barton, 1974). It will be noted that full-scale peak strength is mobilized and exceeded in a "stable" manner, very different to the "unstable" post-peak behavior observed in laboratory-scale tests. Those parts of the failure surface where peak strength is exceeded (generally the central slope region where overburden is maximum) would be allocated the minimum shear resistance (ϕ_r).

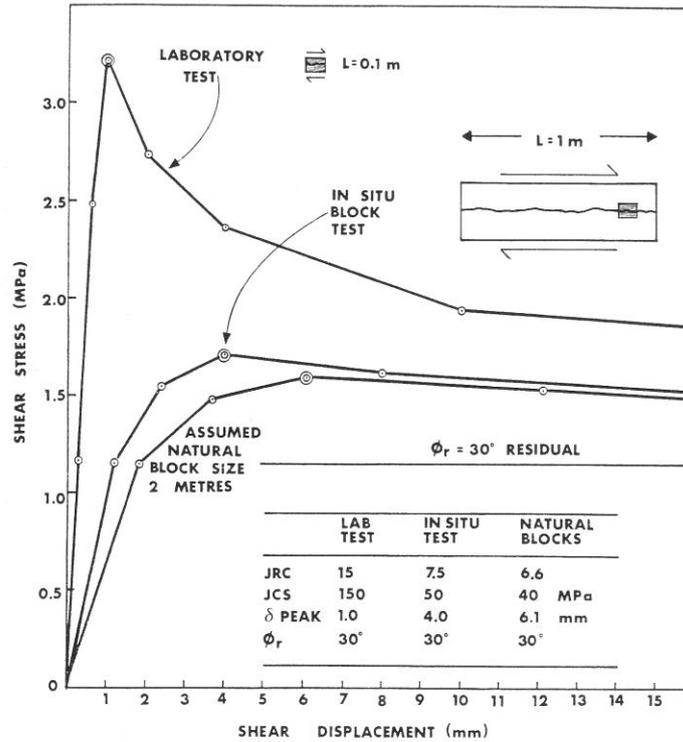


Fig. 10. An illustration of the importance of large scale test data for rock joints. Scaling of JRC and JCS was based on Figure 5, and scaling of δ_{peak} on equation 4. A constant normal stress of 2 MPa was assumed.

STABILITY OF WASTE ROCK DUMPS

The linear reduction of ϕ' with the logarithm of effective normal stress observed in rock joints (equation 1) is also observed in triaxial tests of angular rockfill and gravels (Leps, 1970). Consequently, the adoption of a single value of c and ϕ to design a major waste rock or tailings dump gives an erroneous factor of safety, just as it does for a jointed rock slope.

In fact, for both rock joints and waste rock (rockfill), values of ϕ' are dependent on sample size, stress level, surface roughness, and on the uniaxial compression strength of the rock. Friction angles are therefore higher for smaller samples, and very high where stresses are low, as at the toe or near the face of a slope. Barton and

Kjaernsli (1981) show that the value of ϕ' for rockfill can be quantified by an equivalent roughness (R), and an equivalent particle strength (S), as illustrated in Figure 11. The value of (R) depends on the porosity of the fill and on the particle angularity and surface roughness.

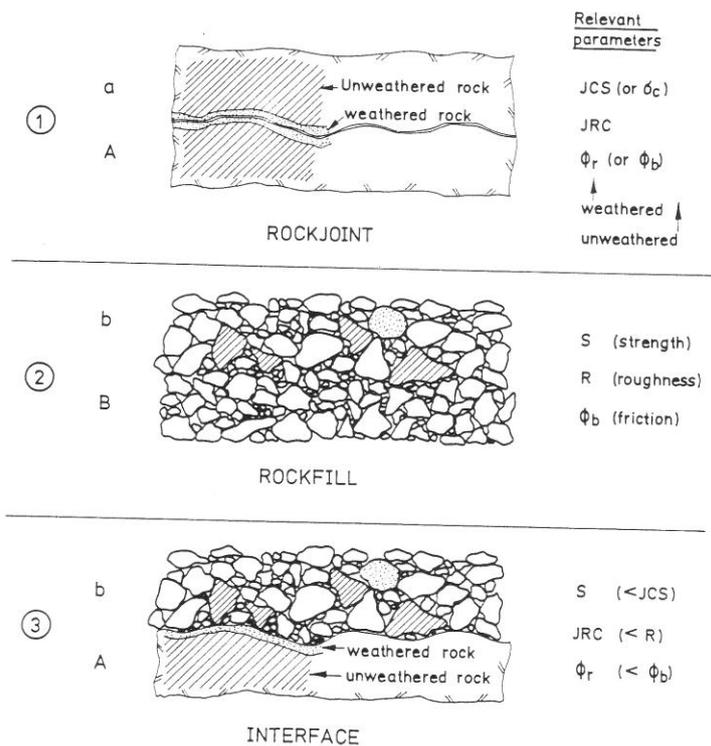
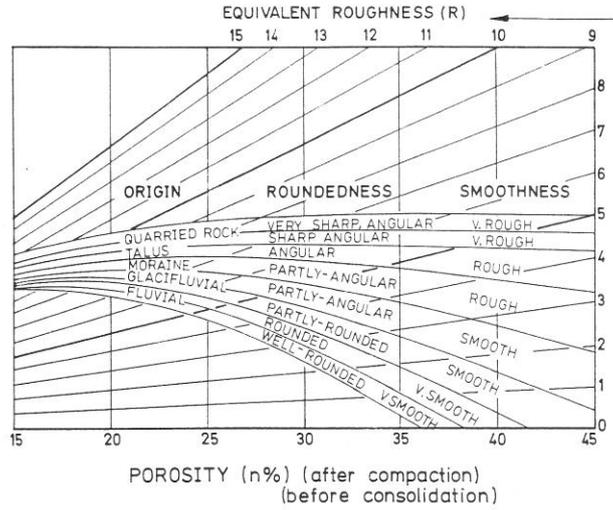


Fig. 11. Empirical approach to shear strength estimation for rock joints, rockfill (or waste rock) and any interface between the two.

The equivalent roughness (R) which is analogous to the JRC value of a rock joint, can be estimated from Figure 12. For example, dumped rock with an in-place porosity of about 35% will probably have an equivalent roughness (R) of about 5 to 6 (sharp, angular particles).

An empirical method of estimating the equivalent strength (S) of rock particles is shown in Figure 13. This parameter is analogous to the joint wall compression (JCS) of rock joints and is also scale dependent. The peak drained friction angle (ϕ') of rockfill or waste rock can be estimated from equation 5, which is exactly analogous to equation 1 for rock joints:

3RD STABILITY IN SURFACE MINING



EXAMPLES SHOWING DEGREE OF ROUNDEDNESS				
QUARRIED ROCK	TALUS	MORaine	GLACIFLUVIAL MATERIAL	FLUVIAL MATERIAL

Fig. 12. Method of estimating the equivalent roughness (R) of rock-fill or waste rock, based on the porosity of the dump, and on the angularity of the particles, after Barton and Kjaernsli, 1981.

$$\phi' = R \cdot \log (S/\sigma_n') + \phi_b \tag{5}$$

Example: porphyry waste dump:

$$\sigma_c = 150 \text{ MPa}, d_{50} = 250 \text{ mm}, S \cong 30 \text{ MPa (Figure 13)}$$

$$n = 35\%, \text{ sharp angular particles, } R \cong 6 \text{ (Figure 12)}$$

$$\phi_b = 30^\circ \text{ (obtained from tilt tests on sawn blocks)}$$

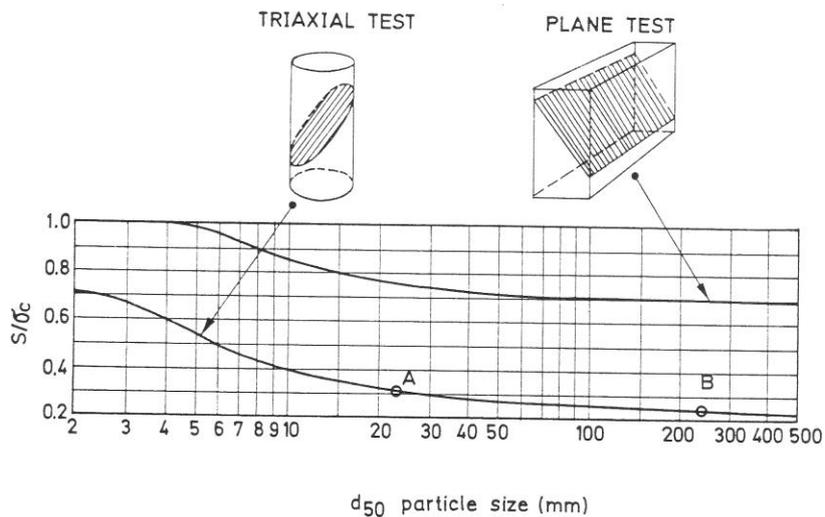


Fig. 13. Method of estimating the equivalent strength (S) of rock-fill or waste rock, based on the uniaxial compression strength (σ_c), and on the d_{50} particle size, after Barton and Kjaernsli, 1981.

The following values of ϕ' would be obtained from equation 5.

σ_n' (MPa)	ϕ'
0.1	45°
1.0	39°
10	33°

It will be noticed from this tabulation and an earlier one for rock joints that the value of ϕ' varies by R or JRC degrees, for each ten-fold change in stress level. Comparisons with published data (Barton and Kjaernsli, 1981) indicate that this degree of stress-dependency extends over at least five orders of magnitude. Figure 14 illustrates how this stress dependency influences the values of ϕ' available in different parts of a waste dump, when assuming a simple triangular (self-weight) distribution of vertical stress.

The lesson to be learned from this type of example is that the excellent surface stability of a dump, and the steep angles toler-

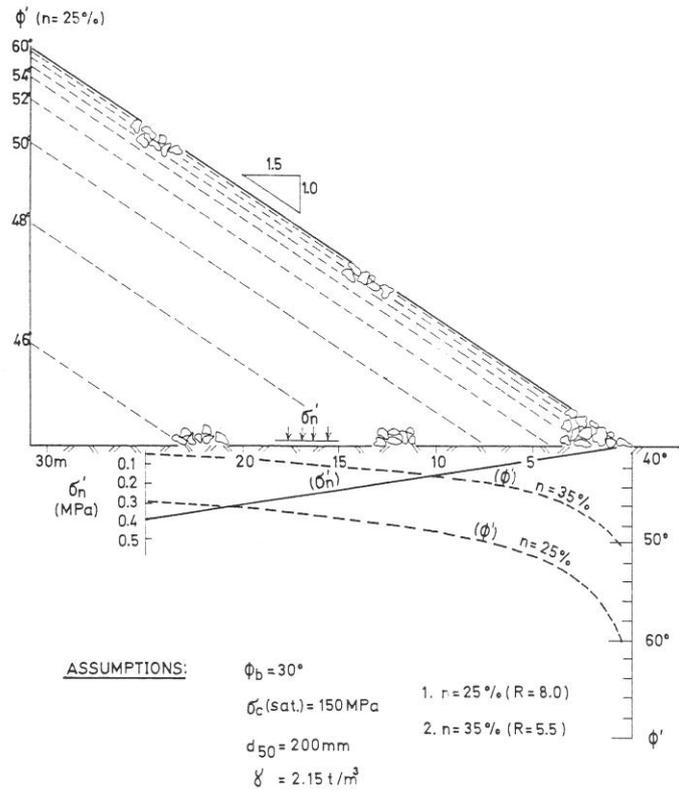


Fig. 14. Estimated variation of ϕ' under the toe and beneath the slope of a waste dump or dam.

ated when slopes are of moderate height, may each be misleading. Some tens of meters beneath a waste dump the available shear strength may be 10-15° lower than at the surface or at the toe. A deep seated failure through the waste rock (or partly along an underlying foundation-waste interface (Figure 11) is therefore more likely than surface manifestations of instability.

Tilt-Test for Waste Rock

A serious limitation of laboratory shear strength investigations is the inability to test full-scale rockfill or waste rock samples with as-built gradings and porosities. Development of this equivalent roughness method, with its potential for accurate extrapolation of strength over many orders of magnitude of stress, provides a means of

interpreting large scale tests on wasterock performed under extremely low stress, in a similar manner to tilt tests on jointed blocks of rock. The principle of a method for tilt-testing waste rock is illustrated in Figure 15. A tilt frame of several meters length is suggested. The one end can be elevated by hydraulic rams, in the manner of a dumper truck.

The maximum angle of tilt (α°) tolerated before failure of this size of sample will probably be of the order of 55° to 65° under the extremely low effective normal stress operating across the failure surface. The angle $\alpha^\circ = \phi'$, can be extrapolated to design stresses by estimating values of S and ϕ_b from index tests, and back-calculating the value of R from equation 5. The inevitable errors in estimating S and ϕ_b are automatically compensated by the values of R back-calculated. Under-estimates of S or ϕ_b are compensated by over-estimates of R , and vice versa. Final estimates of ϕ' are therefore unusually accurate. This error compensation is also common to the tilt tests on rock joints shown in Figures 2 and 3. Tilt tests have the added advantage that the non-uniform strain and progressive failure common to conventional shear box tests on rockfill are much reduced, due to the inherently more uniform nature of gravitationally induced shear and normal stress.

CONCLUSIONS

1. Shearing between the two interlocked walls of a rock joint, and between the interlocked particles of dumped waste rock or rockfill, results in the mobilization of quite similar values of peak shear strength. However, rock joints generally reach their peak strength at much smaller strains than rockfill or dumped waste rock.
2. The peak drained friction angles of rock joints and rockfill (or dumped waste rock) can be quantified by an equivalent roughness (JRC or R), an equivalent asperity or particle compression strength (JCS or S), and by the residual or basic friction angle (ϕ_r or ϕ_b) respectively. The latter are generated by non-dilatant surfaces of the given rock types, following large displacements. Each of these six parameters can be estimated by simple index tests, using the Schmidt hammer and various forms of tilt test.
3. The analogous behavior of jointed rock and fragmented waste rock extends to size-effects on strength, and to log-linear relationships between the effective normal stress and the peak drained friction angles (ϕ') of each material. Values of ϕ' tend to be high at the toe and close to the face of rock slopes and waste-dumps. This apparent stability should not be misinterpreted.
4. Size-effects can probably be almost eliminated by conducting self-weight tilt tests (or regular direct shear tests) on jointed rock blocks of natural size. In the case of waste rock or rockfill, tilt tests should be performed on full-size gradings whenever possible.

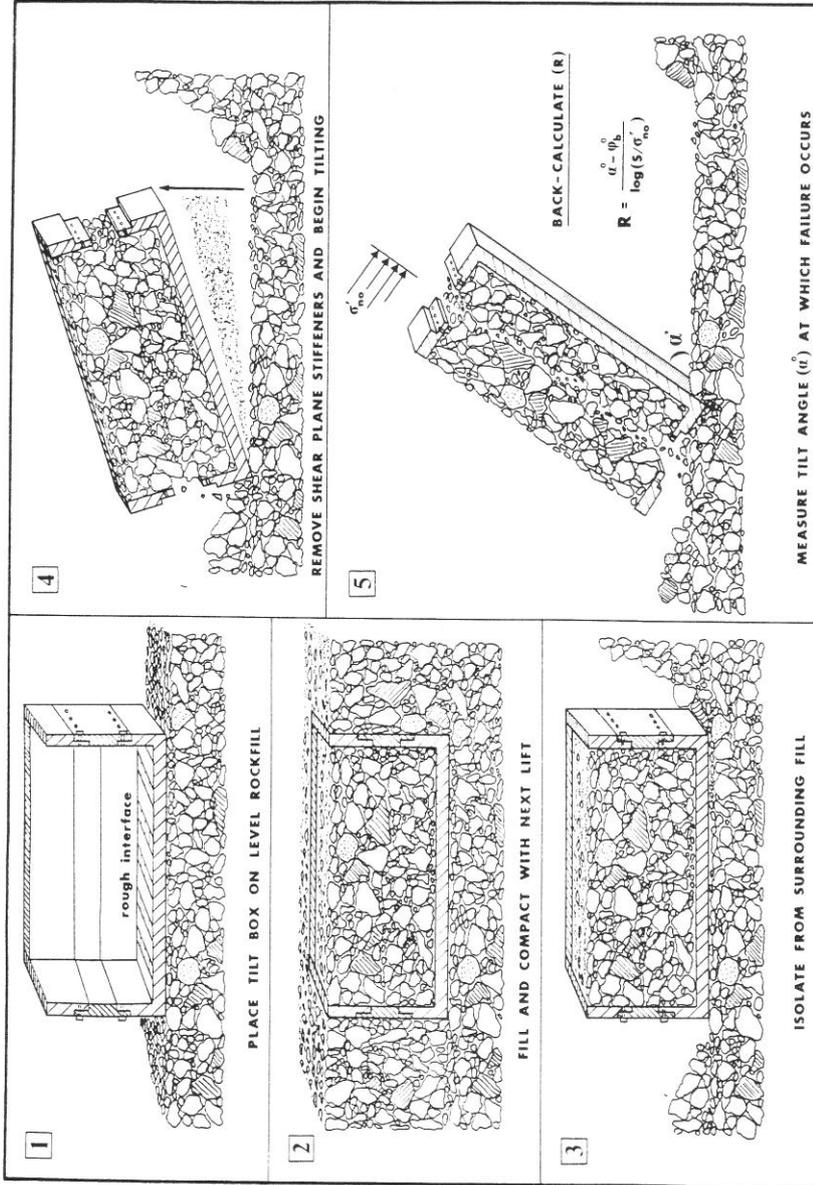


Fig. 15. A method for measuring the shear strength of dumped waste rock after Barton and Kjaernsli, 1981.

5. In cases where large samples are unavailable, suggested methods are described for scaling down the values of JRC and JCS for rock joints (Figures 5 and 6) and for scaling down values of S for waste rock or rockfill (Figure 13).
6. It is suggested that the full-scale value of peak shear strength for rock joints can be used in those parts of a potential failure surface where the shear stress does not exceed peak strength. Recent tests indicate that the full-scale value of peak strength is often lower than the ultimate or assumed "residual" strength measured on laboratory-scale samples.
7. Complete shear strength-displacement and stress-strain modeling of rock joints and rockfill can be achieved using the concept of roughness mobilization. The term JRC (mobilized) is used for rock joints, and the term R (mobilized) is used for rockfill. Good agreement with experimental strength-displacement and stress-strain curves are achieved with these methods.
8. A major uncertainty remains in the analysis of rock slope stability. The continuity of individual joints is difficult to estimate and the possibility of limited interaction between joint sets hard to quantify. The assumption of zero cohesion (no intact rock "bridges") is probably a wise precaution when joints have adverse orientations.

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Question

The joint roughness as measured by you appears to assume dry surfaces. Have you tried any tests using a wet surface. Would it make a significant difference to the roughness coefficient by acting as a lubricant.

Answer

Our tilt tests were performed dry to avoid negative pore pressures (suction) delaying the tilt failure. In the field, moisture is probably present anyway, so tilting should be performed slowly, i.e. drained. However, as a point of definition, the value of JRC is back-calculated using the saturated value of the joint wall compression strength (JCS), which is usually 10-15% lower than the dry value. I believe water might act as a lubricant for some layer-lattice minerals, i.e. rocks rich in mica, etc. It would then presumably tend to reduce JRC, but I suspect most of the water effect would be seen in strongly reduced JCS values. Rocks rich in quartz with massive crystal lattices, apparently suffer an anti-lubrication effect. I believe they also show the least change in JCS, with saturation. The 140 rock joints that I developed the tilt test hypothesis on, were all tested saturated in the shear box, following the index tests. The correlations were therefore developed between dry tilt or pull tests, and wet shear box tests. I don't recall any correlation

SHEAR STRENGTH INVESTIGATIONS**193**

problems specifically related to the gneiss or slate.

Question

If ϕ_r reduces with increase of σ_n' , and ϕ_r reduces with increase of size of shear surface, can you recommend a σ_n' for testing laboratory samples.

Answer

Firstly, a point of definition. Maybe we should talk of ϕ_u (ultimate) as measured at the end of a given test, because the true minimum ϕ_r (residual) should not change with σ_n or size of shear surface. So answering the question with regard to ϕ_u ; yes, I think a suitable testing level for σ_n' could be recommended which would force the laboratory size joint to give the same lower ϕ' (peak) value as a large sample. The necessary value of σ_n' could be calculated if JRC and JCS were known - it would depend on both. However, I don't think this would be the right way to proceed. Wouldn't it be more logical to test at the design level of σ_n' , and extrapolate JRC and JCS to the correct size values? Best of all, do tilt or pull (self-weight) tests on the correct size of surface in the first place, where this is practical.

Question

Does your tilt box test for aggregate require a particular ratio between the size of aggregate and the size of the box. Can a similar approach be used for evaluating a heavily jointed cliff face with several sets of Joints.

Answer

The objective of the large scale tilt test for aggregate (wasterock) or rockfill) is to overcome the normal test limitations. We specifically want to be able to test full-scale material without the usual requirement of using model gradings. A box length of at least $5 \times d_{max}$ or at least $30 \times d_{50}$ is perhaps reasonable. It would be worth experimenting with a small box and model gradings before designing the large tilt box frame. The upper parts of a waste dump will probably have an entirely different grading curve from the lower parts, so the size of tilt frame will not be ideal for either case.

The heavily jointed cliff face with several sets of joints - if sampled "undisturbed" - with joints still interlocked - will possibly display too high a ϕ' (peak) value; the tilt angle at which failure occurs might be too high (i.e. 75°) such that tensile rather than shear failure occurs. The empirical relationship between ϕ' (peak) JRC and JCS might otherwise work reliably since a modified version involving equivalent roughness (r) and equivalent strength (S) works well for rockfill. Judging by the steep and strongly curved envelopes obtained by Jaeger for Bougainville (Panguna) andesite, I would expect quite high values of JRC (or R), unless failure is joint controlled.

194 3RD STABILITY IN SURFACE MINING

Question

How do your JCS and JRC shear strength relationships vary with different rock types.

Answer

This is a difficult question to give clear-cut answers to. Joints of different roughness (JRC) obviously occur in the same rock type, i.e. bedding joints and cross joints in sandstone, foliation joints and tension joints in gneiss. Similarly, in the same rock type one joint may be tightly interlocked, "non-conducting" to water, and essentially unweathered (high JCS), compared to the major weathered, water-conducting set with lower JCS. The study we conducted in Norway some years ago involved fifteen different types of joints sampled from seven rock types. A wide range of behaviour was exhibited - from ϕ' peak of about 80° to about 30° , for exactly the above reasons. The chief cause of this scatter was roughness, i.e. $JRC \approx 18$ for bedding joints in hornfels, and $JRC \approx 1$ for cleavage joints in slate. Joint compression strength (JCS) ranged from about 140 MPa in granite to 20 MPa in soapstone, according to the Schmidt hammer testing.

Recent work on size effects by Bandis et al. (see references in paper) show that these wide differences in JRC and JCS are effectively reduced at large scale, but will still give ϕ' (peak) values as different as $10 - 30^\circ$, depending on joint type.

Question

Did all small blocks fail at the same (or nearly the same) tilt angle. Is there a fundamental basis for your shear strength equation.

Answer

The eighteen small blocks cut from the large jointed slab give a tilt angle of 69° (3 x 18 tests in all). Some were too strong to tilt test, so were pulled horizontally under self-weight. From memory, they showed quite a wide range of tilt (or pull) angles; perhaps $55^\circ - 80^\circ$. Beside experimental scatter, some small samples were essentially shearing down the dip of major asperities, and others up-dip. So there would need to be a big difference. It was noticeable that the large sample (before division) gave a tilt angle of 59° each of the three times. Lower and more consistent JRC values seem to be a fundamental feature of larger size samples. That is one of the reasons we recommend tilt (or pull) tests on blocks of natural (large) size.

The "fundamental basis" question would take time to answer fully. I'll try a shortened version. First of all, roughness (JRC) is related to asperity height (a) and sample length (L) (Figure 6). Values of $\Delta a / \Delta L$ obviously represent crude approximations to Patton's (i) values - which can be added to ϕ_r in a fundamentally justifiable manner to give ϕ' (peak). Secondly, we have found from our and other's results (Iwai, Bandis, et. al.) that the ratio of true to assumed contact areas (A_1/A_0) at or prior to peak strength, are

SHEAR STRENGTH INVESTIGATIONS

195

approximately equal to the particular value of σ_n'/JCS . In other words, asperities are just about at failure.

A review of a large number of high pressure triaxial tests that I undertook some years ago, indicated that the Mohr failure envelopes for a wide range of rocks were horizontal when the "critical" stress state $\sigma_1' = 3\sigma_3'$ was reached. This condition is the same as $(\sigma_1 - \sigma_3)/\sigma_n' = 1$ on the failure plane. The "confined strength" $(\sigma_1 - \sigma_3)$ has proved to be useful estimate of JCS at stress levels higher than $\sigma_n' = \sigma_c$ (unconfined compression strength). By implication, our high stress "critical" state with $JCS/\sigma_n' = 1$ also implies total crushing of asperities, with $A_1 = A_0$, and, intuitively, total suppression of dilation. These more or less logical patterns in the JRC, JCS behaviour constitute something like a "fundamental basis", at least for me. Most important is the fact that the equation works, over many orders of stress magnitude, for rock joints, rockfill, and for interfaces between the two. So far, I haven't been able to understand why ϕ' (peak) for these materials changes by approximately JRC or R degrees ($^\circ$) for each ten-fold change in effective normal stress. This is a simple, fundamental result that needs to be explained.

Question

Do I understand you to mean that a highly fractured model (2D) has higher strength than a less fractured model. What happens when you tessellate a volume as opposed to a plane. Does the relationship still hold?

Answer

The highly fractured (4000 block) models do have higher strength than the less fractured (1000, and 250 block) models. But they also have lower deformation modulus, which seems to be the reason why the small blocks have the freedom necessary to slightly rotate and register the smaller scale (higher JRC) roughness. I think the result is chiefly a useful experimental justification for saying that block size effects shear strength. In the real world, highly fractured rock masses will often not have as rough joints as the more massively jointed rock masses, so the same result is less likely to be seen. But when you get down to the case of a heavily fractured, multiple joint set material, where the shear strength is no longer anisotropic (perhaps), I suppose interaction of blocks comes into the picture to cause rather high strength, low modulus behaviour. This was not the case in my tension crack model tests, with just two intersecting sets.

I believe the block-size controlled result is likely to persist to the three-dimensional case, but with the same provisos as I've mentioned.

Question

As one of the main authors of the rockmass Q-system approach in tunneling design, could you elaborate on your hope, or the

196 3RD STABILITY IN SURFACE MINING

potential in the long term, to develop the same approach to rock slope stability.

Answer

As you probably know, Bieniawski has suggested that his RMR system can be applied to both tunnels and slopes, by modifying the original orientation weighting factor. If one has a good eye for slope instability, I suppose this could work. As far as the Q-system is concerned, I would prefer to see it used in estimating the relative strengths of individual discontinuities (J_r/J_a gives a realistic estimate of $\tan \phi$), or for helping to estimate the shear strength of a randomly jointed mass (as done by Hoek and Brown). The orientation question is so important for slopes that are structurally controlled, that I feel it may be dangerous to use classification systems on their own, to classify good, poor, or very poor (presumably failed) slopes. Where does one draw the line between stability and instability? However, I would encourage use of the Q-system parameters for a general classification of the degree of jointing, clay fillings, etc. The first four parameters RQD/J_n , J_r/J_a give a very detailed description of rockmass conditions, useful for communication in reports, and understood by increasing numbers of engineers. As in the case of tunnels, judgment would need to be exercised in classifying J_r and J_a for the discontinuities most likely to allow failure to initiate.