Effect of Rock Matrix with Circular Opening on Strength Characteristics Using Triaxial System with Partial Unconfined Condition

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ABSTRACT: Every ground/rock mass has a pre-existing stress. Stress on the rock mass changes from point to point, practically this brings considerable influence on shear strength characteristics of the rock mass. It is requisite to understand the behavior of the rock with varying stress concentration for the analysis and design of the civil structures, specially tunnel. This research work is an attempt to scrutinize the effect of uneven pre- existing stress under different confining pressures. The partial confinement across the sample will create an uneven stress across the sample. Partial confinement of the rock on triaxial system simulates the uneven stressed ground condition better than conventional rock triaxial. This research is to analyze the change in shear parameter due to partial confinement. The circular opening of 6 mm and 10 mm diameter is drilled on the cylindrical specimen of 45 mm diameter and aspect ratio 2, creating a partial confinement with the help of firm material. Study contains comparisons of the intact rock with the partially confined condition at different position on the rock with various size of circular openings. The results obtained are analyzed with linear approach of Mohr-Coulomb as well as non-linear approach for jointed rocks with discontinuity by Hoek and Brown (1980a, b). The Jointed rock by Hoek and Brown(1983) is also taken into account. The rock chosen for the work is from Dhrangadhra town near Surendranagar district in Gujarat (India). The primary objective is to inter-relate the strength parameter, material properties and shearing pattern after introducing of partial confinement.

Keywords: Partial confinement; Rock triaxial test; Circular opening

1. Introduction

Rock is being used as a contruction material since centuries. The rock shelters made by mankind thousands of years ago are the examples of it. Rock has umpteen variation of different properties because of joints, folds, faults and fissures. Till date, there are difficulties preparing structures upon and below it. Nowadays rock mechanics is being used for different heavy structures such as military bases, nuclear powerplants, tunnels for transporting fluids and mobility of vehicles. As the constuction on the rock in time consuming, it requires a rigorous exploration of the properties of the ground. The problem of concern is that non-homogenity of the strength across the rock mass and difference of stresses after the constuction.

Present study focuses on the non-homgenous stresses produced across the rock. For simulating such condition on electronic Rock triaxial test apparatus a sleeve is used with intact rock samples and samples with different openings.

2. Materials and Experimental Method

Sandstone is a sedimentary rock composed mainly of sand-sized mineral particles or rock fragments. Most sandstone is composed of silicates because they are the more resistant minerals to weathering processes at the Earth's surface. Sandstone may be any color due to impurities within the minerals, but the most common colors are red, grey, pink, white, and black, tan, brown, yellow. They are formed from cemented grains that may either be mono-minerallic crystals or be fragments of a pre-existing rock. The formation of sandstone is divided in two stages. First, the layer of sand accumulates as a result of sedimentation due to water (streams and rivers) or air (in deserts). Finally sandstone is formed by the overburden pressure and cemented by minerals from seeping water.

The common minerals are cacl2 and silica. All sandstones are composed of the same minerals. These minerals make up the framework components of the sandstones. Such components are feldspars, quartz and lithic fragments. Sandstones are divided based on mineralogy and texture. Sandstones have many

application in our daily life such as domestic construction. It is relatively soft rock which can be carved into different shapes such that it is used as temples, homes and artistic ornamental statues from ancient times.

Sandstone as junk is procured from Dhrangadhra town of Gujarat(India) and core samples are taken out consisting L/D ratio as 2 to 2.5 with the help of core cutter machine confirming to IS: 9179-2001.

The sleeve is fabricated with the high grade aluminium which holds the rock and doesn't allow the oil to create pressure on the sample. The clamp type sleeve rest on the rock with the help of bolts of 6 mm on both the sides. It holds the rock such a way that there is no space between the sample and sleeve. The partial confinement is accomplished on the specimen with the help of the sleeve on whichever position required. Silicon gel was then applied on the joints of the sleeve and the rock for relieving pressure from the sample shown in Fig. 3.

Rock triaxial is performed confirming to IS: 13047-2010 and the shear parameters were observed for the set of 3 test on different confining of 1 MPa, 1.5 MPa, 2 MPa respectively for the intact rock samples as well as pattern rock matrix samples at the strain rate of 0.125 mm/min. The intact rock samples with fully confinement were tested conventionally and samples with the partial confinement were tested firm aluminium sleeve which is 1 mm loose from the rock sample. The pressure of the oil was sealed from using silicon gel which will not let the pressure to act on the sample. The sample was to left for 10-12 hours to dry silicon gel. The test was then started conventionally by applying σ3 around the cylinder and measuring stress while applying constant strain. The electronic rock triaxial (a) with digital load cell (b) and Constant pressure system(c) used for the study are shown below in Fig. 1.

3. Laboratory Testing

For this experimental study, different types of matrix has been prepared shown in Fig. 2. The Table 1 shows the notations for the different rock matrix. The preliminary test according to IS: 13030-1991 shows the results as shown in Table 2.

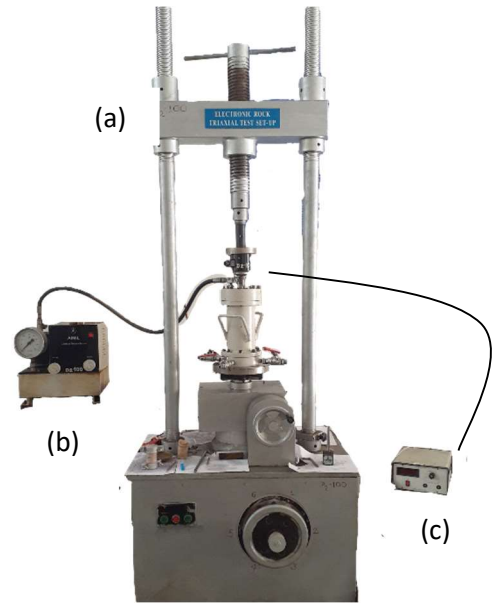

Figure 1. Rock triaxial system.

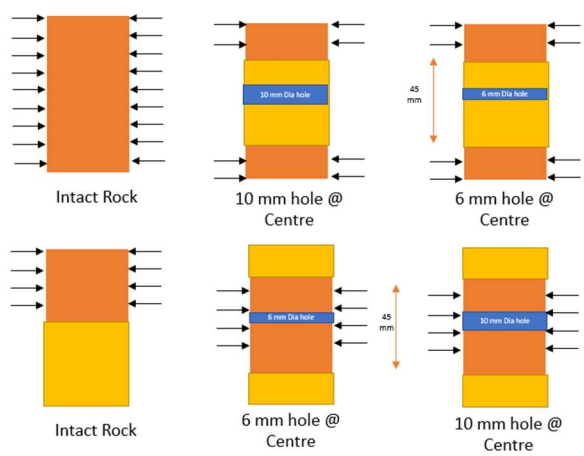

Figure 2. Types of matric prepared for triaxial test.

Table 1. Notations for rock matrix.

Sr. No.	Specification	Notation
1	Intact rock	A_IR
2	Intact rock with unconfined portion of h/2 from top	A_IR_UT
3	6 mm hole with unconfined portion of h/2 at centre	A_6C_UC
4	10 mm hole with unconfined portion of h/2 at centre	A_10C_UC
5	6 mm hole at centre with unconfined portion of h/4 at top and bottom	A_6C_UTB
6	10 mm hole at centre with unconfined portion of h/4 at top and bottom	A_10C_UTB

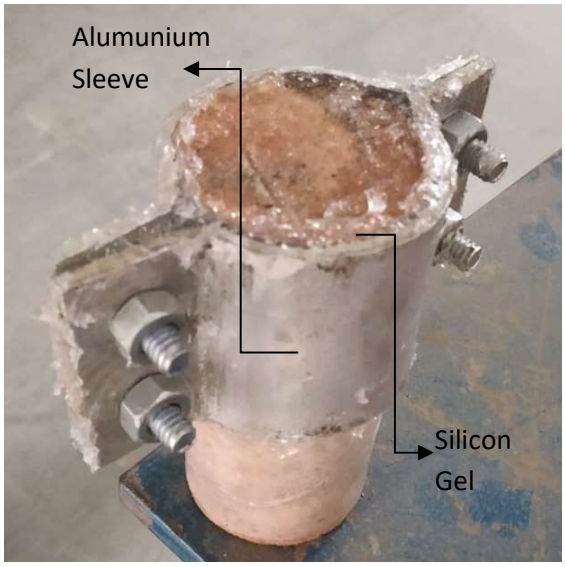

Figure 3. Sample preparation for partially unconfinement.

Table 2. Preliminary test on rock sample.

Type of matix	Sample No.	Water Content (%)	Void Ratio e=vv/vs	Density kN/m3 (ρ=M/V)
A_IR	1	1.12	0.12	24.23
	2	1.24	0.13	24.8
	3	1.18	0.13	24.12
A_IR_UT	4	1.17	0.13	23.36
	5	1.29	0.12	24.45
	6	1.26	0.14	24.62
A_6C_UC	7	1.18	0.12	23.52
	8	1.23	0.13	23.63
	9	1.23	0.12	23.20
A_10C_UC	10	1.20	0.12	23.68
	11	1.17	0.13	23.37
	12	1.22	0.13	23.46
A_6C_UTB	13	1.17	0.12	22.86
	14	1.22	0.12	22.09
	15	1.25	0.13	22.26
A_10C_UTB	16	1.19	0.14	22.82
	17	1.25	0.12	22.56
	18	1.28	0.12	23.03

4. Results and Discussion

From the plots of Fig 7. and Fig 8., the general inferences are discussed below.

i. The effect of confining pressure both on full length of sample was determined using triaxial system and shear parameters obtained were 'c' value of 5.86 and Φ value of 36.24 under confining pressure from 1 MPa to 2 MPa with interval of 0.5 MPa.

ii. The effect of partial confinement/ confining pressure was done for samples viz. A_IR, A_IR_UT, A_6C_UC, A_10C_UC, A_6C_UTB, A_10_UTB and it was observed that 'c' value decreased from 5.86 MPa to 4.26 MPa while the Φ value decreased from 36.24 to 24 respectively.

iii. Referring to figure 1.1 to 1.4 and comparing A_IR with A_IR_UT ,it is seen that the 'c' value decreses from 5.86 to 4.26 and Φ value decreases from 36.24 to 31.35.

iv. Comparing A_6C_UT with A_IR and A_IR_UT, the 'Φ' value decreases 22.73% and 10.68% respectively.

v. Comparing A_10C_UC with A_IR and A_IR_UT, the 'Φ' value decreases 33.77% and 23.44% respectively.

vi. Comparing A_10C_UC with A_6C_UC, the 'Φ' value decreases 14.29% and Comparing A_10C_UTB with A_6C_UTB , the 'Φ' value decreases 12.46% respectively.

vii. The above changes in shear parameter indicate that sandstone due to its intermetabolic structure of grain conglomeration and the bonding structure does not make any major change in cohesion value both under full confinement and partial confinement . similarly the higher course grain partical size sandstone the higher angle of internal friction is obtained for unconfined condition and it decreases with increase in confinement coverage area.

Referring to the Fig 4., Fig 5. and Fig 6. it is observed that nature of all the plots is curvilinear asympotic to x-axis. The initial portion of the plot indicates that strain values are governing the rock behavior almost for both confined and un-confined conditions. Sandstone being semi-elastic in nature, strain acceptance values are higher and diffraction of grain-to-grain is making initial portion of plot more concave, while the later portion of plot reflects quasi-plastic behavior of rock. When confining pressure are absent along certain length of rock, internal stress distribution phenomena into that zone is acting as non-linear which makes mobilization of strength more complex and it is this reason that partial confined/confinement of applied pressure is completely

changing the failure behavior of rock sample. It is seen from the plot in Fig 4., Fig 5. and Fig 6. that at higher confinement the major principle stress (σ1) increases.

Referring to Fig 7. and Fig 8., comparisons of cohesion values shows the decrement for partially confined samples although the decrement is not too high to make major change in the shear strength. While angle of internal friction changes from 36.24° to 24° which shows considerable decrement in shear strength of the rock.

The Mohr circles for A_IR is shown in Fig 10., similar plots were obtained for all other samples. The failed sample of triaxial test are shown in Fig 9.

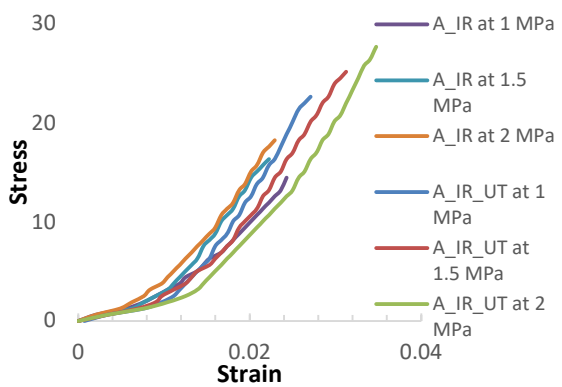

Figure 4. Stress strain curve for A_IR and A_IR_UT samples.

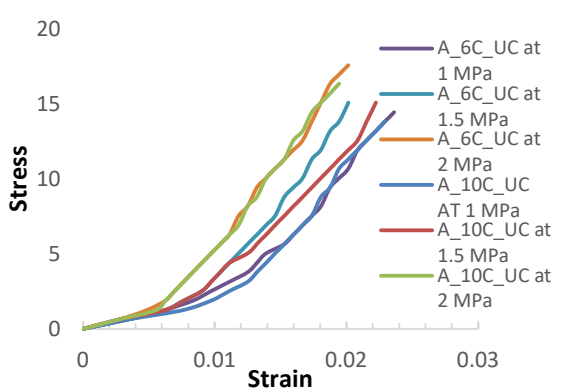

Figure 5. Stress strain curve for A_6C_UC and A_10C_UC samples.

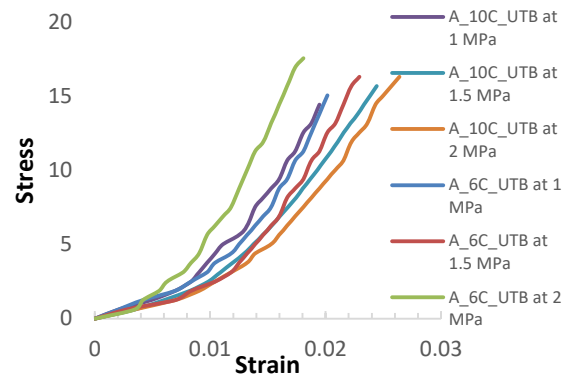

Figure 6. Stress strain curve for A_6C_UTB and A_10C_UTB samples.

Referring to Table 4., it shows the results obtained theoritically by the Hoek Brown theory, when compared to practical results it shows the errors due to neglectance of pore water pressure in the rock samples for the triaxial test. While for partially confined/confinement shows more than 30% higher major principle stress than practically observed because hoek brown theory does not have any constants which could define partially confined/confinement condition.

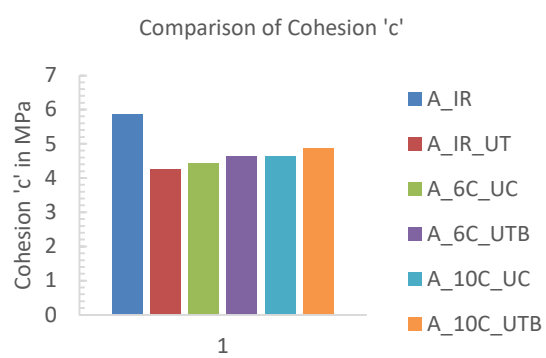

Figure 7. Comparison of 'c' value.

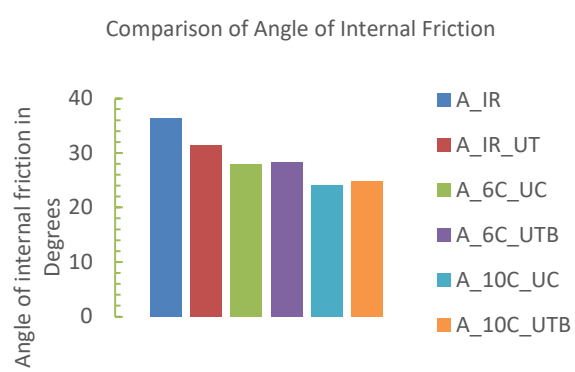

Figure 8. Comparison of 'Φ' value.

Figure 9. Failure pattern of the various samples.

The failure pattern in the Intact rock was observed to be brittle with and without partial confinement as there partial confinement does not change type of failure. The rock with 10 mm circular opening shows more strain than 6 mm as more opening brings more strain to the rock sample. The sample with 10 mm circular opening shows gradual failure while the rock sample with 6 mm circular opening shows sudden failure of the rock sample.

The peak values of stress increases with the increase of the confinement pressure and the samples with the circular opening has lower modulus of Elasticity compared to the intact sample.

The samples with the opening shows vertical failure pattern passing from the opening. The sample with 10 mm circular opening was shows minor crack as compared with sample consisting of 6 mm and intact rock. The sample with 10 mm opening shows higher strain and losses strength and the breakage was seen to be less brittle and more strain governed.

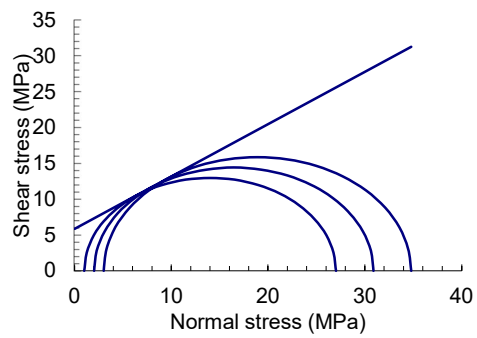

Figure 10. Mohr circle for A_IR.

Table 4. Theoritical results based on Hoek brown theory

Samples	σ3 (MPa)	Tσ1 (MPa)	Pσ1 (MPa)	% Error
A_IR	1	27.24	27.00	0.89
	1.5	30.89	30.89	0.00
	2	34.23	34.78	-1.60
A_IR_UT	1	27.24	18.33	32.72
	1.5	30.89	21.5	30.39
	2	34.23	24.66	27.94

When the unconfined sleeve of 22.5 mm height is at bottom and top, the c increases while the Φ remains approximately same. The weaker portion of the rock sample is a circular opening which is confined, as the higher confinement gives higher strength and the results shows the same. When the sample with circular opening is introduced, the value of c and Φ both decreases which can also be seen in results.

5. Conclusions

- The failure pattern of an intact rock specimen with partial confinement starts from the unconfined top portion which is seen in all samples of A_IR_UT.
- For samples with circular opening failure starts from the opening with higher strain.
- The errors for the Hoek Brown analysis is due to neglectance of pore water pressure.
- Hoek Brown shows more than 30% higher principle stress than practically observed for the samples with Partial confinement.
- With increasing confinement, there is tendency of decrement the modulus of elasticity.
- Cohesion decreases by 37% and angle of internal friction also decreases by 15.6% in the specimen with partial confinement compared with A_IR.
- This allover brings 64% decrement in shear strength of the rock sample with partial confinement compared to A_IR.

6. References

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6.1. Standards:

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