

## Bolt design based on shear strength

NICK BARTON & KHOSROW BAKHTAR  
Terra Tek Engineering, Salt Lake City, Utah, USA

**ABSTRACT:** Laboratory tests for determining the shear strength of bolted rock joints indicate that peak values of strength are developed when a grouted bolt is installed at an angle of about 35-50° to the plane of the joint. This range of angles generally corresponds to the peak, post-peak, or pre-residual region of behavior. Solution of practical problems, such as the bolting of unstable slopes or of major wedges in underground caverns, indicates that minimum bolt capacities are required when the bolt is angled perpendicular to the frictional resultant, corresponding to the arctangent of ( $\tau/\sigma$ ) relative to the joint, as above. Methods are described for generating appropriate shear force-displacement curves for rock joints so that a bolt of a given stiffness can be installed at the appropriate angle of mobilized friction. Bolts of lower stiffness require smaller installation angles and correspondingly increased capacity. The use of bolts of lower stiffness, for example partly grouted bolts, may be justified if displacements are irresistible, or if other components of support are of reduced stiffness. Bolt design should always be compatible with the expected deformation.

### 1 INTRODUCTION

The correct design of rock bolt reinforcement is dependent on compatible strength-deformation properties of both the rock mass and the rock bolts. It is as easy to design a rock bolt system that is too stiff for the rock mass as it is to design a system that allows the rock to strain soften, thereby losing the interactive capacity of the reinforced system. Timing of support installation is of importance both for rock slope reinforcement and for tunnels. If fully grouted bolts are installed before any shear stresses are developed, the high stiffness of the bolts at each "joint-crossing" will cause the bolts to be subjected to the full excavation-induced shear stress, with little if any assistance from the inherent strength of the joints. The difficult-to-achieve ideal is the bolt that reaches yield just when the joint has fully mobilized its shear strength.

Some few millimeters of shear displacement may typically be required to fully mobilize shear strength, unless the feature being bolted is already subjected to a significant shear stress prior to excavation.

The frequent need for careful characterization of the joints is apparent.

### 2 REVIEW OF LABORATORY TEST DATA

Laboratory tests for determining the shear strength of bolted rock joints indicate that peak values of strength are developed when a grouted bolt is installed at an angle of about 35-50° to the plane of the joint. Test results for 450 mm (17.7 inches) long jointed blocks reported by Bjurström (1974) are reproduced in Figure 1. The most significant result is the existence of an optimum installation angle. Also of interest are the dowel and tensile components of strength and the added shear strength due to a normal stress increment caused by the inherent resistance of the bolt to dilation.

A comprehensive series of tests on different types of bolts was recently reported by Haas (1981). Both perpendicular and 45° installation angles were studied. The higher peak strengths achieved with 45° installation angles are noticeable, but the more direct tensile load also appears

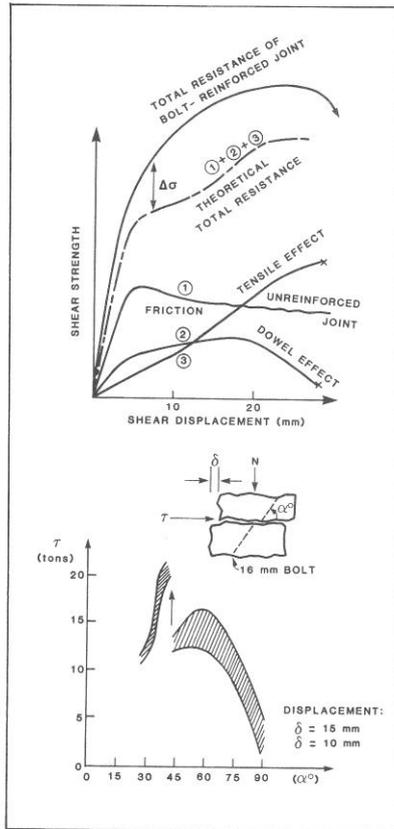


Figure 1. The strength components, and influence of installation angle on the shear resistance of a bolted joint. After Bjurström (1974).

to cause failure at smaller shear displacements as shown in the two diagrams reproduced in Figure 2.

Failure of the bolts after about 1 inch of shear compared to 2½-3 inches in the case of the perpendicular installation, emphasizes the advantage that might sometimes be gained by using a low-alloy strain-hardening steel as a bolt material, if large displacements need to be tolerated

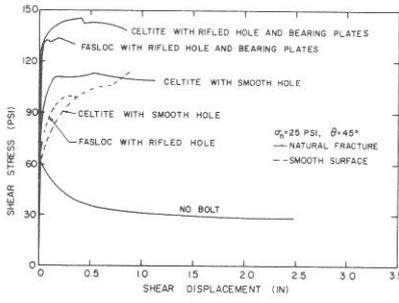
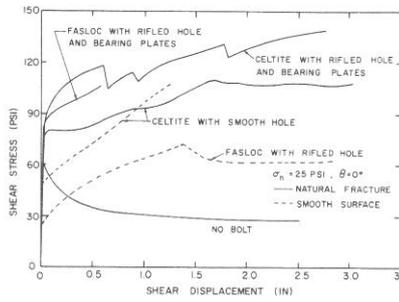
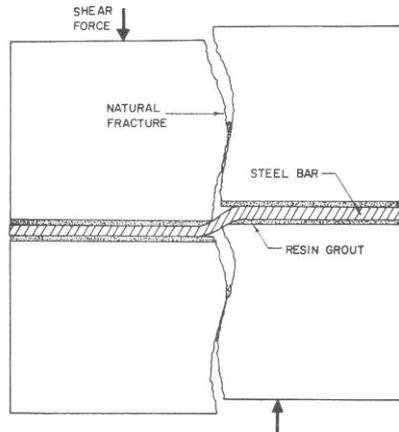


Figure 2. Comparison of shear strength developed with perpendicular and 45° bolt installation angles, after Haas (1981).

without failure. Such may be the case in tunnels undergoing squeezing, or in defense related structures that must "ride" with more or less irresistible block motions.

Tests reported by Azuar, et al. (1979) also show greatest shear resistance when the bolt is installed at 30°, with successively reduced resistance when the angle is increased to 60° and 90°.

Recent tests on fibre reinforced sand reported by Gray and Ohashi (1983) indicate that the fibres should be "ideally extensible" inclusions. Fibre-reinforced sand and bolt-reinforced ruptured rock have obvious points in common. The authors' shear tests indicated optimum fibre reinforcement angles of 60° relative to the shear plane. Reportedly, this corresponded to the principal tensile strain direction in a dense sand, tested at stress levels of about 5-30 psi.

### 3 THEORETICAL BOLT DESIGN USING FORCE DIAGRAMS

The requirement for specially designed bolting or anchoring of a rock cutting or underground opening is usually associated with the presence of an adversely dipping discontinuity, or with a set of persistent joints. Frequently such features act in combination to define an unstable wedge.

In the two-dimensional representations shown in Figure 3, it will be assumed that the unfavorable feature strikes parallel to the rock cutting and parallel with the underground opening. Such a situation may frequently arise in the case of an engineering construction joining point A with point B. However, it would normally be avoided or minimized by different orientation in the case of an underground powerhouse or similar "short" excavation.

In the cutting illustrated in Figure 3, a water-filled tension crack and water seepage along the discontinuity have been assumed as a worst case for purposes of design. The associated forces T and U are represented in the force diagram, and are seen to increase the required bolting force B. All forces including weight W can be estimated per running meter of slope.

Other key parameters in the force diagram are the direction of the frictional resultant R, which depends both on the shear

strength ( $\arctan \tau/\sigma'$ ) and on the dip ( $\alpha$ ) of the discontinuity. It is readily shown that the force diagram is closed by the minimum value of B, when the bolts or anchors are installed at an angle  $\arctan (\tau/\sigma')$  relative to the plane of the discontinuity. For the moment  $\arctan (\tau/\sigma')$  will be referred to as the effective mobilized angle of friction ( $\phi'$  mob). Its value may correspond to pre-peak, peak, or post-peak shear strength, depending on the stiffness of the anchor or bolt installation.

Note that while force equilibrium is satisfied by appropriate closure of the force diagram, overall equilibrium of the system will not be satisfied unless net moments are zero. In the diagram of the cutting (Figure 3) the forces T, U, W and R have all been drawn through the centroid of the section of slope. This is clearly an ideal case. Anchor hole locations could be chosen so that moment equilibrium is satisfied anyway, but this may not always be consistent with other design considerations, such as anchor length.

### 4 DESIGNING FOR LARGE SHEAR DISPLACEMENTS

It has already been suggested that installation of fully grouted bolts prior to the development of excavation-induced shear stress may cause a disproportionately large load to be transferred to the bolts. A finite shear displacement is required for the shear strength of rock joints to be fully mobilized. The objective should be to share the load between the joints and the bolts.

Similar ideals should apply when bolting is just one part of a comprehensive support system. In the case of a tunnel undergoing squeezing, it makes little sense to have compressible slots in the shotcrete and sliding joints in the steel ribs, when the bolts are fully grouted and effectively much stiffer than the other components of support. A major tunnel known to the first author suffered failure of approximately 80% of the bolts and essentially uncontrolled closures, due to the incompatible stiffnesses of the various support components (Barton, 1982a).

An extension of the above load-sharing-compatible-stiffness concept is needed in defense related block-motion problems. Dynamic loading and associated block-motion may be so severe that the displacements cannot be resisted, and must there-

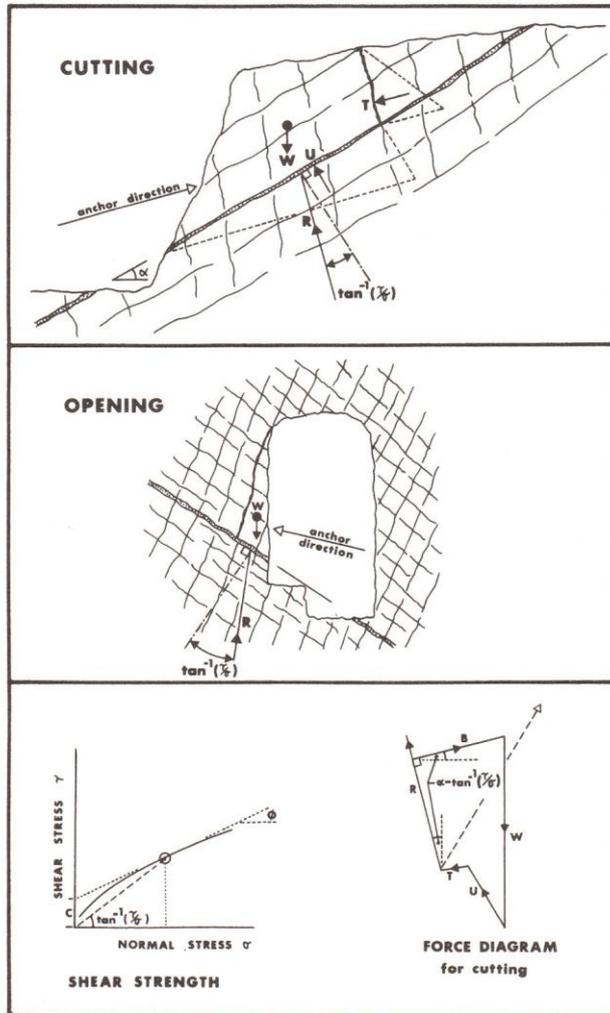
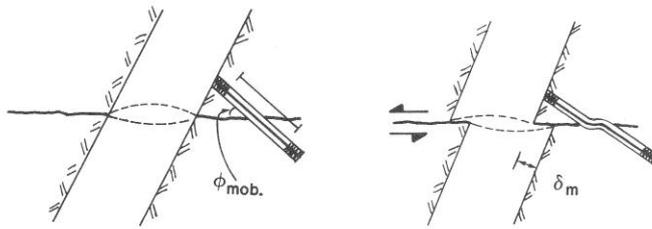


Figure 3. Optimum anchor or bolt orientation should be based on the shear strength of the adverse discontinuity, with anchor deformability compatible with the strength-deformation properties of the discontinuity, under the appropriate level of effective normal stress. After Barton, 1973.



ROCK ANCHORS DESIGNED TO TOLERATE ELONGATION  $\delta_m/L \leq 0.20$

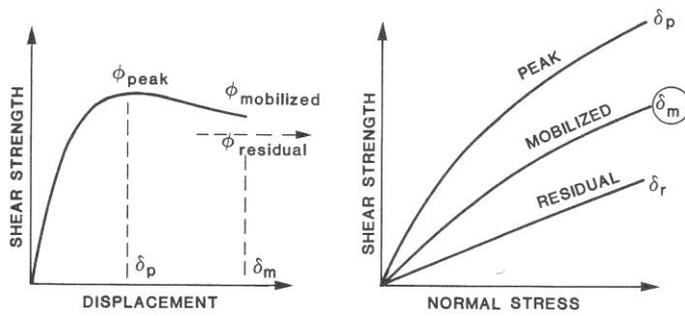
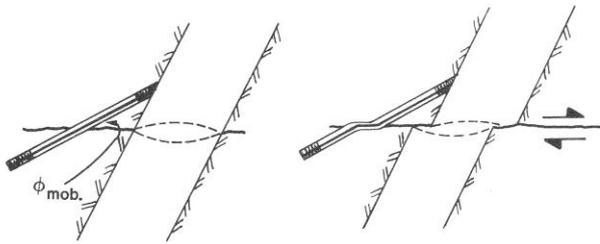


Figure 4. Bolts designed to tolerate large displacements should be inclined at an angle  $\phi$ (mobilized), the shear strength available after a displacement  $\delta_m$ .

fore be absorbed by the support system. In such cases flexible, strain hardening bolting is called for, as typified by low-alloy steels, which can absorb local strains in excess of 20%.

An idealized, flexible design for absorbing large displacements is illustrated in Figure 4. In essence, long bolts or anchors would be grouted only at their ends, to allow for considerable extension in the central ungrouted region. Damage

to the bolt would be minimized by using a lubricated sleeve in this ungrouted region.

In such cases, the design displacement ( $\delta_m$ ) determines the shear strength remaining along the joint or bedding plane. The value ( $\phi'$  mob.) is assumed to be relevant to post-peak, pre-residual conditions, and its value will determine the optimum angle of bolt installation. Calculating the appropriate value of  $\phi'$  mob. requires that shear stress-displacement curves are generated for shear under appropriate levels of effective normal stress. Both the geometry of the structure, any water pressures, and the stress increment caused by the presence of the bolt determine the appropriate effective stress level.

Figure 5 illustrates hypothetical force diagrams that are relevant to the rock cutting shown in Figure 3. Three potential values of shear strength; peak, mobilized or residual are shown. The frictional resultants  $R_p$ ,  $R_m$  and  $R_r$  which are perpendicular to the minimum bolt forces  $B_1$ ,  $B_2$  and  $B_3$ , intersect the relevant strength envelopes at three different levels of effective normal stress. The actual stress level is determined by the magnitudes of the total weight  $W$ , by the water forces  $U$  and  $T$ , and by the relevant friction angle. These in turn determine the required bolting force. It is readily seen that allowing a larger displacement to occur increases the required bolting force, increases the effective normal stress, and changes the optimum angle of

installation. The important design question is how to estimate the appropriate strength envelope for optimizing the bolt design. Guidelines are provided in the remainder of this paper.

#### 5 ESTIMATING APPROPRIATE STRENGTH ENVELOPES

A simple method of estimating the peak shear strength of rock joints was described by Barton and Choubey (1977). The empirical formulation includes three easily determined index parameters; the joint roughness coefficient (JRC), the joint wall compression strength (JCS) and the residual friction angle ( $\phi_r$ ). These values can be determined by performing self-weight tilt tests on jointed core or jointed blocks, by measuring the Schmidt (L-hammer) rebound values on the relevant joint surfaces, and by sliding pieces of core in line contact. All but the Schmidt hammer tests are illustrated in Figure 6. Peak shear strength ( $\tau$ ) is related to effective normal stress ( $\sigma_n'$ ) as follows:

$$\tau = \sigma_n' \tan (\text{JRC} \log \text{JCS} / \sigma_n' + \phi_r) \quad (1)$$

The peak strength envelope generated with this equation will only be relevant if the bolt stiffness is designed to limit displacements to the peak strength region. Furthermore, the values of JRC and JCS are scale dependent. A more general description of behavior is required that incorporates scale effects, and that also

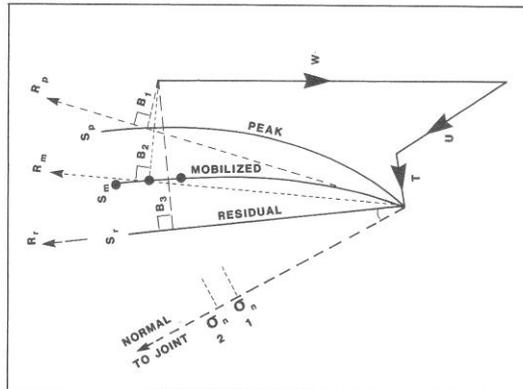


Figure 5. Optimum bolt installation angles and load capacities will vary with the amount of shear displacement to be tolerated.

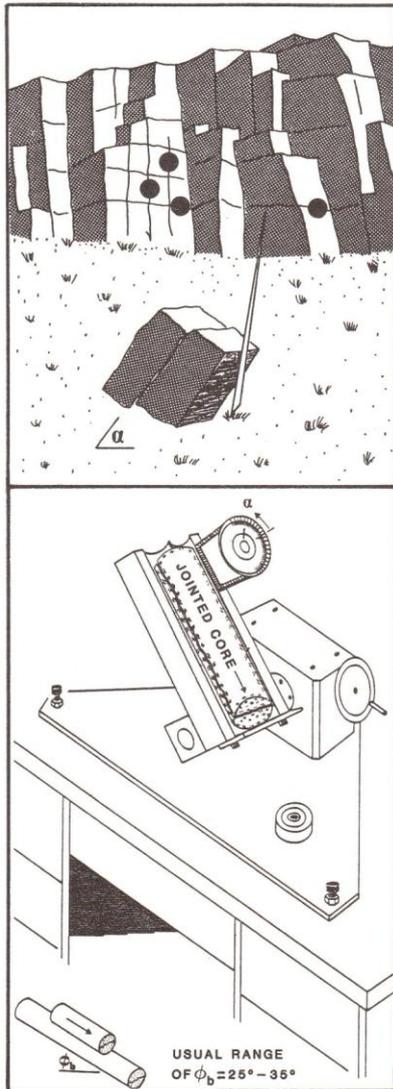


Figure 6. Simple index tests for determining the joint roughness coefficient

(JRC) and the basic friction angle  $\phi_b$ . These parameters form the basis for estimating shear strength.

addresses the important question of strength mobilization at different shear displacements.

Barton and Bandis (1982) discussed methods of allowing for the scale effects associated with different in-situ block sizes. The capability for modelling shear strength at different displacements was developed by considering the associated mobilization and reduction of joint roughness. Examples of joint displacement modelling are shown in Figure 7. The

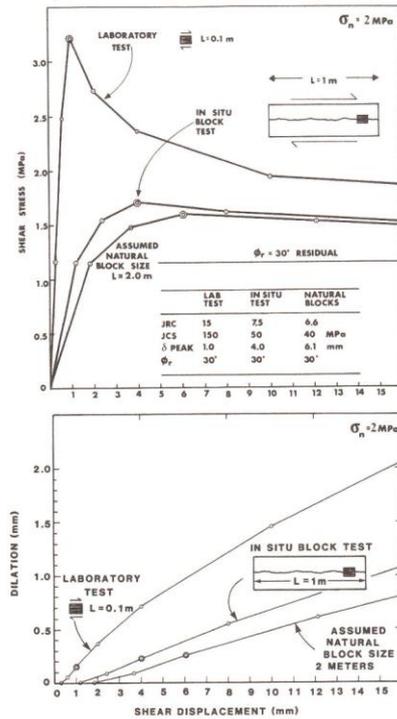


Figure 7. Shear stress-displacement and dilation-displacement behavior predicted for different joint sample sizes, after Barton, 1982b.

method was developed by detailed analysis of a large body of shear test data, as described by Barton (1982b) and Barton and Bakhtar (1983).

In practice it will be necessary to perform a preliminary analysis such as that shown in Figure 5, to determine the approximate range of effective normal stress levels. This stress range will then be used to generate an appropriate set of stress-displacement curves, using full scale values of roughness ( $JRC_n$ ) and wall strength ( $JCS_n$ ). Examples of typical size reductions in these parameters are shown in the inset to Figure 7. (Reduced scale effects are evident when smoother joints are involved.)

The remaining step is determined by engineering constraints in that the properties of the bolt material and the length of ungrouted bar determine the design displacement. The appropriate displacement is marked on each stress-displacement curve and the relevant pairs of values of mobilized shear strength ( $\tau_{mob}$ ) and effective normal stress ( $\sigma'_n$ ) are used to generate the appropriate envelope. The circles drawn on the "MOBILIZED" curve in Figure 5 would be typical examples of these pairs of values.

Example output from a numerical model designed to generate stress-displacement data at different levels of effective normal stress is shown in Figure 8. In this example index data has been derived from 0.1 m long joints in drill core and resulted in nominal laboratory-scale values of  $JRC_o = 10$  and  $JCS_o = 150$  MPa. These values represent a medium rough joint in competent rock. Scaling has been performed such that the curves represent the behavior of in-situ jointed rock, with an assumed natural block size ( $L_n$ ) of 2 meters.

At the largest displacements considered (500 mm, filled circles) the shear strength is obviously very close to the residual value of  $30^\circ$ . The huge design displacements chosen here obviously far exceed the values that will normally be considered in a rock slope, tunnel, or dam abutment bolt design. However the exaggeration of displacement helps to illustrate the design concepts described above.

## 6 BOLT DESIGN CONCEPTS FOR FILLED DISCONTINUITIES

Bolt design, as discussed above, makes allowance for different installation angles for dilatant rock joints and for almost non-dilatant clay-filled discontinuities, since the respective strength envelopes include the effect of dilation, if present. However, the estimation of appropriate strength envelopes for the case of clay-filled discontinuities is one of the more difficult tasks likely to be faced by an engineering geologist. Time-effects and unloading-effects may have a surprising influence on the available long-term shear strength.

A hypothetical example of a rock cutting excavated above an adverse discontinuity is illustrated in Figure 9. It is assumed that the discontinuity has a heavily over-consolidated clay filling and is partly unloaded by the major new highway cutting.

Several time effects are set in motion by excavating such a cutting. Negative pore pressures may be set up in the clay filling from two causes. Firstly the total normal stress is locally reduced by excavation, and the resulting limited expansion is initially resisted by the pore water. If the clay filling is heavily overconsolidated, additional negative pore pressures may be developed due to the tendency for O.C. clays to dilate when subjected to shear strain. However, following its deceptively high short-term strength, the clay will slowly soften as water is sucked into the dilating zone. The dissipating negative pore pressure will cause a gradual reduction in effective stress with time, as the steady seepage condition is approached.

A very conservative anchor design would be required, with anchor installation angles determined by the slow, drained residual shear strength of the particular clay filling. This might easily result in installation angles inclined as little as  $10-15^\circ$  from the dip of the discontinuity. A review of typical drained shear strength parameters for a wide variety of clay-filled discontinuities was given by Barton (1973).

## 7 CONCLUSIONS

Shear tests of bolted joints show that the optimum installation angle for maximum

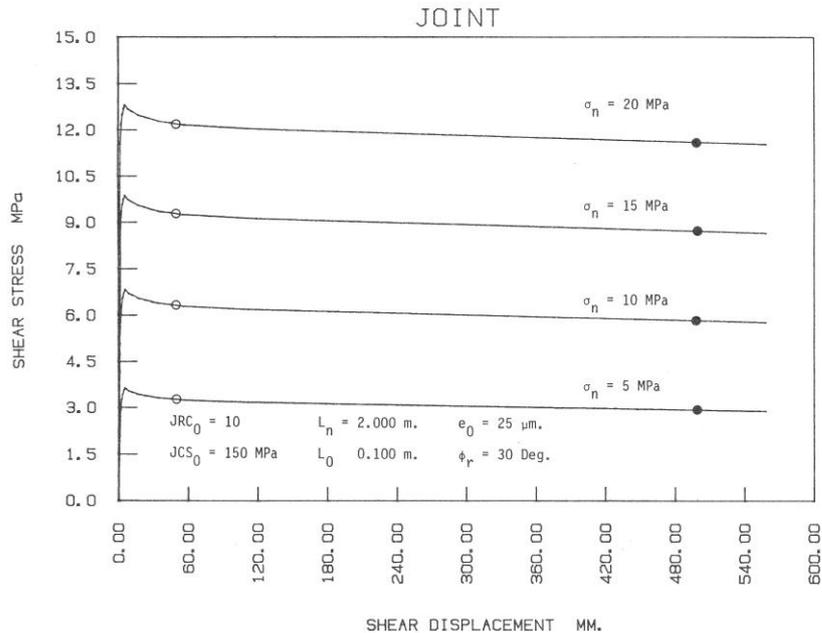


Figure 8. Modelling stress-displacement behavior at different levels of normal stress will allow appropriate  $\phi$ (mobilized) envelopes to be derived for use in Figure 5. In this example the strength remaining after 50 mm (2 inches) of displacement is shown by the open circles, and at 500 mm (20 inches) by the filled circles.

bolt capacity should be inclined at about  $35^\circ$ - $50^\circ$  from the plane of the joint. This corresponds to the mobilized friction angle. Theoretical force diagrams, with or without joint water pressures, also indicate optimum inclination angles equal to the mobilized friction angle. The exact value of the mobilized friction angle will depend on the shear displacement that occurs. This in turn will depend on the designed stiffness of the bolting system. Stiffness requirements will vary from project to project.

It is shown how joint properties can be characterized by simple index tests, so that shear stress-displacement behavior can be predicted for any given in-situ block size. Compatible design requires that stress-displacement curves are gener-

ated at effective stress levels appropriate to the particular engineering problem. The shear strength mobilized at the design displacements may be pre-peak, peak or post-peak, depending on the bolt stiffness. The appropriate bolt installation angles should be based on the relevant mobilized strength envelope, which can be generated by analyzing the appropriate set of stress-displacement curves.

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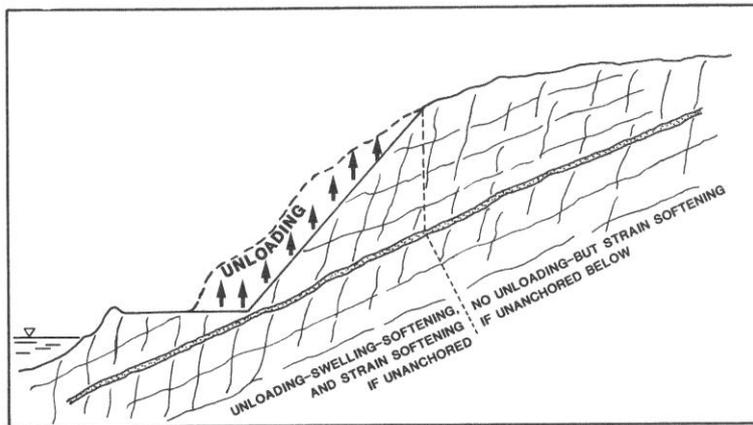


Figure 9. Excavation above a clay filled discontinuity is likely to cause a long-term stability problem, particularly if the filling is initially over-consolidated.

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