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Evaluation of Shear Strength Parameters in Jointed Rock Mass

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ABSTRACT

This paper deals with a unique experience in interpretation of shear strength parameters of jointed rock mass determined by the results of sixteen in situ shear tests conducted in left and right bank drifts of proposed bridge abutments. The main rock type found in the area was blocky dolomitic limestone. The shear strength parameters were determined by conducting in situ shear tests for rock to rock interface. The shear strength parameters i.e., values of cohesion (c) and friction angle (ϕ) were interpreted from eleven blocks at left abutment and five blocks at right abutment. The shear strength parameters of rock mass were further evaluated by using data of all sixteen blocks from both abutments. As the rock mass was jointed and fractured with few shear zones, the shear strength parameters were evaluated only for residual values because the peak values could not be differentiated, which was clear from the shear stress versus shear displacement curves of all sixteen blocks. Because of variations in the shear strength parameters, it was recommended to utilise data of eleven blocks for analysis from both the abutments at the exact locations of bridge.

Keywords: In situ shear test; Shear strength parameters; Jointed rock mass; Dolomitic limestone

1. INTRODUCTION

It was proposed to construct an arch bridge for Udhampur-Srinagar-Baramulla Rail Link (USBRL) project, across Anji, a seasonal tributary of Chenab River, near Reasi district, Jammu & Kashmir, India. The proposed Anji Bridge is 186m high from river bed level 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 Central Soil and Materials Research Station (CSMRS) undertook the investigation work of determining the deformability and shear strength characteristics of rock mass required for the design of railway bridge foundation at Anji Khad (CSMRS 2009a, 2009b and 2009c).

In general when in situ shear tests are conducted under constant normal load, the peak shear strength is attained first and then residual shear strength is determined. In shear stress versus shear displacement plot, the shear stress reduces suddenly just after achieving peak stress and then comes residual stress after applying shear stress continuously and getting shear displacement without getting much ups and down in shear stress. In the present investigation, as the rock mass was jointed and fractured with few shear zones, the shear strength parameters were evaluated only for residual value as peak values could not be differentiated

(Fig. 3). This phenomenon was noticed in all the eleven blocks on the left abutment and five blocks on the right abutment. These tests were conducted inside the drifts on the rock slopes of left and right abutments of the Anji Khad. It appeared from the test results that peak strength must have already been attained during natural shearing on the rock slopes as the shear stress was applied perpendicular to the rock slope.

This paper deals with interpretation of shear strength parameters of jointed rock mass using the results of sixteen in situ block shear tests conducted in the left and right bank drifts of railway bridge for USBRL project. An important observation of tests will be discussed along with in situ shear test results of eleven blocks on the left abutment and a comparison with in situ shear test results of five blocks on the right abutment.

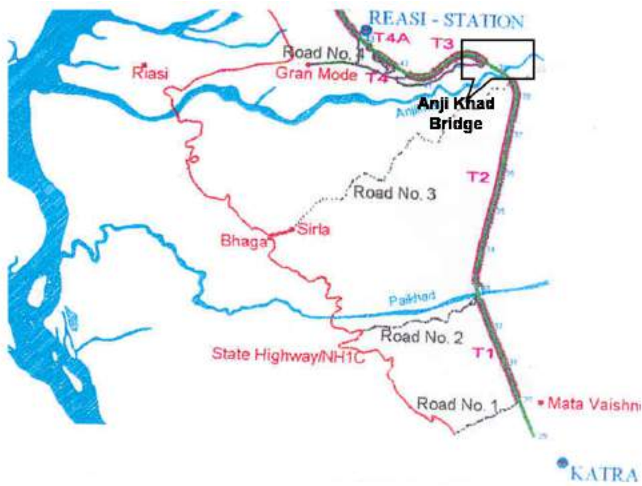

Fig. 1 - Location map of Anji Khad bridge, Reasi District, Jammu & Kashmir, India

Fig. 2 - Schematic view of proposed Anji Khad bridge

2. GEOLOGY OF THE AREA

The proposed railway line alignment between Katra and Qazigund generally passed through Shiwaliks and Pre-Tertiary rocks overlain by unconsolidated sediments of Recent to Sub-

Recent ages. The study area was within the Sub-Himalayan zone, with outcrops of unfossiliferous limestone, Sirban limestone of Hazara formation of presumably Permian or Permo-Carboniferous/Meso-Proterozoic age as inliers.

Massive to blocky dolomite is exposed along both the banks of Anji river 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 were characterized by prominent one sub-horizontal foliation/bedding joint and two sub-vertical joints. The foliation/bedding joint strikes roughly N-S and dips 200 to 300 in East direction. The first joint strikes roughly NE-SW and dips 800 to 850 in NW direction and the second joint strikes SE-NW and dips 600 to 700 in SW direction. A few sub-vertical random joints were also present. The strata had major three sets of discontinuities, which were continuous and persistent. At most of the places, the foliation joint and the other two joints intersected each other forming cubical structure. In weathered and fractured dolomitic limestone and limestone, the average spacing of the foliation joint was 5 to 10cm and of the other joints was 10 to 15cm. These joints were 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.

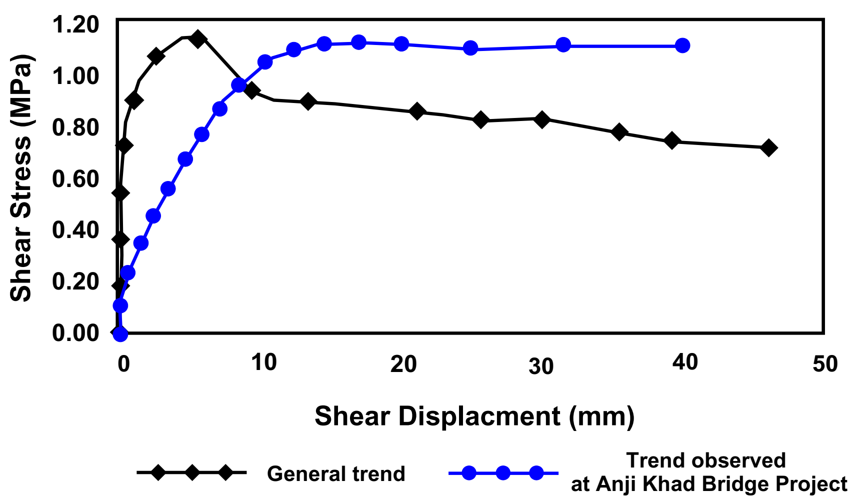

Fig. 3 - Shear stress versus shear displacement with trends in general and at present project

2.1 Geology of Left Bank Drift

The left bank drift (Katra end) was of 30m length. The main rock type found in the drift was blocky dolomitic limestone, which was criss-crossed by sub-vertical joints and sub-horizontal foliation joints. Sub-vertical random joints were also present at few places, which resulted in forming blocky / cubical structure. In the drift at different locations, few shear zones of different thickness were observed. Yellow coloured bedding shear zone of thickness 3 to 6cm was observed between RD 12 and 13m. Chert band of thickness 2 to 4cm was seen on the right wall of the drift between RD 19 to 22m and between RD 23 to 25m. Bedding shear zone of thickness 3-10cm was passing through the RD 21 and 22m. Similarly transverse shear zone of thickness 4-20cm was passing through the RD 25 and 26m.

This drift was further extended up to 68m length so as to reach the central line of bridge. The main rock type found in the extended drift was also blocky dolomitic limestone, which was criss-crossed by sub-vertical joints and sub-horizontal foliation joints. A cherty dolomite band of thickness 15-20cm was seen on the left wall of the drift between RD 31 to 34m and another band of thickness 5-20cm was passing through RD 36m. Bedding shear zone of thickness 28-30cm was passing through the RD 41m. Similarly another bedding shear zone of thickness 30-40cm was passing through the RD 60m and shear zone of thickness of 30-55cm was passing between RD 63 and 64m. Thin bedding shear seam of thickness 3-10cm was passing between RD 35.5 and 40.5m and seam of thickness 5-8cm was passing through the RD 49m.

2.2 Geology of Right Bank Drift

The right bank drift was of 32m length. The central line of the bridge was approximately around 17m from the portal of the drift. The main rock type found in the drift was blocky dolomitic limestone. The rock type was criss-crossed by sub-vertical joints and sub-horizontal foliation joints. At few places sub-vertical random joints were also present, which resulted in forming blocky/cubical structure. In the drift at different locations, few shear zones of different thicknesses were observed. Bedding shear zone of thickness 30-40cm were passing through the RD 29m from the portal of the drift. Similarly shear zone of thickness 2-10cm was passing between the RD 10 and 11m. Three transverse shear zones were also observed passing through around RD 3m, 7m and 22m.

3. IN SITU SHEAR TEST

In situ shear tests were conducted for the determination of shear strength parameters of rock to rock. Tests were conducted at a location of similar rock mass. For each set of tests, generally 5 to 6 blocks are sheared by varying the normal stress on each block. The peak and residual values of shear strength with reference to each block were noted from a shear stress versus shear displacement curves. The normal stresses from all blocks were plotted corresponding to peak and residual shear stresses. The shear strength parameters were evaluated from lines of best fit by linear regression analysis for peak and residual stresses. In this paper rock to rock interface shear test results are presented.

3.1 Test Procedure

The general procedure consisted of bringing the normal load of the specified intensity over test block by loading the system normally and then applying the shear load in increments until the failure occurred. Displacements of block were observed after each increment of load in the directions of normal, lateral and shear. Minimum of five blocks were prepared for this purpose for one rock type and each block was tested for different but constant normal stress.

For conducting this test, the rock surface was prepared by careful manual chiselling. The rock blocks were prepared by drilling 0.35m deep overlapping holes in such a way that the area of rock block was 0.70m x 0.70m. To avoid any disturbance in the test block during shearing, the block was surrounded by filling concrete around the block with the help of steel frame along with an angle of 15° on the upstream side of the block. The same steel frame was used during test to avoid any disturbance of the block.

Before conducting tests, a channel approximately 20mm deep and 80mm wide was cut around the base of the block to allow freedom of shear and lateral displacements. The vertical reaction was taken against a reaction pad cast at the crown. The reaction for shear load was taken against a concrete pad cast at the upstream face of the drift. The direction of the shear load was kept the same as anticipated shearing in the structures, in the case of the bridge abutment, the shear load was applied towards inside the hill slope.

The normal load was applied using 100 tonne (T) jack and the shear load was applied using another 200 T hydraulic jack. The hydraulic jack for shearing was set up at an inclination of 15° to the horizontal on the vertical side of the block so that the shearing load passes through the centre of the base of the block without causing any overturning movement (Fig. 4). The displacements were measured in the directions of normal, shear and lateral to the sheared plane. The displacements can be measured either with dial gauges or LVDT (Linear Voltage Differential Transducers) with an accuracy of 0.01mm and a travel of at least 70mm.

Fig. 4 - In situ shear test assembly inside drift

In the consolidation stage of testing, pore water pressure in the rock and filling material adjacent to the shear plane was allowed to be dissipated under full normal stress before shearing. The consolidation stage was considered to be completed when the rate of change of normal displacement recorded at each of the four normal gauges was less than 0.05mm in 10 minutes. The normal stress was kept constant for each block and shearing stress was applied in small increments and corresponding horizontal shear displacement of the block was noted to an accuracy of 0.01mm with the help of dial gauges along with displacements in normal as well lateral directions.
The rate of shear displacement should be less than 0.1mm/minute in 10 minute before taking a set of reading. This rate may be increased to 0.5mm/minute between sets of reading provided that the peak strength was adequately recorded (ISRM, 1981). After reaching peak strength, reading should be taken at increment of shear displacement from 0.5mm to 5mm. The residual strength value was achieved when the block was sheared at a constant normal stress and at least 4 consecutive sets of reading were obtained which showed not more than 5 % variation in shear stress over a shear displacement of 1cm. The observations were continued continued even after the failure to the extent possible to get the information regarding residual frictional resistance.

After first value of residual strength was established, it is possible to obtain more values of residual strength from the same block. However, only one peak stress was obtained from one block. The normal stress on the block was either reduced or increased and another new value of residual stress was obtained against new normal stress by repeating the test on new block. The block should be consolidated under each normal stress before shearing. After the completion of test on each block, the shear block was overturned to measure the correct contact shear area. The contact area was measured by taking a section at every 10cm in the directions of shear and perpendicular to the shear to measure accurately the average length and width of the sheared block.

3.2 Calculations

Shear and normal stresses are computed as follows:

$$Shear\ \ Stress\ (MPa) = \frac{P_s}{A} = \frac{P_{sa}\,\mathrm{Cos}\,\alpha}{A} \tag{1}$$

$$Normal\ \ Stress\ (MPa) = \frac{P_n}{A} = \frac{P_{na} + P_{sa}\,\mathrm{Sin}\,\alpha}{A} \tag{2}$$

where

- P_s = total shear force in N,
- P_n = total normal force in N,
- A = area of shear surface corrected to account for shear and lateral displacements in mm^2,
- α = inclination of shear force to the shear plane at 15°,
- P_{sa} = applied shear force in N, and
- P_{na} = applied normal force in N.

The applied normal stress was reduced after each increase in shear stress by an amount P_{sa} sin α in order to maintain the normal stress approximately constant during in situ shear test. The plots of shear stress versus horizontal displacement and normal stress versus shear stress observed at all locations are plotted. The failure envelopes corresponding to the peak and the residual strength are also drawn. The displacements recorded during the test are averaged to obtain mean values of shear, lateral and normal displacements. The shear and lateral displacements are taken into account while computing the corrected contact area. The shear area of the block was measured after the test by overturning the sheared block.

4. RESULTS AND DISCUSSIONS

In the present investigation, in situ shear tests were conducted on blocks of rock intact with the rock mass for rock to rock interface. Test locations are shown in Table 1.

Wheel saw was used to separate the rock mass of block size (0.70m x 0.70m x 0.35m) from parent rock. Each block was tested for a particular normal load which was kept constant during the test (viz; 20, 40, 60, 80 and 100 T respectively at different RD's). All the tests were conducted as per the procedures described in ISRM (1981) and IS: 7746 (1983).

Eleven and five in situ shear tests were conducted in the left and right abutment drifts, respectively. All tests were conducted on rock to rock interface. In the present investigation the shear strength parameters were evaluated only for residual value as peak values could not be differentiated.

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 in reports of 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 banks. 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).

4.1 Shear Strength Parameters of Rock Mass at Left Bank Drift

In this drift of 30m length, five shear tests were conducted to determine the shear strength parameters of rock mass. It is seen from Fig. 5 that the variations in shear stresses are not proportional to the normal stresses. The maximum shear stress was for a normal load of 80T which was more than the shear stress for 100T of normal load. The shear stress at normal load of 40T was higher than the shear stress at 60T. This has all happened due to variations in number of joints and strength properties of joints in rock mass.

Shear stress versus shear displacement plots are shown in Fig. 5 and the shear stress versus normal stress plot is shown in Fig. 6. The results interpreted from five blocks between RDs of 0.0 to 30m showed that the value of cohesion (c) and friction angle (ϕ) were 0.39MPa and 49°, respectively, for residual shear strength parameters. There was slight deterioration in the shear strength parameters on left bank drift as compared to right bank drift. It was, therefore, decided to extend the drift up to the length of exact bridge abutment location. The drift was extended up to 68m and a set of six more shear blocks were tested in the extended portion of the drift.

Table 1 - Locations of in situ shear tests conducted in the left and right bank abutments

Test No.	Drift Location*	RD (m)	Rock Type
SHT-01	Left Abutment	15.0	Jointed and fractured dolomite
SHT-02		22.5	Jointed and fractured dolomite; Bedding shear zone of thickness 3-10cm is observed on the right and left walls of the drift.
SHT-03		25.0	Jointed and fractured dolomite; Transverse shear zone of thickness 4-20cm is passing through RD 25.0m. Chert bands are seen on the right walls of the drift.
SHT-04		27.0	Jointed and fractured dolomite
SHT-05		29.0	Jointed and fractured dolomite
SHT-06		48.5	Jointed and fractured dolomite
SHT-07		52.5	Jointed and fractured dolomite
SHT-08		55.5	Jointed and fractured dolomite
SHT-09		59.0	Jointed and fractured dolomite; A bedding shear zone of thickness 30-40cm is observed passing through RD 60.0m
SHT-10		61.0	Jointed and fractured dolomite; A bedding shear zone of thickness 30-40cm is observed passing through RD 60.0m
SHT-11		66.0	Jointed and fractured dolomite
SHT-12	Right Abutment	12.0	Jointed and fractured dolomite; Joints are generally rough and tight.
SHT-13		17.0	Jointed and fractured dolomite; Joints are generally rough and tight. Shear zone is observed on the left wall of the drift.
SHT-14		22.0	Jointed and fractured dolomite; Joints are generally rough and tight. Transverse shear zone passes through the test location.
SHT-15		25.0	Jointed and fractured dolomite; Joints are generally rough and tight.
SHT-16		27.0	Jointed and fractured dolomite. Joints are generally rough and tight.

** Central line of the bridge foundation is at RD 53.0m in left bank and RD 17.0m in right bank abutment*

The variations of shear stress versus shear displacement curves at different normal loads for all six blocks on the left abutment (Katra end; Extended Drift, RD 30-68m) are shown in Fig. 7. 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. 8. The results interpreted from six blocks between RDs of 30 to 68m showed that the value of cohesion (c) and friction angle (ϕ) were 0.38MPa and 51.12° respectively for residual shear strength parameters. There was slight improvement in the shear strength parameters after extending the drift. Similarly, it was noted that there was improvement in modulus of deformation and modulus of elasticity with improvement in rock mass in the extended drift with applied stress of 5.66MPa in plate loading test (CSMRS, 2009a and 2009b).

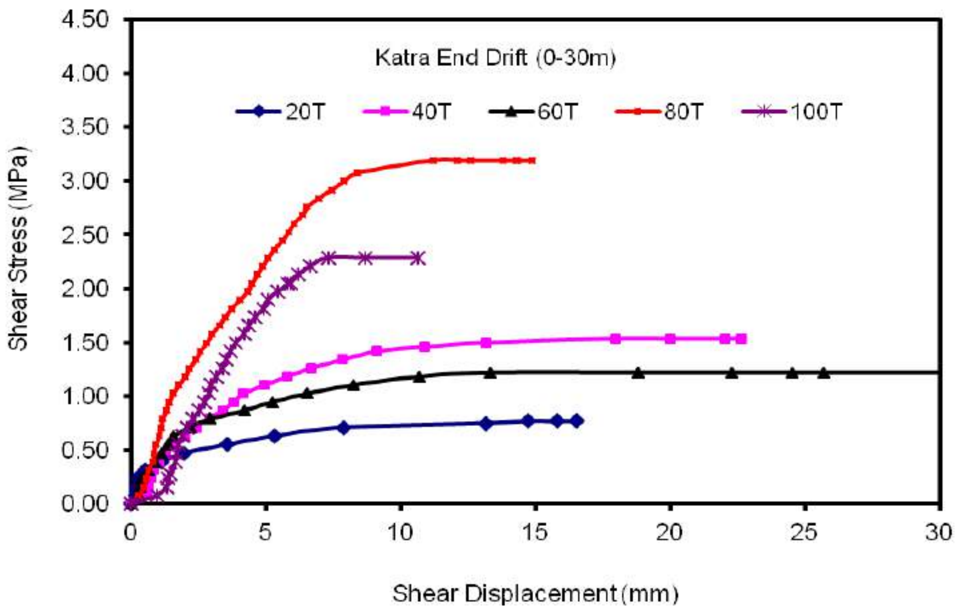

Fig. 5 - Shear stress versus shear displacement of five shear tests conducted at left bank abutment
(Normal load=20T to 100T; RD= from 0m to 30m from the portal)

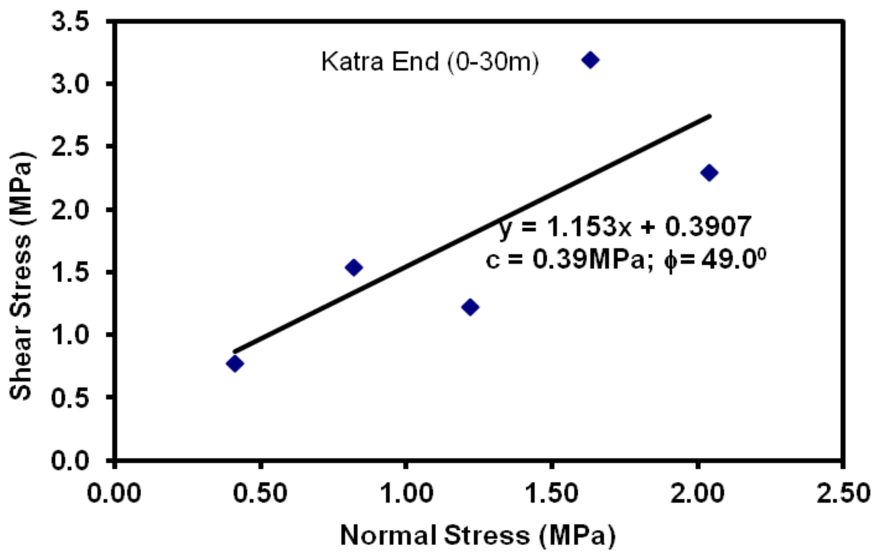

Fig. 6 - Shear stress versus normal stress for left bank abutment

As the rock mass was jointed and fractured with few shear zones, the shear strength parameters were evaluated only for residual value as 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 in situ 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.

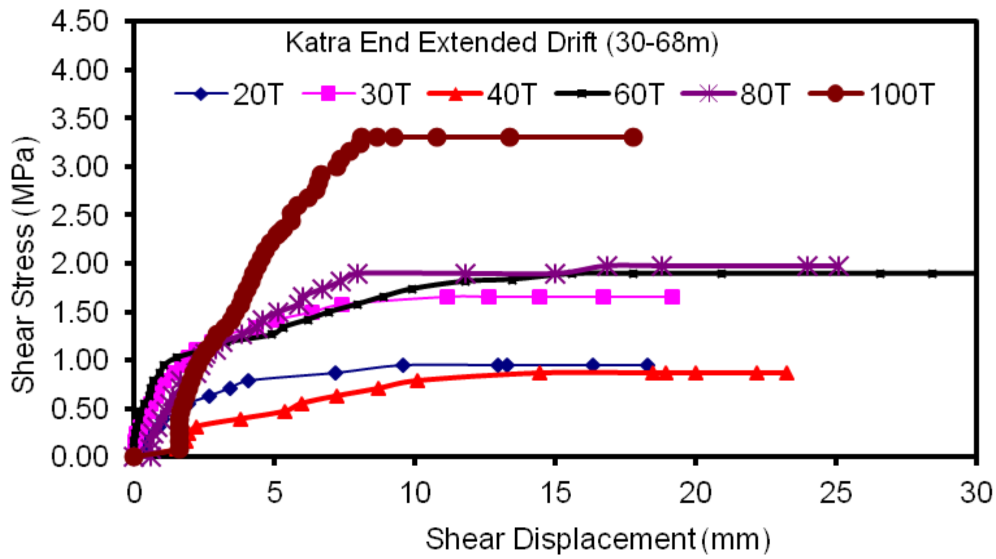

Fig. 7 - Shear stress versus shear displacement of six shear tests conducted at left bank abutment
(Normal load 20T to 100T; RD= from 30m to 68m from the portal)

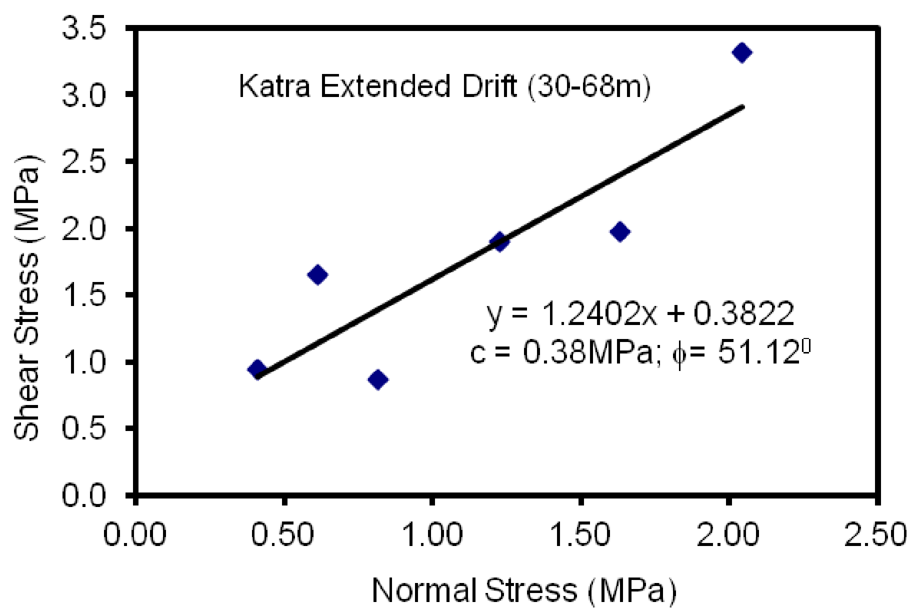

Fig. 8 - Shear stress versus normal stress for six shear tests on left abutment in extended drift

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.

4.2 Shear Strength Parameters of Rock Mass at Right Bank Drift

In right bank drift of 32m length, five shear tests (SHT) were conducted to determine the shear strength parameters of rock mass. Shear stress versus shear displacement plots are shown in Fig. 9 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. 9 that the variations in shear stresses are not proportional to the normal stresses. The maximum shear stress is for a normal load of 100T which is more than the shear stress for 80T of normal load. However, shear stress at normal load of 40T is higher than the shear stress at 60T of normal load. This has all happened due to variations in number of joints and strength properties of joints in rock mass.

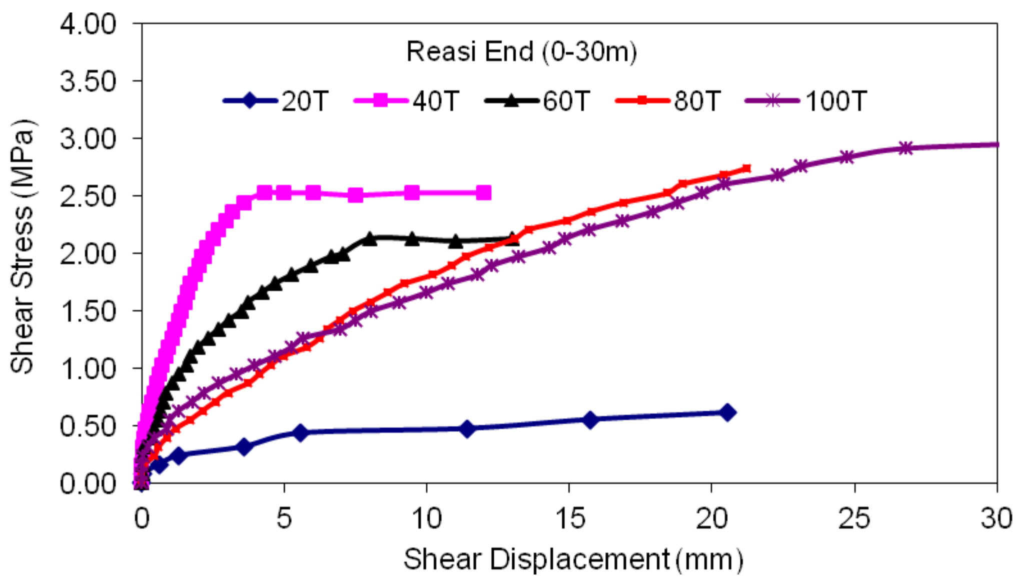

Fig. 9 - Shear stress vs shear displacement of five shear tests conducted at right bank abutment (Normal load=20T to 100T; RD= from 12m to 27m from the portal)

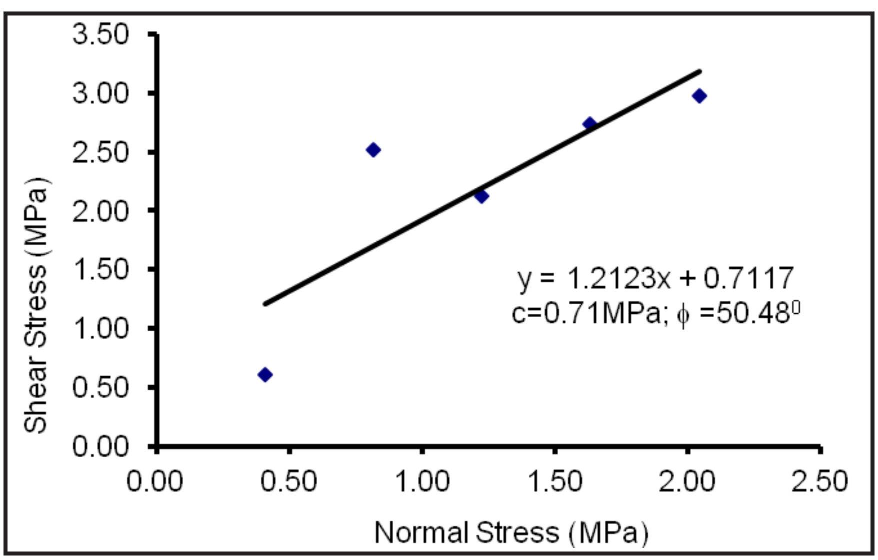

Fig. 10 - Shear stress versus normal stress for right bank abutment

Strata of 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° to 30° in East direction. Generally in a shear test, the peak value is attained first followed by residual stress as shown in Fig. 3, when 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. 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 in situ shearing of the rock mass. It was a unique experience at this project site. It is clear from Fig. 9 that there is no peak stress at this location from all the tests.

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. 9 at different normal stresses for respective blocks. From the "curve of best fit" for shear stress versus normal stress plot as shown in Fig. 10, the value of cohesion 'c' is 0.71MPa and 'ϕ' is 50.48°, as obtained for residual shear strength on rock to rock interface.

4.3 Interpretation of Average Shear Strength Parameters

The shear stress versus normal stress plot is shown in Fig. 11 for three sets of shear tests conducted inside drifts on left and right abutments. The trend lines for 3 sets are almost parallel to each other with slight variations in shear strength parameters of rock to rock interfaces.

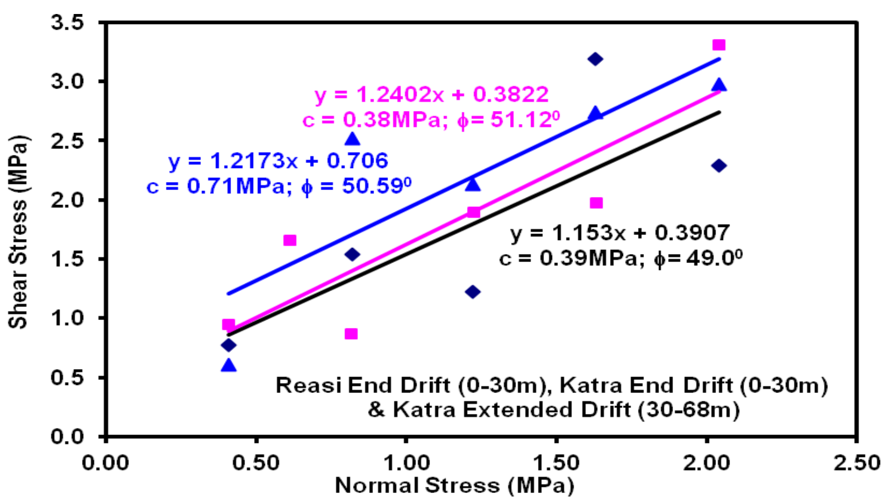

Fig. 11 - Shear stress versus normal stress for 3 sets from left and right abutments

4.4 Shear Strength Parameters from all Shear Block Tests

Test locations with normal, residual shear stresses with displacement are given in Table 2 for all sixteen shear blocks. The shear stress versus normal stress plot is shown in Fig. 12 for all sixteen shear blocks tested in both the drifts. The results interpreted from all sixteen blocks showed that the value of cohesion (c) and friction angle (ϕ) are 0.48 MPa and 50.50°, respectively, for residual shear strength parameters. There is slight deterioration in the shear strength parameters as compared to right bank drift. However, it gives a good interpretation of data from both the abutments.

Table 2 - Test locations with normal and residual shear stresses with displacement

Test No.	RD from Portal (m)	Normal		Residual		Shear Displacement (mm)
		Load (T)	Stress (MPa)	Load (T)	Stress (MPa)	
SHT-01	15.0	60	1.22	62	1.22	30.84
SHT-02	22.5	20	0.41	39	0.77	16.53
SHT-03	25.0	40	0.82	78	1.54	22.64
SHT-04	27.0	100	2.04	116	2.29	10.65
SHT-05	29.0	80	1.63	162	3.19	14.87
SHT-06	48.5	60	1.22	96	1.89	34.94
SHT-07	52.5	100	2.04	168	3.31	17.78
SHT-08	55.5	80	1.63	100	1.97	25.08
SHT-09	59.0	30	0.61	84	1.65	19.16
SHT-10	61.0	20	0.41	48	0.94	18.25
SHT-11	66.0	40	0.82	44	0.86	23.22
SHT-12	12.0	40	0.82	128	2.52	12.00
SHT-13	17.0	60	1.22	108	2.13	13.00
SHT-14	22.0	80	1.63	139	2.74	21.33
SHT-15	25.0	100	2.04	151	2.98	33.04
SHT-16	27.0	20	0.41	31	0.61	20.52

Notation: SHT 01 – 11 were conducted in left and SHT 12 – 16 conducted in right abutment drift

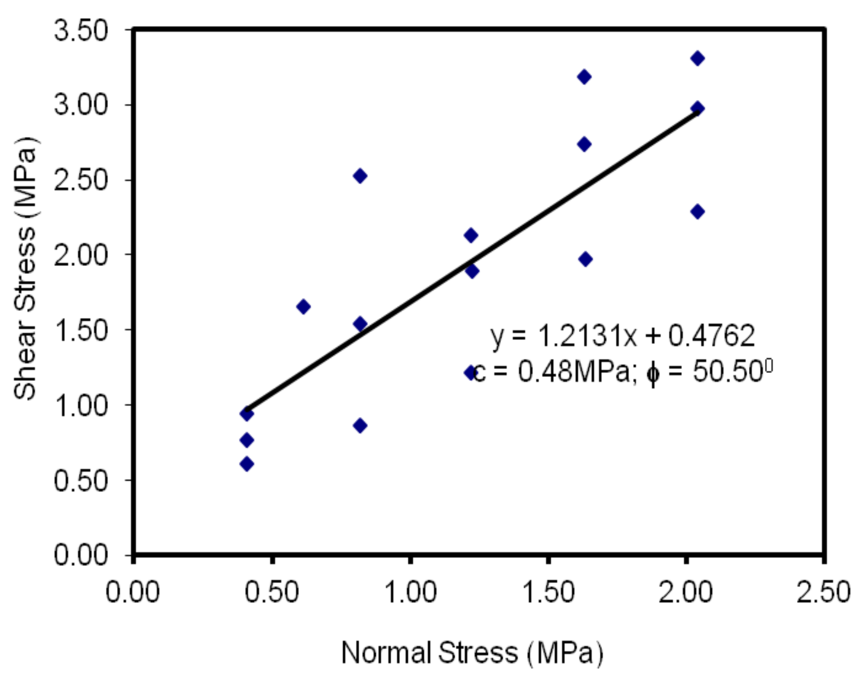

Fig. 12 - Shear stress versus normal stress for all 16 blocks on left and right abutments

4.5 Shear Strength Parameters from Shear Block Tests Aligned to Bridge Abutment

The shear stress versus normal stress plot is shown in Fig. 13 for all eleven shear blocks tested in both the drifts at the exact location of bridge abutments at both banks of the river. The results interpreted from all eleven blocks show that the value of cohesion (c) and friction angle (ϕ) were 0.50MPa and 51.39°, respectively, for residual shear strength parameters. There was slight improvement in the shear strength parameters as compared to right bank drift. However, these common values of shear strength parameters can be utilised effectively for both the abutments.

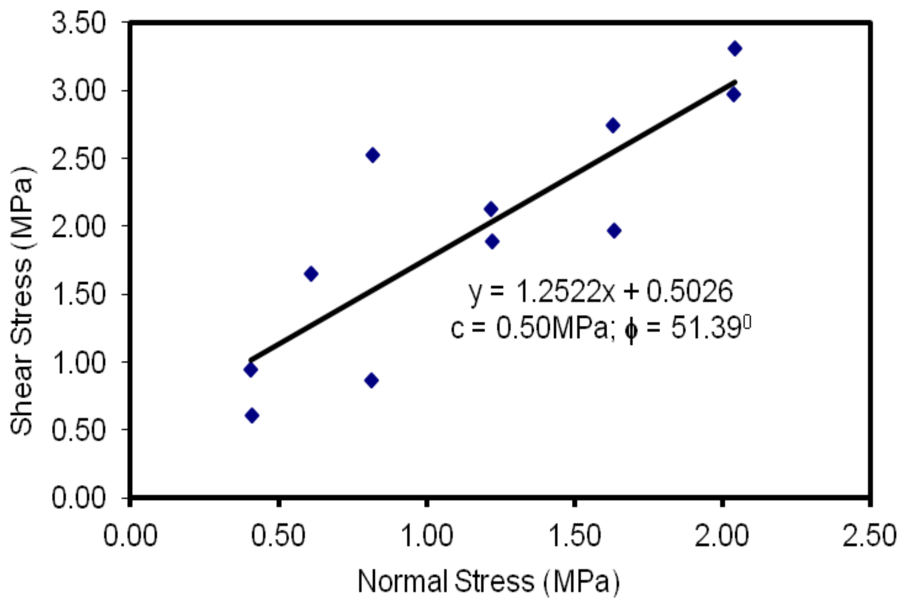

Fig. 13 - Shear stress versus normal stress for 11 shear tests on left and right bridge abutments

All the values of shear strength parameters, cohesion (c) and friction angle (ϕ) in the drifts of left and right abutments are interpreted from the data based on in situ shear tests and are given in Table 3. The shear strength parameters based on using the data of eleven blocks from both the abutment seems to be most appropriate common value for both abutments. It is a new method of interpretation of data to arrive at a common value for both abutments. This method of interpretation may be appreciated in case of large differences in the values from both the abutments.

Table 3 - Shear strength parameters, cohesion (c) and friction angle (ϕ), in left and right abutments

Drift	RD (m)	No. of Tests	Cohesion, c (MPa)	Friction Angle, ϕ (°)	Shear Strength (MPa)
Left bank	15 – 30	5	0.39	48.97	6.89
Left bank - Extended drift at exact location of bridge foundation	30 – 68	6	0.38	51.12	7.40
Right bank	6.5 – 27	5	0.71	50.48	7.57
Results of all the 16 tests conducted at left and right bank	-	16	0.48	50.50	7.35
Results of 11 tests at exact location of bridge foundation in left and right bank	-	11	0.50	51.39	7.59

5. CONCLUSIONS

The following conclusions are deduced based on the basis of present experimental results:

- For getting a fair idea of the shear strength parameters of the rock mass at the bridge site, the results of in situ shear tests carried out in right and left abutment drifts have been compared.
- The shear strength parameters are available only for residual value as peak values could not be differentiated from residual values because the rock mass was highly 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 in situ 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 strength is attained first in all the in situ shear tests and is followed by residual shear strength in the plot of shear stress versus shear displacement.
- From the "curve of best fit" for shear stress versus normal stress plot, the values of cohesion 'c' and friction 'ϕ' are 0.38MPa and 51.12° for left abutment and 0.71MPa and 50.48° for right abutment, respectively, for residual shear strength on rock to rock interface.
- The common values of shear strength parameters cohesion (c) and friction angle (ϕ) based on 11 shear blocks for both the abutments are 0.50MPa and 55.39°, respectively.
- 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 shear test blocks must be prepared with due care particularly in jointed rock mass to minimize the disturbance or displacement in joints.

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References

CSMRS Report (2009a). Report on deformability characteristics and shear strength parameters of rock Mass in right bank drift (Reasi End), Anji Khad Bridge project, Reasi district, Jammu & Kashmir.

CSMRS Report (2009b). Report on deformability characteristics and shear strength parameters of rock mass in left Bank drift (Katra End), Anji Khad Bridge Project, Reasi district, Jammu & Kashmir.

CSMRS Report (2009c). Report on Deformability Characteristics and Shear Strength Parameters of Rock Mass in Left Bank Extended Drift (Katra End), Anji Khad Bridge Project, Reasi District, Jammu & Kashmir.

IS: 7746, (1983). Indian Standard code of practice for In Situ Shear Test on Rock, Bureau of Indian Standards (BIS), New Delhi.

ISRM (1981). Suggested Methods For Determining Shear Strength, International Society for Rock Mechanics (ISRM) Commission On Standardization of Laboratory and Field Tests, In: Rock Characterization Testing and Monitoring, ed. E.T. Brown., 131-140.

Singh, Rajbal, Dev Hari and Dhawan, A. K. (2000a). Characterisation of Foundation Rock for a Concrete Gravity Dam, Proc. of Indian Geotechnical Conference igc-2000, Mumbai, 67-68.

Singh, Rajbal, Dixit, Mahabir and Dhawan, A. K. (2000b). Characterisation of rock mass at Kalpong H.E. project, north Andaman, Proc. of Indian Geotechnical Conference IGC-2000, Mumbai, 69-70.

Singh, Rajbal and Sharma, V. M. (1989). Determination of foundation deformability and shear strength characteristics of a concrete dam, Indian Geotechnical Conference (IGC-90), Bombay, 371-373.

Singh, Rajbal (2007). Field Shear Test. In: Engineering in Rocks for Slopes, Foundations and Tunnels, ed. T. Ramamurthy, 256-264.

Singh, Rajbal (2009). Measurement of in situ shear strength of rock mass, J. Rock Mech. Tunlg. Tech., ISRMTT, 15(2), 131-142.