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Experimental Study of Mechanical Behaviour of Rock Joints Under Cyclic Loading

By

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Summary

Evaluation of the effects of small repetitive earthquakes on the strength parameters of rock joints in active seismic zones is of interest of the designers of underground constructions. In order to evaluate these effects, it is necessary to study the behaviour of rock joints under dynamic and cyclic loadings. This paper presents the results of a systematic study on the behaviour of artificial rock joints subjected to cyclic shearing. More than 30 identical replicas have been tested using triaxial compression devices under different conditions of monotonic and cyclic loading. At the first stage a few samples have been tested in monotonic loading modes under various confining pressures and rate of displacement. In the second series of tests, small cyclic loads were applied on the samples for increasing number of cycles, frequency levels and stress amplitudes. These were then followed by monotonic loading again. The variations of maximum and residual shear strengths for each test have been studied. The results show increase of shear strength as a result of the increase in confining pressure and they display decrease of shear strength due to the increase of rate of loading, number of cycles, frequency levels and stress amplitudes.

Keywords: Roughness, second order asperities, damage, rate of loading, number of cycles, frequency level, stress amplitude.

1. Introduction

About two decades ago, there was a general believe that underground openings are not vulnerable to earthquakes and seismic loads when compared with ground surface structures. Some studies during the last years of the 70$^{\text{th}}$, such as Dowding and Rozen (1978), and first years of the 80$^{\text{th}}$, such as Owen and Scholl (1981), showed that underground openings are not quite safe when subjected to strong earthquakes. During the years after, Peak Particle Velocity (PPV) and Peak Particle Acceleration (PPA) of the ground motions have been used frequently by most of the designers as the key parameter for assessing the dynamic stability of underground excavations, as mentioned by Brady and Ma (1999). Some researchers such as Dowding and Rozen (1978) and Sharma and Judd (1991) have presented experimental relations between PPV and PPA with possible damage levels of underground structures.

During each weak earthquake, some small changes occur in the rock masses along the joint surfaces and discontinuities. Accumulations of these changes during the repetitive seismic events may cause considerable displacements or even catastrophic collapses. Some of the results of the repetitive seismic loading on rock masses are accumulation of small displacements, cyclic fatigue, degradation of the asperities, etc. Considering small repetitive earthquakes, the main question is their possible effects on the properties of rock masses and if the peak parameters such as PPV or PPA are sufficient for evaluation of the stability of underground excavations. In order to find a proper answer, the behaviour of rock joint samples should be studied under cyclic loading. This could be achieved under two different conditions: the behaviour of rock joints before considerable sliding of one rock wall on the other one and the behaviour during sliding.

Most of the works carried out so far, for example Hutson and Dowding (1990), Ghosh et al. (1995) and Armand et al. (1998), have focused on the effect of cyclic loading during sliding. The methods presented in these researches can be used to evaluate the asperity degradation during the strong earthquakes that have enough energy to make considerable relative displacement between two sides of the joint surface. During the small earthquakes however, there is no large relative displacement between the joint surfaces due to the low stress applied in each event, so there would not be significant degradation of the joint surfaces. In these cases mostly accumulations of the small displacements and cyclic fatigue may occur that could decrease the joint shear strength. These conditions have been studied by Hencher (1980) and Barbero et al. (1996) that considered the behaviour of the discontinuities just prior to the occurrence of relative displacements between the contact surfaces.

In this research the effects of small cyclic loading on shear strength of rock joints using identical saw-tooth samples have been studied. In addition some experimental equations that cover the trends of the obtained results would be presented. It is recommended to complete this study with further tests on rock samples with different roughness, mineralogical composition and structure. The accuracies of the presented relations should also be improved by more tests on real samples.

2. Physical Joint Models

2.1 Experimental Constraints

In order to carry out an experimental study on rock joints, it is necessary to prepare identical samples (considering geometrical and constitutive properties) and to evaluate the relative displacements during the tests. The level of confining pressure and cyclic loading conditions should also be determined before the tests begin.

As the total relative displacements in the proposed tests are lower than 1 mm in most cases, the triaxial compression method has been selected to perform all the tests. It is possible in these tests to model jointed rock samples in a state similar to their real underground conditions during cyclic loading. Of course this method has its own limitations. The main problem is the orientation of the joint surface inside the triaxial testing cell.

By arranging the sample inside the testing cell with a proper dip of joint plane (lower than 45 degrees with respect to the sample axis) this limitation can be overcome, as discussed by Goodman (1989). Also it is possible to determine the maximum allowable dip of the joint surface for such tests by using the Mohr-Coulomb criterion as follows, Pellet (1993):

For intact rock

$$\sigma_1 = 2c_r \tan\left(\frac{\pi}{4} + \frac{\phi_r}{2}\right) + \sigma_3\left[1 + 2\tan\phi_r \tan\left(\frac{\pi}{4} + \frac{\phi_r}{2}\right)\right] \tag{1}$$

For jointed rock

$$\sigma_1 = [c_j(1 + \tan^2\theta) + \sigma_3 \tan\theta(1 + \tan\phi_j \tan\theta)]\left(\frac{1}{\tan\theta - \tan\phi_j}\right) \tag{2}$$

where

ϕ_r is the internal friction angle of intact rock,
c_r is the cohesion of intact rock,
θ is the angle of inclination of the joint (as shown in Fig. 1),
ϕ_j is the friction angle of the joint,
c_j is the cohesion on the joint surface that is zero in the tested samples.

c_r can be evaluated from the following equation:

$$c_r = \frac{\sigma_c(1 - \sin\phi_r)}{2\cos\phi_r} \tag{3}$$

where σ_c is the uniaxial compression strength of the intact rock.

By considering different values for θ in the above equations and calculating σ_1 and σ_3 levels in different conditions, it is possible to determine the maximum possible joint surface orientation in which shearing on the joint occurs before failure in the rock.

2.2 Joint Shape

The surfaces of the tested rock joints are given regular triangular saw-tooth shapes with maximum inclination angle of (i-value) 15 degrees having small roughness on the surfaces of the teeth, as shown in Figs. 1 and 2. By using these shapes it is possible to observe the changes on the shear surface before and after each test. In addition the test results can be analysed easier. The joint surface area for all the samples is 30 cm^2, having 5 main asperities with wavelength 1.5 cm and amplitude 0.2 cm. It should be considered that the number and geometrical specifications of the teeth have been chosen based on practical and analytical evaluations. Other geometrical parameters of the joint surface and sample are shown in Figs. 1 and 2.

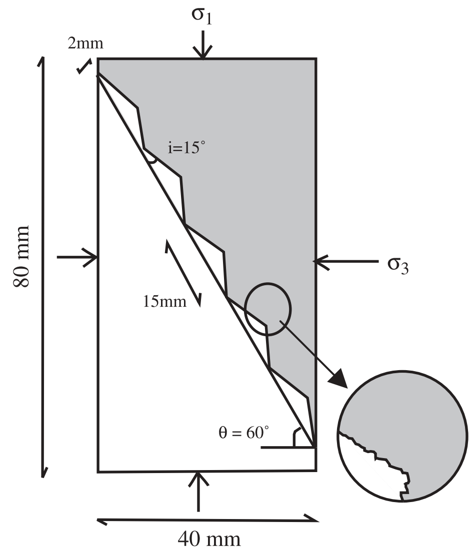

Fig. 1. Schematic section and dimensions of the prepared saw-tooth sample

2.3 Material of the Physical Model

Several studies have been carried out to develop the best possible physical models for testing intact and jointed rocks. These studies have been performed in two main groups. In the first group Stimpson (1970), Huang and Doong (1990), Indraratna (1990), Mostyn and Bagheripour (1998), and Kusumi et al. (1998) developed their physical models using natural and artificial materials (mostly a mixture of cement or plaster with other materials such as sand). In the second group Hutson and Dowding (1990), Boulon (1995), and Armand (2000) have used rock samples with natural joint surfaces or have made artificial joints in natural rock samples using sawing machines, hydraulic or Brazilian fracturing.

Although both of the above mentioned methods have their own advantages and disadvantages, it seems that the first method would be ideal for modelling rock samples with low to medium strength (unconfined compressive strength below 60 MPa) and the second method for medium to hard rock (unconfined compressive strength above 60 MPa). Also it should be considered that the first method is easier than the second one as sawing rock materials in precise dimensions needs special devices such as CNC (Computer Numerical Control) machines, that are not easily available in rock mechanics laboratories.

In the present study the first method has been used. A special cement based mortar, called Rapidex (Lafarge Company) has been used to produce the replicas. Its uniaxial compression strength is about 55 MPa and its tensile strength (using Brazilian test) is about 8 MPa after about 24 h. For evaluation of the friction angle and cohesion, three samples have been compressed triaxially at different levels of confining pressure (2, 4 and 8 MPa) and the evaluated friction angle was about 40 degrees and cohesion was about 16.8 MPa. In order to prepare each sample, the mortar has been passed through a sieve of 0.5 mm, to remove its large particles, and then has been mixed with water in 5 to 1 weight ratio.

2.4 Sample Preparation

To produce samples as physically similar as possible, a special casting procedure has been developed. The casts consisted of PVC tubes and prismatic moulds made from silicon rubber called Silastic (Rhone-Poulene Company). The prismatic moulds of each side of the joint were put into the PVC tube and filled with the mixture of mortar and water. In each stage of sample preparation, three additional samples without joint were also prepared to determine the uniaxial compression and tensile strength of each individual sample in its same age. After 24 h the two ends of the samples were smoothed with a surface grinding machine to produce smooth and parallel surfaces perpendicular to their cylinder axis. One of the final prepared samples is shown in Fig. 2.

3. Testing Devices

The main part of experiments has been conducted using the triaxial compression machine at Laboratory 3S (Lab. 3S) of University of Joseph Fourier, in Grenoble. In addition, some of the cyclic tests were also conducted with the triaxial machine in the Rock Mechanics Laboratory (LMR) in the Swiss Institute of Technology (EPFL), in Lausanne. Photographs of these two units are shown in Fig. 3a and b.

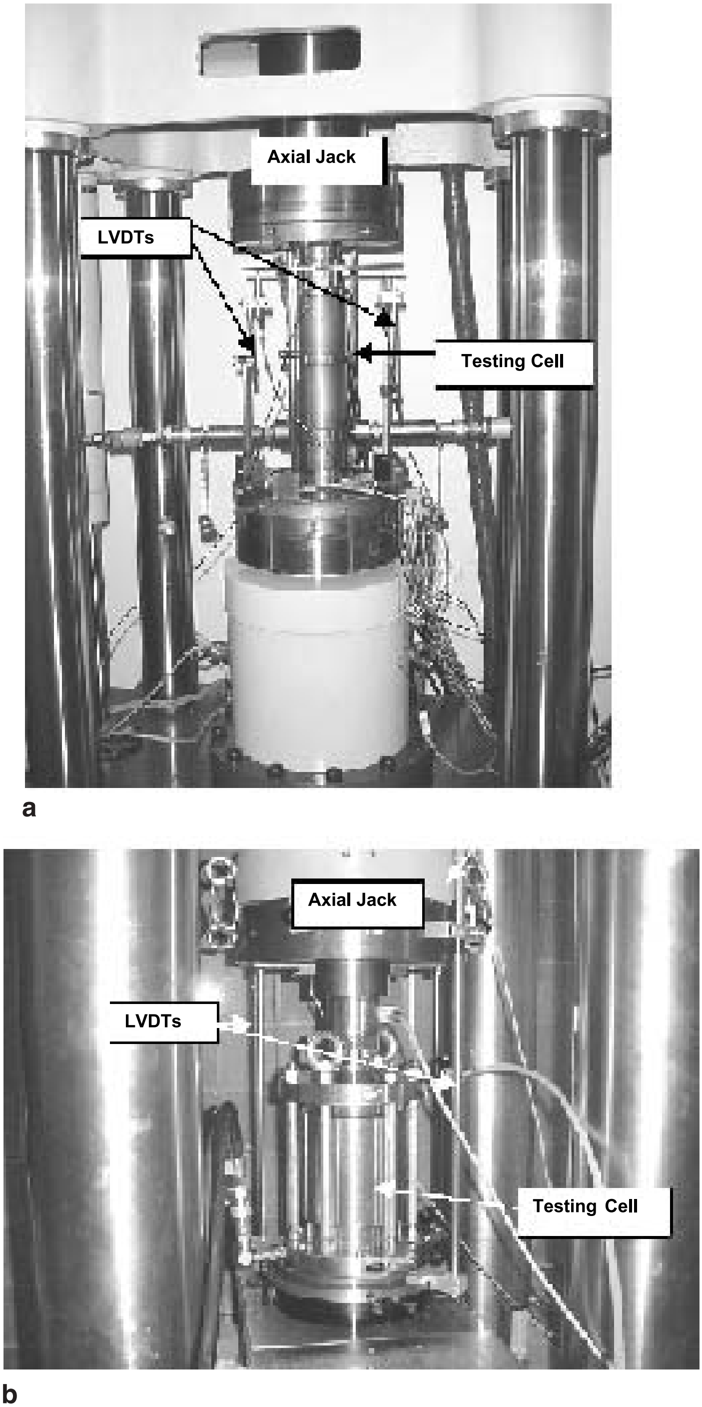

Fig. 3. Testing cell and axial jack; **a** laboratory 3S; **b** laboratory LMR

3.1 Testing Cells

The testing cell in Lab. 3S was a Boehler cell consisting of three parts: base, hollow cylinder and upper piston. The sample was placed on the base after greasing its two ends (to limit the effects of friction) and then covered with a membrane. After filling the cylinder with oil and being sure that there is no air inside the cylinder and its oil passages, then the upper piston was pushed on the upper side of the hollow cylinder. In LMR the testing cell had some differences and the hollow cylinder had a larger diameter. Other procedures of installing a sample into the cell were nearly the same except positioning the upper part and piston (Fig. 3).

3.2 Testing Machine

The testing machine in Lab. 3S involved two different hydraulic equipments for applying the axial load (Schenck Corporation) and the confining pressure (SBEL) and two separate control systems for axial and lateral pressures. In LMR the testing machine involved two different systems for axial loading and confining pressure (Walter + Bai). The total loading capacity of the axial hydraulic jack in Lab. 3S is 1000 kN and in LMR is 2000 kN and for confining pressure it is about 100 MPa in Lab. 3S and 20 MPa in LMR respectively. All loading systems have been equipped with a function generator to control monotonic and cyclic loads. Different modes of loading could be applied, however in the performed tests the monotonic loading and complete cyclic loading (between positive and negative values) with sinusoidal shape have been used. The maximum frequency of axial loading applied in the cyclic tests was 1 Hertz.

All measurements of displacements, force, confining pressure and time, were collected and recorded with IBM PC based systems, with the appropriate sampling rates (0.1 second for cyclic tests and 0.5 second for monotonic tests). In Lab. 3S, 4 vertical LVDTs (as shown in Fig. 3a) have been used to measure simultaneously the vertical displacements. Three of them were fixed around the testing cell, between the moving upper piston and its fixed lower part, and the last one was an individual LVDT connected to the main control machine. In LMR 3 LVDTs have been used for measuring the axial displacement (Fig. 3b). Although there were not considerable differences in the recorded values of LVDTs, their mean value has been used to analyse the data. Confining pressures were controlled with separate systems but were recorded during the tests with the same computers and data acquisition systems. All displacements have been measured between the moving piston and fixed part of the testing cells, as shown in Fig. 3.

Monotonic loading was conducted under displacement control mode and cyclic loading was performed in a combination of stress control and displacement control modes. In these tests, after each part, the system was adjusted to the other mode manually. Cyclic loads in different number, frequency, and stress amplitude were applied on each sample and then monotonic loads were applied to measure the difference of peak and residual shear strength in different testing conditions.

3.3 *Evaluation of the Effects of Internal Friction of the Testing Cells and Membrane on Shear Strength*

For the evaluation of the effects of piston friction with hollow cylinder, some tests have been performed without sample, with the cylinder full of oil. The results showed that the total effect due to friction is about 10% of the maximum applied load, which is removed during analysis. Also the effects of the membrane stiffness on shear strength were studied by testing the samples with one, two and three layers of membrane. The differences in the results were negligible showing the small effect of membrane stiffness on the final results.

3.4 *Reproducibility of the Test Data*

In order to evaluate the reproducibility of the test results, three identical samples have been tested in monotonic mode in the same conditions. The total differences between the results of these tests were approximately 1% of maximum value of shear strength. However, in order to obtain a better correlation of the testing results and to find the best relation from the trends of data (specially for the cyclic tests), additional tests in each condition were needed.

4. Testing Procedures and Results

The variation of shear strength under different conditions of monotonic and cyclic loading has been studied. The tests were performed in two main groups, monotonic and cyclic modes. In the first group the monotonic effects of the confining pressure and rate of axial displacement on shear strength have been considered, whereas in the second group the cyclic effects of the number of cycles, frequency and stress amplitude on peak and residual shear strengths have been studied.

4.1 *Testing Program*

A summary of the tests performed, the parameters considered in each test, and the results of some of the tests are given in Table 1. In this table the peak normal stress (σ_n) is the normal stress value corresponding with the maximum axial stress. This could be calculated by the following equation (Goodman, 1976):

$$\sigma_n = \sigma_3 + (\sigma_1 - \sigma_3)\sin^2\theta \tag{4}$$

4.2 *Effect of Confining Pressure*

The first series of tests explored the effect of confining pressure on the shear strength of the artificial jointed samples. Different levels of confining pressure from 0 to 6 MPa applied on different identical samples and then monotonic axial loading at controlled displacement with a rate of 0.05 mm/min were applied. The selected levels of confining pressure are similar to the levels normally encountered in underground excavations at depth lower than 300 m.

Table 1. Summary of the results of performed tests

Test no.	Type of the test	Changing parameter	Axial displacement at peak point mm	Peak shear stress MPa	Peak normal stress MPa	Residual shear stress MPa	Shear displacement at peak point mm
1	Monotonic (effect of confining pressure)	Confining pressure (1.2 MPa)	0.17	1.81	2.25	1.31	0.15
2	Monotonic (effect of confining pressure)	Confining pressure (4 MPa)	0.20	3.39	5.96	3.05	0.17
3	Monotonic (effect of confining pressure)	Confining pressure (6 MPa)	0.17	4.33	8.50	3.33	0.15
4	Monotonic (effect of rate of loading)	Rate of loading (0.05 mm/min)	0.20	3.39	5.96	3.05	0.17
5	Monotonic (effect of rate of loading)	Rate of loading (0.1 mm/min)	0.39	3.04	5.76	2.66	0.34
6	Monotonic (effect of rate of loading)	Rate of loading (0.2 mm/min)	0.34	2.66	5.53	2.51	0.30
7	Monotonic (effect of rate of loading)	Rate of loading (0.4 mm/min)	0.28	2.45	5.42	2.45	0.24
8	Cyclic – monotonic (effect of number of cycles)	Number of cycles (25 cycles)	0.09	3.30	5.91	2.94	0.08
9	Cyclic – monotonic (effect of number of cycles)	Number of cycles (50 cycles)	0.15	3.23	5.86	2.84	0.13
10	Cyclic – monotonic (effect of number of cycles)	Number of cycles (100 cycles)	0.27	3.15	5.82	2.75	0.23
11	Cyclic – monotonic (effect of number of cycles)	Number of cycles (300 cycles)	0.15	3.09	5.78	2.72	0.13
12	Cyclic – monotonic (effect of number of cycles)	Number of cycles (500 cycles)	0.11	3.04	5.75	2.68	0.09
13	Cyclic – monotonic (effect of number of cycles)	Number of cycles (1000 cycles)	0.08	3.00	5.74	2.64	0.07
14	Cyclic – monotonic (effect of number of cycles)	Number of cycles (3000 cycles)	0.11	2.99	5.73	2.63	0.10
15	Cyclic – monotonic (effect of frequency of cycles)	Frequency (1 Hz)	0.27	3.15	5.82	2.75	0.23
16	Cyclic – monotonic (effect of frequency of cycles)	Frequency (0.5 Hz)	0.50	3.26	5.88	2.88	0.43
17	Cyclic – monotonic (effect of frequency of cycles)	Frequency (0.2 Hz)	0.15	3.34	5.93	2.94	0.13
18	Cyclic – monotonic (effect of amplitude of cycles)	Amplitude of cycles (0.5 MPa)	0.42	2.93	5.69	2.93	0.36
19	Cyclic – monotonic (effect of amplitude of cycles)	Amplitude of cycles (1.6 MPa)	0.23	2.65	5.52	2.65	0.20
20	Cyclic – monotonic (effect of amplitude of cycles)	Amplitude of cycles (1.8 MPa)	0.15	2.32	4.83	2.32	0.13

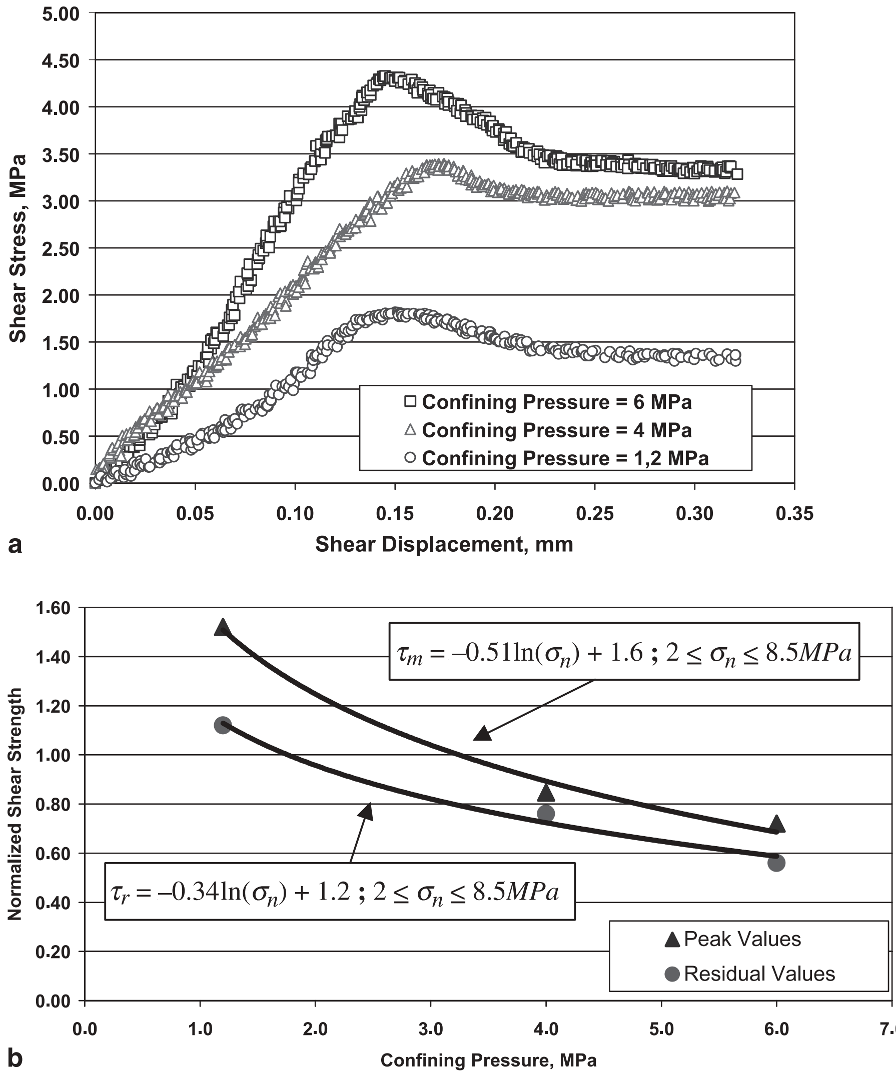

Fig. 4. **a** Shear strength versus shear displacement at different levels of confining pressure; **b** normalized peak and residual shear strength (normalized by confining pressure) versus confining pressure

The resulting shear strength versus shear displacement responses in Fig. 4a show a peak and residual value of shear strength. Goodman (1976) characterised this type of curve as representative of rough unfilled joints. There are three distinct regions for each curve in Fig. 4a, identified as elastic, peak and residual or plastic parts. In this paper, the maximum shear stress, τ_m, is called the peak shear strength and the minimum value of post peak shear stress is called the residual shear strength, τ_r. Figure 4b gives plots of normalized peak and residual shear strength (normalized by confining pressure) versus confining pressure. As shown in this figure, there are non-linear trends for increasing shear strength with confining pressure. The best fit of the data gives the following relations:

Peak shear strength:

$$\tau_m = 1.82 \ln(\sigma_n) + 0.30 \quad \text{for} : 2 < \sigma_n < 8.5\,\text{MPa} \tag{5}$$

Residual shear strength:

$$\tau_r = 1.48 \ln(\sigma_n) + 0.34 \quad \text{for} : 2 < \sigma_n < 8.5\,\text{MPa} \tag{6}$$

where τ_m is peak shear strength, τ_r is residual shear strength and σ_n is the normal stress.

It can be concluded that the confining pressure has an important effect on the shear strength of the jointed sample, mainly for the lower range of values. By increasing the level of confining pressure, this effect decreases nearly logarithmically, which means that for very high values of confining pressure the effect of confinement on shear strength would be of a second order of importance. Hutson and Dowding (1990) and Armand et al. (1998) have reported similar results by using direct shear tests. It also should be noted that for all the samples, observations done after each test showed small changes in first order asperities over the shear surface in comparison with their initial shapes. These small changes were due to the limited applied shear displacements and relatively low level of confining pressure. There was no evidence of severe damage on the main teeth after each test, so it can be concluded that the changes occurred only on the second order asperities and roughness on the surfaces of the main teeth.

4.3 Influence of Rate of Axial Displacement

The second series of tests were performed in order to investigate the influence of the axial displacement rate on the shear strength. Four monotonic loading tests at different displacement rates equal to 0.05, 0.1, 0.2 and 0.4 mm/sec have been performed with a confining pressure of 4 MPa. The results obtained are shown in Fig. 5a. The variation of the shear strength due to the rate of axial displacement is shown in Fig. 5b, as normalized peak and residual shear strength (normalized by confining pressure) versus rate of axial displacement. The logarithmic trends between the data can be represented by the following equations:

Peak shear strength:

$$\tau_m = -0.46 \ln(\dot{w}) + 1.98 \quad \text{for} : 0.05 < \dot{w} < 0.4\,\text{mm/min} \tag{7}$$

Residual shear strength:

$$\tau_r = -0.28 \ln(\dot{w}) + 2.12 \quad \text{for} : 0.05 < \dot{w} < 0.4\,\text{mm/min} \tag{8}$$

where $\dot{w}$ is the rate of axial displacement.

Figure 5b shows that the shear strength decreases as the axial displacement rate increases. These new peak and residual values can be called fast peak and fast residual as used by Fearon (1999). Similar results have been reported by Scholtz (1990) and Boulon (1995). Another important result is the reduction of the effect of roughness for high values of shear velocity. As shown in Fig. 5a, for higher levels of displacement

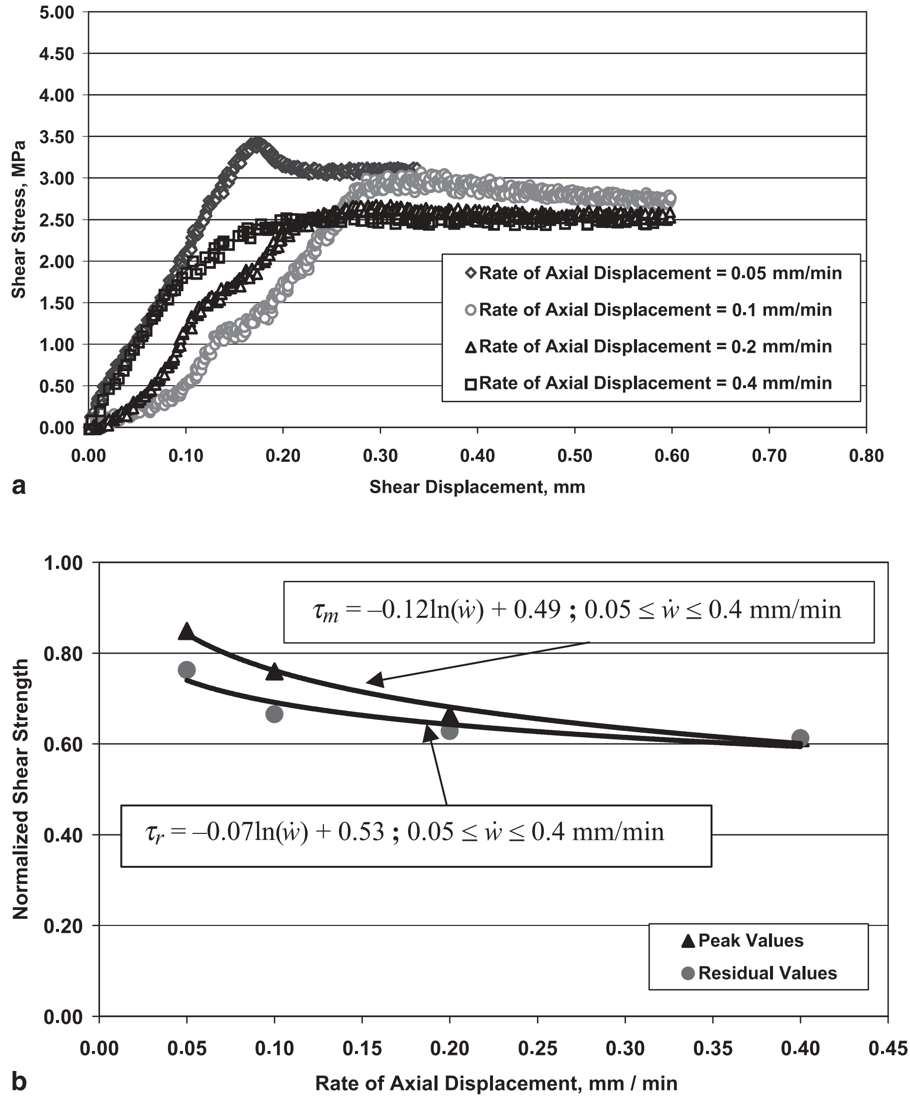

Fig. 5. **a** Variations of shear stress with rate of axial displacement under 4 MPa of confining pressure; **b** normalized peak and residual shear strength (normalized by confining pressure) versus rate of axial displacement

rates, the differences between peak and residual values become negligible. It probably means that when the rate of axial displacement and thus the shearing velocity increases, the secondary asperities do not play an important role in defining the shear strength of the jointed rocks. This remark should be taken into account for evaluating the rock mass maximum shear strength at high shear velocity (as in the case of seismic loading). The results obtained are in accordance with the previous studies reported by Crawford and Curran (1981) and Hutson and Dowding (1990). Of course, based on the presented results it seems that the importance of shear velocity is less than other parameters such as joint roughness, confining pressure and uniaxial strength of intact rock.

The reduction of shear strength due to shearing velocity can be evaluated by considering the adhesion theory, which states that the frictional resistance is proportional to the true area of contact. As discussed by Sture et al. (1984), the true area of contact may be smaller for surfaces that are only in contact for short period of time due to a lag in elastic or plastic deformation. Thus rapidly shearing hard surfaces might exhibit lower shear strength.

4.4 Effect of Experiencing Small Cyclic Loads on Shear Strength

The effects of three different cyclic parameters including the number of cycles and their frequency and stress amplitude have been studied. For modelling of small repetitive earthquakes, small cyclic loads were applied to identical samples (stress control mode), which were then subjected to monotonic loading (displacement control mode) to evaluate the new peak and residual shear strength of the samples after experiencing small cyclic loads. Brief descriptions of the testing procedures and of the results obtained are discussed in the following.

4.4.1 Effect of the Number of Cycles

The first series of cyclic-monotonic tests focused on the effect of the number of cycles of loading (in 25, 50, 100, 300, 500, 1000 and 3000 cycles). These tests have been performed using the triaxial machine in Lab. 3S under a confining pressure of 4 MPa. The stress amplitude in all the tests was about half of the maximum value attained in monotonic tests, with a rate of 0.05 mm/min. Also the frequency of the cyclic loading in all the tests was about 1 Hz.

The method of loading was a combination of stress control and displacement control modes as already discussed. The rate of axial displacement during the displacement control tests was about 0.05 mm/min. Figures 6 to 10 show the results of these tests. The small observed variations of axial displacement corresponding with the peak values can be related to the possible experimental constraints, as taking place after the first part of the test (controlled stress) and before starting the second part (controlled displacement).

Figure 11 depicts the normalized peak and residual values of shear strength versus the number of cycles. The following equations can be used to fit the trends of behaviour:

Peak shear strength

$$\tau_m = -0.06 \ln(\mathrm{NC}) + 3.43 \quad \text{for} : 1 < \mathrm{NC} < 3000 \tag{9}$$

Residual shear strength

$$\tau_r = -0.06 \ln(\mathrm{NC}) + 3.10 \quad \text{for} : 1 < \mathrm{NC} < 3000 \tag{10}$$

where NC is Number of Cycles.

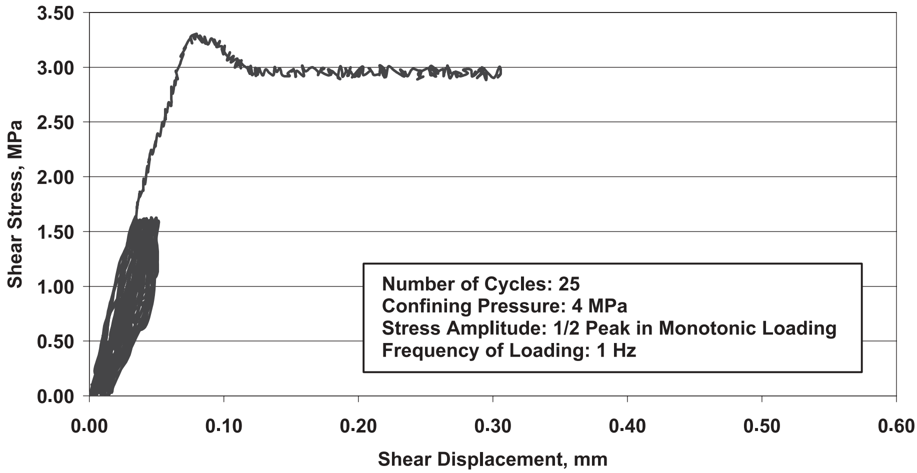

Fig. 6. Variations of shear stress with shear displacement for 25 cycles of loading

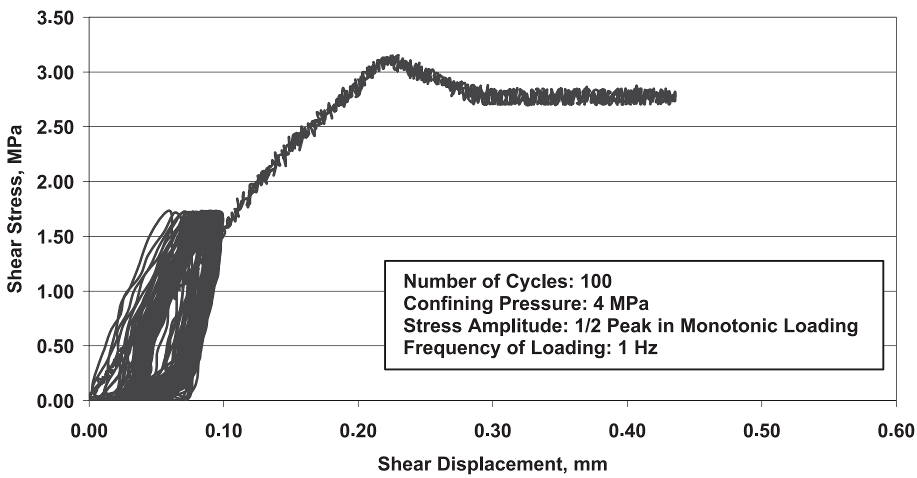

Fig. 7. Variations of shear stress with shear displacement for 100 cycles of loading

As shown in the Fig. 11, there are some differences between the levels of shear strength in the first part of the curves (lower than 300 cycles), whereas these differences become nearly negligible after 1000 cycles. It can be concluded that the shear strength of rock masses would be nearly constant after experiencing a high number of cycles, if the applied cycles have stress amplitudes lower than 50% of that at their maximum strength in monotonic loading. As discussed by Hencher (1980), dynamic loading can affect the static friction angle, so the observed reduction of shear strength can be related to reduction of friction angle.

The other important result that is apparent from Fig. 11 relates to the total effect of the number of loading cycles on shear strength. In the range of loading amplitudes

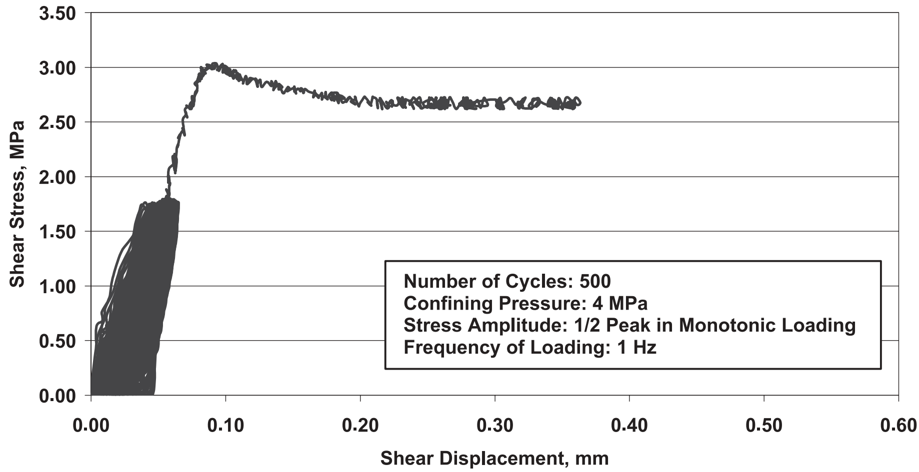

Fig. 8. Variations of shear stress with shear displacement for 500 cycles of loading

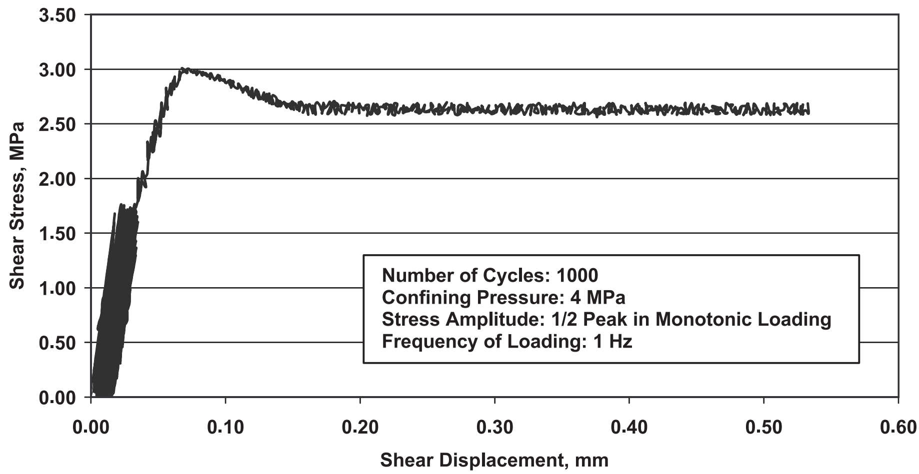

Fig. 9. Variations of shear stress with shear displacement for 1000 cycles of loading

considered, the total effect of the number of cycles on shear strength is to lower the maximum values attained in monotonic loading by no more than 20%. So it seems that by considering an appropriate factor of safety for engineering evaluations, this effect can be accounted for.

4.4.2 Effect of History of Frequency of Small Cyclic Loads

Although the frequency does not play an important role in assessing the shear strength of intact rocks, as shown by Burdine (1963), Haimson and Kim (1972) and Ray et al. (1999),

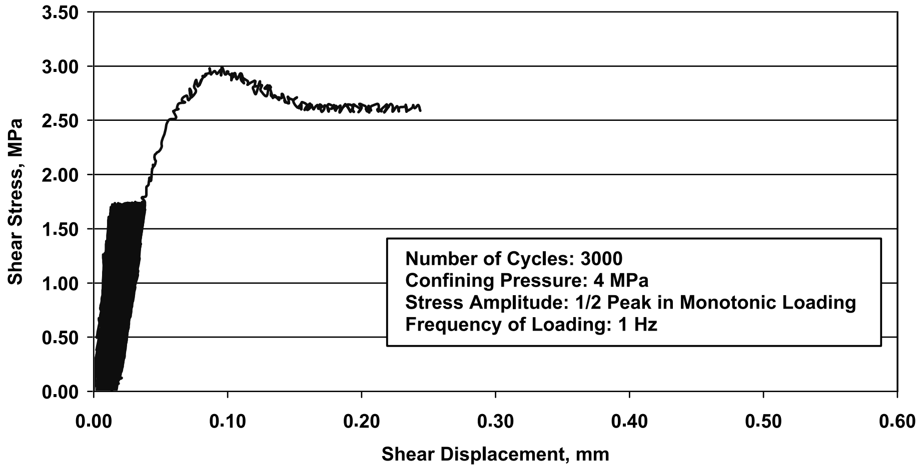

Fig. 10. Variations of shear stress with shear displacement for 3000 cycles of loading

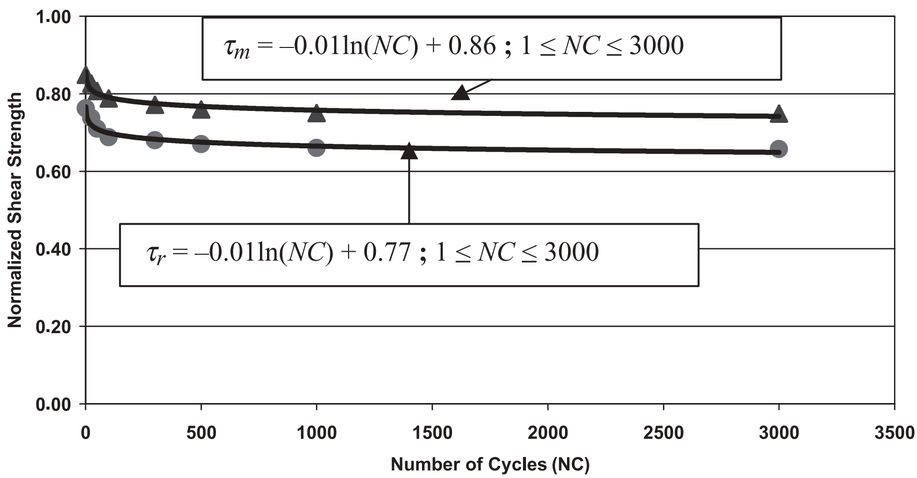

Fig. 11. Normalized peak and residual shear strength (normalized by confining pressure) versus number of cycles of loading

this could not be the case for jointed rock. This can be concluded by reviewing the reports on damaged underground structures in jointed rock during seismic events or explosions.

The effect of frequency of cyclic loading was studied with frequency between 0 to 1 Hertz for 100 cycles under 4 MPa confining pressure. Some of the results are shown in Figs. 12 and 13. Figure 14 presents the results of some of these tests as normalized peak and residual shear strength versus frequency levels. As shown in these figures, a continuous reduction of shear strength with increase of frequency has been experienced. For a better evaluation of the effects of frequency, it is necessary to perform more tests with higher levels of frequency as can be expected in near field of earthquakes or explosions.

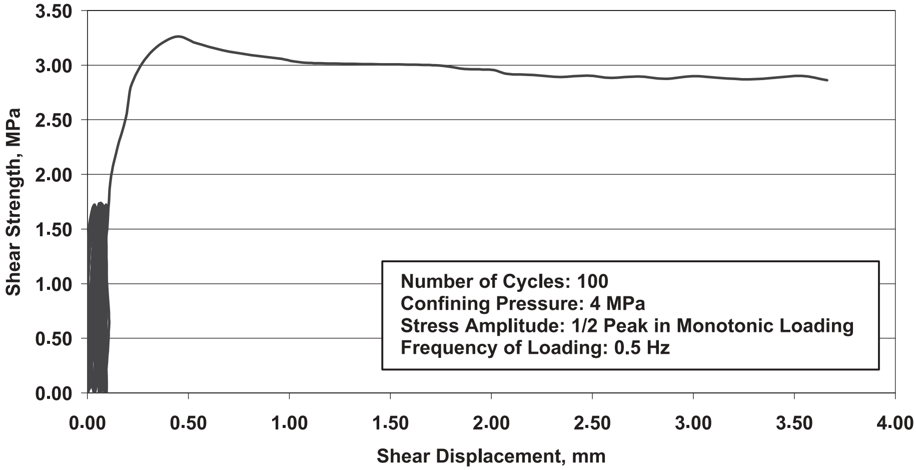

Fig. 12. Variations of shear stress with shear displacements for cyclic loading with 0.5 Hertz frequency under 4 MPa confining pressure

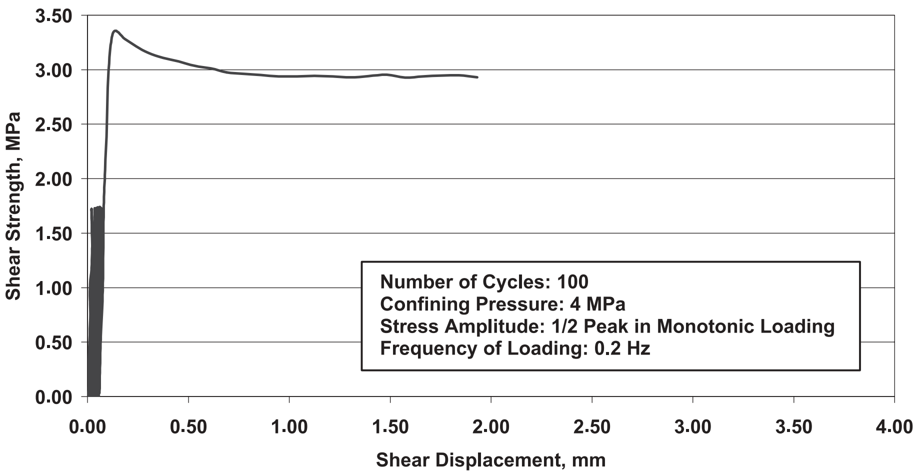

Fig. 13. Variations of shear stress with shear displacements for cyclic loading with 0.2 Hertz frequency under 4 MPa confining pressure

4.4.3 Effect of Stress Amplitude

The effect of stress amplitude was studied in LMR in Switzerland. Different levels of stress amplitude have been applied on the identical samples and shear strength have been measured after applying 100 cycles. The frequency of the cycles was 1 Hertz and the rate of axial displacement for the displacement control part of the tests was about 0.6 mm/min. All the tests have been performed under 4 MPa confining pressure. The results of some of the tests are presented in Figs. 15 and 16. A continuous trend of

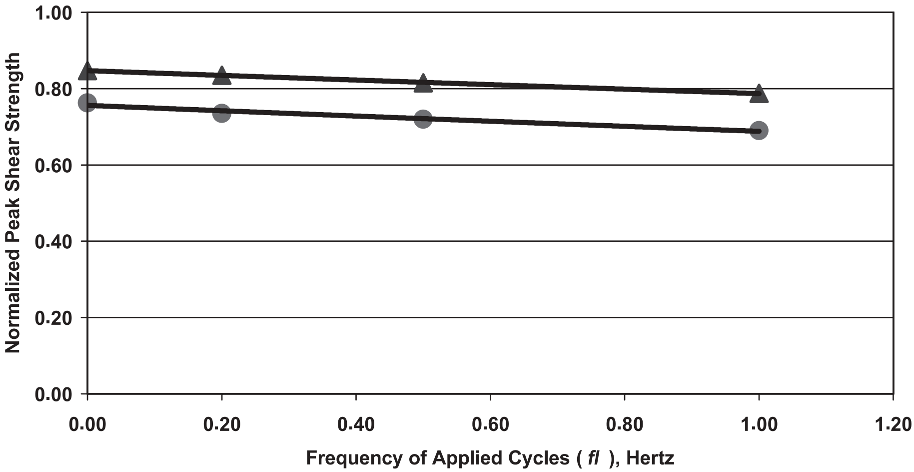

Fig. 14. Normalized peak and residual shear strength (normalized by confining pressure) versus frequency of loading

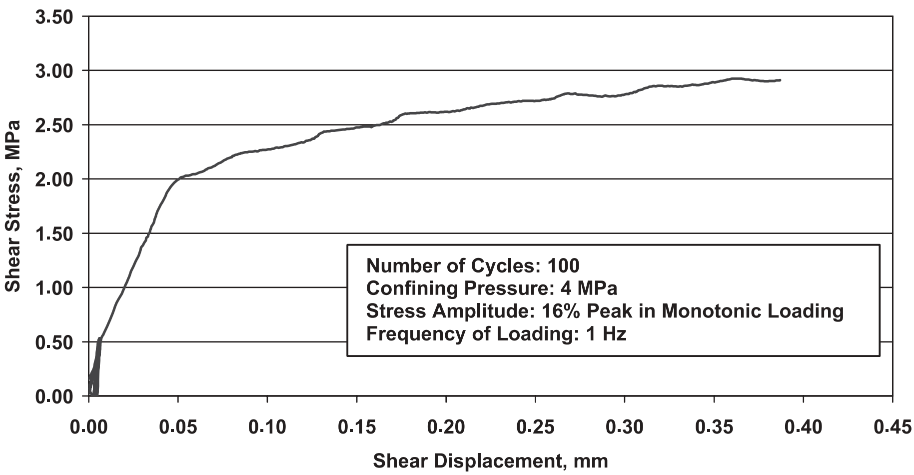

Fig. 15. Variations of shear stress with shear displacements in 0.5 MPa stress amplitude

reduction of the shear strength due to stress amplitude, as shown in these figures, is confirmed in Fig. 17. The data in Fig. 17 show an important effect of stress amplitude on shear strength of the tested samples. In fact by applying cyclic loads with amplitude more than 50% of static shear strength, shear strength decreases sharply, but below this level there is no important effect of stress amplitude on shear strength. The level of critical stress amplitude is concerned with the relative displacement and asperity condition.

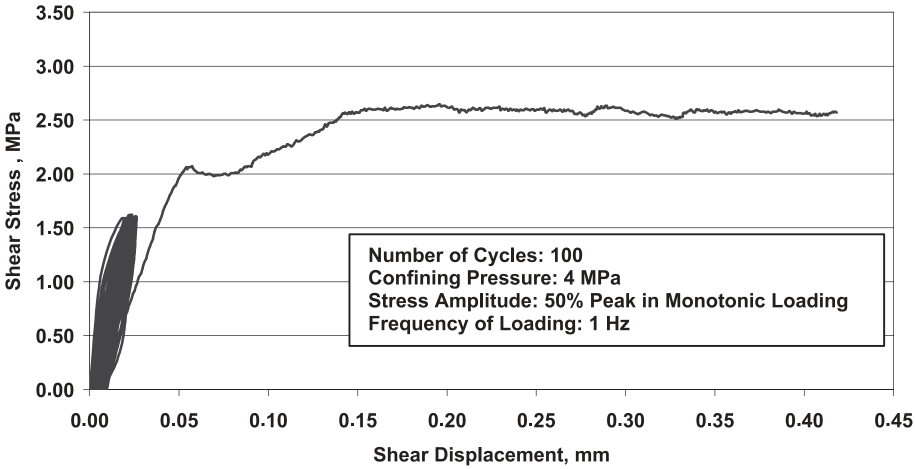

Fig. 16. Variations of shear stress with shear displacements in 1.6 MPa stress amplitude

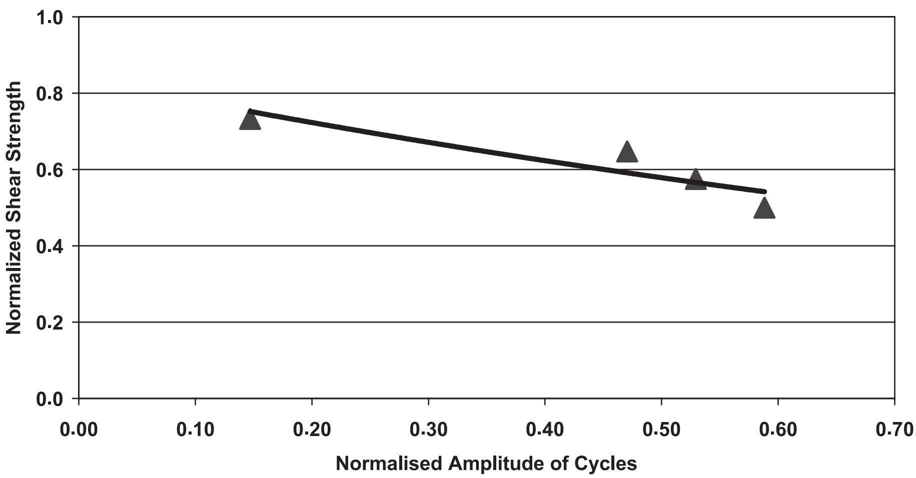

Fig. 17. Normalized mean shear strength (normalized by confining pressure) versus normalized amplitude of cyclic loading (normalized by peak value in monotonic loading condition)

The performed tests in Lab. 3S with higher levels of stress amplitude (more than 70% of static shear strength) showed that during the cyclic part the shear strength reaches its critical level and two sides of the samples start to move on each other after a few cycles. It can be concluded that during repetitive seismic loading on jointed rocks, if the stress amplitude is more than a critical value, displacement and finally instability on the under/above ground structures in rock masses may occur. This critical value can be evaluated by laboratory testing, considering several parameters such as confinement, roughness, rock type, etc.

5. Conclusions

Results of the laboratory study of the artificial rock joints presented herein show that 50's to 1000's of repetitive cyclic motions producing shear stresses of 50% of peak strength may reduce peak joint shear strength by 5 to 15%. Other main conclusions are as follows:

1. When shearing velocity along the joint surface increases from 0.04 mm/min to 0.35 mm/min in monotonic loading, the peak shear strength of artificial rock joints may reduce by 25%. In addition the effects of the second order asperities on shear strength can be eliminated when shearing velocity increases from 0.04 mm/min to 0.35 mm/min.
2. After 500 repetitive cyclic motions having shear stresses of 50% of peak strength, shear strength of artificial rock joints remains nearly constant.
3. Increasing of the frequency of cyclic motions producing shear stresses of 50% of peak strength from 0.2 to 1 Hertz in stress control stage may reduce the peak shear strength of artificial rock joints by 3 to 10% for 100 cycles.
4. If stress amplitude of the repetitive cyclic motions increases from 16% to 60% of peak strength, the shear strength of the artificial rock joints subjected to 100 stress cycles may reduce by maximum 25%.

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