THE SHEAR BEHAVIOUR OF ROCK JOINTS

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SUMMARY

The methods of prediction of rock joint behaviour have traditionally relied on empiricism or over-simplified theoretical approaches. In addition only limited boundary conditions are considered in many cases. Recent advancements into rock socketed piles have resulted in a theoretically based roughness model for the shear behaviour of concrete-rock interfaces. As there are many similarities between rock joints and the concrete-rock interface of socketed piles it is likely that the new theories are also applicable to rock joints. This paper describes the beginning of a project to expand socketed pile research to rock joints.

INTRODUCTION

The behaviour of a rock mass is generally controlled by the strength of its weakest components. These are usually any discontinuities present within the rock mass such as joints, faults and bedding planes rather than the intact rock. When a rock mass is subject to loading from foundations or unloading from excavation and tunnelling, movement at the discontinuities is possible. For the purpose of this project only discontinuities that are mated (no displacement) will be considered. Such discontinuities are known as joints. Under opening a joint is assumed to have zero tensile strength while under closing the intact strength of the rock is usually taken. It is therefore the shear behaviour of a joint that is of most concern to the rock engineer.

To date researchers have tackled this problem by adopting over-simplified theoretical approaches or have relied heavily on empiricism. Recent work at Monash University has resulted in an experimentally based theoretical model for the shear behaviour of concrete-rock interfaces of rock socketed piles. This problem has many similarities to rock joints so it is likely that these models are also applicable to rock joints.

This paper will provide a background to the area of rock joints, a brief description of the models developed at Monash for socketed piles and the current project on the behaviour of rock joints.

ROCK JOINTS

The shear behaviour of rock joints is complex and depends on several factors including roughness, rock strength, boundary conditions and presence of infill. A brief description of the effect of these factors is given below.

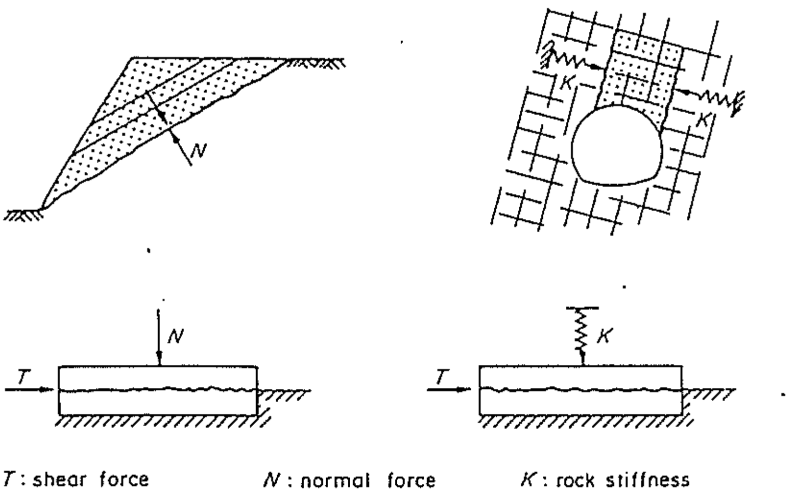


Figure 1 : Boundary conditions for rock joints (after Leichnitz, [6])

Boundary Conditions

There are basically two different boundary conditions under which a joint will be subject to. These are known as the constant normal load (CNL) and constant normal stiffness (CNS) conditions, Fig. 1. The laboratory shear test approximations are also shown.

The CNL condition is applicable to rock slopes where sliding may occur along a joint under the constant load of the overlying rock block. The CNS condition is applicable to rock socketed piles and underground excavations where sliding along a rough joint causes a dilation against the surrounding rock mass. Since the rock has a stiffness, additional normal load is applied to the joint. In CNS cases a load usually acts on the joint prior to shear displacement. This is known as the initial normal stress, σ_0.

Roughness

The effect of roughness is of major importance to the shear behaviour of rock joints, Seidel and Haberfield [9]. Joint roughness has a direct influence on the strength of the joint and on the amount of dilation developed during shear displacement. Roughness can be considered as a parameter that increases the friction angle above the base friction angle by an amount i, where i is the "asperity" or "dilation" angle, (see Eq. 1). Seidel and Haberfield, [9], also describe the scale dependence of roughness. For example, small scale roughness may be present on a larger scale roughness. Under small shear displacements peak shear strength will be governed by small scale roughness and vice versa.

$$\tau = \sigma_n \tan(\phi_b + i) \tag{1}$$

where τ = shear strength
σ_n = normal stress
ϕ_b = base friction angle
i = angle of asperities

Infill

The presence of infill affects the frictional resistance of the sliding surfaces. Infill generally reduces the friction angle however some infills can bond the joint making it stronger than the parent rock. It has been suggested by Goodman, [2], that where the depth of infill is 1.5 times the asperity height the shear behaviour of the joint may be governed entirely by the shear strength of the infill material. For the purpose of the current project infill will not be considered. The joints are assumed to be mated and clean.

PREVIOUS RESEARCH

Previous research has followed two separate paths. The first is to reduce the problem to simple theoretical models. The second approach is to rely on empirical methods. Key examples of these approaches will be described below. It should be noted that much of the previous research has tended to concentrate on CNL boundary conditions. The importance of CNS boundary conditions has been recognised by many researchers only quite recently.

Ladanyi and Archambault Model [5]

Ladanyi and Archambault used an energy approach to model joint behaviour. In this model the total shear stress, τ, is given by Eq. 2.

$$\tau = \frac{\sigma(1-a_s)(\dot{v} + \tan\phi_\mu) + a_s(\sigma\tan\phi_0 + S_0)}{1-(1-a_s)\dot{v}\tan\phi_f} \tag{2}$$

where, $\dot{v}$ = rate of dilation at failure
ϕ_μ = angle of frictional sliding resistance
ϕ_f = average value of sliding friction
ϕ_0 = internal angle of friction
S_0 = intact rock strength
a_s = area ratio

It can be shown that under high or low stress this model can be reduced to the appropriate sections of the bilinear model proposed by Patton [7], which is derived from statics. While an improvement on Patton's model, there are several limitations. These include the assumptions that shear occurs on a horizontal plane, the normal stress acts equally on all asperities and asperities are rigid.

Barton Model [1]

Barton's original model was based on the results of extensive shear tests on natural joint samples of varying strength and roughness. The empirical expression shown in Eq 3 is the result of this work.

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

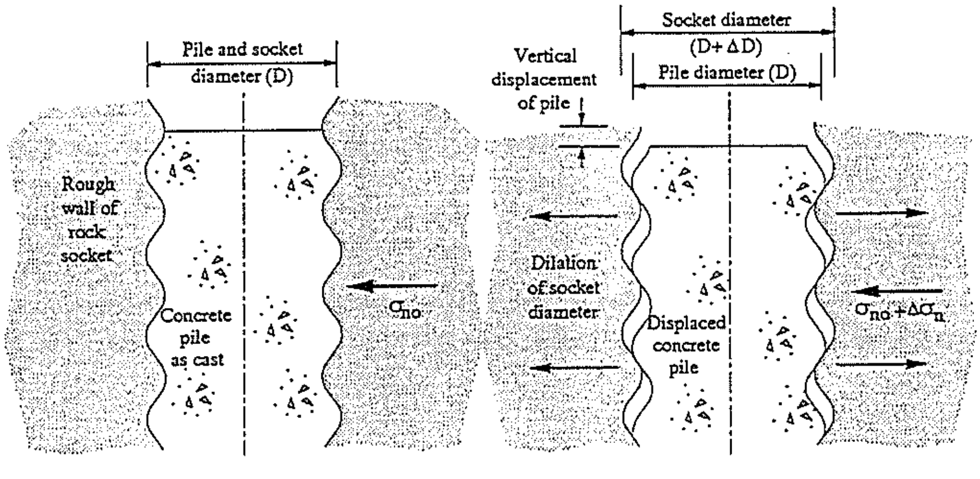
where

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

JRC is a measure of roughness determined by tilt tests or by visual observation with standard profiles and ranges from 0 for planar surfaces to 20 for very rough surfaces. It can be seen that the Barton model is the same as Eq. 1 but with an empirical expression for i (i= JRC $\log_{10}$ (JCS/σ_n). The factor $\log_{10}$ (JCS/σ_n) is introduced to account for normal stress suggesting that as the normal stress increases the dilation angle decreases. While the use of this model is very popular there are several shortcomings. Although the basic model has been retained, later publications have added various empirical correction factors to account for effects such as scale. In addition, the determination of JRC by visual observation can be difficult and there is no cohesive component to account for shearing of asperities.

RESEARCH AT MONASH UNIVERSITY

Since the mid 70's the Geotechnical Group at Monash have conducted extensive research into the behaviour of concrete piles socketed into rock. The construction process of a socketed pile results in a rough interface between the concrete and rock. When the pile is loaded and displaced vertically, dilation occurs against the surrounding rock mass, Fig. 2. Socketed piles are therefore governed by the CNS condition. The concrete-rock interface of socketed piles has many obvious similarities with rock joints. As the purpose of this project is to apply the advancements in socketed pile models to rock joints it is necessary to briefly describe the history and current stages of development into socketed piles.



(a) After casting and before displacement (b) After vertical displacement

Figure 2 : Rock socketed piles (after Johnston and Lam, [4])

The work into socketed piles is the result of several researchers' efforts culminating, so far, with the work of Seidel, [8]. Early work evolved from rock joints by representing joint roughness as regular triangular asperities.

Later work was extended to include elasticity effects and irregular triangular asperities. While these models enabled the basic failure mechanisms of single asperities to be established they still did not represent the behaviour of natural rock joints adequately. Seidel [8], used the concepts of fractal geometry to generate profiles with a more realistic roughness. To make practical use of these profiles Seidel developed methods to relate the fractal dimension (the fractal dimension of a rough line is between 1 and 2) of the profile to roughness statistics such as chord angle and height. The fractal model provides a basis to verify if the mechanisms established from regular triangular asperities can be used to predict the behaviour of more complex and realistic profiles.

The work of Seidel has been incorporated into a Windows based computer program called Rocket. Rocket is able to predict the full response of piles socketed into weak rock. While the program is specifically designed for Melbourne mudstone it has been found to be accurate for a wide variety of rock types and strengths. Another feature of Rocket is a prediction of CNS direct shear tests on which the testing program was based. It is this function that will be used to compare with the testing on rock joints and modified as necessary.

CURRENT PROJECT

As mentioned previously the main objective of this project is to extend the successful research into rock socketed piles to rock joints. Since the socketed pile work originated from rock joint research the extension of the new models back to rock joints is natural, however some differences between the two cases are expected. The basis of this investigation will be a series of direct shear tests. Analysis and modifications to the model will be undertaken after the completion of testing.

The experimental testing program is being conducted on two types of sedimentary rocks, Melbourne mudstone and Hawkesbury sandstone. The rocks are of significant economic importance in the Melbourne and Sydney areas. Due to the difficulty of obtaining intact samples of mudstone a reconstituted and reconsolidated mudstone - known as "Johnstone", [3] - will be used. This provides a sample that is homogeneous, reproducible and has well established properties. Natural sandstone will be used. The unconfined compressive strengths of Johnstone (8MPa) and sandstone (17MPa) provide a reasonable range of rock strengths.

The most recent direct shear device at Monash provides an opportunity to test larger samples sizes. A maximum sample size of 600×200mm, significantly greater than other devices, is possible. This allows a large number of segments that are needed for fractal profiles to represent natural behaviour. Hydraulic actuators of 250kN capacity apply the shear and normal loads. The device is capable of both CNS and CNL boundary conditions and is fully computer controlled.

Regular Triangular Asperities - 10 degree

Fractal Profile - 10 degree mean angle

Figure 3 : Profiles (vertical scale exaggerated)

The direct shear tests will be used to determine the effects of the following on joint behaviour :

- joint roughness
- joint normal stress
- joint normal stiffness
- joint scale

The shear tests are to be performed on Johnstone and sandstone rock samples with artificial profiles comprising both regular triangular asperities and fractal profiles. Profiles are cut by waterjet techniques. As with previous research, regular triangular profiles will be used to establish the basic failure mechanisms. In addition, time lapse video images will be utilised to observe the failure mechanisms. Angles of 5, 10 and 20 degrees have been adopted for the regular triangular asperities with base lengths of 16 and 48mm. Three fractal profiles with mean absolute angles of 5, 10 and 15 degrees and base lengths of 16mm have been chosen. Figure 3 gives an example of a regular profile and a fractal profile. While the testing program is yet to be completed some results on regular triangular profiles are presented in Figures 4 and 5.

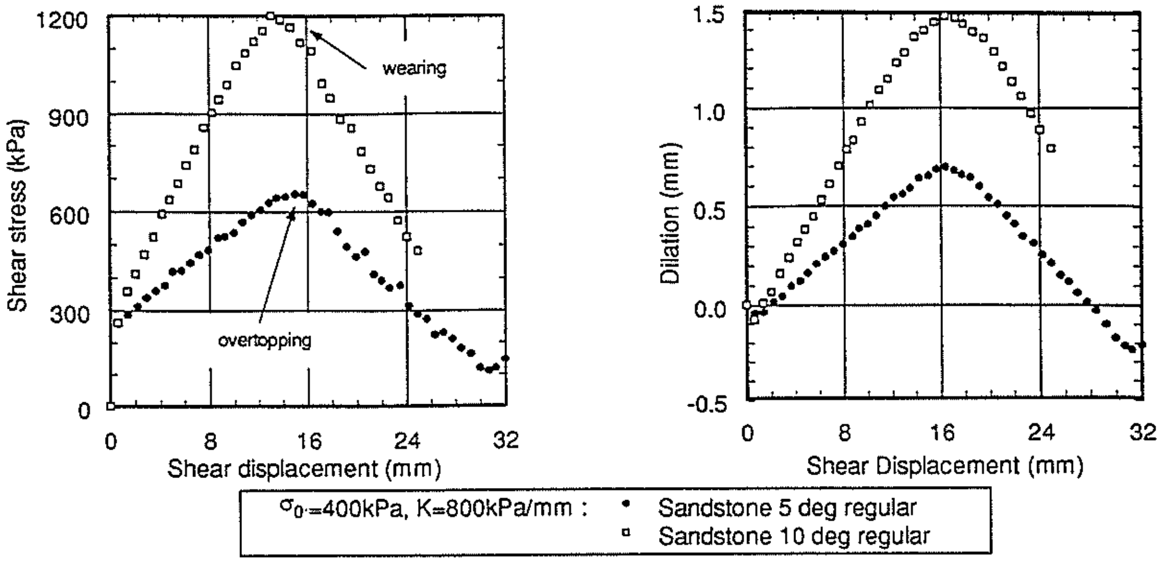


Figure 4 : Typical results - effect of roughness

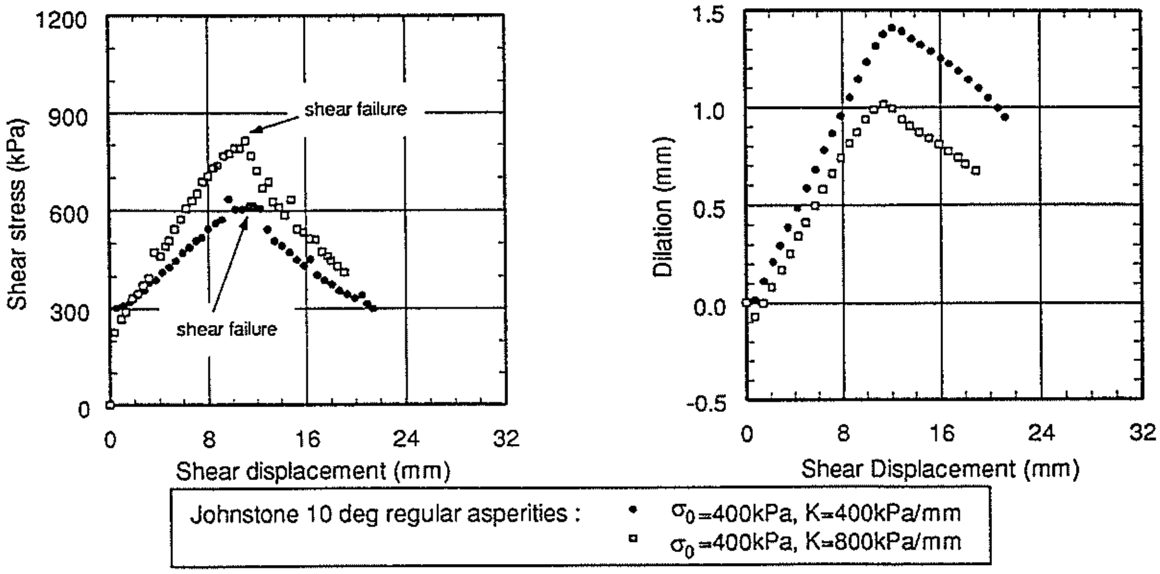


Figure 5 : Typical results - effect of CNS boundary conditions

Figure 4 shows a comparison of typical test results for sandstone samples with 5 and 10 degree regular triangles with 16mm base lengths. The initial normal stress and normal stiffness are identical at 400kPa and 800kPa/mm respectively. The 10 degree sample can be interpreted as being a "rougher" profile. From the shear stress and dilation versus shear displacement graphs it can be seen that the greater angle has a significant influence on the shear stress and dilation developed. The greater shear strength experienced by the 10 degree sample is due to two effects. Firstly the greater asperity angle produces a greater friction angle, (ϕ_b + i). Secondly as the tests were carried out under CNS conditions the greater dilation of the 10 degree sample increases the normal stress acting on the joint. Therefore as both the friction angle and normal stress acting on the joint are greater for the 10 degree profile the shear resistance is also greater.

Figure 5 shows a comparison of typical test results for Johnstone samples with identical profiles of 10 degree regular asperities. The initial normal stresses are also identical however, one has a greater stiffness. As shearing progresses the dilation of the sample with higher stiffness is more restricted however the increase in normal load due to stiffness is still greater resulting in a greater shear stress. It can be seen that both roughness and boundary conditions have a significant influence on the shear behaviour of rock joints.

Comparisons between Johnstone and sandstone are also seen in Figures 4 and 5. For the tests on the 10 degree profiles at an initial normal stress of 400kPa and stiffness of 800kPa/mm the sandstone sample achieves a much greater shear strength. This is a reflection of the higher strength and friction angle of the sandstone.

Comparisons of the completed shear tests to the existing concrete-rock model using the Rocket program have been performed. Currently the Rocket program consistently overestimates the shear strength of the rock joint tests. This is to be expected since the Rocket program was modelled on concrete-rock interfaces. As one surface (concrete) is much stronger than the other, dilation of the joint is greater and compression of the sample is less, hence normal stress increases, leading to greater shear strengths (for CNS conditions). This is one obvious modification necessary to the Rocket program which will need to be incorporated in the near future. However, it is observed from time lapse photography that the fundamental mechanisms of failure for Johnstone samples are similar, suggesting that the models are valid. Further investigation of the sandstone samples will be necessary. A full review of the models will take place after completion of the testing program.

CONCLUSIONS

This paper describes the beginning of a research project into the shear behaviour of rock joints. The project is an expansion of the related work into rock socketed piles that has achieved significant advances in recent years. The basis of investigation is a series of direct shear tests on two types of sedimentary rocks currently underway. The results will be used to adapt the existing model to predict the shear behaviour of mated rock joints under CNS and CNL boundary conditions.

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