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"STRENGTH CHARACTERISTICS OF JOINTED ROCK MATRIXES WITH CIRCULAR OPENING UNDER TRIAXIAL COMPRESSION"

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ABSTRACT : *In mining and tunneling operation,cracks propagation,existing fissures into a rock mass is most common process.This process of widening joints into a rock mass can be simulated into matrix formation.This study has been done to to overcome this problem by postulating this crack, fissures pattern into various matrix forms into cylindrical rock specimen (sandstone) using triaxial test system to obtained true strength characteristics viz. shear parameters and modulus of elasticity. The paper contains comparison of different matrix pattern. The results were also compared with to those for corresponding intact cylindrical rock specimens. The results indicate that intact specimen delivers more strength and higher modulus of elasticity as compared to various matrix patterns.*

KEY WORDS: *Rock Matrix, Sand stone, Tri-Axial test ,Circular Opening ,Non-jointed specimen, Jointed specimen*

1. INTRODUCTION

Rock differs from most other engineering materials in that it contains discontinuities such as joints, bedding planes, folds, sheared zones and faults which render its structure discontinuous. Rock mechanics may be applied to many engineering applications ranging from dam abutments, to nuclear power station foundations, to the manifold methods of mining ore and aggregate materials, to the stability of petroleum wellbores and including newer applications such as geothermal energy and radioactive waste disposal.

Rock mechanics is mostly concerned with rock on the scale that appears in engineering and mining work, and so it might be regarded as the study of the properties and behavior of accessible rock due to changes in stresses or other conditions. Two distinct problems are always involved: (i) The study of the orientations and properties of the joints, and (ii) The study of the properties and fabric of the rock between the joints. In any practical investigation in rock mechanics, the first stage is a geological and geophysical investigation to establish the lithologies and boundaries of the rock types involved. The second stage is to establish, by means of drilling or investigatory excavations, the detailed pattern of jointing, and to determine the mechanical and petrological properties of the rocks from samples. With this information, it should be possible to predict the response of the rock during excavation or loading. There are two basic reasons for an engineer to understand stress in the rockWhich includes (a) there is a pre-existing stress state in the ground and we need to understand it, both directly and as the stress state applies to analysis and design. It is emphasized again here that there can be circumstances when, during the engineering, no new loading is applied, e.g. when driving an unsupported tunnel in rock. In this latter case, the pre-existing stresses are redistributed.(b) Stress state can be changed dramatically, because rock, which previously contained stresses, has been removed and the loads have to be taken up elsewhere. So these unsupported excavation surfaces are principal stress planes. Furthermore, most engineering criteria are related to the strength of the rock and the analysis of these subjects involves stresses. For example, almost all failure criteria are expressed as a function of certain stress quantities. During excavation of rock we are dealing with a reverse type of construction where the rock material is being taken away, rather than added, to form a structure. On the mining side, rock may be excavated in an open pit and we will then be concerned with the stability of the sides of the open pit. Joints are the most significant discontinuities in rocks. Joints are breaks of geological origin along which there has been no visible relative displacement. A group of parallel or sub-parallel joints is called a joint set, and joint sets intersect to form a joint system. Joints may be open, filled or healed. Discontinuities frequently form parallel to bedding planes, foliations or salty cleavage, and they may be termed bedding joints, foliation joints or cleavage joints. Sedimentary rocks often contain two sets of joints approximately orthogonal to each other and to the bedding planes. These joints sometimes endat bedding planes, but others, called master joints, may cross several bedding planes. "Since joints are among the most important causes of excessive over break and of trouble with water, they always deserve careful consideration" as told by Karl Terzaghi (1946). So some research works are required in the area of rock matrix. There is no general equation that exists, which adequately defines completely matrix properties of all types of rock. This property varies from rock to rock and other factors also. Lots of research work has already been done on intact rock (Bieniawski (1967), Hudson (1971), Jaeger (1976), Hoek (1980),Brown (1980) Stephansson (1985), Kulatilake (2001), X.G.Zhao (2010), Alejano (2012), Arzua (2012), Walton(2012), etc.) but few researchers (Hudson (1971), Stephansson (1985), Hoek (1980), Brown(1980), Alejano (2012), Arzua (2012)) focuses has been made towards rock matrixes, that's why efforts has been made to study the behavior of different rock matrix of different patterns which actually simulates crack and joints patterns when rock mass is under stress due any external disturbance. The different types of matrixes are available on site such as (1) Polyhedral block (2) Equidimensional block (3) Prismatic block (4) Tabular block (5) Rhombohedral block (6) Columnar block.

In present investigation sandstone is used for laboratory investigation to know the shear and compression capacity of this rock samples with simplest matrix pattern viz.

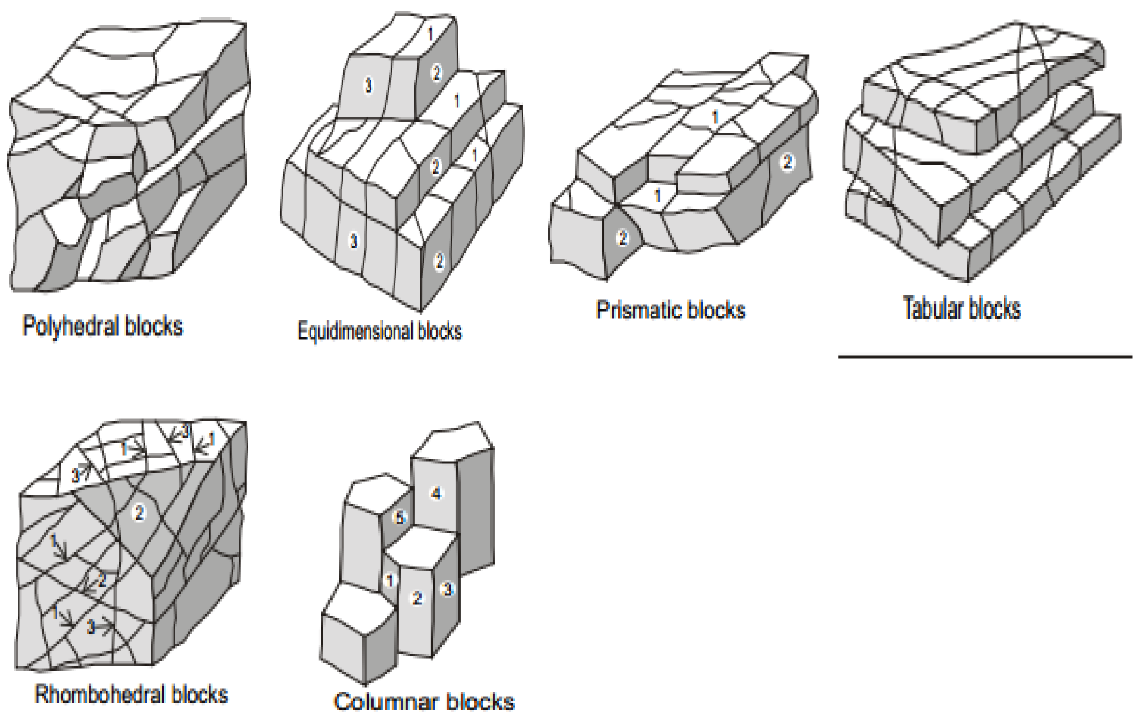


Figure 1 Types of matrix

present research work is to study shear parameters (C &Φ); & stress-strain characteristics, Modulus of elasticity (E) is to be determined for different rock opening such as (1) Intact (2) Circular Opening 6 mm at Center (H/2) (3) Circular Opening 10 mm at Center (H/2|) (4) Circular Opening 14mm at Center (H/2) (5) Circular Opening 6 mm at H/3 and 10mm at center(H/2) (6) Circular Opening 6mm at 3H/4 and 10mm at H/2(7) Circular Opening 10mm at H/3 and 10mm At center(H/2) (8) Circular Opening 10mm at 3 H/4 and 10mm At center (H/2)(9) Circular Opening 10mm at H/3 and 10mm At 3H/4(10) Circular Opening 10mm at H/3 , 10mm At H/2 and 10mm At 3H/4(11) Circular Opening 10mm at H/2 and Horizontal Cut (12) Circular Opening 10mm at H/2 and Inclined Cut 20' total 72 test by triaxial system on different types of soft to medium hard rock viz. *Rajasthan sandstone* and *Dhrangadhra sandstone* , using triaxial test system under various confining pressures at constant strain rate.

2. MATERIALS AND EXPERIMENTAL SETUP

2.1 Rock Sample

Source of the sandstonespecimen was procured fromRajasthan andDhrangadhra.

2.2 Test methodology

The rock triaxial test is performed according to IS-13047-2010 and shear parameters are obtained for three different confining pressures viz. 10, 15 & 20 N/mm^2 for the intact and rock matrix pattern specimens of Limestone for the constant strain rate of 0.315 mm/min.The usual procedure for conducting a tri-axial compression test is first to apply the confining pressure σ_3 all around the cylinder is held constant & then to apply axial load σ_1. In a tri-axial compression test, the direction of load is called the maximum principal direction & the direction of the confining pressure applied is the minimum principal direction. Through plunger vertical load is applied which causes failure in the sample. This test also helps in the determination of shear strength parameters of the rock material. Three cylindrical samples of same rock material are subjected to three different chamber pressures. For each chamber pressure sample is loaded to failure. Thus Major and minor principal stresses are known. The Mohr's circles are drawn with three sets of observations. Mohr's envelope gives the value of cohesion "c" and the angle of internal friction "Φ".

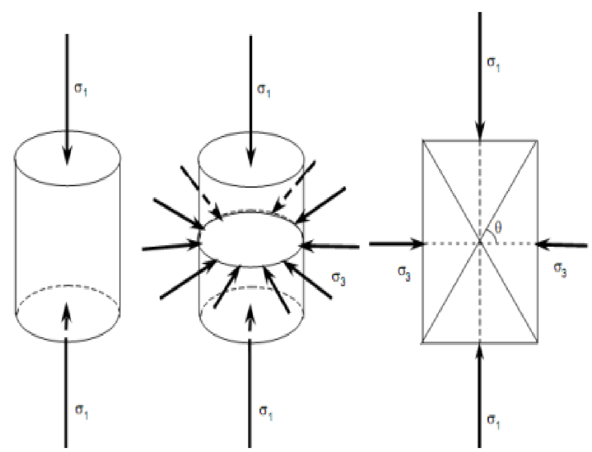


Figure 2 Cylindrical sample under confining pressure

2.3 LABORATORY TESTING

Index Properties

The index properties of intact sandstone samples viz. sample

(1) Intact
(2) Circular Opening 6 mm at Center (H/2)
(3) Circular Opening 10 mm at Center (H/2|)
(4) Circular Opening 14mm at Center (H/2)
(5) Circular Opening 6 mm at H/3 and 10mm at center(H/2)
(6) Circular Opening 6mm at 3H/4 and 10mm at H/2
(7) Circular Opening 10mm at H/3 and 10mm at center(H/2)
(8) Circular Opening 10mm at 3 H/4 and 10mm At center (H/2)
(9) Circular Opening 10mm at H/3 and 10mm At 3H/4
(10) Circular Opening 10mm at H/3 , 10mm At H/2 and 10mm At 3H/4
(11) Circular Opening 10mm at H/2 and Horizontal Cut
(12) Circular Opening 10 mm at H/2 and Inclined Cut 20'

having aspect ratio (L/D) 2 was obtained in accordance with IS13030-1991

Fig 3 Core cutter machin

Fig 3Cutter machin

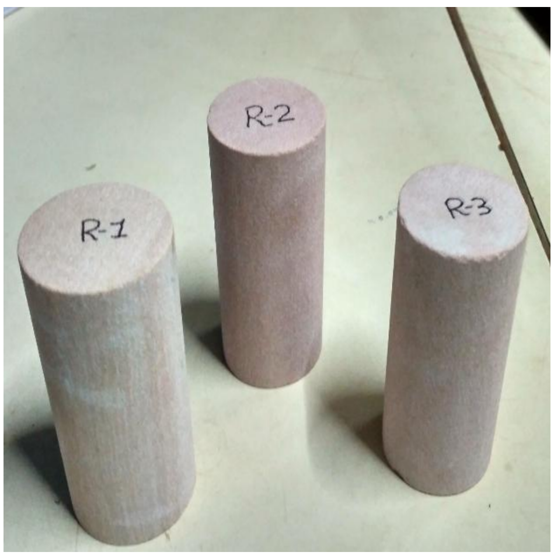


Figure 4 Intact

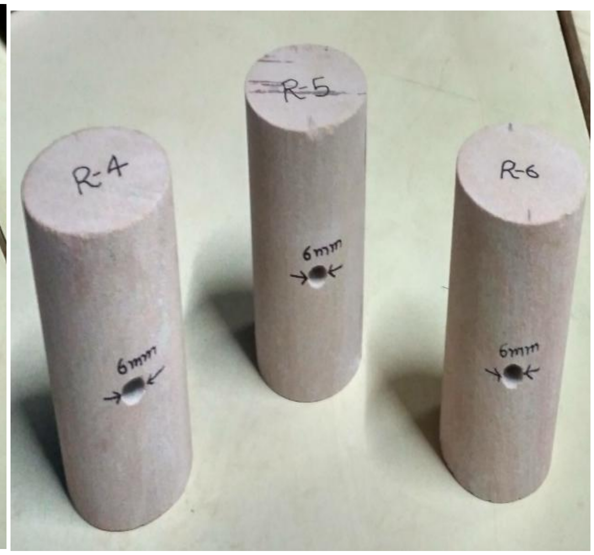


Figure 5 Circular Opening 6 mm at Center

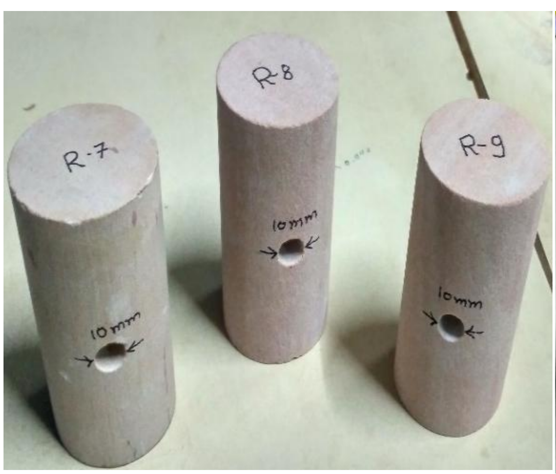


Figure 6 Circular Opening 10 mm at Center

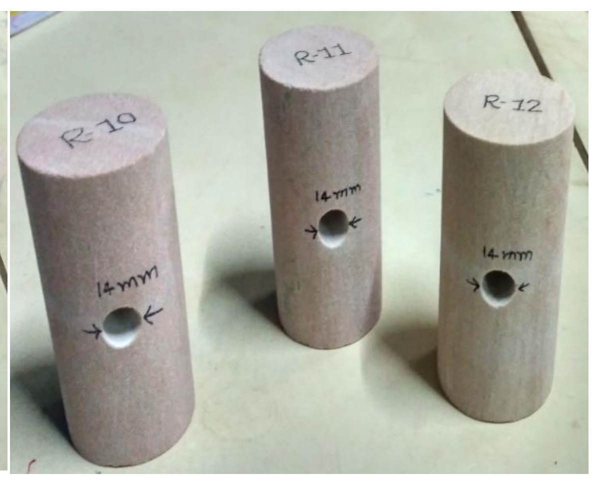


Figure 7 Circular Opening 14mm at Center

Figure 8Circular Opening 6 mm at H/3
and 10mm at center

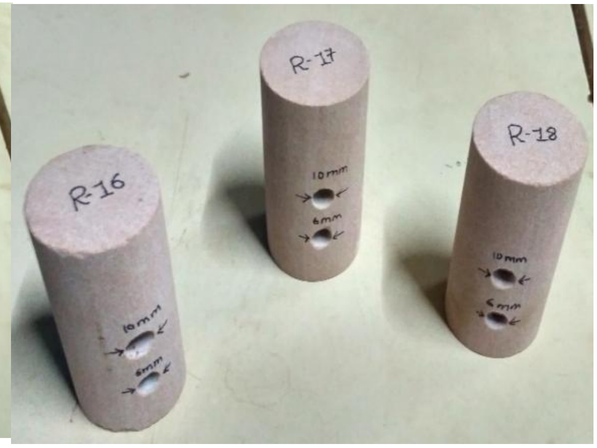


Figure 9 Circular Opening 6 mm at 3H/4
and 10mm at center

Figure 10 Circular Opening 10mm at H/3 center 10mm At center

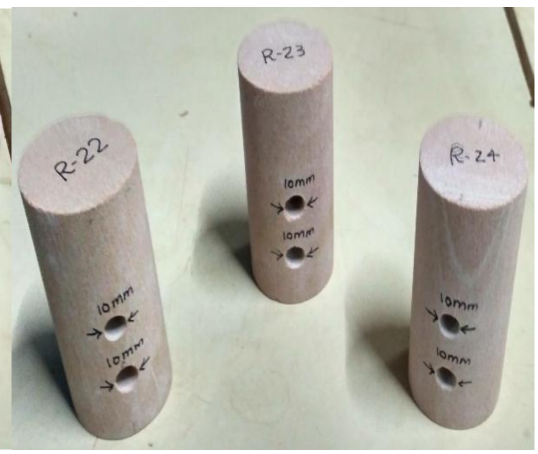


Figure 11 Circular Opening 10mm at 3 H/4 and 10mm At

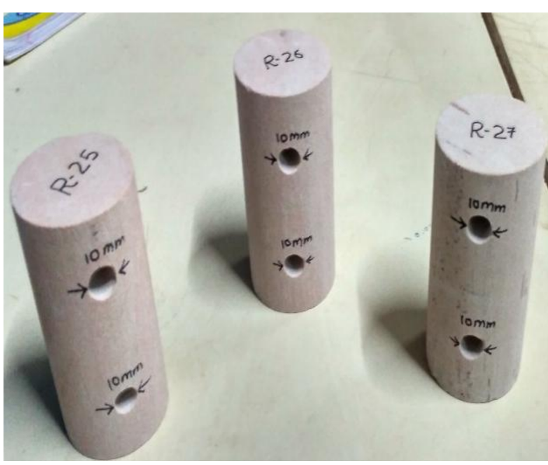


Figure 12 Circular Opening 10mm at H/3
H/3,H/2 and 10mm At 3H/4

Figure 13 Circular Opening 10mm and 10mm At 3H/4

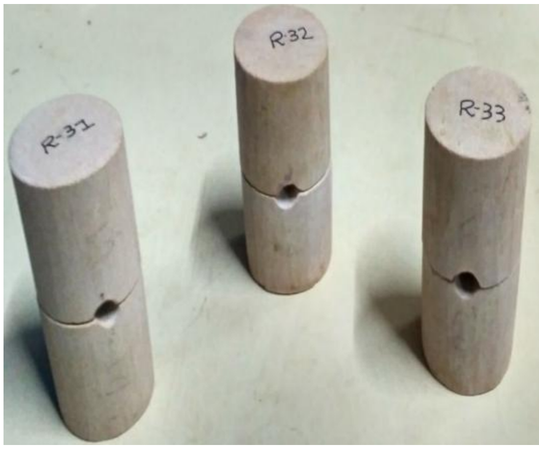


Figure 14 Circular Opening 10mm at H/2 HorizontalCut

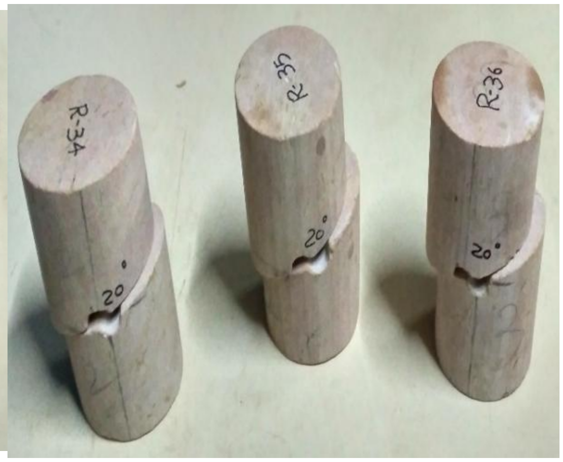


Figure 15 Circular Opening 10mm at H/2 and and Inclined Cut 20'

Type of matrix	Sample no.	Height (mm)	Dia. (mm)	Water content(%) w=(Mw/Ms)*100	Void Ratio e=Vv/Vs	Density(kN/m³) ϱ=M/V
R_IR	1	90	45	2.08	0.09	24.26
	2	90/	45	2.15	0.09	24.95
	3	91	45	2.73	0.11	24.08
R_6C	4	90	45	2.62	0.10	23.88
	5	93	45	2.30	0.10	23.01
	6	92	45	2.45	0.10	22.90
R_10C	7	93	45	2.42	0.10	22.86
	8	94	45	2.29	0.10	22.82
	9	93	45	2.48	0.10	22.32
R_14C	10	94	45	2.60	0.12	22.15
	11	95	45	2.75	0.12	22.7
	12	93	45	2.18	0.11	22.11

Table 1 Physical properties of Rajasthan sandstone

R_6T_10C	13	92	45	0.54	0.028	22.44
	14	93	45	0.82	0.035	22.03
	15	91	45	0.54	0.028	22.78
R_10T_6B	16	90	45	0.55	0.036	22.72
	17	91	45	0.80	0.043	22.92
	18	93	45	0.80	0.035	25.29
R_10T_10C	19	94	45	0.80	0.034	21.29
	20	93	45	0.55	0.035	21.69
	21	92	45	0.79	0.042	21.48
R_10C_10B	22	91	45	0.54	0.028	21.65
	23	90	45	0.54	0.036	21.21
	24	91	45	0.26	0.021	21.40

Type of matrix	Sample no.	Height (mm)	Dia. (mm)	Water content(%) w=(Mw/Ms)*100	Void Ratio e=Vv/Vs	Density(kN/m³) ϱ=M/V
$R_{\_10T\_10B}$	25	92	45	0.54	0.028	21.44
	26	93	45	0.82	0.035	21.03
	27	91	45	0.54	0.028	21.78
$R_{\_10T\_10c\_10B}$	28	90	45	0.55	0.036	20.72
	29	91	45	0.80	0.043	20.92
	30	93	45	0.80	0.035	20.29
$R_{\_10C\_90}$	31	94	45	0.80	0.034	25.29
	32	93	45	0.55	0.035	24.69
	33	92	45	0.79	0.042	25.48
$R_{\_10C\_20}$	34	91	45	0.54	0.028	25.65
	35	90	45	0.54	0.036	24.21
	36	91	45	0.26	0.021	26.40

3 .RESULTS AND DISCUSSION

3.1 RESULTS

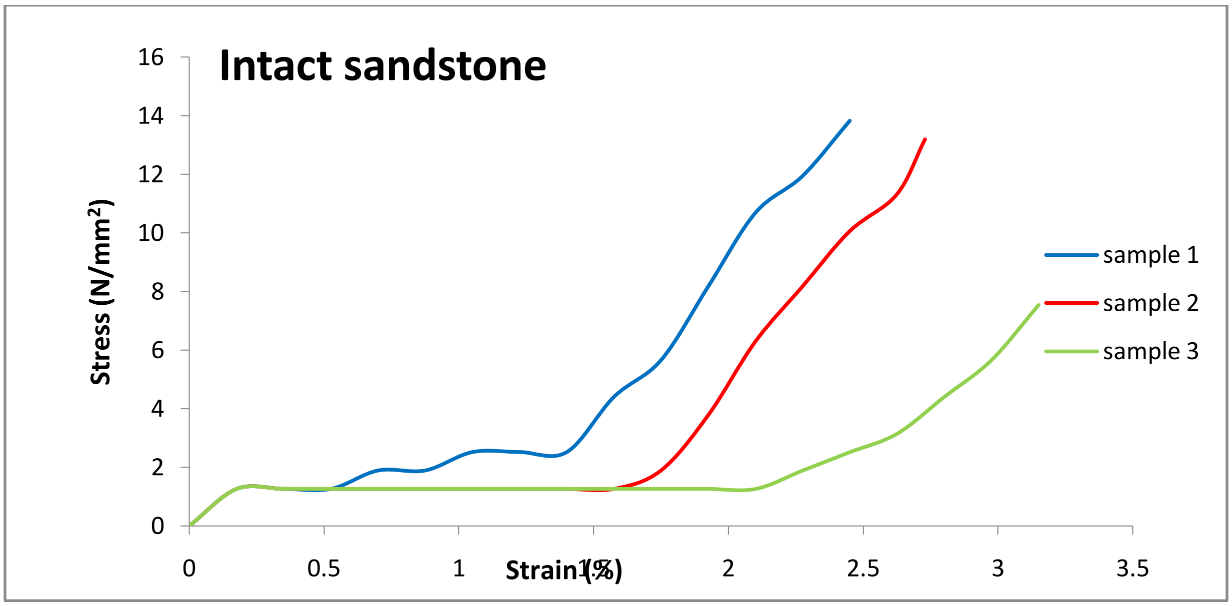
Table 2 Triaxial test results for $R_{\_IR}$

Load (P) (N)	Deviator Stress $\sigma_d = \frac{Load}{Area}$ (N/mm^2)	Confining Pressure σ_3 (N/mm^2)	Normal Stress $\sigma_n = \sigma_d + \sigma_3$ (N/mm^2)	c (N/mm^2)	Φ (°)	E (N/mm^2)
53000	33.32	10	43.32	7.51	33	1002.26
62000	38.98	15	53.98			968.5537
70000	44.01	20	64.01			898.2554

Table 3.2 Theoretical results based on relationship for $R_{\_IR}$

Griffith's **theory** $\sigma_1 = \sigma_3 + (\sigma_c m_i \sigma_3 + S\sigma_c{}^2)^{0.5}$ (N/mm^2)	**Percentage of error** $P = \frac{100 * (T\sigma1 - P\sigma1)}{T\sigma1}$ **(%)**	**Hoek-Brown failure criterion** $\sigma_1=\sigma_3+\sigma_{ci}((m_i*\sigma_3)/\sigma_{ci})+1)^{0.5}$ (N/mm^2)	**Percentage of error** $P = \frac{100 * (T\sigma1 - P\sigma1)}{T\sigma1}$ **(%)**
44.32	2.256	44.19	1.968
54.84	1.568	54.31	0.607
65.36	2.065	65.09	1.659

FIFURE 16 Different types of failure observed in different types of pattern



3.2 DISCUSSION

- From the above graph it is observed that as the confining pressure increasing from 10 to 20 N/mm^2 by difference of 5 N/mm^2, the percentage increment in stress is observed respectively and the percentage decrement in modulus of elasticity (E) is observed as respectively.
- The stress vs. strain curves were not steepen with increasing confining pressure, that was to say, the elastic modulus of rock decreased with increase in confining pressure, which is due to the development of microstructures, and they were compressed and closed under the action of confining pressure.
- Peak values of stress and corresponding strain values increased due to the effects of confining pressure. With increasing confining pressure, the overall elastic modulus of the rock has a tendency to decrease.
- As the confining pressure increases from 10 to 15 N/mm^2, the elastic modulus decreases slowly. However, as the confining pressure increases from 15 to 20 N/mm^2, the elastic modulus decreases rapidly.

3. CONCLUSIONS

- The value of cohesion (c) is observed to be decreasing and angle of internal friction angle (Φ) is also decreasing with increases in number of joints and increases in angle of joints because as number of joints increases, friction between two rock specimens decreases.
- The failure has been observed instantaneously because it's a brittle material.
- Observed that shear angle was dependent on confining pressure.
- At higher confining pressure, the load carrying capacity increases due to restrain provided by the surrounding confinement which does not allow the specimen to deform.
- Strength of jointed rock is dependent on the direction of applied loading with respect to orientation of joints.
- The normal stress is found to be decreasing as numbers of joints and angle of joints are increasing
- Observed that internal friction angle (Φ) was dependent on confining pressure and the spacing of joint in the specimen.
- The percentage of error in practical result with Mohr-Coulomb Failure theory and Hoek-Brown failure criteria is due to the neglecting pore pressure for all samples.
- Peak values of stress and corresponding strain values increased due to the effect of confining pressure.
- With increasing confining pressure, the overall elastic modulus of the rock has a tendency to decrease.
- In jointed rock matrixes a minute hair cracks were developed near the vicinity of treated portion of jointed area due to partly resistance of load, which is parallel or at some angle to the vertical direction.
- In non-jointed specimen the failure is observed in terms of broken pieces of specimen.
- The most noteworthy seen from stress-strain curve is the occurrence of the downward concave behavior in the early stages of loading indicating the development of non uniform normal stresses.
- During the formation of limestone, pore structure are generated which is due to presence of marine spices. When these spices get decomposed a pore structure is developed which results into decrease in strength of rock.
- Due to cut the relative density by weight gets reduced. The strength due to this cut gets reduced compared to intact rock due to reduction in density of rock.

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