

A REVIEW OF THE SHEAR STRENGTH OF FILLED DISCONTINUITIES IN ROCK

Oversikt over skjærfastheten hos fylte diskontinuiteter i fjell.

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SUMMARY

Rock discontinuities that are filled with plastic materials represent one of the greatest problems in rock engineering. The wide range of properties and variety of occurrences make it extremely difficult to estimate the shear strength in anything but crude terms – for instance “low” ($\phi_f = 12^\circ - 20^\circ$), or “very low” ($\phi_f = 6^\circ - 12^\circ$). Even the ability to classify in this manner may be extremely valuable when designing the optimum anchoring or bolting required to stabilize surface cuttings or the walls of large underground openings. The most complicated and critical filled discontinuities may need to be tested in situ, if the cost of failure is sufficiently high.

If direct shear tests are to be performed it is extremely important that the test conditions are as relevant as possible to field conditions. The soil mechanics principles relevant to shearing and unloading problems are briefly reviewed. It would seem that slow drained tests will be the most relevant test method for all cases involving unloading above the critical filled discontinuities.

An increasing degree of complexity is introduced into the problem when the clay fillings are less thick than the roughness amplitude of the wall rock. A limited shear displacement will then result in a marked stiffening when opposed rock asperities make contact.

Both idealized laboratory models and engineering examples of rock wall interaction are reviewed, in an attempt to clarify the relative importance of filling behaviour and rock contact. Shear test results reported in the literature for filled discontinuities are tabulated in an appendix.

INTRODUCTION

For various reasons the rock joints of Norway, both clean and clay-filled, have hardly ever been tested in direct shear, either in the laboratory or in situ. Among the most important reasons for this apparent failure are:

- (i) the unusually high strength of most of the rock
- (ii) the relative lack of surface weathering, due to recent glacial erosion
- (iii) generally widely spaced and discontinuous jointing
- (iv) extremely varied and complicated occurrences of filled joints

The first three factors combine to make shear testing “unnecessary” in the view of the optimistic design engineer. The last factor makes testing “impossible”, or at least very expensive. The object of the next few paragraphs is to show that shear strength parameters can play an important role in rock engineering.

The two stability problems illustrated in Figure 1 are examples of engineering constructions that result in unloading of two hypothetical clay filled joints. If these clay fillings are in a heavily over-consolidated state it is quite possible that their initial shear strength (combined cohesion and friction represented by $\tan^{-1}(\tau/\sigma)^{\circ}$) will exceed their angle of dip (α) $^{\circ}$. Their apparent short term stability will be increased by the marked drop in pore water pressure as a result of unloading and increased shear strain. The visible signs of complete stability at the end of construction might easily lead to inadequate anchoring or bolting, which in any case could not be designed economically without knowledge of the approximate long term shear strength of the clay filled joints.

In time, as a result of increasing pore water pressures and softening and swelling of the clay, the stability will deteriorate and an increasing load will be thrown onto the existing bolts or anchors. It is quite possible that tension cracks above a surface cutting could be filled with water during unfavourable weather conditions.

All these factors should be taken into account when designing optimum reinforcement. A simple force diagram such as that illustrated in Figure 1 will provide the designer with the required anchor loads, for instance in units of tons per metre of wall. An estimate of the long term shear strength ($\tan^{-1}(\tau/\sigma)^{\circ}$) is absolutely necessary, since this is the angle that the anchors should be inclined, relative to the dip (α) $^{\circ}$ of the critical clay filled joints. Relative to horizontal the anchors or bolts should be inclined at an angle (β) as follows:

$$\beta = \alpha - \tan^{-1}(\tau/\sigma)^{\circ} \quad \begin{array}{l} (+) \text{ upwards} \\ (-) \text{ downwards} \end{array} \quad (1)$$

The above examples illustrate that there are simple everyday uses for shear strength information. Perhaps more important than anchor design, is the ability to estimate that it is often unnecessary. The force diagram illustrated in Figure 1 (stippled line) indicates that anchoring may not be required if the line of action (R) intersects the line of action (W), unless a certain factor of safety is required in addition to the pessimistic water pressure assumptions.

Methods of testing filled discontinuities

It may be possible to extract clay samples from simple clay filled discontinuities, and thereby perform relatively inexpensive shear tests in the laboratory. More complicated occurrences involving clay and decomposed rock may need larger scale tests and more careful recovery using the method of "integral sampling" described by Rocha (1). An ingenious method of wire sawing described by Londe (2) can be used for recovering larger blocks containing the filled zone. Finally there are large scale in situ tests which are often justified for important dam foundations. Figure 2 illustrates three possible test set-ups which have been used for testing block samples from about 40 cm to 300 cm in length. Such tests are very expensive and time consuming, but justified if the cost of failure is sufficiently high. Unfavourable tectonic shear zones beneath the 100 metre high Krasnoyarsk dam in Russia qualified for 8 m x 12 m tests. Each block weighed at least 1700 tons. Details of these tests – probably the worlds largest – are given by Evdokimov and Sapegin (6).

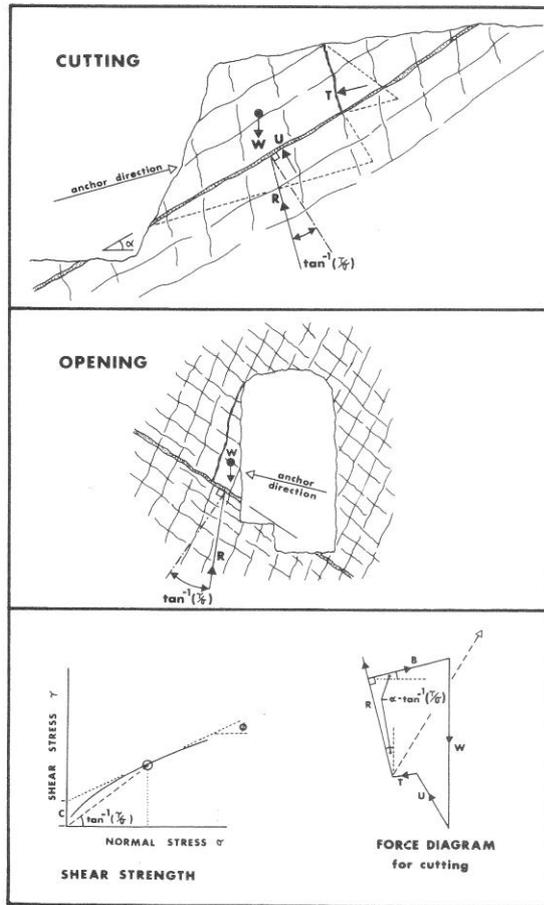


Fig. 19-1 Engineering occurrences of filled discontinuities in which knowledge of shear strength is necessary for the design of optimum reinforcement.

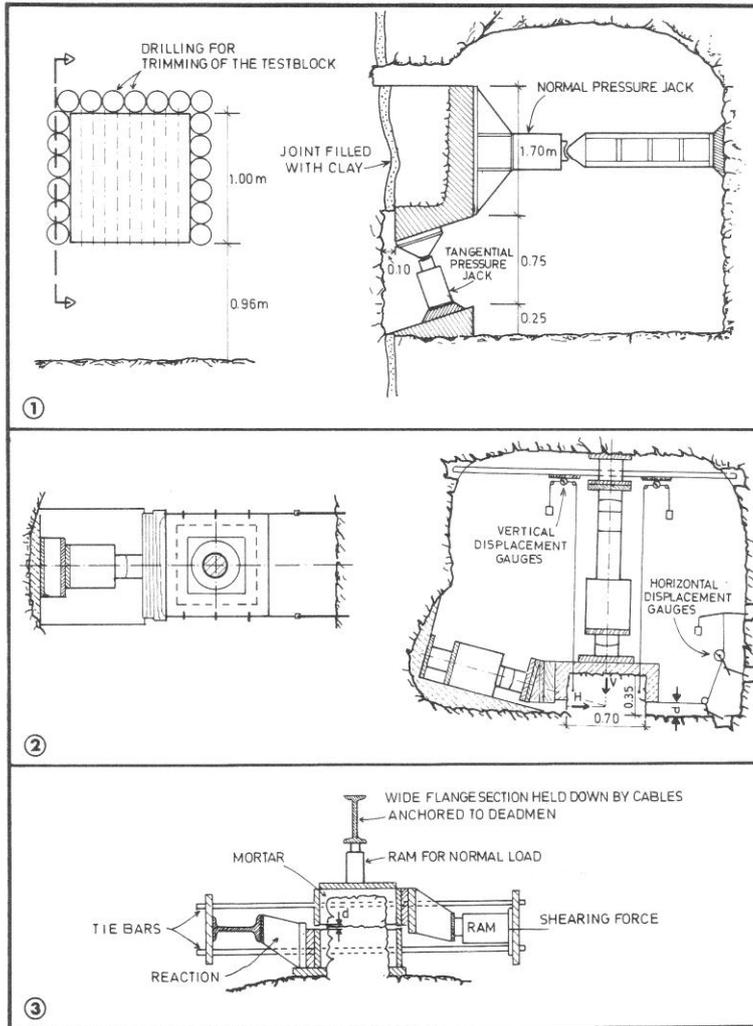


Fig. 19-2 In situ direct shear equipment for testing vertical and horizontal discontinuities exposed in an adit, and for testing a horizontal joint where a reaction frame is needed to apply the necessary forces. After Romero (3), Rocha (4) and Dodds (5).

SLIDING ALONG FILLED JOINTS – SIMPLIFIED GEOMETRICAL EFFECTS

The behaviour of discontinuities that are filled with a thick layer of clay, where there is no complication due to rock/rock interaction, can be largely understood from well tested soil mechanics principles. Complications arise due to fillings that consist of widely graded materials, for instance rock breccia down to clay, and for discontinuities that have a filling no thicker than the amplitude of the wall roughness. Unfortunately, these complications seem to be the rule rather than the exception.

The range of behaviour that can be exhibited by different filled discontinuities is so vast that it is necessary to build up a picture in small steps starting with the simple geometric effects.

If a filled joint is sheared under conditions of no lateral (normal) displacement, there will be no change in the volume of the filling. Closing asperities will be exactly balanced by diverging voids. If, under these conditions, rock contact does eventually occur and dilation is then allowed, there will be an increase in volume which might cause negative pore pressure to be developed unless the rate of shearing is very slow. A thinly filled rough discontinuity may dilate and increase in volume enormously, but this effect will steadily reduce as the thickness of the filling increases. The effect will theoretically disappear when the thickness of the filling just exceeds the amplitude of the largest asperity.

Figure 3 is an idealized picture of a rough, undulating joint that has four hypothetical thicknesses of clay filling. The shear characteristics of these four examples can be grossly simplified as follows:

- A. Almost immediate rock/rock asperity contact. Shear strength will be very little different from the unfilled strength because the rock/rock contact area at peak strength is always small. Normal stresses across the contact points will be sufficiently high to dispel the clay in these critical regions. Slight reduction in dilation component of peak strength may be more than compensated by "adhesive" action of the clay in zones which would be voids during shear of the unfilled joints. Dilation due to rock/rock contact will cause negative pore pressures to be developed in filling if shearing rate is fast.
- B. May develop same amount of rock/rock contact as in A, but required displacement will be larger. Dilation component of peak strength greatly reduced since new position of peak strength is similar to position of residual strength for unfilled joints. Similar "adhesion" effect as in A. Less tendency for negative pore pressures due to reduced dilation.
- C. No rock/rock contact occurs anywhere, but there will be a build up of stress in the filling where the adjacent rock asperities come closest together. If the shearing rate is fast there will be an increase in pore pressure in these highly stressed zones and the shear strength will be low. If on the other hand the shearing rate is slow, consolidation and drainage will occur, the drainage being directed towards the low stress pockets on either side of the consolidating zones. The net result will be a marked increase in shear strength as compared to the fast shearing rate.
- D. When the discontinuity filling has a thickness several times that of the asperity amplitude, the influence of the rock walls will disappear. Provided the filling is uniformly graded and predominantly clay or silt the shear strength behaviour will be governed by straight-forward soil mechanics principles.

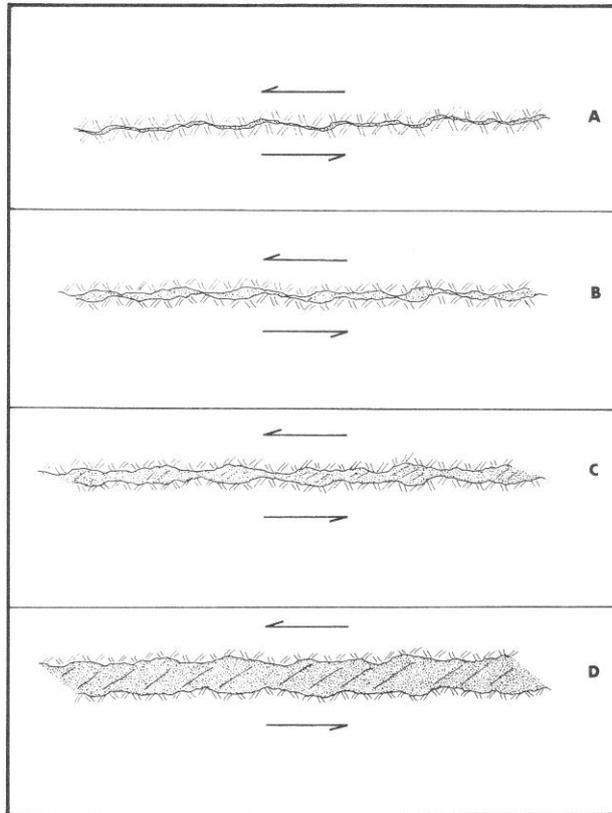


Fig. 19-3 Four categories of discontinuity filling thickness.

Rocha (4) has suggested that if the stresses present in the joint filling are very low in comparison with the overburden stresses, it can be safely assumed that the influence of the filling material on the shear strength is slight. He has also observed that joints are frequently closed in certain lengths and filled with weak material in others, and has suggested that this intermittent type of filling need not be taken into account if it does not occur continuously for a considerable proportion of the foundation surface. In other words, in such cases, in situ test blocks would be prepared where the joints were closed and the filled pockets would be ignored. These observations appear to be valid in principle, but in practice would require a very thorough investigation programme, perhaps to the extent of tracing partially filled joints by means of galleries.

OCURRENCES OF FILLED JOINTS AND INFLUENCE OF DISPLACEMENT AND LOADING HISTORY

The simplified geometrical effects discussed above take little or no account of the real complexity and range of occurrence of filled joints. To understand the significance of all the different varieties it is again necessary to simplify greatly, but on a more realistic level.

To start with it is convenient to divide the numerous varieties of filled joints into two parts: Those that have suffered earlier shear displacement and are therefore at, or near residual strength, and those that have not been displaced.

The first group of occurrences involving previous displacement is typified by faults, old slide surfaces in rock masses, shear zones, clay mylonites, and bedding plane slips. These last two occurrences probably involved slip across discontinuities which were already clay-bearing and merely represented weak horizons during folding or gravitational sliding during basin formation. In contrast, the faults and prehistoric slide surfaces probably became filled with breccia and gouge during the sliding process itself. Subsequently, the finer material may have been altered to clay with particle orientation more or less parallel to the shear planes due to the favourable stress system. These weak zones have probably suffered many periods of displacement as a result of subsequent stress changes. In all these cases one must assume for design purposes that sufficient shearing has occurred for favourable clay particle orientation and for smoothing of rock protrusions. Close to the surface there may be instances where silty-clay materials have subsequently been washed into voids as a result of near-surface weathering. These zones will obviously not be at their residual strength. Nevertheless, the shear strength of the whole will be low, particularly in view of the additional softening that may occur due to increased water content.

The second group of occurrences, specifically those involving no previous displacement, is typified in sedimentary rocks by alternating beds or seams of clay and weak rocks such as shales, sandstone or limestone. In igneous and metamorphic rock masses there will be many varieties of filling as a result of alteration, for instance the alteration of a diabase dyke to amphibolite and finally to clay, and the alteration of feldspar to clay. A very large group of filled discontinuities can be broadly described as hydrothermally altered. Sometimes these will be the end product of hydrothermal alteration of an old fault zone, and a weak alteration product such as montmorillonite will be found within a crushed zone perhaps several metres wide. In other cases the hydrothermal alteration product may be relatively strong and the zone may also be quite narrow, as typified by quartz or calcite filled joints. The formation of clay particles as a result of hydrothermal alteration would seem unlikely to be accompanied by preferential particle orientation, due to the probable hydrostatic nature of stresses during crystallization. As regards near-surface excavation, there are two further potential classes of undisplaced clay/rock interfaces which can have an important influence on stability. Both are a direct result of surface weathering. The finer products of weathering such as silty clays may be washed into open, water conducting joints and gradually precipitate as a weak unconsolidated clay with a correspondingly high water content. In other cases the products of weathering may remain in situ and result in a weak interface between two dissimilar rocks which are differently weathered. Patton and Deere (7) have discussed several practical examples and their adverse influence on rock slope stability.

The division of occurrences into displaced and undisplaced categories is illustrated in Figure 4. An additional and very important category is also shown: normally- or over-consolidated. If previous displacement has occurred then the weakened state will be

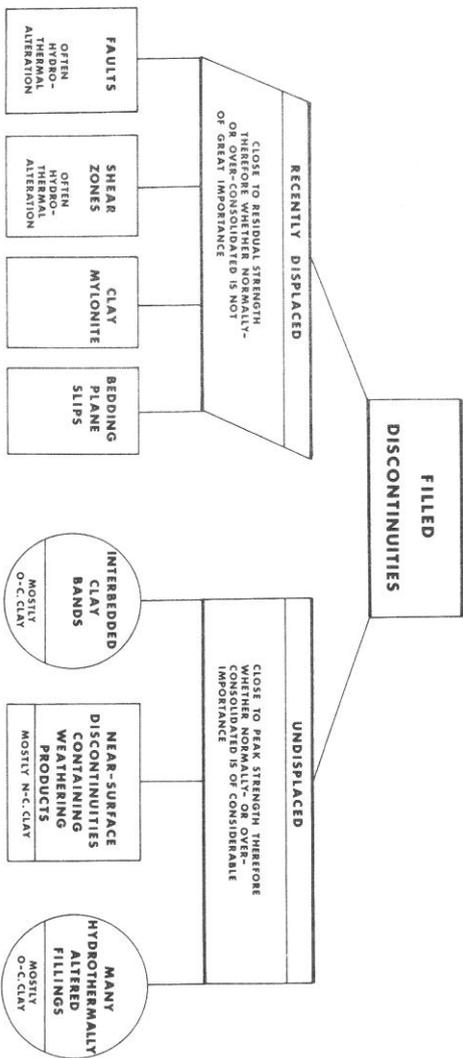


Fig. 19-4 Simplified division of filled discontinuities into displaced and undisplaced, and normally- and over-consolidated categories.

relatively easy to detect or recognise due to the softening and particle orientation that has occurred. In the actual shear zone the cohesive bonds associated with over-consolidation are destroyed, and this remoulded state is equivalent to the normally consolidated state as far as residual strength is concerned.

If the fillings are in an undisplaced condition then the differences in strength between the normally- and over-consolidated state may be very large. The degree of consolidation of many clay fillings probably exceeds even the most over-consolidated deposits that are familiar in soil mechanics practice. Therefore the real danger lies in the under-estimation of softening, swelling and pore pressure changes that are possible when unloading occurs. The high initial strength exhibited immediately after excavation can seldom be used as a design for long term stability.

The significance of the above categories: displaced, undisplaced, normally-consolidated and over-consolidated can be largely understood from soil mechanics principles. It seems essential that these principles should be followed before embarking on a programme of shear strength investigation. Consequently the next section of this paper is devoted to a brief summary of those soil properties that would seem to have most significance to methods of estimating or testing the shear strength of clay filled discontinuities. Complications arising from rock wall or rock fragment interaction will be discussed after this section on soil properties.

PRINCIPLES CONCERNING THE SHEAR STRENGTH OF SOIL FILLINGS

The shear strength of plastic filling materials is contributed by a complicated interaction of mineralogy, particle size and loading history together with various physico-chemical effects such as the composition of the pore fluid and the nature of the absorbed ions. For rock mechanics purposes a rough guide to the range of shear strength may be obtained from the following index quantities:

1. Degree of over-consolidation (if any)
2. In situ water content ($w\%$)
3. Plasticity index (= liquid limit – plastic limit) ($I_p\%$)
4. Clay fraction ($\% < 2\mu$)

The measured strength will depend on all these quantities together with several external experimental factors of which the following are perhaps the most important:

- (i) The degree of sample disturbance and the stress condition within the test sample in relation to the in situ stress and anisotropy condition.
- (ii) Degree of drainage or volume change allowed during test.
- (iii) Rate of shearing in relation to the field problem.

(a) Mineralogy and particle size

Horn and Deere (8) and Kenney (9) have made detailed investigations of the shear strength of rock forming minerals, and of the influence of mineral composition on the shear strength of natural soils. Kenney was able to conclude that mineral composition was even more important than the grain size or the plasticity characteristics of the soil. For the case of

pure minerals the following ranges of drained, residual angles of friction were obtained from tests conducted at effective normal stresses from 0.2 to 8.0 kg/cm²:

1. Massive minerals: quartz, feldspar, calcite: $\phi'_r = 29^\circ - 35^\circ$
2. Micaceous minerals: mica, muscovite, hydrous-mica and illite: $\phi'_r = 17^\circ - 26^\circ$
3. Montmorillonitic minerals: Na-, Ca-: $\phi'_r = 4^\circ - 10^\circ$

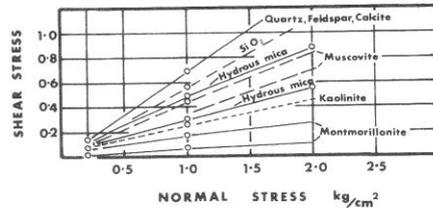


Fig. 19-5 Dependence of residual shear strength on mineralogy, after Kenney (9).

The range of values obtained by Kenney for pure minerals is reproduced in Figure 5. The residual strength of natural soils was found to be strongly dependent on the relative amounts of the above minerals. For instance, soils containing large amounts of montmorillonite or mixed-layer minerals containing montmorillonite exhibited small values of ϕ'_r . Soils containing large quantities of massive non-clay minerals, small quantities of the montmorillonite minerals, and large quantities of the clay minerals of the mica family exhibited the highest values of ϕ'_r . The mineralogic control of shear strength is also reflected in part by the clay fraction of a given discontinuity filling. The range of values of residual strength obtained from a variety of normally- and over-consolidated clays is shown in Figure 6. When the clay fraction drops to zero and the soil is more or less a silt or sand, the friction angle corresponds to that of the massive minerals. On the other hand, when the clay fraction

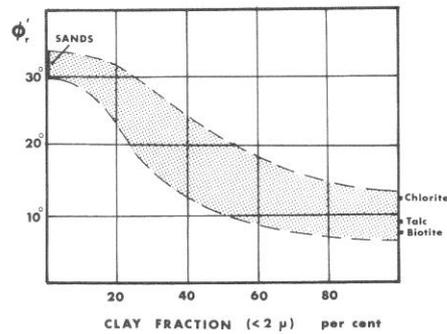


Fig. 19-6 Dependence of residual shear strength on clay fraction, after Skempton (10).

is very high, the strength corresponds to Horn and Deere's (8) results of the minerals chlorite, talc and biotite which Skempton (10) compared with the clay minerals illite and kaolinite.

In many cases a discontinuity filling may consist of particles of clay, silt and perhaps sand, and the coarser particles will tend to increase ϕ'_r above that of the clay particles. The full orientation of the clay particles will be inhibited and the coarse particles may contribute some measure of their own higher strength to the overall value. Terzaghi and Peck (11) gave the following range of effective friction angles for silts, sands and gravels for effective normal stresses less than 5 kg/cm².

Tab. 19-1 Frictional angles for silts, sands and gravels after Terzaghi and Peck (11).

Material	ϕ' degrees	
	Loose	Dense
Sand, round grains, uniform	27,5	34
Sand, angular grains, well graded	33	45
Sandy gravels	35	50
Silty sand	27–33	30–34
Inorganic silt	27–30	30–35

The residual shear strength will be almost independent of the history of loading of the discontinuity. The residual shear strength obtained from drained tests on a clay in a normally-consolidated or over-consolidated state will be more or less identical since the residual state is by definition the strength along a slip surface in which the clay particles are completely reorientated, with all original cohesive bonds broken. However, the history of loading has an enormous influence on the peak strength of clays and this aspect will now be summarized.

(b) Normally- and over-consolidated clay fillings

A discontinuity filling is normally-consolidated if the existing effective normal stress in situ (σ'_{n0}) equals or exceeds the maximum effective pre-consolidation pressure (p_c) that the filling has ever been subjected to. The filling is over-consolidated if (σ'_{n0}) is less than (p_c). It is probable that almost all discontinuities, filled or unfilled, will be in an over-consolidated condition when exposed at the surface. There may be an additional over-consolidation effect between the stages "undisturbed" and "post-construction", since the undisturbed in situ effective normal stress (σ'_{n0}) may exceed the post-construction effective normal stress (σ'_{n1}), particularly in the case of slope excavation.

In the terms of the present discussion, the only filled discontinuities that are likely to be normally-consolidated are those resulting from surface weathering processes, as described earlier. The relative strengths of normally- and over-consolidated clays have been described by Skempton (10), and are reproduced in Figure 7. The idealized peak and residual strength envelopes are in terms of effective stress, so are equivalent to the results that would be obtained from slow, drained shear tests.

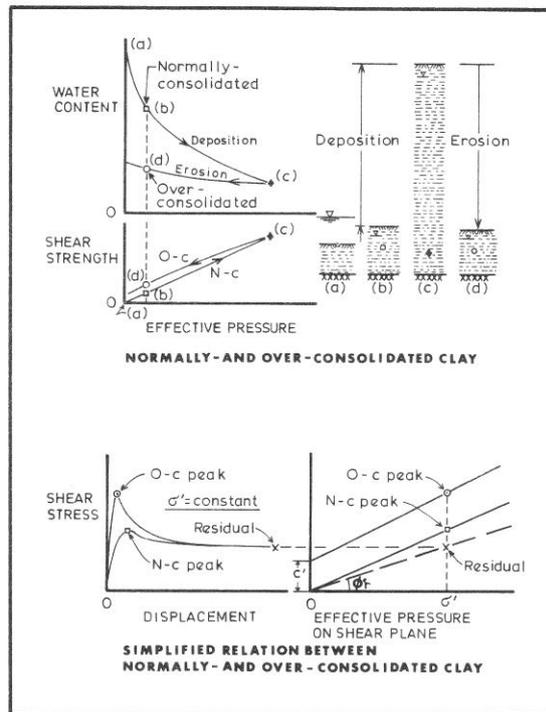


Fig. 19-7 Origin and strength effects of normally- and over-consolidated clay, after Skempton (10).

(c) The effect of drainage on the shear strength of clay fillings

Drained or undrained tests are employed for effective stress or total stress analyses respectively. In all testing the aim is to simulate the field conditions as closely as possible, and consequently, these tests will be used to simulate long-term (drained) and short-term (undrained) conditions. The permeability of a clay filling will generally be very low and consequently, in a true drained test the shearing rate will have to be very slow to prevent the development of pore pressures. If it were possible to monitor the changing pore pressure during an in situ shear test, then this slow testing requirement could be dispensed with, and the total stresses would be simply corrected to the effective stresses, by subtracting the measured pore pressures.

Drained strength

The drained strength of normally and over-consolidated clays were illustrated diagrammatically in Figure 7. Over a limited range of effective stresses the peak and residual strength can be represented by linear Coulomb envelopes:

$$\text{peak } \tau = \tau' = c' + \sigma'_n \tan \phi' \quad (\text{i})$$

$$\text{residual } \tau = \tau_r = c_r' + \sigma'_n \tan \phi_r' \quad (\text{ii}) \quad (2)$$

It should be noted that the value of (c') is not a constant since it depends on the rate of shear.

For over-consolidated clays both (c') and (ϕ') depend on the range of effective stresses, as shown for heavily over-consolidated clay by Bishop, Webb and Lewin (12). Values of (c') between approximately 1.3 and 8.0 kg/cm² and values of (ϕ') between 26° and 10° were obtained between the low stress and high stress portions of typical Mohr envelopes for this clay. The strongly curved shape of the peak strength envelopes is of exactly the same character as that for rough undulating joint surfaces in extremely weak rocks. For low ranges of effective normal stress below about 5 kg/cm² the values of (c') are frequently in the range 0.1 to 0.6 kg/cm² according to results reported by Skempton and Petley (13). However, it seems possible that very heavily over-consolidated filled joints which have suffered tectonic stressing could have significantly higher cohesion intercepts. At present there does not appear to be any reliable data, devoid of the complicating influence of rock wall interaction.

The value of (c_r') denoting residual strength is normally very small or zero. Thus, in moving from peak to residual strength in over-consolidated clays the cohesion intercept disappears completely, and the angle of friction decreases, in some cases by only 1° or 2°, but in others by up to 10° or 11°. Over-consolidated clays show this marked difference between peak and residual strength due to reorientation of clay particles within narrow bands next to the shear surfaces, and also due to the dilation accompanying shear, which, in a slow drained test allows an increase in water content and consequent softening to occur. These effects increase with clay content and the degree of over-consolidation (Skempton and Hutchinson (14)). Soft silty clays may show little difference between peak and residual strength.

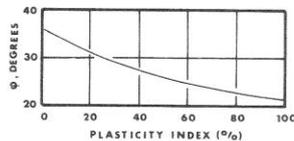


Fig. 19-8 Approximate relation between ϕ' and plasticity index for normally-consolidated clays under drained conditions, after Terzaghi and Peck (11).

For normally-consolidated clays both the peak (c') and the residual (c_r') Coulomb parameters are zero for all practical purposes. This in no way implies that the clays are non-cohesive in nature. The real strength components are to some extent misrepresented by simple Coulomb parameters. The value of (ϕ') is strongly related to the plasticity index (I_p),

as shown by Figure 8, which is taken from Terzaghi and Peck (11). However, the scattering may be greater than 5° and will be very dependent on the rate of shearing during the drained test, particularly for very plastic clays. (Time effects will be discussed shortly). Soft silty clays may show little difference between peak and residual strength, but with higher clay contents the difference tends to increase. Skempton (10) suggested that the residual strength of both normally- and over-consolidated clays would be the same at any given effective pressure, depending only on the nature of the particles. The variation of (ϕ_r) with clay fraction shown in Figure 6 applies to both normally- and over-consolidated clays.

Undrained strength

The undrained shear strength of an over-consolidated clay may be smaller or larger than the drained strength depending on the value of the over-consolidation ratio. If it lies between 1 and 4 or 8 (which would be unusually low for an over-consolidated discontinuity filling), the volume tends to decrease during shear and the undrained strength is less than the drained strength. However, for values of the over-consolidation ratio greater than this, the clay tends to increase in volume during shear, the pore pressure correspondingly decreases and the undrained strength exceeds the drained value. We can therefore expect the short-term stability of slopes excavated above heavily over-consolidated filled discontinuities to be greatly superior to their long-term stability. The strong negative pore pressures associated with unloading would tend to draw water into the discontinuity fillings, causing the filling to soften and swell, whereupon the strength would steadily be reduced. This aspect was discussed earlier in connection with the two anchoring examples.

The undrained shear strength of a normally-consolidated clay is lower than the drained strength because of a decrease in volume during shear. However, as a result of slope excavation there is a tendency for negative pore pressures to be developed, which take time to dissipate. Consequently, although in the case of *loading* the short-term stability of clay-filled discontinuities that have soft normally-consolidated fillings will be critical compared to the long-term condition, in the case of *unloading* due to slope excavation, the long-term drained condition is critical. Thus for both normally- and over-consolidated fillings the long-term drained condition seems to be most critical for design of rock reinforcement. This generalization should apply to all cases of unloading caused by excavation of slopes or large openings. However it may not apply to dam foundations.

(d) Effect of shearing rate on the shear strength of clay fillings

The influence of shearing rate on the shear strength of clay-filled discontinuities is not only associated with the reduced possibility of drainage when the rate is too fast, or on the softening process that accompanies the slow shearing of heavily over-consolidated fillings. There is a fundamental shearing rate effect due to the viscous nature of the cohesive component of shear strength. Bjerrum (15) has given a detailed description of this phenomenon for the case of soft, plastic, normally-consolidated marine clays. It appears that the rate effect is a function of the time required to reach the critical shear strain at which failure will take place. A clay filling subjected to a high shear stress will show a very high rate of straining, resulting in an early shear failure, but at a relatively high stress level. To each stress level applied to a clay filled discontinuity, there corresponds a length of time over which the filling is able to sustain the stress before shear failure will occur.

Tests to investigate the effect of time to failure on the *undrained* strength of a plastic marine clay, indicate that in the range between a few minutes and a few days to failure, the

shear strength falls approximately 10% with each ten-fold increase in time. If undrained tests were performed to estimate short-term stability of a clay filling under increased loading, then this order of magnitude correction would obviously need to be applied, at least for the case of soft, plastic normally-consolidated clay fillings. The actual magnitude of the correction is dependent on the plasticity index of the clay.

The effect to time to failure on the drained shear strength of clays is little known, due to the experimental difficulties arising from the consolidation and decrease in water content that is likely to occur during the few days duration of each test. Bjerrum (15) has suggested that during unloading, for instance that resulting from slope excavation above clay-filled discontinuities, the effect of time will be of the same order of magnitude for drained as undrained tests. It also appears from the limited data available that the time effect will operate on both normally- and over-consolidated clay, since the cohesive property (plasticity index) is the controlling parameter, and its value is fundamentally the same for the normally-consolidated and over-consolidated states.

The slow drained tests recommended for both normally-consolidated and over-consolidated clay fillings will induce failure in a time period that is several orders of magnitude smaller than the so called "long-term". Based on limited data, Skempton and Hutchinson (14) suggested the use of a 10 to 15% reduction in drained strength for long-term conditions. The same authors reported only a small influence of rate of shear on the residual strength of cut planes in over-consolidated clays, amounting to 0.5% to 2% decrease in strength per log cycle of time. The rates of shear employed in these tests ranged from 20 cms/day down to 2 cms/year. In general, all the above rate effects depend on the plasticity of the clay, in fact on the cohesive component of shear strength. A discontinuity filling having a high content of silt and coarser material may not exhibit any appreciable rate effect, due to the essentially frictional strength.

(e) Sample disturbance and anisotropy

When taking "undisturbed" samples of clay from filled discontinuities, or when preparing test blocks for direct shear tests on the rock/clay/rock sandwich, the in situ total normal stress (σ_{n0}) will be temporarily reduced — probably to zero. If the clay fillings are saturated, as is likely, this reduction in stress will cause negative pore pressures to be developed. If the clay is in an over-consolidated state and of the swelling variety then all available moisture will tend to be absorbed, both from the atmosphere and from the outer disturbed zone which will probably have been compressed during sampling. The net result is that the water content will tend to be higher within the sample than in the undisturbed state in situ. In his review of these problems, Bjerrum (15) laid great emphasis on the necessity for reconsolidating samples before testing:

1. to replace the field stresses with an identical set of effective stresses in the sample,
2. to squeeze out the additional water absorbed since sampling.

In problems involving unloading caused by slope excavation, softening and swelling due to water uptake will probably occur in practice, and should therefore be allowed to occur before shear testing. When preparing test blocks or samples of fillings containing swelling clays, the design stresses should be applied as soon as possible after the unloading. If in practice it is considered that the excavated slope can be so well drained that access to additional water can be prevented, then shear testing should be performed as soon as possible to retain the undisturbed state. This optimism may not be justified in many

situations, in which case the most realistic simulation of in situ conditions will be to allow the sample to swell under the design stresses with free access to water. The sample can then be sheared drained or undrained according to the design method employed.

An additional factor in the disturbance of clay samples is the strength anisotropy that has been observed in undrained in situ tests on clays of low plasticity. Samples reconsolidated anisotropically before shear testing also show similar anisotropy, which cannot be reproduced by the normal method of reconsolidation.

If possible the samples should be aligned in a direct shear machine in the relevant direction, and the filling reconsolidated at the normal and shear stress that it carried before sampling — if these can be estimated from stress measurements or overburden depth. The sample can be brought to failure by reducing the normal stress and increasing the shear stress to simulate slope excavation under drained conditions. If, after reconsolidation, the filling is sheared in the same direction as the in situ stress condition (active case) as recommended, the strength will be maximum and the failure strain about half of that in the opposite direction (passive case). Strength anisotropy ratios (active/passive) of at least 3 have been measured on clays having low plasticity.

The difference in strength between the "active" and "passive" directions is caused mainly by the disturbance of the structure in reversing the stresses. The clay particles are orientated to resist shear in the "active" direction but not in the "passive". It seems probable that anisotropy effects will be active in drained as well as undrained tests, and in over-consolidated as well as normally-consolidated clays, though relatively little is known about these effects at present.

The importance of possible strength anisotropy in filled discontinuities is that it may explain certain inconsistencies that can arise. It may be quite impossible for rock mechanics investigators to estimate the undisturbed stress condition in the discontinuity fillings, even with the advantage of adjacent rock stress measurements. Consequently, knowledge of anisotropy will merely give the investigator a healthy suspicion of the accuracy of test results.

(f) Residual strength condition of sheared discontinuity fillings

The widespread occurrence of previously displaced filled discontinuities was discussed in the introduction. Heavily over-consolidated fillings that have been sheared at some time in the past, for instance tectonic shear zones, may be at or very close to residual strength. For clays of low plasticity having a small clay-size fraction, the residual strength which is represented by ϕ_r' may be as high as 30° . However, highly plastic clays having a large clay-size fraction may have ϕ_r' as low as 5° to 12° (Terzaghi and Peck, 11). Because of the nearly complete destruction of the structure of the natural clay along the surface of sliding it is likely that the value of ϕ_r' will be the same irrespective of the stress history. The residual strength of an over-consolidated filling could therefore be estimated with sufficient accuracy by residual tests on the remoulded material, taking care not to include any of the coarser materials in the filling.

The residual shear strength of an artificial cut surface would be represented by curve (1) in Figure 9, which is reproduced from Skempton and Petley (13). The shear strength of a slip surface in situ may however exhibit a small peak (2) possibly due to a non-planar surface, or incomplete particle orientation, or a slight "bonding" effect. However, the true

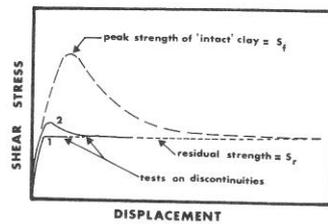


Fig. 19-9 Stress-displacement curves for tests on a discontinuity and on intact clay, after Skempton and Petley (13).

residual strength of intact samples, cut samples and slip surfaces will be essentially the same when sufficient displacement has occurred, as shown in the figure.

The recent descriptions of tectonic shear zones in over-consolidated beds of clay described by Skempton (16) and Fookes and Wilson (17) can be compared to thickly filled discontinuities that have suffered earlier displacements. When the clay fillings are sufficiently thick relative to the amplitude of roughness of the rock walls, multiple slip surfaces (displacement shears and Riedel shears) can form almost unhindered, and the smooth polished slip surfaces will be at or close to the residual strength of the clay. The combined effect of the various sets of slip surfaces illustrated in Figure 10 is to divide the shear zone into numerous lenses, whose size will depend to some extent on the thickness of the clay band. The shear zones discovered during the Mangla Dam Project, in heavily over-consolidated clay beds, were up to 50 cms thick and the result of folding of the inter-bedded clays and sandstones.

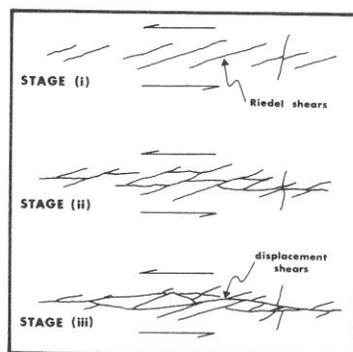


Fig. 19-10 Successive stages in the development of a shear zone in clay, after Skempton (16) and Skempton and Petley (13).

Stimpson and Walton (18) described a small scale variation of these shear zones in English Coal Measure rocks. In this case the clay bands were mostly only 1.0 to 2.5 cms thick. The bands were not at residual strength, but due to the presence of numerous lenses and small shear planes, the residual angle of friction ϕ_r' was only 11° , despite the relatively low plasticity of 21%.

(g) Swelling clay and the problems caused by unloading

Discontinuities that are filled with the products of hydrothermal alteration can display a very wide range of properties. The exceptionally low strength of some of the zones crossing underground constructions frequently causes slides and roof falls.

Brekke and Selmer-Olsen (19) listed several causes of instability of which the following are a direct result of the filling material:

1. the especially low cohesion in joints and fissures carrying chlorite, talc, graphite etc.,
2. the solubility of calcite, especially when the calcite is porous or flaky,
3. washing out and downfall of clayish materials from joint and fault fillings,
4. the swelling capacity of montmorillonite clay minerals.

It appears that montmorillonite can occur in several different ways:

1. as a filling in joints and faults, being strictly confined to the cracks,
2. as an alteration product of feldspar in the side rock of joints and faults,
3. as an alteration product of mica, with thick crushed zones containing montmorillonite and carbonate of high porosity.

It appears that in all occurrences the montmorillonite is saturated, but because of the wide variations in stress, a considerable range of water contents is found. Brekke and Selmer-Olsen (19) laid particular emphasis on the danger of evaluating the stability of montmorillonite occurrence from the dry condition, if there could ever be access to water at a later date.

From the point of view of ease of testing (in direct shear or swelling), it is unfortunate that montmorillonite seldom occurs alone. Rock powder and fragments of the side rock are usually present, together with one or more of the secondary minerals: carbonates, quartz, pyrite, chalcocopyrite, pyrrhotite, chlorite, talc, serpentine, epidote, asbestos, graphite, zeolite, kaolinite, vermiculite. It is also frequently the case that montmorillonite is locally concentrated in an area, with decreased content on moving away from the area. In addition, younger or older discontinuities crossing a montmorillonite-bearing discontinuity may not necessarily contain any swelling clay. Consequently, any in situ tests performed may only be relevant locally, and uncertainties in the design strengths will be unusually high.

When a discontinuity containing swelling clay is partly unloaded and has access to water a marked softening will occur. In underground openings a swelling pressure may develop due to the unyielding surroundings. The only direction for yield to occur is into the opening, and blocks may be sufficiently loosened for fall-out to occur, even though the swelling pressures may be dissipated after initial loosening. Swelling tests reported by Bjerrum et al. (20) demonstrated that swelling pressure decreased rapidly when even a small increase in volume was allowed. For example, three specimens that developed swelling pressures between 1.6 and 3.6 kg/cm² when zero increase in sample height was allowed, showed swelling pressures of only 0.1 to 1.0 kg/cm² when a 5% increase in sample height was allowed.

In recognition of this displacement-unloading effect, security measures in swelling clay zones are designed to allow some swelling to occur behind the support. In the case of tunnels crossed by swelling clay fault zones a design allowing the clay to swell outwards 5% of the fault width has been suggested by Eurenus (21).

The significance of swelling clay occurrences beneath surface cuttings is somewhat different from that around tunnels, since the overlying rock is relatively unconfined and normal displacements can occur without necessarily reducing the stability. The main factor causing instability is the softening and reduction in cohesion.

When a slope is excavated above a clay filled discontinuity there will be a strong tendency for negative pore pressures to be developed from two causes. Firstly, the total normal stress will be reduced with a tendency for limited expansion which is resisted initially by the pore water. Secondly, if the clay filling is heavily over-consolidated and additional shear stress is thrown onto the joints as a result of the excavation, there will be a further tendency for negative pore pressures to develop since heavily over-consolidated clays tend to expand during shear. Such a situation is illustrated in Figure 11.

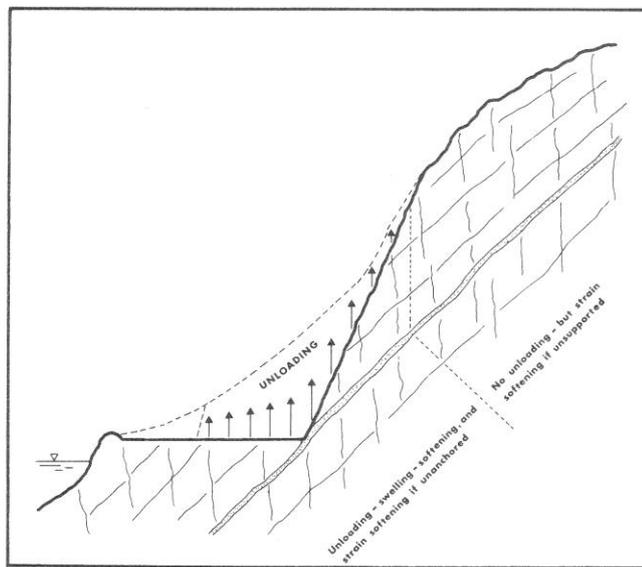


Fig. 19-11 Excavation above a clay filled discontinuity is likely to cause a long term stability problem, particularly if the filling is initially over-consolidated.

The strong negative pore pressures associated with the short-term stability will tend to draw any available water into the unloaded zone. The consequences of this process will be seriously accentuated if the filling contains significant amounts of swelling clay such as montmorillonite. Not only will the clay gradually soften and swell. The dissipating negative pore pressures will cause a reduction in the effective stress with time as the steady seepage condition is approached.

The best engineering solution to this problem is debateable. The zone of swelling and softening associated with the unloading could be stabilized by anchoring as outlined in Figure 1, using a conservative estimate for the shear strength. In view of the uncertainties it would probably be wise to allow only for residual strength, and to make an additional time correction for long-term strength as discussed earlier.

It is very difficult to imagine how the clay filling higher up the hillside will react to the cutting lower down the slope. It is probably best if the anchoring is installed as soon as possible during or after excavation, so that the high initial strength is not dissipated. Increased shear strain under higher parts of the slope may then be avoided. This in turn would help to prevent softening higher up the slope. Therefore a successful engineering solution calls for early recognition of the presence of the clay filled discontinuity.

INTERACTION BETWEEN ROCK WALLS AND FILLING MATERIAL

The majority of direct shear tests performed in situ are used to investigate the shear strength of weak zones, typically clay-filled discontinuities. Despite this relatively large body of data and the acknowledged complexity, there appears to be a marked gap in contributions from the research side. Undoubtedly, this is partly due to the experimental problems. A particular difficulty, and one referred to by Drozd (22) is that soft plastic fillings tend to be squeezed out of the joint during the course of a test. The same thing is not likely to occur in practice due to continuous upper and lower rock faces.

(a) Idealized studies of geometrical effects

Tests by Kanji (23) which were referred to by Patton and Deere (7) showed that a smooth rock/soil interface could have a lower shear strength than the soil tested alone. Artificially sawn and polished surfaces of limestone were used to represent the rock surfaces, and remoulded kaolinite-, illite- and montmorillonite-rich soils were used for the fillers. The test conditions simulated by Kanji are not unlike those present in nature between fault gouge and a slickensided fault surface. For less polished rock surfaces such as sawn surfaces, the effect was less marked. It therefore seems probable that shear tests that are performed on a clay filling that has been extracted from a discontinuity will produce a minimum strength value in all cases, except those involving slickensided rock walls.

In a more idealized study reported by Goodman (24), regular sawtooth surfaces cast in a plaster-celite model material were used to represent non-planar rock walls, and crushed mica was used as the filler. The sawtooth geometry is illustrated in Figure 12. Crushed mica fillings of three different thicknesses were used, ranging from approximately 1.5 mm to 5.0 mm. The graph showing shear strength plotted against percent joint filling illustrates the interesting fact that the thickness of the filling needs to be at least 50% greater than the amplitude of the undulations, for the strength of the composite sandwich to be as low as the filler alone. In the case of saturated clay fillings, the reported effect would probably be

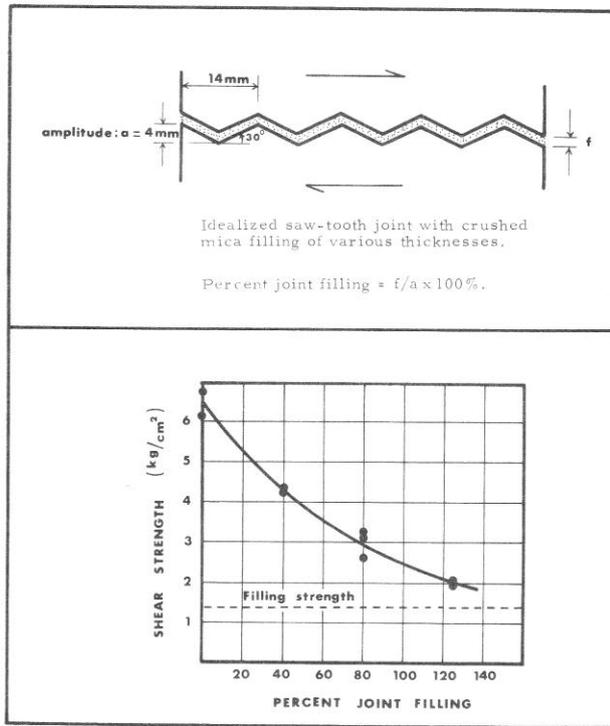


Fig. 19-12 Effect of thickness of filling on the shear strength of an idealized discontinuity, after Goodman (24).

different for fast and slow rates of shear, and also different for normally-consolidated and over-consolidated fillings.

The geometrical effects associated with regular saw-tooth wave forms are obviously quite different from those associated with rough-undulating natural joints which generally have a rather random pattern of troughs and ridges. For instance, for the saw-tooth wave form, rock/rock contact will occur after a horizontal shear displacement of:

$$d = f / \tan i \quad (3)$$

where (f) is the thickness of filling, and (i) the inclination of the teeth. Equation 3 is valid provided the thickness of filling is less than the amplitude of the teeth ($f < a$) and provided there is no volume change up until the instant of rock/rock contact.

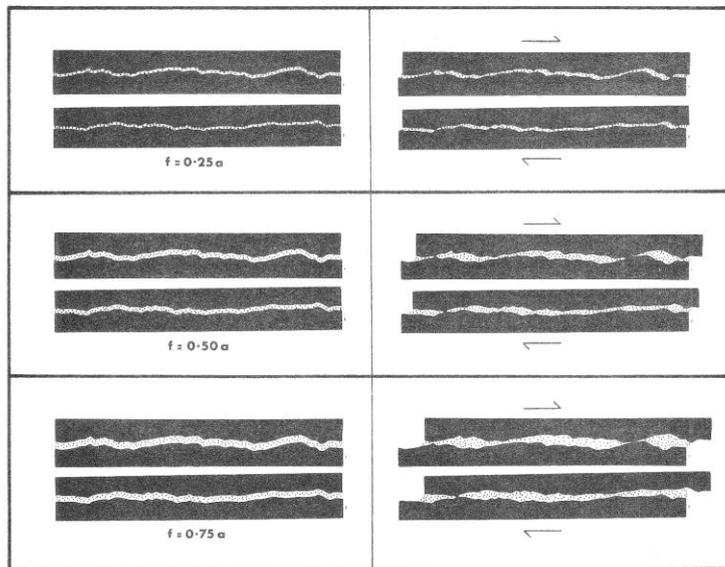


Fig. 19-13 Hypothetical discontinuities containing various thicknesses of "filling" can be used to estimate the displacement required for rock asperity contact to occur, assuming there is no change in volume during shear.

For a natural joint the effective (i) value is not a constant and is dependent on the thickness of filling. Figure 13 illustrates one approach to this problem.

The grey areas represent cross-sections through hypothetical tension joints in rock. These roughness profiles were actually measured from model surfaces by means of photogrammetry, using pairs of overlapping photographs. Between the matching "rock" surfaces are three hypothetical thicknesses of clay filling. These model filled discontinuities were "sheared" under conditions of no volume change, to find out what displacement was required for "rock" contact to occur.

The data tabulated and plotted in section 4 of Figure 14 was obtained from "shear tests" on a number of such profiles. Hypothetical thicknesses of filling (f) equal to 1.0, 0.75, 0.5 and 0.25 times the mean amplitude of roughness (a) were "sheared" under conditions of no volume change, until contact of the asperities occurred. The required distances (d) were averaged between left-handed and right-handed shearing, for a variety of these rough-undulating surfaces.

Application of these results can be illustrated by the following example. Suppose a filled discontinuity was exposed in situ, which over the visible exposure had a mean amplitude (a) of wall roughness of 10 cm. If the mean thickness of filling (f) was 4 cm, then:

$$f/a = 0.4$$

From Figure 14 : $d/a = 0.9$ (approx.)
and : $d = 9$ cm (approx.)

If shearing occurred with no volume change, asperity contact would not occur for the first 9 cm of shearing. In practice the filling would consolidate slightly if

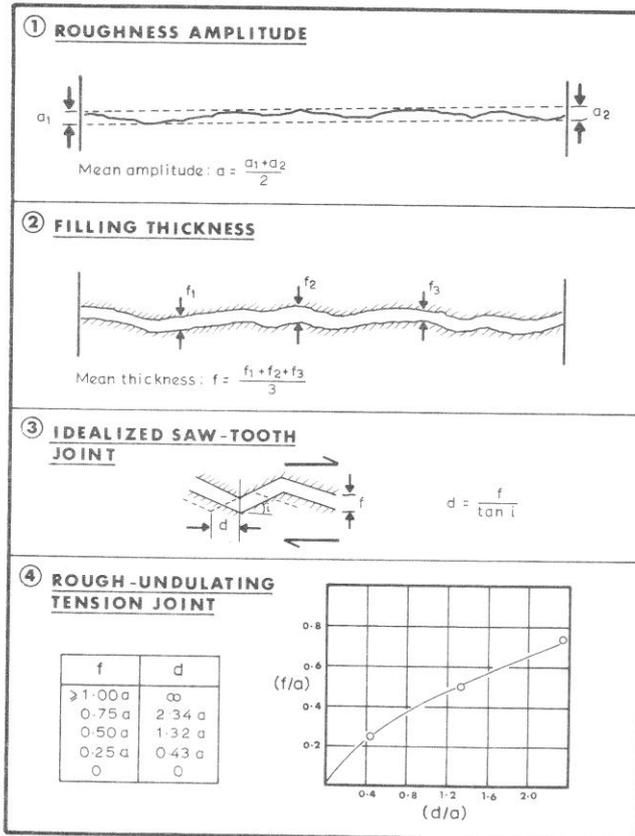


Fig. 19-14 Displacements needed to achieve rock asperity contact when shearing of the filled discontinuity involves no volume change.

normally-consolidated, and dilate if heavily over-consolidated. These effects might be significant if the surface undulations were small. The values obtained from the graph will tend to be minimum values, since when rock/rock contact is approached, the highly consolidated pockets of filling may force the discontinuity to dilate and thereby delay actual contact. In the case of very thin fillings, the small steep asperities come into contact first. As the filling thickness is increased, the flatter slopes of major irregularities are the ones that finally make contact. In the case of these rough-undulating joints, the effective (i) values (equation 3) ranged from 30° for the thin fillings, down to about 18° when the filling thickness was threequarters of the amplitude of roughness.

A series of direct shear tests using real rock joints and remoulded clay fillings is planned at NGI, in order to investigate these relatively simple filled discontinuities. Various degrees of over-consolidation will be simulated. The high initial strength produced by unloading will be compared with the much lower long-term strength simulated in slow drained tests. Various filling thicknesses will be investigated, and a comparison will be made with a parallel series of tests on unfilled rock joints of various roughnesses.

(b) Engineering examples of geometrical effects

Observations of geometrical effects obtained in practice are described by several authors, and some of these will be briefly reviewed. Firstly, on the subject of smooth rock walls, Eurenus and Fagerström (25) found that the contact zone between bentonitic clay layers and chalk walls could be weaker than the internal parts of the clay seam. The tests were performed in situ and were approximately 1 m^2 in area. These observations confirm the findings of Kanji (23) described earlier.

An interesting series of in situ tests reported by Romero (3) were used to estimate the variable shear strength of open clay filled joints, which were vertically orientated in the limestone walls of a steep sided valley. These rough discontinuities, which are typical in cases of decompression in deep valleys, were approximately 10 cm wide high in the valley walls, and reduced to zero at depth. In the context of earlier discussions it is probable that this clay filling resulted from the products of surface weathering and was in a normally-consolidated state. In most of the reported tests on the deeper parts of the discontinuities, there was interlocking between opposing asperities after a certain displacement. Shear force displacement curves showed a marked kink and subsequent stiffening after rock/rock contact occurred.

The displacements needed to develop peak strength in the first test on each block reveals the influence of reducing thickness of clay filling with increasing depth. For normal stresses in the range 2.4 to 5.4 kg/cm^2 , the peak displacements were 3.0, 1.8, 0.9 and 0.5 mm. The discontinuity with the thinnest clay filling and a great deal of rock/rock contact displayed Coulomb strength parameters of $c = 5.5 \text{ kg/cm}^2$, and $\phi' = 49^\circ$. This is broadly similar to the approximate c , ϕ values expected from rough, undulating joints containing no filling.

It is known from model studies (26) that rock contact occurs over a relatively small area for unfilled joints. The stresses at the contact points are so high that if any clay is present it is readily squeezed away from the contacts, if the filling is sufficiently thin. If the filling is thick, these high stress areas may not arise, and consequently there may be no possibility for the thicker filling to be squeezed aside. In this connection should be mentioned the frequent occurrence of higher strength for second and third sliding tests of the same filled discontinuity. Relatively large displacements may be required for rock contact, and the real

peak strength may not be mobilized in the first tests performed because of insufficient displacements.

Some large in situ tests reported by Krsmanovic and Popovic (27) provide some interesting data on the effect of pre-consolidating filled discontinuities in limestone, before shearing under lower levels of stress. Test areas of 280 x 180 cm were used. A test on a discontinuity with approximately 1 mm of a clayey filling can be taken as an example.

A conventional rock mechanics test using no pre-consolidation load produced a shear strength of 8 kg/cm² when the normal stress was 15 kg/cm². When time was allowed for consolidation under 15 kg/cm², the shear strength was 9 kg/cm². When the block was pre-consolidated under a normal stress of 25 kg/cm², before shearing under 15 kg/cm², the shear strength was 12,5 kg/cm². This effect occurred despite slight smoothing of the rock contacts due to continued shear. Before concluding that these results are due to the usual squeezing out of water and consequent strengthening of the filling, it should be noted that the above preconsolidation procedures caused only a slight increase in strength when the filling thickness was in the range 1 to 10 mm, and no increase whatsoever when 10 to 20 mm thick. One must therefore assume that the improved rock/rock contact produced the greatest strength increase. The author noted that the strength increase due to "consolidation" was greatest for discontinuities with rough walls, and least for those with smooth walls. Nevertheless, it is perhaps possible that incomplete consolidation of the filling occurred, and that the water content did not reduce as much as might be expected during the pre-consolidation of the thicker fillings. The length of time of pre-consolidation was not reported.

It may be frequently the case that a discontinuity filling has the consistency of a very dense rock fill with the voids filled with clay and decomposed or crushed rock. Ruiz et al. (28) described this type of situation, and the results of a huge in situ test involving an area of 35 m². The normal stress operating was only 1.7 kg/cm². It is interesting to observe that the "undisturbed" and first sliding tests gave peak values of $\tan^{-1} (\tau/\sigma_n)$ of 69° and 68° respectively, yet the peak dilation angles were 39° and 32°.

The shear strength of a cohesionless material such as sand or gravel can be approximated by an equation of the following type, as suggested by Newland and Allely (29):

$$\tau = \sigma'_n \tan (\phi + i) \quad (4)$$

where σ'_n is the mean effective normal stress acting across the sliding surface,

ϕ is the intergranular friction angle

i is the angle of movement (dilation) with respect to the direction of the applied shear stress

If one can assume that the discontinuity filling tested by Ruiz et al. (28) was behaving like a cohesionless mass and that equation 4 can be applied, then one can say that the *frictional strength* increased from 30° to 36° as a result of continued shearing. Improved rock particle contact would explain such an increase.

(c) Strength effects of grout-filled joints

An important man-made filling which frequently influences the stability of dam foundations is cement grout, which is generally used to reduce the mass permeability of the

foundations. As a side effect it may increase or decrease the shear strength of the joints, the positive or negative effect depending on the joint roughness and on the compressive strength of the rock.

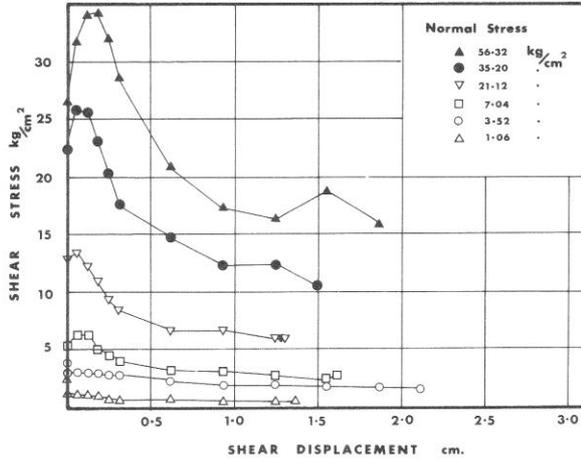


Fig. 19-15 Shear strength-displacement curves for saturated, grouted joints in fine-grained granite, for a grout thickness of 0.8 mm, after Coulson (30).

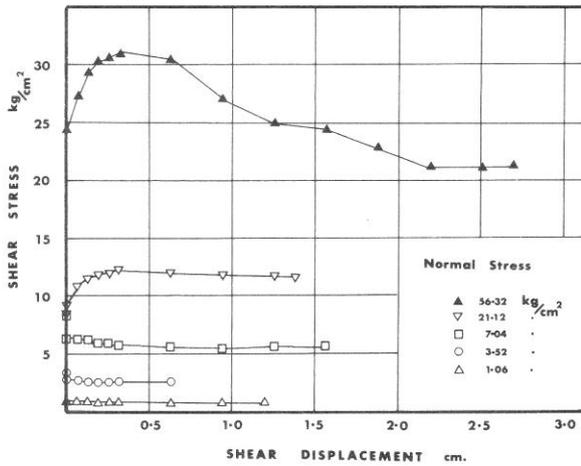


Fig. 19-16 Shear strength-displacement curves for saturated grouted joints in fine-grained granite, for a grout thickness of 3.2 mm, after Coulson (30).

An important investigation on the effects of grout thickness on shear strength has been reported by Coulson (30). Artificial extension fractures in a fine-grained granite were filled with a variety of thicknesses of grout, ranging from 0.8 mm up to 6.4 mm. The effect of grout thickness on the shear strength-displacement curves is clearly illustrated in Figures 15 and 16. The "brittle" type of behaviour for the thinnest grout filling is very similar to that for clean, rough joints. It is probable that in this laboratory study there was rock/grout adhesion across both the upper and lower interfaces, hence the strength at zero displacement.

The relative strength effects of grouted and natural joints in fine-grained and coarse-grained granite are illustrated in Figure 17. In all cases but one, the upper side of each shaded envelope is the strength limit for the coarse-grained granite, and the lower side the

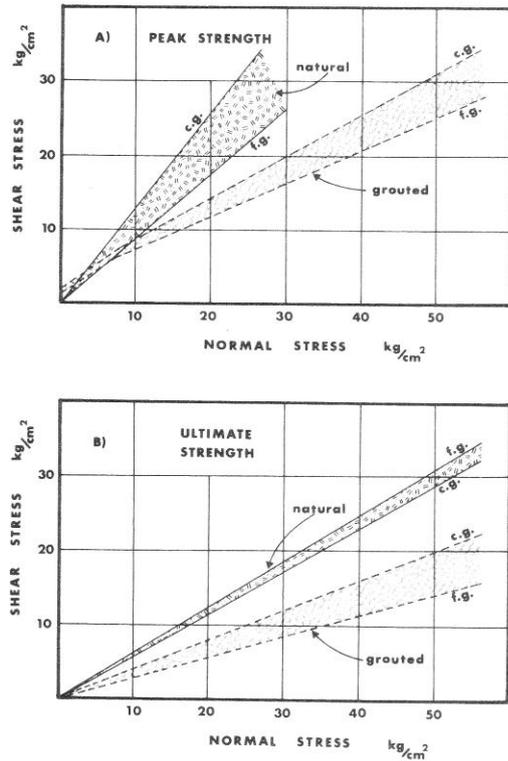


Fig. 19-17 Comparison of peak and ultimate strengths for natural and grouted joint surfaces. The data is for coarse-grained (c.g.) and fine grained (f.g.) granite, after Coulson (30).

minimum strength of the fine-grained granite. The one exception is the ultimate strength of natural joints in fine-grained granite which showed slightly higher strength ($\phi_r' = 32^\circ$) than those in coarse-grained granite ($\phi_r' = 30^\circ$).

The really significant feature of these test results is that except at normal stress levels below about 4 kg/cm², the shear strength of grouted joints is much inferior to those of natural undulating tension joints. At all stress levels the ultimate or residual strength of grouted joints was drastically low, the residual angle of friction, ϕ_r' , ranging from only 16° to 22°. It is possible that the fine powder generated during shearing behaves like a silty clay when saturated, and this would explain the surprisingly low values of residual friction.

An investigation by Barroso (31) showed that planar artificial surfaces in granite would benefit from grouting, at least as far as peak strength was concerned. The coefficient of friction μ rose from 0.47 (25°) to 0.57 (30°) with the addition of grout. However, for natural joints in shale there were few advantages if any from grouting. Values of compressive strength for the cement grouts used in this study ranged from 50 to 460 kg/cm² when cured at atmospheric pressure, and from 130 to 570 kg/cm² when cured under a confining pressure of 30 kg/cm².

If the adhesive strength component is ignored or the normal stresses are greater than a few kg/cm², it is likely that rock walls having strengths in excess of the compressive strength of the grout will suffer a reduction in shear strength. The above ranges of compressive strength which depend on the water/cement ratio, limit improvements by grouting to the very weakest rocks. A further important point is that a rock/grout/rock sandwich will have a shear strength corresponding to the "weakest link". When the value of the compressive strength for the grout exceeds that of the rock wall, the rock will control the shear strength. Consequently, grouting can never be advantageous except where normal stresses are very low, and grout/rock adhesion is guaranteed. These same principles will also apply to joints containing the stronger products of hydrothermal alteration such as calcite.

SUMMARY OF TESTING PRINCIPLES FOR FILLED JOINTS

There are two conventional approaches when applying the results of shear strength measurements to stability analyses:

- (a) effective stress analysis: usually employed for long-term or drained conditions,
- (b) total stress analysis: short-term or undrained conditions.

The long-term condition is separated from the short-term condition by a period of pore pressure redistribution. This may take years in the case of clay-filled discontinuities cutting through impermeable rock, but only a matter of hours or days if the fillings are predominantly sand or silt with little or no clay-size particles.

The loading and drainage conditions that are simulated by shear tests of filled discontinuities, either in the laboratory or in situ, must be relevant to conditions expected in the field during or after excavation. The examples illustrated in Figures 1 and 11 involve unloading and possibly increased shear stresses. Therefore from two sources there may be a tendency for the discontinuity filling to expand. This expansion is resisted by the pore water. The length of time over which this resistance is effective, and the length of time needed for subsequent pore pressure changes will depend on the permeability and loading history of the fillings. The following three simplified conditions of filling are convenient as a basis for discussing the most relevant test method:

1. heavily over-consolidated clay, very impermeable ($< 10^{-10}$ cm/sec?),
2. normally-consolidated clay, probable permeability range 10^{-5} to 10^{-8} cm/sec,
3. decomposed, crushed rock and clay, probable permeability range 10^{-1} to 10^{-5} cm/sec.

(1) The long-term, drained condition is the only one of concern to the first example. This over-consolidated filling will develop strong negative pore pressures when resisting shear in addition to those resulting from the excavation, and this will lead to a slow process of softening and swelling (very marked if the clay is sensitive), and eventually to a reduction in effective stress as the filling becomes more permeable and the long-term drained distribution of pore pressures is established. For this type of situation Skempton and Hutchinson (14) suggested very slow drained tests using the range of effective stresses which will operate in the long-term field condition. If the filling is sensitive, the greatest care will need to be taken to prevent access to water until these effective stresses are applied. Reconsolidation of a free-swelled sample will not reconstitute the original structure. If the stability of a long-term condition is to be "back-analysed", with the clay filling already in its final softened state, then undrained tests can be performed, and a total stress analysis employed.

(2) Normally-consolidated clay, which will occur in open joints or tension cracks that have been filled with the finer products of surface weathering, would normally tend to develop increased pore pressures when the shear stress/normal stress ratio is increased. However, due to the unloading there will be a stronger tendency for negative pore pressures to be developed. Consequently, the long-term condition after drainage occurs is likely to be critical for stability, particularly in view of the time effect. Slow drained tests will be the most relevant approach.

(3) The more coarse-grained fillings containing crushed rock will tend to have the highest permeability. Even if there are initially no voids due to clay filling, there will be the possibility of greatly increased permeability due to geometrical rearrangement when shear strain develops as a result of excavation. The dilation which is likely to accompany shearing will mean that the "long-term" drained condition is likely to be the critical condition for stability analysis. However, due to the increased permeability it may take only a matter of days or weeks for the "long-term" condition to be established. Slow drained shear tests will again be the correct approach.

In all the above cases the shear apparatus should be designed to apply shear stress in the same direction as that applied by the slope excavation. In addition, since the effective design stresses may be lower and in some cases higher than the undisturbed effective stresses, the fillings to be tested drained should be given time to swell or consolidate under these stresses, while having free access to water. Several days should be allowed before applying the shear loads.

APPENDIX*Peak and residual shear strength for filled discontinuities*

In this final section, the reported shear strengths of filled discontinuities are tabulated. A brief description of the filling and rock is given, together with the peak and residual values of c' and ϕ' , and the range of effective normal stress. The assumption is made that all these shear tests were performed at sufficiently slow rates of shear to be considered drained. It is clear that this assumption is rather an optimistic one. Only occasionally do authors give any indication of the time taken over each test. Laboratory tests of extracted clay-gouge fillings lasting only a few minutes, and in situ tests of clay filled discontinuities lasting only a few hours are probably only "partly-drained" tests. It is difficult to imagine how such test results can be used for design, though undoubtedly they often are.

This possible discrepancy should be taken as a warning concerning indiscriminate use of these parameters for other, apparently similar geological environments. Each location would seem to have its own peculiarities, and therefore requires individual in situ testing. The expense involved in testing would be justified if the risk and expense of failure was sufficiently high.

Tab. 19-2 The shear strength of plastic fillings extracted from discontinuities, and related materials, as obtained from laboratory triaxial and direct shear tests.

Rock	Description of filling	c' (kg/cm ²)		φ'		σ _n ' (kg/cm ²)	References
		peak	resi- dual	peak	resi- dual		
weakened granite	sandy-loam fault filling	0.50		40°		<3	Nose (32)
diorite granodiorite and porphyry	clay gouge (2% clay) (P.I. = 17%)	0		26,5°		0-7	Brawner (33)
clay-shale	stratification surfaces		0		19°-25°	0-5	Leussink and Müller-Kirchen- bauer (34)
"paint-rock" soft, fine grained mass quartz, kaolin, pyrolusite	saturated and remoulded to in situ void ratios: e = .56 (triaxial tests): e = .49	0.42 0.91		36° 38°		?	Coates McRorie and Stubbins (35)
slates: finely laminated and altered	saturated and remoulded e = 1.06 (triaxial tests)	0.5		33°			
clay-shale	(triaxial	0.6		32°		<3.5	Sinclair and
bentonitic-shale	tests)	0		29°		<3.5	Brooker
»	»	2.7		8,5°		10-35	(36)
bentonite	»	0.6		13°		<3.5	
»	»	1.0		9°		10-35	
bentonitic-shale	(direct shear tests)		0.3		8,5°	0-25	
clays (over-consolid.)	slips, joints or minor shears	0-0.18	0-0.03	12°-18,5°	10,5°-16°	0-5	Skempton and Petley (13)
montmorillonite clay		3.6	0.8	14°	11°	2-12	Eurenius (21)

Tab. 19-3 The shear strength of filled discontinuities as obtained from direct shear tests. The results appearing above the line in the upper part of the table were obtained from laboratory tests. All the remainder were obtained from in situ tests.

Rock	Description of filling	c' (kg/cm ²)		φ'		σ'_n (kg/cm ²)	References
		peak	resi- dual	peak	resi- dual		
coal measure rocks	clay mylonite seams (1.0 to 2.5 cm thick)	0.13	0	16°	11°	0-7	Stimpson and Walton (18)
		0.11	0	16°	11,5°		
granulite lamproschist	clay filled joints clay within dyke	0	0	25°	22°	?	Henkel et al. (37)
		0		25°			
chalk	bentonite clay: 8 cm seams (mostly montmorillonite)	0.16		7,5°		?	Underwood (38)
		0.22		11,5°			
greywacke	1-2 mm clay in bedding plane		0		21°	0-25	Drozd (22)
limestone	marlaceous joints, 2 cm thick, 7% in situ water content	0	0	25°	15°-24°	10-30	Bernaix (39)
lignite	layer between lignite and underlying clay	0.14		17,5°		0-1.5	Schultze (40)
		0.3 (lab.)		15° (lab.)			
granite (?)	clay filled (30% 5 μ clay) faults (40% 5 μ clay) (40% 5 μ clay)	1.0		45		1-10	Rocha (4)
		1.0		27		1-10	
		0		24		1-10	
limestone	6 cm clay layer		0		13°	8-25	Krsmanovic Tufo and Langof (41)
limestones, lignites and marls	interbedded lignite layers lignite/marl contact	0.8		38°		0-20	Salas and Uriel (42)
		1.0		10°		?	
limestone	v. thin clay fillings (<1 mm) thin clay fillings (1-2 cm)	0.5-2.0		21°-17°		1-25	Krsmanovic and Popovic (27)
		1.0		13°-14°		1-25	

Tab. 19-4 The shear strength of filled discontinuities as measured during in situ direct shear tests.

Rock	Description of filling	c' (kg/cm ²)		φ'		σ_n^* (kg/cm ²)	References
		peak	resi- dual	peak	resi- dual		
basalt	clayey basaltic breccia: wide variation from clay to basalt content	2.42		42°		0-25	Ruiz, Camargo, Midea and Nieble (28)
basalt	contact between compact basalt and clay filled breccia	(τ/σ)		(4.5/1.7)		1.7	« «
chalk	bentonite seam	0.15		7,5°		?	Link (43)
bentonite	thin layers	0.9		12°		3	
	thin layers	1.2		17°		3	
granite	tectonic shear zone: schistose and broken granites, disintegrated rock and gouge	2.6		45°		4-7	Evdokimov and Sapegin (6)
schistose quartzite	stratification planes with: 1) thin film of clay	7.4		41°		3-9	
	2) thin film of clay	6.1		41°		5-11	Serafim and Guerreiro
	3) rock completely separated by clay	3.8		31°		2-4	
schists	10-15 cm thick clay filling	0.8		32°		3-8	(44)
quartzites and siliceous schists	10-25 cm thick clay filling (of altered rock and clay)	0.3		32°		3-12	
dolomite	altered shale bed, approx. 15 cm thick	0.41	0.22	14,5°	17°	0-7	Pigot and Mackenzie (45)

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