Site investigations and rock mass characterisation A Unique Phenomenon of Shear Strength Parameters of Jointed Rock Mass

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Abstract

This paper deals with in-situ shear strength parameters of rock mass determined by conducting a set of five shear tests in right abutment drift and two sets of five/six shear tests in left abutment drift of proposed bridge for Udhampur Srinagar Baramulla Rail Link (USBRL), across Anji, a seasonal tributary of Chenab River, near Reasi. The shear strength parameters for rock over rock interface i.e. cohesion (c) and friction angle (ϕ) were interpreted for right and left abutments. As the rock mass was jointed and fractured with few shear zones, the shear strength parameters were evaluated only for residual values and peak values could not be differentiated, which was a unique phenomenon at this site.

Introduction

It is proposed to construct an arc bridge for Udhampur Srinagar Baramulla Rail Link (USBRL), across Anji, a seasonal tributary of Chenab River, near Reasi district, Jammu & Kashmir, India. The proposed Anji bridge is 186m high and 657m long. The location map of the project is shown in Fig. 1. Schematic view of proposed Anji Khad bridge is shown in Fig. 2. The view of already

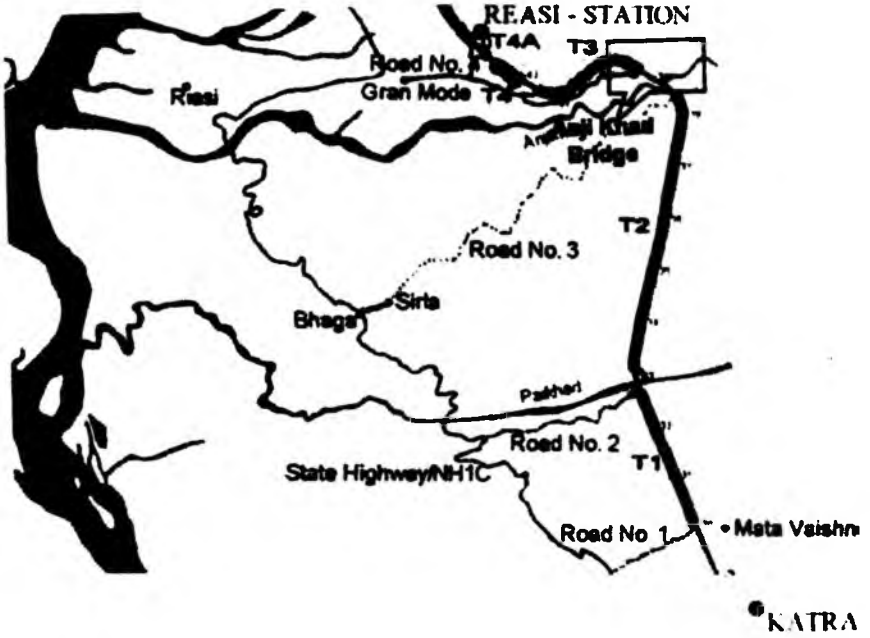

Fig. 1: Location map of Anji Khad Bridge, Reasi District, J&K

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Fig. 2 Schematic view of proposed Anji Khad bridge

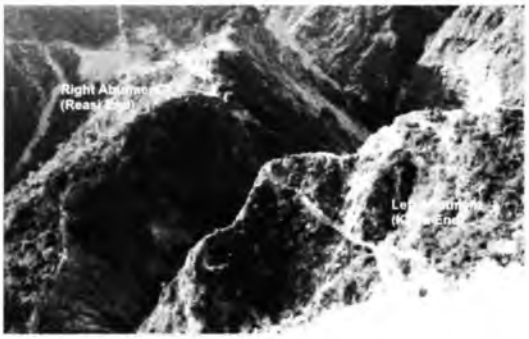

Fig. 3 View of foliation joint in rock mass along with tunnel portal

constructed tunnel and piers at right bank (Reasi end) along with foliation joints in rock mass of Anji Khad Bridge is shown in Fig. 3.

The Central Soil and materials Research Station (CSMRS) undertook the investigation work of determining the shear strength characteristics of rock mass required for the design of bridge foundation at Anji Khad (CSMRS 2009a and 2009b).

This paper deals with evaluation of shear strength parameters of jointed rock mass determined by conducting in-situ shear tests on rock to rock interfaces in both the right (Reasi end) and left abutment drifts (Katra end) of Anji Khad bridge.

Geology

Geology of the Area

The proposed railway line alignment between Katra and Qazigund generally passes through Siwaliks and Pre-Tertiary rocks overlain by unconsolidated sediments of Recent to Sub-Recent ages. The study area comes within the sub-Himalayan zone, with outcrops of unfossiliferous limestone, Sirban limestone of Hazara of presumably Permian or Permo-Carboniferous/Meso-Proterozic age as inliers.

Massive to blocky Dolomite is exposed along both the banks of Anji Khad below the proposed arch foundation up to a height of 50m with wide range of colours (white, light grey, dark grey and pale brown) and different degrees of weathering (fresh, slightly and moderately) and fracturing (moderately and intensely); rolled-down boulders and chips of dolomitic limestone and limestone with silt and clay material and siliceous limestone. The strata in the area are characterized by prominent one sub-horizontal foliation joint and two sub-vertical joints. The foliation joint strikes roughly N-S and dips 20^0 to 30^0 in East direction as shown in Fig. 3. The first joint strikes roughly NE-SW and dips 80^0 to 85^0 in NW direction and the second joint strikes SE-NW and dips 60^0 to 70^0 in SW direction. A few sub-vertical random joints are also present. The strata have major three sets of discontinuities, which are continuous and persistent. At most of the places, the foliation joint and the other two joints intersect each other forming cubical structure. In weathered and fractured dolomitic limestone and limestone, the average spacing of the foliation joint is 5 to 10 cm and of the other joints is 10 to 15 cm. These joints are smooth to rough or irregular, planar to undulating and unaltered with occasional infilling of calcareous or siliceous material along them or very minor joint surface staining.

The right bank ridge at the top is covered with thin cover of debris, i.e. dolomite chips and loose blocks. A thick shear zone can also be seen along the upstream ridge slope and form the part of ridge also. Most of the ridge lengths form the part of the via-duct, the many piers of which have already been constructed and founded on crushed dolomite. Both the arch foundations of the proposed bridge are made up of fresh, hard and competent jointed to blocky dolomite with thin shear seam as revealed by the exploratory drifts. There

appears no instability along the hill slopes around these arch foundations.

In-Situ Shear Test

In-situ shear tests were conducted on blocks of rock intact with the rock mass for rock/ rock interface. Wheel saw was used to separate the rock mass of block size (70 X 70 X 30 cm) from parent rock. A set of minimum five blocks were prepared. Each block was tested for a particular normal stress which was kept constant during the test. In the present investigation, in-situ shear tests were conducted at the normal loads of 20, 40, 60, 80 and 100 tons respectively at different RD's. All the tests were conducted as per the procedures described in ISRM (1979) and IS (1975).

Interpretation Of Test Data

The shear strength parameters obtained from different project sites in India and Bhutan have been discussed by Singh and Sharma (1989), Singh et al. (2000a), Singh et al. (2000b), Singh (2007), Singh (2009) and reports from CSMRS (2009a and 2009b) for rock to rock and concrete to rock interfaces based on in-situ testing data. The data showed a large variations among the shear strength parameters i.e. cohesion and friction angle. There were variations in peak and residual shear strength parameters. The variations were mainly due to change in rock mass properties from one project to another project and orientation of rock mass bedding planes at a particular site. There were variations at one project site with same rock type on left and right bank. However, the shear strength parameters were almost similar in magnitude on the left bank and the right bank of a few projects for both rock to rock and concrete to rock interfaces (Singh 2009).

Drift at Right Abutment

Shear stress versus shear displacement plots are shown in Fig. 4 for all five tests on the right abutment. In general, shear stress increases with the increase in normal stress. However, the different magnitudes of peak and residual shear stresses were obtained exceptionally due to change in rock profile at same magnitude of normal stress (Singh 2009). It is also seen from Fig. 4 that the variations in shear stresses were not proportion to the normal stresses. The maximum shear stress was for a normal load of 100 tons which was more than the shear stress for 80 tons of normal load. However, shear stress at normal load of 40 tons was higher than the shear stress at 60 tons. This has all happened due to variations in number of joints and strength properties of joints in rock mass.

The strata in the area are characterized by prominent one sub-horizontal foliation joint and two sub-vertical joints. The foliation joint strikes roughly N-S and dips 20^0 to 30^0 in East direction as shown in Fig. 3. Generally in a shear test, the peak value is attained first and shear displacement is obtained without increase in shear stress. However, shear stress gets reduced even after regular hydraulic pumping and thus the residual stress is obtained when there is almost no change in shear stress with the increase in shear displacement. It is clear from Fig. 4 that there was no peak stress at this location from all the tests.

Lots of shearing must have already taken place at this location along the rock slope and peak values must have been attained during this natural shearing of the rock mass. It was a unique experience at this project site. In general, peak stress is attained first in all the in-situ shear tests and peak shear stress is followed by residual shear stress in the plot of shear stress versus shear displacement.

In the present case study, the blocks were sheared in horizontal directions which were almost along the foliation joints. Due to highly jointed nature of rock mass and shearing almost along the foliation, only residual stresses were obtained from Fig. 4 at different normal stresses for respective blocks. From the "curve of best fit" for shear

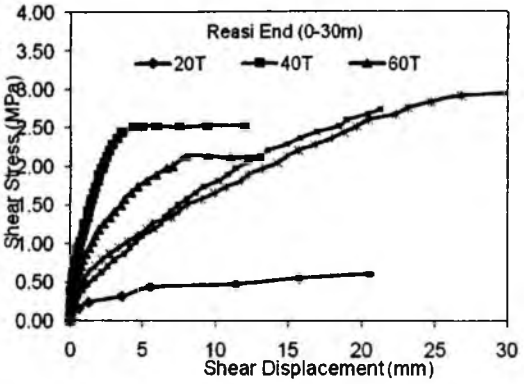

Fig. 4: Plot of Shear stress v/s Shear displacement of all five SHT

(Normal load=20T to 100T; RD= from 12m to 27 m from the portal)

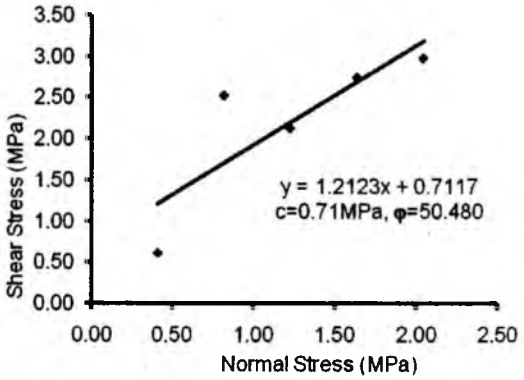

Fig. 5: Plot of shear stress versus normal stress for right abutment

stress versus normal stress plot as shown in Fig. 5, the value of cohesion 'c' is 0.71 MPa and 'ö' is 50.48° obtained for residual shear strength on rock to rock interface.

Drift at Left Abutment

The variations of shear stress v/s shear displacement curves at different normal loads for all five blocks on the left abutment (Katra end) are shown in Fig. 6. It is clear from left abutment data also that no peak shear stress was attained during testing. The shear stress versus normal stress plot is shown in Fig. 7. The results interpreted from five blocks between RDs of 0.0 m to 30 m showed that the value of cohesion (c) and friction angle (j) were 0.39 MPa and 49°, respectively, for residual shear strength parameters. There was slight decrease in the shear strength parameters on left bank drift as compared to right bank drift. This was also visible from rock mass conditions at both the bank. However, this drift was extended further to reach the actual bridge abutment site and the in-situ tests were performed at actual location also.

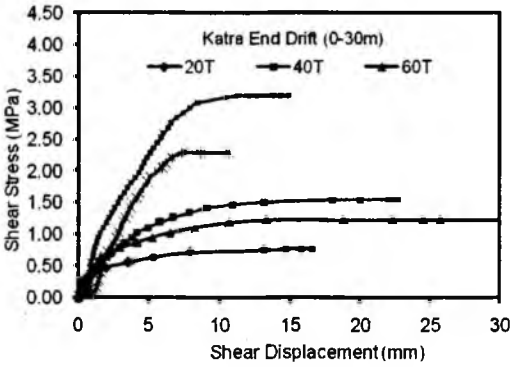

Fig. 6: Plot of shear stress versus shear displacement of all five SHT

(Normal load=20T to 100T; RD= from 0m to 30m from the portal)

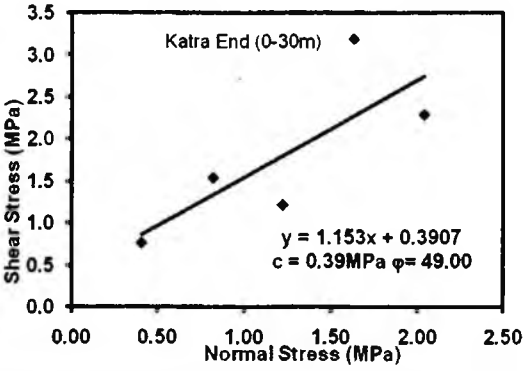

Fig. 7: Plot of shear stress versus normal stress for right abutment

The variations of shear stress v/s shear displacement curves at different normal loads for all six blocks on the left abutment (Katra end) in the extended drift from 30m to 68m have been shown in Fig. 8. It is clear from left abutment data also that no peak shear stress was attained during testing in all the six blocks. So, this unique phenomenon was

noticed in all the 16 blocks on both the banks of Anji Khad. The shear stress versus normal stress plot is shown in Fig. 9. The results interpreted from six blocks between RDs of 40 m to 67 m showed that the value of cohesion (c) and friction angle (j) were 0.38 MPa and 51.12°, respectively, for residual shear strength parameters. There was slight improvement in the shear strength parameters in the left bank extended drift as compared to right bank drift.

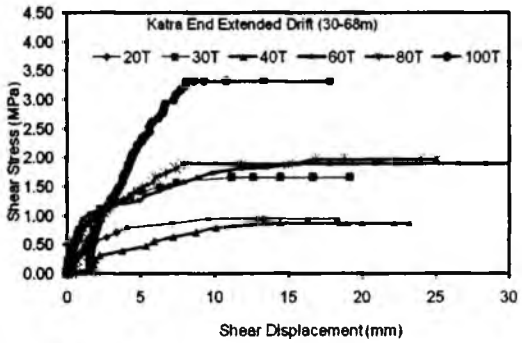

Fig. 8: Plot of shear stress versus shear displacement of all six SHT

(Normal load=20T to 100T; RD= from 30m to 68m from the portal)

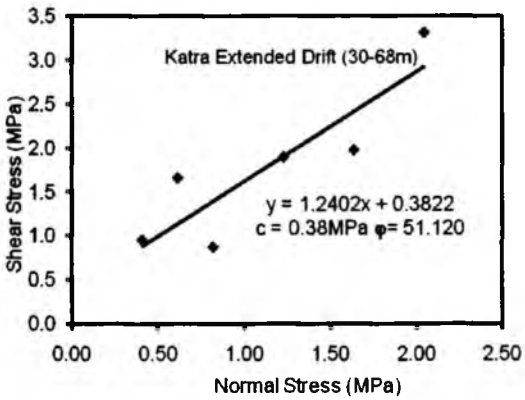

Fig. 9: Plot of shear stress versus normal stress for 3 sets of left and right abutments

The results were further interpreted as shown in Fig. 10 from joint plots of eleven blocks between RDs of 40 m to 67 m at left abutment and five blocks from right abutment. The joint results showed that the values of cohesion (c) and friction angle (j) were 0.50 MPa and 51.39° respectively for residual shear strength parameters. The results based on 11 shear blocks from both abutments are more reasonable and can definitely be used as one common value at this site.

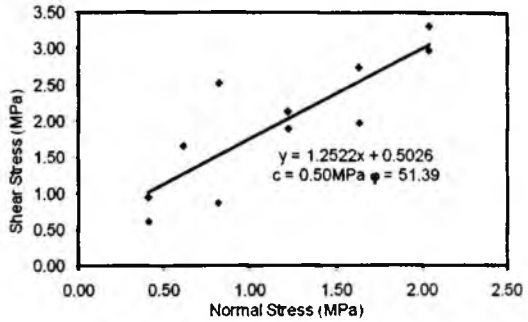

Fig. 10: Joint plot of shear stress versus normal stress for left and right abutments

As the rock mass was jointed and fractured with few shear zones, the shear strength parameters were evaluated only for residual value and peak values could not be differentiated, which was clear from the shear stress versus shear displacement curve. Lot of shearing must have already taken place at this location along the rock slope and peak values must have been attained during this natural shearing of the rock mass. It was a unique experience at this project. In general, peak stress is attained first in all the in-situ shear tests and peak shear stress is followed by residual shear stress in the plot of shear stress versus shear displacement.

It is, therefore, suggested that the in-situ testing must be conducted at the exact location of the structure under construction to determine the shear strength parameters for specific purpose. One must be conscious enough to evaluate peak and residual strength parameters in highly jointed rock mass in Himalayan region near rock slopes.

Conclusions

The following conclusions are deduced from the experimental results:

- From the "curve of best fit" for shear stress versus normal stress plot, the values of cohesion 'c' and friction 'ö' were 0.38

MPa and 51.12° for left abutment and 0.71 MPa and 50.48° for right abutment, respectively, for residual shear strength on rock to rock interface.

- The joint results from plots of 11 blocks from both right and left abutments predicted the values of cohesion (c) and friction angle (j) were 0.50 MPa and 51.39° respectively for residual shear strength parameters on rock to rock interface. The results based on 11 shear blocks from both abutments are more reasonable and can definitely be used as one common value at this site.
- The disturbance in the shear test block was observed by overturning the sheared block after completion of the test to measure the actual sheared area of the test block. It is, therefore, recommended to observe the behaviour of the block by overturning it after shearing and to measure the correct area of shear block.
- For getting a fair idea of the shear strength parameters of the rock mass at the bridge site, the results were compared with the results of in-situ shear tests conducted in the right and left bank drifts of the project.
- The shear strength parameters were evaluated only for residual value and peak values could not be differentiated as the rock mass was jointed and fractured with few shear zones. Lot of shearing must have already taken place at this location along the rock slope and peak values must have been attained during this natural shearing of the rock mass. Another reason for getting only residual shear stress may be due to shearing of the blocks almost along the foliation joints. It was a unique experience at this project. In general, peak stress is attained first in all the in-situ shear tests and is followed by residual shear stress in the plot of shear stress versus shear displacement.
- It is, therefore, recommended that the in-situ testing must be conducted at the exact location of the structure under construction to determine the shear strength parameters for specific purpose. One must be conscious enough to evaluate peak and residual strength parameters in highly jointed rock mass in Himalayan region near rock slopes. The blocks must be prepared with due care particularly in jointed rock mass to minimize the disturbance or displacement in joints.

Acknowledgements

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