

# Shear Behaviour of Rock Joints under CNS Boundary Conditions

## Comportement en cisaillement de joints rocheux en condition de raideur normale constante

Shrivastava A.K.

*Delhi Technological University, Delhi, India*

Rao K.S.

*Indian Institute of Technology, Delhi, India*

**ABSTRACT:** The shear behaviour of rock joints depends up on many factors, the correct evaluation of this is possible only if these parameters are properly considered during experimental investigation, constitutive modelling and numerical modelling. Which is important for safe and economical design of underground openings in jointed rocks, stability analysis of rock slopes, risk assessment of underground waste disposal, design of foundation on rock and design of rock socketed piles. These concerns invite accurate quantification of shear strength of unfilled and infilled joints, proper understanding of the basic mechanics of discontinuity and the principles involved in their shear deformation. This can be done through in-situ or laboratory large scale testing on natural rock or laboratory testing on physical model. In the present paper the detail account of test results of direct shear tests performed on large size modeled unfilled and infilled rock joints under different boundary conditions is systematically presented. It is observed that the constant normal stiffness (CNS) conditions better simulate the field conditions of the loading and shear strength predicted under CNS condition is more than the constant normal load (CNL) conditions for both unfilled and infilled joints.

**RÉSUMÉ :** Le comportement en cisaillement de joints rocheux dépend de nombreux facteurs, dont l'identification n'est possible que par approche expérimentale numérique ou rhéologique. Cela est important pour la conception sécuritaire et économique de cavités souterraines dans les roches fracturées, l'analyse de stabilité des talus rocheux, l'évaluation du risque d'élimination des déchets souterraine, la conception de fondation au rocher et la conception pieux. Une quantification précise de la résistance au cisaillement des joints remplis ou non, ainsi qu'une bonne compréhension des mécanismes de base de la discontinuité et des principes appliqués à leur déformation en cisaillement sont nécessaires. Ceci est possible grâce à des essais en vraie grandeur in situ ou en laboratoire sur modèle physique. Dans le présent document, le détail des résultats des essais de cisaillement direct effectués en vraie grandeur avec ou non un remplissage des joints sous différentes conditions aux limites sont systématiquement présentées. On a observé que les conditions de la rigidité normale constante (CNS) simulent mieux les conditions sur le terrain et que la résistance au cisaillement prédite sous condition de CNS est plus grande que pour les conditions de charge normale constante (CNL) pour les deux types de joint.

**KEYWORDS:** Shear Behaviour, Rock Joints, CNL, CNS, Direct Shear, Infill, Unfill, Dilation, Shear Strength, Deformation.

## 1 INTRODUCTION

Rock joints are mechanical discontinuities having geological origin. These discontinuities are present in the form of joints, faults, bedding planes or other recurrent planar fractures in the rock mass. In general, strength and deformability properties of these discontinuities are quite different from those of intact rock, and in many cases, the discontinuities completely dominate the shear and deformation behaviour of the in situ rock mass in a given stress conditions.

The presence of infill or gouge material in the joints further reduces the shear strength. The sources of infill material include products of weathering or overburden washed into open joint, water conducting in discontinuities, precipitation of minerals from the ground water, by-products of weathering alterations along joint walls, crushing of parent rock surfaces due to tectonic and shears displacements, and thin seams deposited during formation. In general, infill materials may consist of partially loose to completely loose cohesionless soil or fine grained clay. Normally fine-grained clays are more frequently found as fillers and are more troublesome in terms of structural stability. Thickness of the infill material varies from micrometers to several meters and it plays an important role in shear behaviour. In tectonically crushed zones, the infill thickness may exceed several meters.

These rock joints unfilled or infilled are the weakest plane which tries to slide or shear one over the other due to construction of foundations of a structure and tunnels or

highways and railways on the rock slopes. Hence, for safe and economical analysis of all the above cases it is important to understand the strength and deformation behaviour of the rock joints under direct shearing conditions. Shrivastava and Rao (2009) discussed in details the influence of factors like (a) boundary condition (b) shear rate (c) joint roughness (d) size of joint i.e. scale effect (e) joint condition i.e. unfilled joint/in filled joint on the direct shear strength of rock joints.

There are two boundary conditions i.e constant normal load (CNL) and constant normal stiffness (CNS) boundary conditions under which the shear behaviour of rock joints can be studied. The planar rock joints can be investigated in the laboratory by using a conventional direct shear apparatus where the normal load is kept constant (CNL) during the shearing process. This particular mode of shearing is suitable for situations where the surrounding rock freely allows the joint to shear without restricting the dilation or there is no dilation during the shearing process, thereby keeping normal stress constant during shearing process. Shear testing under a constant normal load (CNL) boundary condition is only beneficial for cases such as non-reinforced rock slopes or planar rock joints, but natural rock joints are seldom planar.

However, for non-planar discontinuities, shearing results in dilation as one asperity overrides another, and if the surrounding rock mass is unable to deform sufficiently, then an inevitable increase in the normal stress occurs during shearing. At any time  $t$  if the normal stress  $P_n(t)$  then increase in normal stress

on the shear plane at any time  $t + \Delta t$  is equal to  $P_n(t) + k_n \Delta \delta v(t + \Delta t)$ , where  $k_n$  is the stiffness of the surrounding rock mass and  $\Delta \delta v(t + \Delta t)$  is the dilation restricted in the given interval of time. Therefore, shearing of rough joints under such circumstances no longer takes place under constant normal load (CNL), but rather under variable normal load where stiffness of the surrounding rock mass plays an important role in the shear behaviour. This particular mode of shearing is called as shearing under constant normal stiffness (CNS) boundary conditions. For analysis and design of tunnels, foundations and rock slopes, shear tests results under CNL condition are not appropriate. A more representative behaviour of joints would be achieved if the shear tests were carried out under boundary conditions of constant normal stiffness (CNS).

In past decades numerous shear models have been proposed based on experimental, analytical and numerical study to find out the shear behaviour of rock joint. These models available in the literature fail to appropriately determine shear behavior of rock due to limitations of boundary condition i.e. CNL boundary condition is used for modeling like (Patton 1966, Barton 1973 and 1976, Haberfield and Johnston 1994 and Yang and Chiang 2000).

But very few studies are available under CNS condition i.e. constant normal stiffness conditions. To study the shear behaviour under CNS conditions, the conventional direct shear test apparatus working under CNL boundary condition is modified by different researchers like, (Obert et al. 1976, Ooi and Carter 1987, Johnston et al. 1987, Indraratna 1998, Gu et al. 2003 and Kim et al. 2006) to be used for CNS boundary conditions.

Despite frequent natural occurrence of infill material, filled discontinuities have been studied much less, perhaps because of the difficulties arising from sampling, testing or due to increased number of variable parameters for constitutive and numerical modelling. Due to limited research, it is a common practice to assume the shear strength of an infilled joint equal to the infill material alone, regardless of its thickness. Kanji 1974 reported that the shear strength of the infilled joint is lower than that of the infill material. Hence this assumption will lead to unsafe designs. These uncertainties in estimation have motivated the present work.

## 2 PHYSICAL MODELLING OF ROCK JOINTS

It is difficult to interpret the results of direct shear test on natural rock because of difficulty in repeatability of the sample. To overcome this problem a model material is searched which can easily be handled and reproducibility of the sample can be ensured. To achieve this different brands of plaster of Paris and dental plasters at different moisture content and curing period in isolation or combinations have been tried. Finally, plaster of Paris is selected because of its universal availability and its mould ability into any shape when mixed with water to produce the desired joints and also long term strength is independent of time once the chemical hydration is completed. To characterize model material a series of physical and mechanical tests on a number of specimens prepared by mixing the prescribed quantity of water with plaster of Paris powder were carried out. The prescribed percentage of water is decided so as to achieve proper workability of the paste and required strength to simulate the soft rock. Different water cement (POP) ratios were tried in order to obtain desired strength and workability. The ratio which is finally selected is 0.60.

The physical and engineering properties of the model material were determined in the laboratory as per the suggested methods of ISRM 1977 and 1979. The average uniaxial compressive strength and tangent modulus at 50% of peak axial stress of model material at 0.60 water cement (POP) ratio and after 14 days of air curing is 11.75 MPa and 2281 MPa respectively. Thus, the material can be classified as 'EL' based

on Deere and Miller 1966 classification chart, indicating that the material has very low strength (E) and low modulus ratio (L). The cured plaster of Paris samples showed a consistent uniaxial compressive strength ( $\sigma_c$ ) in the range of 10.58 to 13.22 MPa and a Young's modulus of 1856 to 2631 MPa. These ranges of strength and modulus values are suitable for physically and mechanically simulating the behaviour of jointed rocks like siltstone, sandstone, friable limestone, clay shale and mudstone.

### 2.1 Preparation of unfilled rock joint samples

The asperity plate of 15<sup>o</sup>-15<sup>o</sup> angle designed and fabricated by Rao and Shrivastava 2009 has been used to produce desired asperity in the sample as shown in Fig. 1(a). The plaster of Paris with 60% of the moisture is mixed in the mixing tank for 2 minutes and then the material is poured in the casting mould which is placed on the vibrating table. Vibrations are given to the sample for a period of 1 minute and then the sample is removed from the mould after 45 minutes and kept for air curing for 14 days before testing.

### 2.2 Preparation of infilled rock joint samples

The infill material is selected to simulate the field conditions. In the present work combination of fine sand and mica dust both passing through 425micron sieve and plaster of Paris is selected. The selected composition is plaster of Paris 40%, fine sand 50% and mica dust 10% mixed together with water 45% by weight of total mass of the material. The uniaxial compressive strength of the 7 days air cured infill material is 3.47 MPa and direct shear tests carried on the infill material gave friction angle and cohesion, 28.8<sup>o</sup> and 0 respectively.

The infill joint with required thickness as shown in Fig. 1 (b) is created on the sample with the help of infill mould developed by Shrivastava et al. 2011.

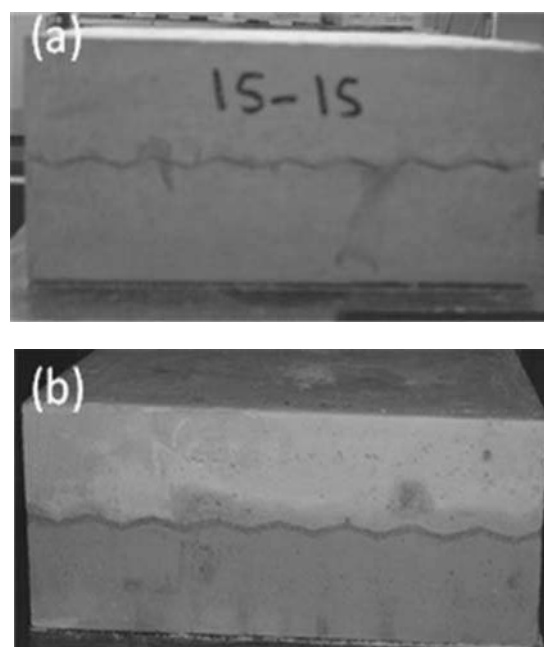


Figure 1. Photograph of simulated rock joints (a) unfilled (b) infilled.

The samples are placed on the mould and tighten at suitable point so that the required thickness of the infill material is created. The infill material is spread over the lower sample and the asperity plate is put over the infill material and the asperity plate is compressed from the top with the help of C-clamps so that the uniform pressure is applied on the sample and the same

asperity is created on the infill material. The upper mould is now placed over the lower mould with the help of the guide rod and movable screw the correct placement and thickness of the infill material is insured. The whole assembly is now compressed from the top with the help of C- clamp, after 30 minutes the sample is removed from the mould and kept for air curing for 7 days before testing.

### 3 SHEAR BEHAVIOUR

To study the effect of CNS boundary condition and infill material on the shear behaviour of the rock joints the extensive tests were planned and conducted under different boundary conditions on the equipment designed and developed by the authors as shown in Fig. 2, on  $15^0$ - $15^0$  asperity unfilled and infilled joint at different initial normal stress ( $P_i$ ) ranging from 0.1 to 2.04 MPa.

In this equipment Normal and shear load is applied through an electro hydraulic servo actuator unit which works on closed loop principle. The displacements are measured by LVDT's mounted on the specimen. The data acquisition system has 16 channels, 2 channels for load cell, 6 channels for LVDTs and remaining 8 channels are free for additional input. The data acquisition system converts the mechanical and electrical signals in to the digital data. The output of signal is connected to CPU via cord. The load and deformation values are stored at desired intervals as note pad data. The direct shear software developed is having the facility to collect data and plot online graphs.

In this apparatus CNL and CNS boundary conditions are reproduced by an electro hydraulic servo-valve which under the control of an electronic controller controls the application of hydraulic power to a linear actuator to provide the programmed force to the test specimen.

The thickness of the infill material ( $t$ ) and height of asperity ( $a$ ) is maintained at 5mm for the present case. The normal stiffness ( $k_n$ ) of surrounding rock joints are set to be 8 kN/mm for CNS boundary condition.



Figure 2 Close up view of large scale direct shear machine (Rao and Shrivastava 2009).

The effect of shearing rate for different asperity joint under different boundary conditions have been studied by Rao et al. 2009 and they found that the effect of increasing shear rate for shearing rate  $> 0.5$ mm/min is to increase the peak shear stress for the same initial normal stress and for shearing rate  $\leq 0.5$  mm/min, the effect of shearing rate is not much on the peak shear stress. Hence, for the present case rate of shearing is maintained as 0.5mm/min during each test.

The shear behaviour of  $15^0$  - $15^0$  asperity unfilled and infill joint under CNL ( $k_n=0$  kN/mm) and CNS ( $k_n=8$  kN/mm) boundary condition is plotted as shown in Fig.3 and Fig.4 respectively. The stress – displacement behaviour is characterized by a well defined peak. It is clear from the test result that CNL boundary condition always under predicts the shear strength of the joint as compared to CNS boundary condition for the same initial normal stress. This is due to increase in normal stress at the shearing surface during the shearing because of restriction in dilation imposed by simulated surrounding rock stiffness.

Shrivastava and Rao 2011 reported the variation of normal stress with shear displacement under CNL and CNS boundary condition for similar type of synthetic rock joints. The normal stress on the shear plane remains constant during testing for CNL conditions. However, for CNS conditions normal stress increases as asperity slides on over the other. Variation of normal stress under CNS conditions exactly follows the shape of the asperity, but angle of inclination is different.

The shear strength of the infill joint is less than that of unfill joint for both CNS and CNL condition, when tested under the same  $P_i$ . But for CNS boundary conditions % decrease of shear strength of infill joint is lower at higher  $P_i$ . It may be due to failure of infill material under increased compression.

The shear stress and displacement behaviour curve of modelled rock joint can be divided into three zones. In the zone I predominantly sliding of the sample take place without shearing of the asperity. The limit of the zone-I depends upon the shear strength of the material and shear stress increases at higher rate with small shear displacement in this zone. In zone-II, shearing of the asperity is more predominant than the sliding. The limit of the zone-II is up to maximum shear stress, in this zone rate of increase in shear stress decreases with shear displacement. Zone-III is the last zone where all the asperity is sheared off. Due to deposition of the crushed material on the joints, shear stress decreases or increases slightly with shear displacement depending upon CNL or CNS conditions.

Probable strength envelope is found by joining the peak shear stress of different stress path and plotted as shown in Fig.5.

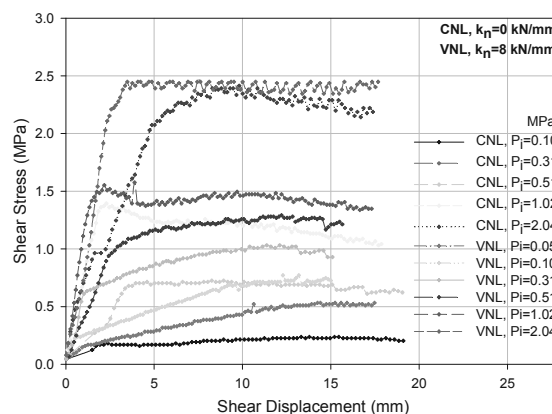


Figure 3. Shear behaviour of  $15^0$ - $15^0$  unfilled joint under CNL and CNS boundary condition.

The shear test result on  $15^0$  -  $15^0$  asperity modelled rock joint reflects that the strength envelope for both CNL and CNS boundary condition is curvilinear and curvature is same up to low  $P_i$  i.e  $P_i < 0.09 \sigma_c$  of the sample and after that the curvature of the strength envelope is change. The change in the slope of the strength envelop indicates that the complete shearing of the asperity at that normal stress and sliding of samples takes place after that normal stress. At low normal stress, shear strength significantly increased because of the enhanced shearing resistance offered by the angular asperities. However, at higher

normal stress increased degradation of the asperities is associated with decrease in increase of the shear strength and at very high normal stress the complete shearing of the asperity takes place and there is no effect of boundary conditions on shear strength.

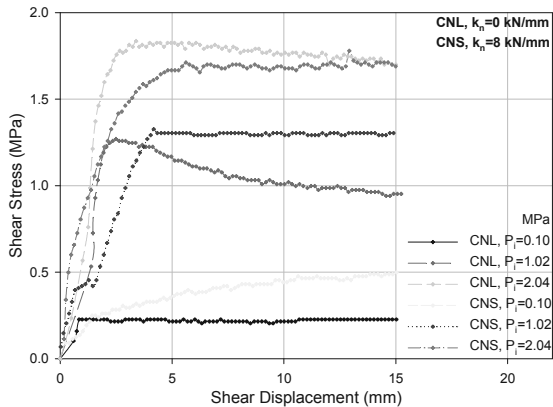


Figure 4. Shear behaviour of 15°-15° infilled joint under CNL and CNS boundary condition.

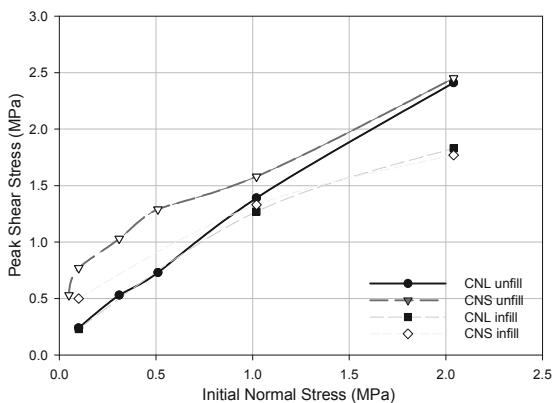


Figure 5. Strength envelope of 15°-15° unfilled and infilled joint under CNL and CNS boundary condition.

The increase in shear strength for unfilled joints for CNS conditions varies from 221% to 6% of the CNL conditions when  $P_i$  increases from 0.10 MPa to 2.04 MPa.

The curvilinear strength envelope for infilled joint is observed as presented in Fig.5 at all range of  $P_i$ . But peak shear stress of the infill joint is always less than that of unfilled joint. Maximum reduction in shear strength of the infill joint as compared to unfill joint for CNS condition is observed to be 35% at  $P_i=0.01$  MPa and % reduction in shear strength decreases with increase in the  $P_i$ . At high  $P_i$ , the close look on the sheared sample reflected breakage of the infill material, which has resulted into more participation of the joints, hence less reduction in shear strength.

#### 4 CONCLUSIONS

The experimental studies on physically modeled unfilled and infilled rock joints have been conducted to understand the effect of boundary conditions and infill thickness on shear behavior. The conclusions made from the test results are summarized below:

- I. CNL boundary condition is not suitable for non planar rock joints and it under predict the shear strength, which makes the design uneconomical.

- II. The % increase in shear strength of unfilled joint under CNS conditions as compared to CNL conditions is as high as 221 for  $P_i=0.10$  MPa.
- III. The effect of the infill material in the joint is to reduce the shear strength and a maximum reduction of 35% is observed for 5mm infill thickness under CNS conditions at  $P_i=0.10$  MPa.
- IV. The effect of boundary conditions on the shear strength of non planar unfilled/ infilled rock joints decreases with increase in  $P_i$ , the effect is almost nil for  $P_i \geq 0.18\sigma_c$ .

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