APPLICATIONS OF LARGE SCALE DIRECT SHEAR TESTING

C.M. Haberfield[12] and J. Szymakowski[2]
[1]Golder Associates, [2]Department of Civil Engineering, Monash University

1 INTRODUCTION

Direct shear testing is a common and well-established method for determining the strength of geomaterials. In particular, it is ideally suited to measuring the strength of interfaces between geomaterials and structures, e.g. at the rock-concrete interfaces of concrete dams and concrete piles drilled into rock; and within geomaterials, e.g. natural rock joints.

One area that has been of significant interest to researchers at Monash University since about 1980 has been the behaviour of the interface formed between a concrete pile and the surrounding rock. The performance (capacity and displacement response) of the pile is dominated by the behaviour of this interface. Axial loading of the pile produces slip displacement at the rock-concrete interface, in much the same way as shearing of a rough rock joint. In order to investigate the behaviour of this interface, a large direct shear testing apparatus was designed and constructed. Design criteria for the shear apparatus and associated split shear box were:

- To have the capability of testing large samples so that scale effects could be investigated
- To be as rigid as possible to prevent sample rotation and minimise compliance while keeping friction losses to a minimum
- To be able to replicate in-situ boundary conditions such as normal stress and stiffness as accurately as possible
- To be able to apply any loading pattern (stress and displacement control) so that both cyclic and monotonic loading conditions could be investigated

The construction of the shear rig was completed in 1991 and has since been used almost continuously on a number of research and consulting projects. A photograph of the shear rig is shown in Figure 1. These projects have included the testing of rock-concrete interfaces and rock-rock interfaces, jointed rock masses, base coarse and rock fill material and the dynamic testing of pile-soil interfaces. The rocks used to date have varied in uniaxial strength from 1 MPa to 200 MPa and included siltstone, Johnstone (a synthetic siltstone), sandstone, calcarenite, basalt and granite.

Figure 1: Shear testing rig

A short description of the capabilities of the direct shear rig and some of the applications for which it has been used are presented in this article.

2 TESTING CONDITIONS

To obtain meaningful test results, in-situ conditions, such as stresses and constraints, should be accurately replicated during testing. If these conditions are not present, the laboratory test may not be able to match the response observed in-situ. Boundary conditions that are appropriate in the field can usually be approximated to either constant normal load (CNL) or constant normal stiffness (CNS) conditions.

2.1 CONSTANT NORMAL LOAD

Constant normal load (CNL) conditions occur where the rock mass is free to dilate as shear occurs, such as when slope failure occurs (e.g. see Figure 2). In this case, the normal force acting on the shear plane is a function of the mass of the overburden mobilised during the failure.

2.2 CONSTANT NORMAL STIFFNESS

Constant normal stiffness (CNS) conditions occur where the rock mass dilates against adjacent rock as shear occurs, e.g. during axial displacement of a rock socketed pile (see Figure 2). In this case, the normal force acting on the shear plane is a function of the stiffness of the adjacent rock. As much of the work on rock-concrete joints was carried out to investigate socketed pile behaviour, it is perhaps worthwhile to discuss this particular application in more detail. To model the rock-socket pile interface in a laboratory test, it is necessary to understand the conditions at the pile socket. As shown in Figure 3, when the pile is constructed in rock, the concrete of the pile and the surrounding rock are initially in intimate contact. Initial normal stresses caused by the fluid concrete act across the interface between the pile and rock mass. On application of structural surface loads, the pile and rock mass undergo elastic deformations until the shear stress at the interface leads to axial slip of the pile. As a result of axial slip displacement, the surrounding rock must dilate due to the roughness between the pile and rock. This dilation, Δr, is constrained by the surrounding rock mass, and therefore causes an additional normal stress. This additional normal stress may be estimated using well established thick walled cylinder theory as follows:

$$\frac{\Delta\sigma_n}{\Delta r} = \frac{1}{r}\frac{E_m}{(1+\nu_m)} \approx K \qquad 1$$

where r, $\Delta\sigma_n$, E_m and ν_m are the pile socket radius, the increase in normal stress due to dilation of the pile socket, the Young's modulus and Poisson's ratio of the rock mass respectively. As the increase in the radius due to dilation (Δr) is much smaller than the initial radius of the pile, r, the normal stiffness, K, is effectively constant. The behaviour of piles socketed in rock can therefore be modelled as being governed by a constant normal stiffness (CNS) condition.

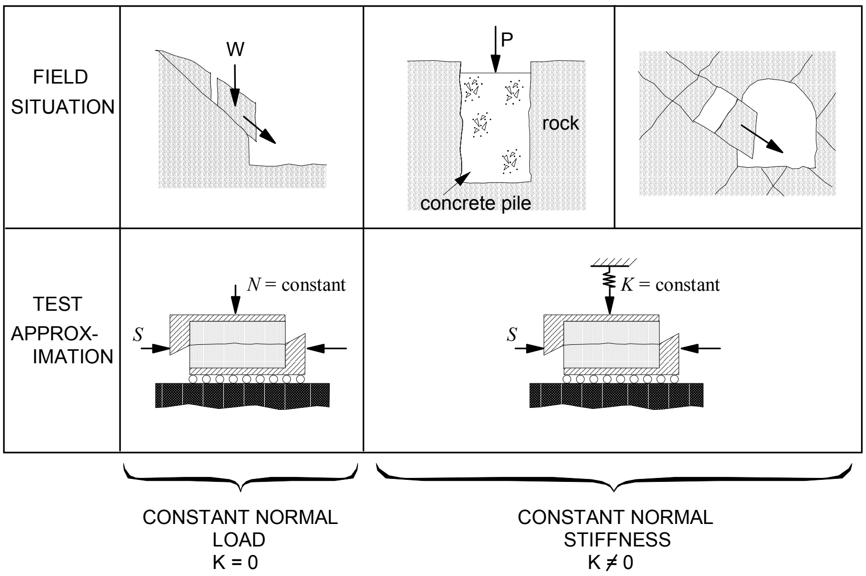


Figure 2: Boundary conditions for testing and their field equivalent

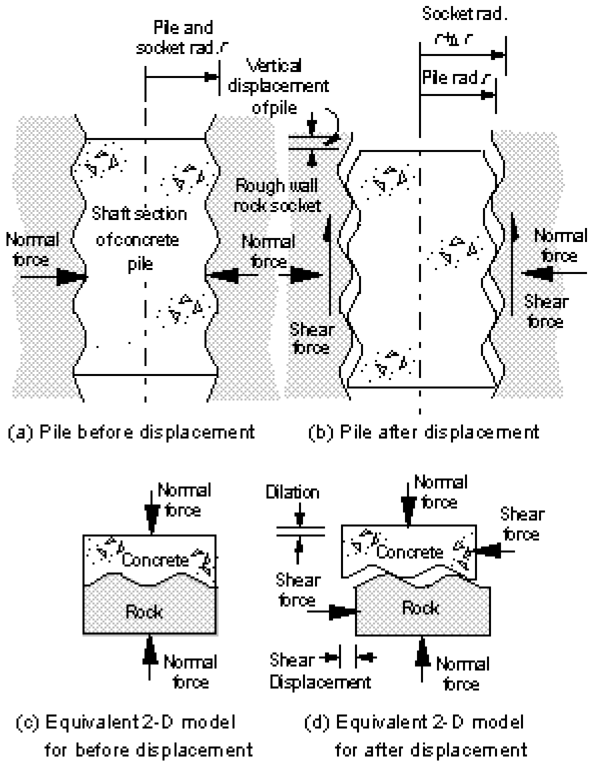


Figure 3: Forces acting on displaced pile

3 SHEAR RIG CAPABILITIES

The direct shear rig is approximately 4 m in length and 4 m in height and has the following features and capabilities:

- Three split shear boxes:

  - Box Type "A" that can accommodate samples up to 600mm by 200mm in plan and 135mm in depth,

  - Box Type "B" that can accommodate samples up to 450mm by 280mm in plan and 320mm in depth,

  - an open ended shear box 555mm by 160mm in plan for testing soil

- Computer control using PC with onboard 12 bit digital input/output card, which also automatically logs displacement and load in both shear and normal directions. Analog inputs are monitored from the load cell in the shear direction, two shear displacement transducers, the load cell in the normal direction and three normal displacement transducers. All test measurements are displayed in real-time.
- 250kN servo-controlled Instron hydraulic actuators for normal and shear load application, (extended to 500 kN for dynamic loading). The servo-controlled capability allows testing under a wide range of stress paths and boundary conditions including constant normal load, constant normal stiffness and variable normal stiffness.
- Monotonic and cyclic loading capability with facility to vary waveforms and periods for realistic loading patterns.
- Load control for normal stress, with automatic shear area correction.
- Displacement- or load-control for application of interface shear.
- PC-based digital control and real-time display of results, and
- Facility to monitor the shear process with using time lapse photography.

Loads are measured by Instron load cells placed in series with the actuator. Loads can be resolved to 0.12 kN, (2.4 kPa over the typical specimen size) and representing a resolution of less than 0.6% of typical interface shear strengths. Displacements are measured to an accuracy of ± 0.01 mm in the shear direction, and ± 0.002 mm in the normal direction. Because of inevitable (albeit small) compliance effects in the test frame, all displacement transducers are mounted as close to the shear plane as possible. For the normal direction, the displacement transducers are bolted directly to the top half of the specimen split box, and slide on a Teflon foot along a reference plate attached to the bottom half. Area correction of normal and shear loads to account for the reducing contact area is automatically applied in the control software.

The split shear box holding the two halves of the sample allows shearing along a predetermined plane. The two Type “A” box halves are locked together during specimen preparation by means of steel strips on either side which physically separate the half-boxes by 25 mm, while the Type “B” box halves are separated by 10 mm. The separator strips are removed prior to testing to allow observation and recording of the interface processes. Further details of the device and the procedure for conducting the tests are described in Haberfield et al. (1994).

It is noted that one of the most important considerations in design of the CNS device was the design of the bearing systems in the normal and shear directions. It is important that rotation of both the top and bottom parts of the shear box is effectively restrained. Without such restraint, stress distributions at the specimen interface would be adversely affected. This restraint must be imposed whilst load is transferred from the actuators to the specimen with minimal frictional loss. Frictional losses in the bearings were measured at less than an equivalent 10 kPa for the typical specimen size.

4 SHEAR BOXES

Two split shear boxes are available, one for testing Type “A” samples and one for testing Type “B” samples. The Type “A” shear box is used predominantly for testing concrete/rock interfaces and natural rock joints. This shear box requires that the sample be cast into the shear box halves using cement grout or plaster. The Type “B” shear box is used for testing jointed rock masses. This shear box has a perspex front to allow viewing of the development of the failure plane. The sample is held in the shear box halves using steel plates. These plates bear against the side of the sample and can be used to apply a known horizontal stress. The plates have been configured so that they also can also apply a known horizontal stiffness.

The dynamic shear box arrangement comprises a single, open-ended box that allows contact between the piston and the soil and contact between the soil and the pile surface. The shear box is attached to the piston and holds the soil in place while allowing the development of lateral stresses to occur.

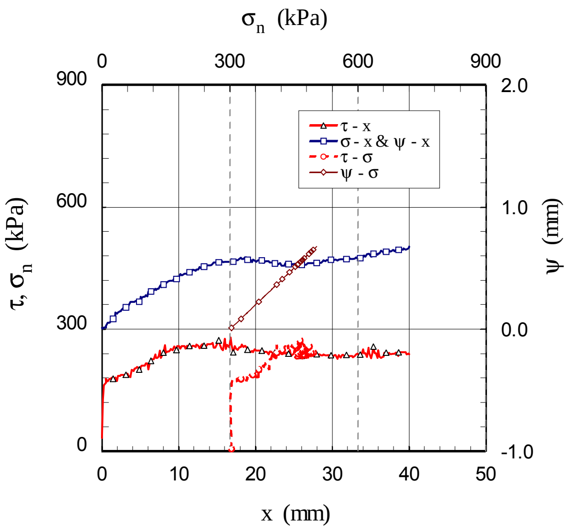
5 APPLICATIONS

5.1 CONCRETE – ROCK INTERFACES

The interface of piles socketed into rock can be modelled in laboratory tests by resolving the axisymmetric pile situation into the two dimensional situation, and then testing under CNS direct shear conditions. As can also be seen in Figure 3, the interface between the concrete (pile) and the rock can be simulated by two half specimens with the required interface roughness. These two half specimens are placed in a split shear box with each sample individually secured into the respective halves of the shear box using either casting plaster or cement grout. The lower half box is mounted in the CNS direct shear machine on a horizontal carriage that is prevented from moving in the vertical direction, but allowed to traverse freely in the horizontal direction due to the applied shear force. The upper half box is bolted to a very large piston that is constrained to vertical movement only. The applied normal force is related to the sample dilation through the feedback control system that simulates the constant normal stiffness condition. The particular value of CNS used can be varied to reflect the influence of different pile diameter, rock modulus and Poisson’s ratio. Typical test results for Johnstone-concrete joints are shown in Figure 4. For further details refer to Seidel (1993) and Haberfield and Seidel (1997).

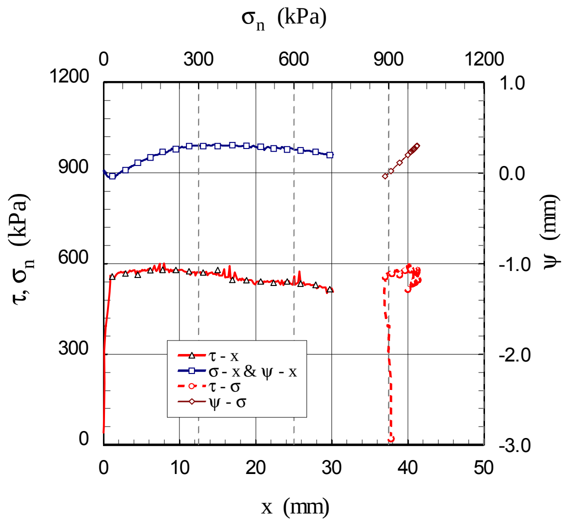
This testing has allowed the basic mechanisms occurring at the interface to be observed and modelled theoretically, even for very complex roughness profiles. As a result, a theoretically based computer program for predicting socketed pile performance is now commercially available (see Seidel and Haberfield, 1995; Haberfield and Seidel, 2000).

Cyclic tests have been carried out on calcarenite-concrete, Johnstone-concrete and sandstone-concrete interfaces (Seidel, 1993; Seidel and Haberfield, 1994). Such tests are important for determining the possible degradation of pile capacity under the cyclic action of wind and live loads (e.g. for offshore oil and gas platforms).



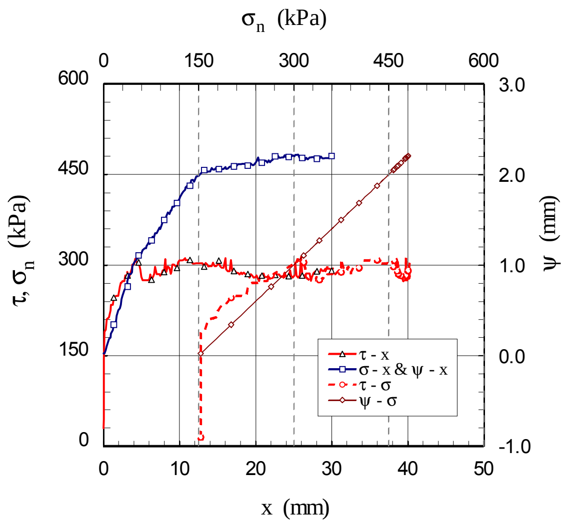
(a) Fractal Class A Profile 1 coarse approximation

(σ_n= 300 kPa, K = 300 kPa/mm)



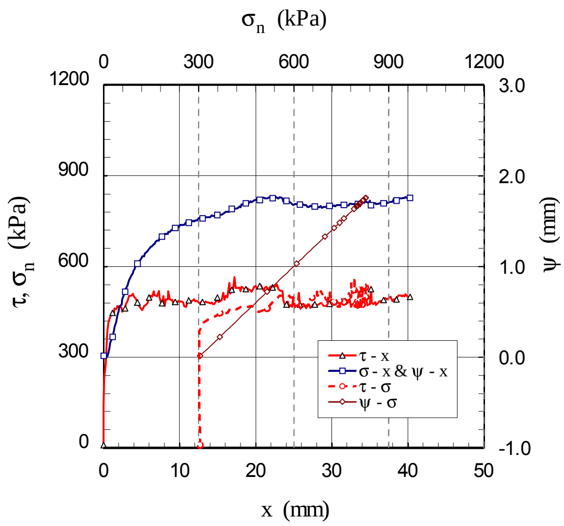
(b) Fractal Class A Profile 1 coarse approximation

(σ_n = 900 kPa, K = 300 kPa/mm)



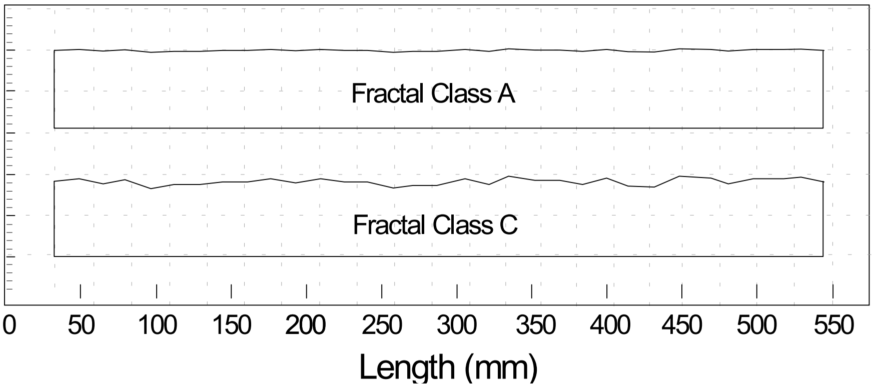
(c) Fractal Class C Profile 1 coarse approximation

(σ_n = 150 kPa, K = 150 kPa/mm)



(d) Fractal Class C Profile 1 fine approximation

(σ_n = 300 kPa, K = 300 kPa/m)



Roughness profiles used in direct shear testing

Figure 4: Typical results from direct shear tests on Johnstone-concrete joints with Class A and C roughness

5.2 ROCK JOINTS

The application of using the direct shear rig to simulate the shear displacement of pile sockets under axial loading has been extended into modelling the shear behaviour of rock joints. The direct shear rig is well suited to testing rock joint samples due to the following capabilities:

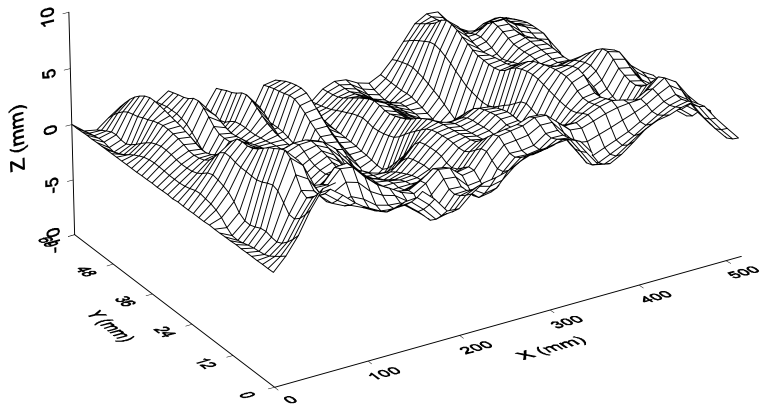
- Its ability to accommodate large samples so that scale effects are minimised. In the direct shear rig, tests can be conducted on rock joint samples up to 560mm long and 180mm wide. The samples are set into the shear box using either plaster for weak materials or high strength cement grout for stronger rock samples.
- Its ability to test under constant normal stiffness conditions allows underground rock situations or constrained rock slopes to be modelled. In these situations the rock blocks cannot slide freely due to constraint provided by the surrounding rock mass. This means that any dilation of the joints causes a reaction from the surrounding rock mass that applies additional normal stress to the rock joint.
- Laboratory testing has indicated that the shear displacement velocity can affect the magnitude of the shear resistance at stress levels applicable to engineering structures. The direct shear rig can apply a range of velocities suitable for the rock types of interest.
- The direct shear rig's capability to test cyclic displacements over a range of velocities and displacements also allows the effect of earthquakes, vibrations and any cyclic movements due to wind loads to be investigated.

As with the other areas of testing the real time display of data allows immediate assessment of the test results and the ability to video the shear interface allows closer monitoring of the shear process. The relatively large scale of the shear box has meant that scale effects and both two- and three-dimensional roughness profiles can be investigated. Typical results obtained from testing two- and three-dimensional joints in siltstone are shown in Figure 5. Refer to Pearce and Haberfield (2000) for further details. The observed similarity between rock-concrete and rock-rock joint behaviour (Haberfield and Seidel, 1999) has allowed the theoretical models developed for rock-concrete joints to be extended to natural rock joints. These models have been incorporated into a computer program (Pearce, 2001). Good predictions have been obtained for a range of rock types, rock strength and joint roughness.

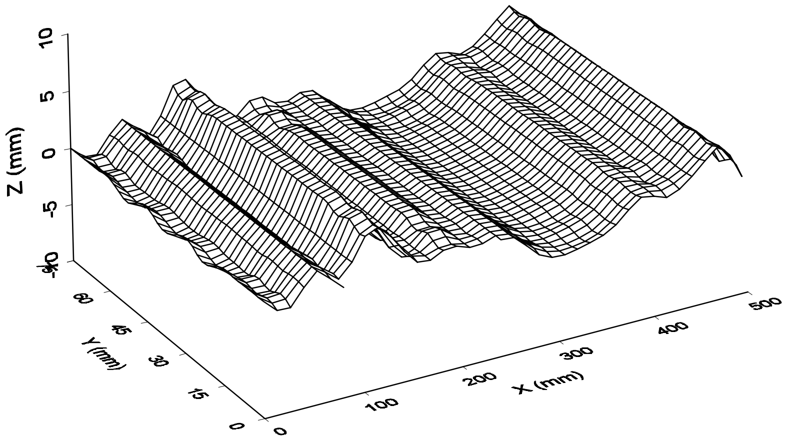
5.3 JOINTED ROCK MASSES

This project uses the direct shear rig to determine the full anisotropic behaviour of jointed rock masses under direct shear. Of particular interest is the behaviour of rock masses under loading from common civil engineering applications such as sheet pile walls and laterally loaded piles. The nature of civil infrastructure is that they involve rock masses that are typically near surface, and hence are usually weak (due to weathering), highly fractured and of low quality and subject to low in-situ stresses. Failure usually involves both the intact rock and the discontinuities. The shear box is ideal for this application as the shear box controls the location of the shear plane, and therefore the shear strength of the rock mass in any direction relative to the joint orientations can be investigated. The influence of boundary constraints imposed by the infrastructure can also be accommodated by using the constant normal stiffness capabilities of the shear rig.

Rock joint surface formed by tensile splitting of an intact siltstone block



Equivalent two dimensional joint profile cut into siltstone block



Direct shear tests results obtained from the two rock joints tested under an initial normal stress of 380 kPa and a constant normal stiffness of 300 kPa.

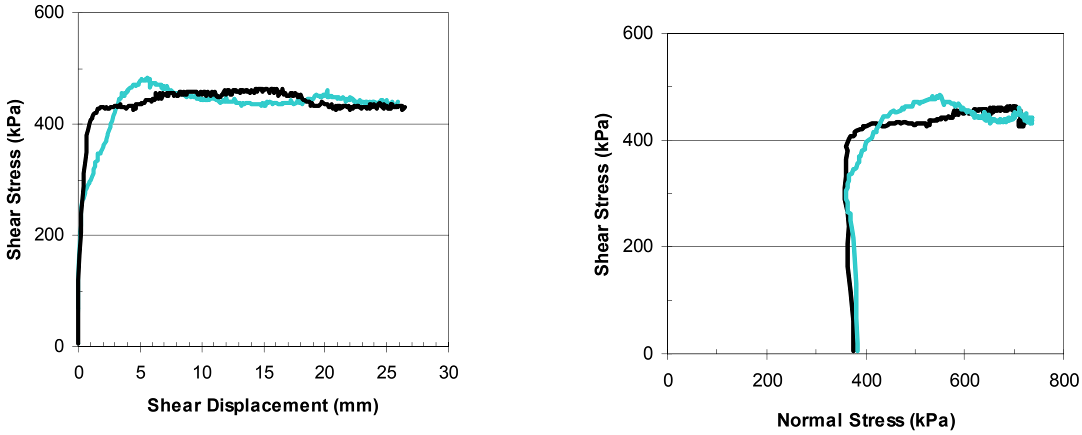


Figure 5: Comparison of 3D and statistically similar 2D joints in a soft siltstone

Both Type "A" and Type "B" Johnstone samples have been produced and tested. The testing program has examined the effects of joint orientation, joint spacing, intact rock strength and boundary conditions on shear strength. A schematic of a typical sample is shown in Figure 6. The test results for rock mass samples containing 2 and 3 joint sets have been plotted in Figure 7. The joints are smooth, planar, tight and unaltered with a spacing of either 32 mm, 50 mm or 70 mm. Initial normal stresses varied between 100 and 400 kPa and initial normal stiffness held constant at 200 kPa/mm. Intact rock strength was typically 3.0 MPa $\pm$ 0.5 MPa.

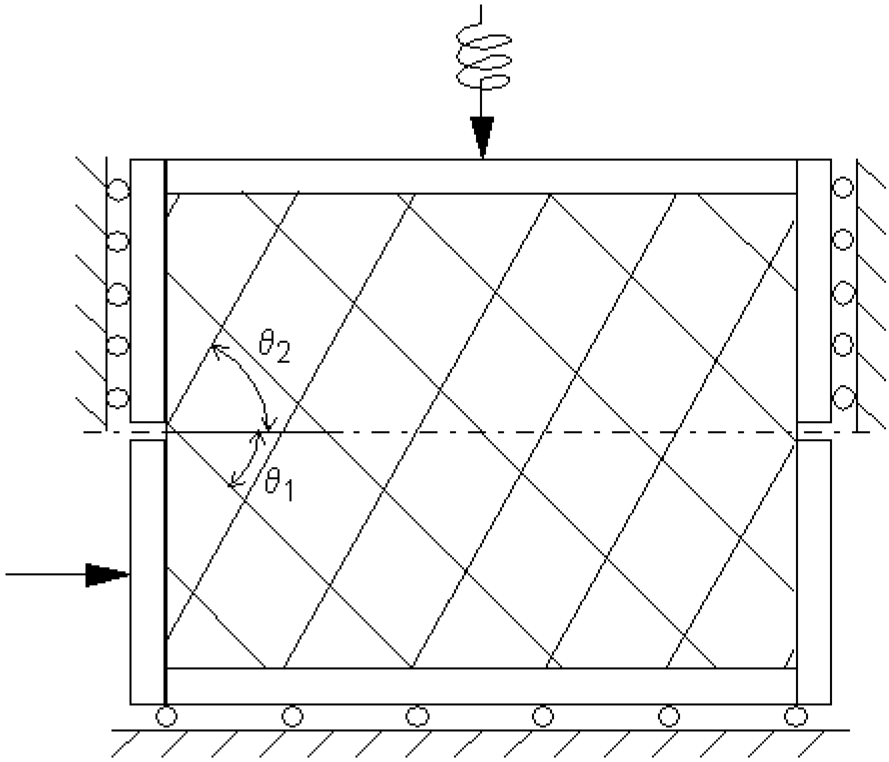


Figure 6: Diagram of boundary conditions and sample configuration.

In Figure 7 the peak strengths measured during testing are compared to the strength envelope predicted by the Hoek and Brown (1997) GSI model. As the rock masses tested were considered to be blocky with smooth but unaltered (fair) joint surfaces, a GSI value of 62 was considered to be appropriate. In this case, the Hoek Brown envelope falls slightly below the experimental results.

The visually observed behaviour of rock mass samples containing shallow joint inclinations (relative to the shear plane) indicated sliding occurred along the joints inclined at θ_1 (as shown in Figure 6). For steeper joint inclinations, sliding was prevented and rotation of rock blocks occurred. See Szymakowski and Haberfield (2001) for further details.

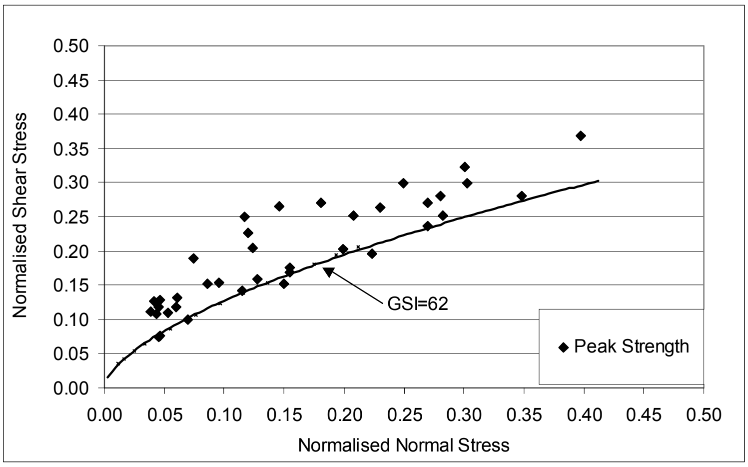


Figure 7: Peak strengths obtained from direct shear tests.

5.4 BASECOURSE AND AGGREGATES

The shear rig has also been used for testing basecourse and aggregate samples compacted at specific moisture contents and dry density conditions so as to determine the friction angle and residual behaviour of the material for design purposes.

5.5 DYNAMIC SIMULATION OF PILE DRIVING IN SOIL

During dynamic testing and pile driving events, the pile is dynamically loaded, mobilising resistance at the pile-soil interface. An investigation into the influence of pile velocity on the dynamic resistance of the pile-soil interfaces is being carried out. As such, the dynamic response of the pile-soil interface has to be simulated under laboratory conditions.

The shear rig has been chosen for the laboratory simulation of the interface behaviour because of special features outlined below. In order to deliver high speeds to the section of the pile interface, some significant additional components have been added to the rig such that the modified rig now has capabilities for both static and dynamic shear tests.

The carriage (or lower half of the shear box) is connected to the horizontal actuator, which is capable of moving at shear rates ranging from 0.01 mm/s to 50 mm/s. As such, a section of the pile interface, 555 mm by 160 mm, that is attached to the surface of the carriage can be sheared across the interface at these low speeds in tests called “quasi-static” and “slow shear” tests.

For dynamic shear tests, a lighter carriage and a high-speed hydraulic ram have been added to the rig in order to accelerate the carriage (onto which the pile interface is attached) to speeds relevant to dynamic events. The ram and the carriage are joined by a connection housing a dynamic load cell. The base of the new carriage is bolted down to the older heavy carriage and the platform of the shear box such that the old carriage cannot move. The ram is capable of delivering a maximum speed of 4 m/s and has an adjustable stroke such that the carriage can be pushed monotonically (or incrementally) for a total shear displacement of 100 mm.

The bearing systems in the vertical and horizontal planes have large rotational stiffness that are essential for ensuring a uniform stress across the pile-soil interface and they allow loads to be transferred vertically and horizontally to the sample and the carriage respectively with minimal friction.

For fast tests, the data acquisition system comprises of accelerometers that are mounted on the carriage. The acceleration thus obtained is integrated to obtain velocity for fast tests. For slow and fast tests, the data acquisition system comprises of LVDT’s to measure the displacement and hence speeds of the carriage, an Instron load cell is used to record normal loads. For slow tests, an Instron load cell interfacing the horizontal Intron actuators and carriage is used to record shear loads. For fast tests, the data acquisition system includes a dynamic load cell interfacing the high-speed ram and the new carriage to record shear loads.

Software has been specifically written to capture data from both the slow and fast tests. A trigger device that activates data acquisition has also been constructed. Preliminary testing has recently been completed.

6 CONCLUSION

A number of applications that can be modelled using a cyclic CNS direct shear rig have been presented. It can be seen that each application requires conditions that are specific and which can be suitably modelled using this rig. The cyclic CNS direct shear rig at Monash University has proved to be an invaluable tool in research activities and for consulting applications.

7 REFERENCES

Haberfield, C.M. and Seidel, J.P. (1997). The behaviour of rock joints in direct shear. *Int. Sym. on Rock Mechanics and Environmental Geotechnology*, April, Chongqing, China. pp. 19-25 .

Haberfield, C.M. and Seidel, J.P. (1999). Some recent advances in the modelling of weak rock joints. *Int. J. of Geotechnical and Geological Engineering*. Vol 17 pp. 177-195.

Haberfield, C.M. and Seidel, J.P. (2000). The role of theoretical models in the analysis and design of rock socketed piles. *John Booker Memorial Symposium*, DW Smith & JP Carter Eds, pp. 465-488.

Pearce, H. and Haberfield, C.M. (2000). Direct shear testing of Melbourne mudstone joints under constant normal stiffness conditions. *GeoEng2000 - International Conference on Geotechnical and Geological Engineering*. November. 6pp.

Pearce, H. (2001). The shear behaviour of rock joints. PhD dissertation. Department of Civil Engineering, Monash University, Australia.

Seidel, J.P. (1993). The analysis and design of pile shafts in weak rock. PhD dissertation, Department of Civil Engineering, Monash University, Australia.

Seidel, J.P., Haberfield, C.M. and Johnston, I.W. (1994). Constant normal stiffness testing of soft rock-concrete interfaces. *Int. Symp on Pre-Failure Deformation of Geomaterials*, Sapporo, Japan, Shibuya, Mitachi & Miura (Eds.), pp. 155-160.

Seidel, J.P. and Haberfield, C.M. (1995). The axial capacity of pile sockets in rocks and hard soils. *Ground Engineering,* March: 33-38.

Szymakowski, J. and Haberfield, C.M. (2001). The behaviour of joint soft rock masses under direct shear. *EUROCK 2001*, Espoo, Finland, Saarka & Eloranta (eds), pp. 307-312.