A MICRO-MECHANICAL APPROACH INTO THE SHEAR BEHAVIOUR OF ROCK JOINTS

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Summary: Current methods to estimate the peak shear strength of rock joints rely predominantly on empirical relationships. The empirical nature of these models requires conservatism on the part of the designer. The Geotechnical Group at Monash University is currently developing a theoretical approach where the behaviour of the rock joint is being modeled. By understanding the mechanisms involved during shearing, it is hoped that the uncertainty of design can be minimized. This paper outlines the difficulties in correctly modeling rock joint behaviour and gives a brief outline of the theoretical approach being taken.

1 INTRODUCTION

Rock masses are inherently intersected by discontinuities that are typically the weakest part of the rock mass and hence may govern its strength. The presence of these discontinuities has been known to cause catastrophic failure in many engineering structures where the design has not incorporated accurate properties of these defects.

To understand the mechanisms of failure within the rock mass, it is first necessary to understand the behaviour of a single discontinuity. Considerable research has been undertaken during the last three decades to determine the behaviour of rock joints under the application of shear load. Much of this research has involved the use of empirical models due to the complexity of the joint interface and shearing behaviour.

This paper outlines the complexities of the joint shearing process and the many factors that may affect its behaviour. A theoretical approach to rock joint behaviour that is currently being investigated by Monash University is discussed.

2 FACTORS AFFECTING ROCK JOINT SHEAR BEHAVIOUR

To accurately predict the shear response of a rock joint, it is important to identify and understand the many factors that will affect the joint behaviour. These factors are often very difficult to quantify.

There are many stresses that act on a rock joint. These can be in-situ stresses; such as gravitational stresses, tectonic stresses, residual stresses from such processes as magma cooling, terrestrial stresses such as seasonal variation; or induced stresses from such processes as mining, drilling or pumping. These stresses are often difficult to accurately measure and with little to compare answers to, unavoidably require large confidence intervals.

The boundary conditions for a rock joint vary according to the deformability of the surrounding rock. The deformability of the surrounding rock can be measured in terms of its stiffness. If the rock surrounding the joint is deformable enough to allow dilation of the joint without reaction forces, then shearing will take place under zero normal stiffness. An example of this would be a rock slope where sliding along a joint occurs under the constant normal load of the block's weight. In most underground rock situations and in rock slopes where rock bolts or cables have been used, rock blocks cannot slide freely due to intimate contact with surrounding rock blocks. Dilation of the joint causes a reaction from the surrounding rock mass that applies additional stress to the rock joint. This is known as Constant Normal Stiffness. It is also possible to have conditions where the normal stiffness may vary.

Rock joint surfaces possess many irregular departures from the plane both on the large scale and on the small scale. These departures of varying angle and width (referred herein as asperities) are dependent on the rock's mode of origin, weathering and mineralogy. Due to the random and irregular nature of the asperities, it is difficult to assign a statistical value to represent the roughness of the surface. This is made more difficult as the roughness is scale dependent. Roughness measured on a 10 cm core sample may not indicate the overlying waviness visible when looking at an exposed joint surface from some distance. At very small stress levels or very small scales the grain size can even be seen to affect the shear behaviour. To further complicate a roughness analysis, the joint surface roughness will also typically decrease during shearing as steeper asperities are sheared.

Shearing of asperities may be dependent on the rock strength. If the rock joint is not weathered then the joint wall strength is that of the intact rock. However, weathering of the joint surface will alter the joint wall strength properties.

Weathering will typically weaken the joint wall strength although in some cases such as the occurrence of iron penetration, the joint wall strength can be stronger than the strength of the intact rock. Laboratory testing can be performed to determine the strength of the joint surface although very small depths of penetration often make this testing very difficult. A field tool, called the Schmidt Hammer, has been developed to provide characteristic data. It involves dropping a spring-loaded plunger onto the joint surface and recording the number of rebounds. From an average of at least ten tests the surface unconfined compressive strength can be estimated. Damage to the surface can cause an inaccurate measurement as can voids or compressive layers hidden in the rock mass and therefore, unless calibrated extensively, the results should not be relied upon.

Joint aperture, a measure of the perpendicular distance separating the adjacent joint faces, can affect the shear response as partially open joints only allow the higher asperities to come into contact and hence become involved in the shearing process. Measuring the aperture can be quite difficult as typical drilling techniques would tend to disturb the joint interface.

Many joint walls are separated by material that will typically reduce the shear strength of the joint (such as carbonaceous material, clay, silt, breccia or minerals). The effect on the shear behaviour will be dependent on the infill's thickness, composition, water content, extent of consolidation, previous shear history and roughness of the joint walls. Laboratory direct shear tests have indicated that a rock joint containing infill will produce two peak shear strengths – failure of the infill followed by failure of the asperities (Checcia de Toledo and de Freitas, 1995).

Laboratory testing has indicated that the shear displacement velocity can affect the magnitude of the shear resistance at stress levels applicable to engineering structures (Crawford and Curran, 1981). The extent of the effects appears dependent on the rock type and level of the normal stress.

The presence of water in the joint has been documented to either reduce, increase or have no effect on the shear strength of the rock joint (Barton, 1973). These results are dependent on the mineralogy and roughness of the joint surface and the resulting development of pore pressures.

In the determination of a realistic but useable rock joint model capable of accurately predicting the performance of the joint under the application of a shear load, an understanding and method of quantifying all factors affecting the joint is required.

3 TRADITIONAL APPROACHES TO ROCK JOINT SHEAR STRENGTH MODELLING

During the past three decades extensive research has been conducted on the behaviour of rock joints under the application of a shear load.

According to the classical law by Amonton, the shear resistance, τ, is related to the normal stress, σ_n, and the coefficient of friction, μ, by the following relationship:

$$\tau=\mu\sigma_n \qquad (1)$$

where $\mu=\tan\phi$
ϕ=friction angle of the material

By investigating over 300 slopes in the Rocky Mountains and performing laboratory tests on synthetic rock profiles with constant angled asperities, Patton, (1966) highlighted the presence of two failing mechanisms - sliding and shearing. He produced a bilinear failure model whereby under a predetermined transition stress, σ_T, sliding occurs and above this stress shearing occurs. He extended Amonton's shear stress relationship to include the asperity angle, i, as shown in equation (2).

$$\tau=\sigma\tan(i+\phi) \qquad (2)$$

Although this model highlights two basic mechanisms of rock joint behaviour, it is simplistic in that real rock joint surfaces are irregular, comprising many different asperity angles. This allows shearing and sliding to occur simultaneously and would produce curved failure envelopes.

Given the non-linear failure envelopes of natural rock joints, Barton, (1973) believed that empiricism was required to correctly describe the shear strength. He proposed the following relationship based on extensive testing and observations of rock joints:

$$\tau=\sigma_n \tan(JRC\log_{10}(JCS/\sigma_n)+\phi_b) \tag{3}$$

where τ = peak shear strength
σ_n = effective normal stress
JRC = joint roughness coefficient
JCS = joint compressive strength
ϕ_b = basic friction angle

This equation can be compared to the Patton model with the asperity angle, $i=JRC\log_{10}(JCS/\sigma_n)$.

The JCS is a measure of the joint compressive strength and can be roughly measured using the Schmidt Hammer. This takes into account any alterations to the joint surface.

The JRC represents a sliding scale of roughness where the roughness is visually assessed from ten roughness profiles. These profiles have been given the ranges 0-2, 2-4, etc up to 18-20 and are 100mm long. Scale effects are therefore not considered in the measurement of the JRC. A further empirical relationship was later added to the model to take into account scale effects.

The standard roughness profiles together with the empirical relationship has been accepted by the International Society for Rock Mechanics as a useful method to estimate peak shear strength in their Commission on Standardization of Laboratory and Field tests (ISRM, 1978). Due to this endorsement and ease of use, this method has gained significant popularity and is perhaps the most widely used approach today. However, its empirical nature may limit its applicability to all situations and rock types.

In an attempt to produce an understanding of the shear behaviour, Ladanyi and Archambault, (1970) used energy principles to extend Patton's bilinear model to take into account various failure modes of rock joints occurring simultaneously. They considered that the total shearing force would comprise of four components:

- component due to external force done in dilating against the external force, N
- component due to additional internal work done in friction due to dilatancy
- component due to the work done in internal friction if the sample did not change in volume in shear.
- component due to shearing through the base of the asperities

The summation of these components combined with several empirical constants was used to predict the total shear force.

Although introducing combined concepts of shear failure of asperities and dilation, the Ladanyi and Archambault model relies on several parameters that are either difficult to predict or rely on empirical methods. Its approach to boundary conditions, rigidity of asperities and elasticity of the rock is also overly simplified.

4 MONASH APPROACH

The Geotechnical Group at Monash University has for the past 20 years being developing a theoretical approach to the determination of rock socket pile performance. This work has concentrated on developing a micro-mechanical approach to the behaviour of the concrete-rock interface during shear. The model uses several of the energy principles adopted in Ladanyi and Archambault but is based on a more fundamental approach without the use of empiricism. The model is reported in detail in Seidel, (1993).

It was considered that the shear model developed for concrete-rock interfaces could be modified to represent rock joints. Initial testing has suggested that the main mechanisms are consistent (Fleuter, 1997). A simplified description of the model details will be briefly outlined.

The model represents the roughness as a series of irregular triangular elastic asperities. When a shear force is applied, sliding will initially occur on the steepest asperity slopes. As the asperities are elastic (in particular with soft rock) sliding will also occur on several of the lower sloped asperities. Sliding on these asperities causes dilation and other asperities are lifted out of contact. This reduces the contact area causing an increase of the normal stress on areas still in contact. If the normal stress increases beyond the intact strength of the asperity then the asperity will shear. Further movements in the shear direction will cause a displacement of the sheared material. Due to the irregular nature of the surface, sliding, dilating and shearing can all occur simultaneously. With some materials the surface can also experience inelastic deformations where the surface is worn away. These simplified components are shown diagrammatically in Figure 1.

To date this model has been developed using simplified roughness profiles on either synthetic rock or reasonably homogeneous sandstone samples. To be able to realistically extrapolate the model to real rock joints of varying rock type and strength, further detailed laboratory testing is currently being undertaken.

5 TESTING OF MODEL

Most of Melbourne is underlain by Silurian and Lower Devonian Age siltstones, sandstones and mudstones called Melbourne Mudstone. This formation can be encountered in a range of weathering states that can vary from very low strength rock to very high strength in its fresh state.

Due to its importance in the Melbourne area, initial testing was conducted on a synthetic rock called Johnstone that has similar properties to Highly to Moderately Weathered siltstone. This rock was developed specifically for testing of the concrete – rock interface in pile sockets (Johnston and Choi, 1986). It has an unconfined compressive strength of approximately 4MPa when its saturated moisture content is approximately 14%.

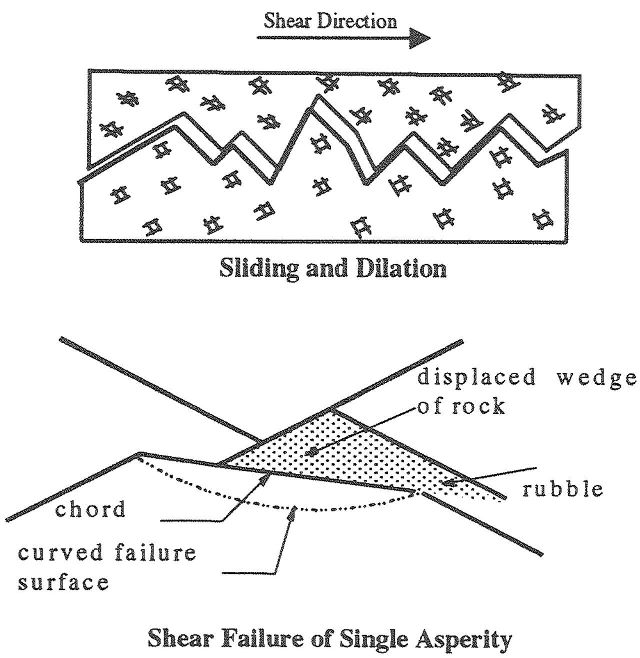

Figure 1 Idealised sketches of sliding, dilation, shearing

Direct shear tests conducted on intact and planar Johnstone samples obtained peak and residual friction angles of 35^0 and 24.5^0 respectively. Tests were undertaken on regular triangular 2-dimensional profiles to be able to observe and model the sliding, shear and wear behaviour. This was then extended into 2-dimensional irregular triangular profiles. All direct shear tests were conducted in a specially designed device that is capable of testing samples up to 600mm long under constant normal stiffness conditions.

These direct shear tests indicated that sliding was initially occurring on the steepest asperities and on some shallower asperities due to elasticity effects. This caused dilation and some asperities moved out of contact. Due to the increase in load on the steeper asperities, failure occurred transferring the load to lower asperities. Due to the irregular nature of the surface, sliding and failure was observed to occur simultaneously. Later inspection of the surface also indicated wear was occurring. With these observations, the basic model developed for concrete-rock interfaces could be extrapolated to rock joints (Fleuter, 1997).

Recent testing has extended this work into 3-dimensional profiles. These profiles were obtained by tensile splitting Johnstone blocks. Several 2-dimensional profiles were taken of each split surface using laser profilometry techniques. The laser device used was the Monash *Socket-Pro* that was developed for pile socket roughness inspections (Collingwood et al., 1999).

A visual comparison of the profiles from each surface indicated a close similarity across the face. Several of the 2-dimensional profiles for one split face are shown in Figure 2.

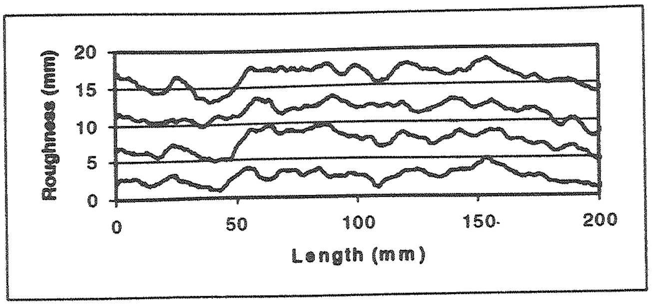

Figure 2 Several laser scans from a Johnstone split face

A statistical analysis was also completed by using the compass walking method. This method, which has been suggested as one method to estimate the fractal dimension of the joint roughness, involves walking a compass opened to a specified chord length along the profile. Considerable debate still rages over the application of this method, amongst others, in determining accurate fractal dimensions. As a result, analysis has been restricted to the determination of the variation of the standard deviation of chord angle with chord length. Correlation coefficients determined for the between each profiles range of standard deviation of chord angle with chord length were in the order of 0.99. This indicates excellent correlation.

These comparisons have suggested that the 3-dimensional surface can be modeled using a 2-dimensional profile. Limited laboratory testing has agreed with these comparisons although further work will be required to confirm this.
By testing Johnstone, a synthetic rock, problems associated with natural variations in the material are avoided allowing the basic model to be developed. Testing is currently being conducted on natural Slightly weathered to Fresh Melbourne mudstone. Samples were obtained from the tunneling work conducted for the Melbourne City Link project. The rock obtained has an unconfined compressive strength of approximately 60MPa and saturated moisture content of approximately 1.5%. These samples are therefore considerably stronger than previous samples tested in the development of the rock socket model.

A series of direct shear tests have been conducted on regular triangular asperities of chord length 16mm at angles of 5^0,10^0 and 15^0. The profiles were cut using waterjet cutting techniques. Waterjet cutting involves spraying water mixed with fine sand through a nozzle under high pressure. The resulting jet is approximately 1.2mm in diameter. The shear test results indicated that under the normal stresses applied (up to 3000 kPa), sliding on the asperity faces occurred with no shearing and very little wear. An example profile and the resulting direct shear test results are given in Figure 3.

Tests were also conducted on 2-dimensional irregular triangular profiles. An example profile together with the direct shear test results is shown in Figure 4. Some shearing of the higher asperities can be seen in these results. Dilation caused some of the interface to move out of contact causing the stresses became highly localized. When these were greater than the strength of the intact rock shearing of the asperity occurred. The results from these tests can be used to confirm the basic model.

Several blocks of siltstone have been split parallel to their bedding direction These surfaces have been compared to natural bedding joints in the slightly weathered to fresh siltstone. The split surfaces were visually and statistically similar to the natural bedding joints. Direct shear tests will be conducted on these samples to extend the existing model into more realistic 3-dimensional profiles.

Siltstone 5^0, 16mm chord length, initial normal stress=800kPa, normal stiffness=800kPa/mm

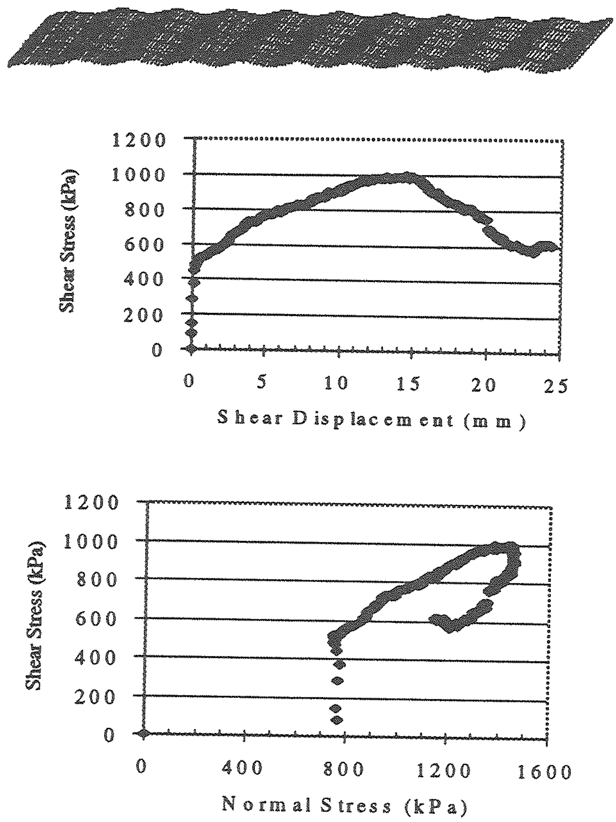

Figure 3 Shear results of 5^0 siltstone sample

6 CONCLUSIONS

Empirical models to predict the peak shear stress of natural jock joints fall short of explaining the behaviour of the joint. Although simple to use, they cannot be confidently extended to all rock joint situations thus requiring considerable conservatism.

The Geotechnical Group at Monash University is currently investigating a theoretical approach to model the behaviour of a rock joint under the application of a shear load. This work is being extended from previous investigations that were related to the performance of pile rock sockets. Testing to date has indicated that the model can be modified for rock joint analysis.

To maximize the potential application of the proposed model, confidence is required in its ability to model all situations. To provide this level of understanding, considerable testing of the many factors that effect the rock joints behaviour is still required. Once this level of understanding is achieved, it is hoped that a computer aided design tool can be developed.

7 ACKNOWLEDGEMENTS

The work described in this paper forms part of the ongoing research into the behaviour of rock joints under the application of a shear load funded through the Australian Research Council. The author is financially supported by the Australian Postgraduate Award Council. Grateful acknowledgement of the support provided by the Department of Civil Engineering, its technical staff and in particular my supervisors Chris Haberfield and Julian Seidel is also given.

Siltstone standard deviation angle = 5^0, 5mm chord length, initial normal stress=800kPa, normal stiffness=800kPa/mm

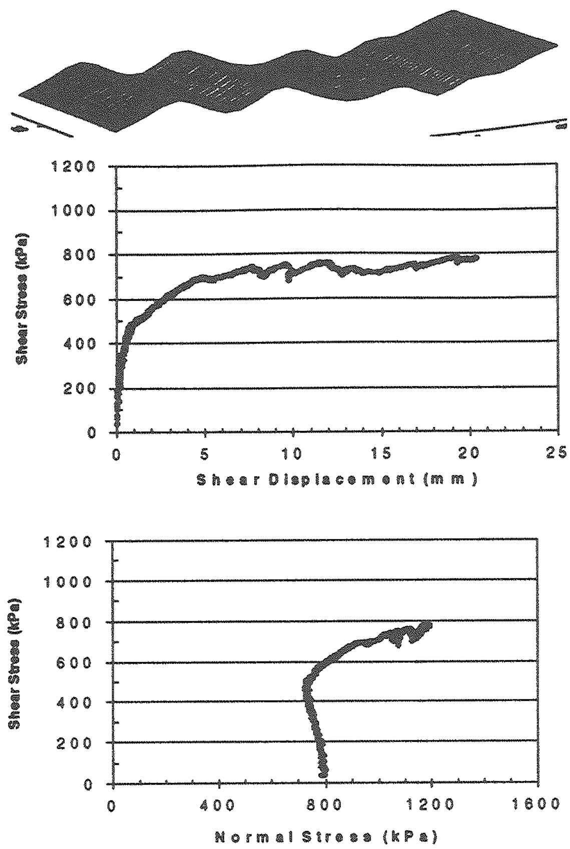

Figure 4 Shear results of s=5^0 siltstone sample

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