“Effect of Cemented Fill on Strength of Jointed Specimen”

Submitted in Partial Fulfilment for the Award of the Degree of

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in
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With specialization in

STRUCTURAL ENGINEERING
By

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CERTIFICATE

This is to certify that the project entitled **“Effect of Cemented Fill on Strength of Jointed Specimen ”** being submitted by me, is a bona fide record of my own work carried by me under the guidance and supervision of Prof. A.Trivedi in partial fulfilment of requirements for the award of the Degree of Master of Engineering (Structural Engineering) in Civil Engineering, from University of Delhi, Delhi.

The matter embodied in this project has not been submitted for the award of any other degree.

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This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

Several, complicated and difficult structures are constructed or in planning stages under complex geological conditions around the world. Even small variation in analysis and design can cost significantly. It is well recognized that the strength of rock masses depends upon the strain history, extent of discontinuities, orientation of plane of weakness, condition of joints, fill material in closely packed joints and extent of confinement. Several solutions are available for strength of jointed rock mass with a set of discontinuities. There is a great multiplicity in the proposed relationships for the strength of jointed rocks. In the present study, the author conceives the effect of increasing stresses to induce permanent strains. This permanent strain appears as micro crack, macro crack and fracture. A fully developed network of permanent deformations forms joint. The joint may contain deposits of hydraulic and hydrothermal origin commonly known as a fill which may have cementing tendency. The joint factor numerically captures varied engineering possibilities of joints in a rock mass. The joints grow as an effect of loading. The growth of the joints is progressive in nature. It increases the joint factor, which modify the failure stresses. After extensive experimentation significant joint properties affecting the strength of jointed rocks with unfilled joints and joints with cemented fill has been evolved. This factor is called joint modified factor in which number of joints per meter length, orientation of joints and strength along joints and strength along joints are clubbed together. As the in situ determinations of jointed rock mass is costly and time consuming attempts are being made to predict the strength and deformation of rock mass through model test under controlled laboratory conditions

In the present work jointed rocks are simulated by preparing specimens of mortar and cemented joints containing PoP were created artificially by inducing paste of PoP inside the joints. The experimental investigations have been carried on PoP cemented at varied joints possibilities specimen. The specimen made of cement and standard sand in the ratio of 1:3 to simulate the rock mass. The samples were cured at the interval of 7 days to create weakly cemented rock mass and at 28 days to make a comparatively stronger rock

mass. The strength changes on 7 days and 28 days are incorporated in relation to their peak compressive strength in presence of PoP cemented joints. The specimen was tested under uniaxial compression to determine the various parameters. The results have been analysed in relation to the modified joint factor Trivedi(40) and a simple empirical approach has been found to predict unconfined compressive strength of jointed rocks with PoP cemented joints. The investigation indicates that the results are in conformity with the recent analyses proposed by Trivedi [40].

CONTENTS

LIST OF SYMBOLS & ABBREVIATIONS

List of symbols

φcn,φpeak	Angle of critical friction and peak internal friction, respectively (°)
φj	Equivalent friction angle for the jointed rocks (°)
C'& C	Initial confining pressure-dependant empirical fitting parameters for jointed rocks
Cg	A modification factor for gouge
da	Reference depth of joint (=sample diameter in mm)
dj	Depth of joint in mm
gd	Correction factor depending upon the density of gouge in joint
Jdj	Joint depth parameter
Jf	Joint factor
Jfg	Joint factor corrected for gouge
Jn	Number of joints in the direction of loading (joints per meter length of the sample)
Jt	Gouge thickness parameter
Lna	Reference length (=1 m)
M and B	Empirical rock constants
n	Joint orientation parameter depending upon inclination of the joint plane [(β)] with respect to the direction of loading
p	Mean confining pressure (kPa)
Pa and sa	Reference pressure (=1 kPa)
Pi	Initial mean confining pressure (kPa)
q	Shear stress (kPa)
Qj and rj	Empirical material fitting constants for gouge
r	Joint strength parameter
RAC	Ramamurthy–Arora criterion
RD	Relative density of gouge
t	Thickness of gouge in the joint (mm)
ta	Reference thickness of gouge in the joint (=1 mm)
λ	Empirical coefficient for joint factor
α	Fitting constant
s1,s3	Major and minor principal stresses, respectively (kPa)
sci,scj,scig	Uniaxial compressive strength of intact, jointed and jointed rock with gouge respectively (kPa)
sci	Strength ratio
scr	Strength reduction factoruring shear along the gouge.

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1.0 INTRODUCTION

The strength of jointed rock mass is important for the design of structures built on rocks such as towers, bridge piers, tunnels, deep-seated nuclear and hazardous waste confinements and dams. The rock masses occur in nature with joints and varying amount of infill material commonly known as gouge. Natural geological conditions are usually complex. The earth topography is varied and complex. Rock mechanism is studied as separate field of engineering from engineering geology. It not only deals with rocks as engineering materials but it also deals with changes in its mechanical behavior in such as stress, strain and movement in rocks brought in due to engineering activities. It is also associated with design and stability of underground structures in rock. Rock itself may be homogenous but when we consider rock mass over which we plan our construction. It may behave altogether in different manner due to its defects in the masses such as jointing, bedding planes, fissures, cavities and other discontinuities.

To predict the behavior of rock mass to a nearest value, “in-situ” tests are done but these tests are very expensive. In such cases modeling is proposed .A fair assessment of strength behavior of jointed rock mass is necessary for the design of slope foundation, underground opening and anchoring system. The uncertainty in predicting the behavior of a jointed rock mass under uniaxial stress is essentially caused by scale effects and unpredictable nature of modes of failure, due to jointing and presence of gouge.

Everywhere rock exists in jointed gouged state. They all contained discontinuities along with gouge. Generally rock mass is an anisotropic and discontinuous medium having varied faults. This discontinuities like cracks, fissures, joint, faults and bedding plane make rock weaker more deformable .In case of a dam it can cause leakage of water and it leads to energy loss and erosion of dam.

The years of great engineering development and the anticipated demand of future societies have necessitated the need of very fine scientific observation and a profound engineering vision. The geotechnical engineers concerned with rocks, a few decades ago depended heavily on the empirical relationship based on the strength of intact rocks. But during the works of great specialization on a variety of rock masses subjected to the rocks of natural and post structural origin as in the case of tunnel, open pit dam foundation, undergrounds chambers for storage of oil, disposal of nuclear waste material and underground transportation system to protect/ avoid surface environment, engineers are able to visualize that very fine presence of some fragmented material in the discontinuities influences the strength of rocks and several times it can become critical to the design. The strength of jointed rock with gouged discontinuities depends upon the nature of joints and that of gouge.

Gouge materials is the term used for the filling materials separating the adjacent rock walls of discontinuities e.g. calcite, chlorite, clay, fault & gouge, etc. It consists of the sediments due to the hydro thermal deposition similar in Strength to the enclosing rocks or may be partially loose cohesive or cohesion less soils like clay, sand, coarse, fragmentary material, deposited into open joints or formed in place due to weathering of the joint surface. Filling material into gouge can be divided into following five types:

1. Loose material from tectonically crushed zone.
2. Product of decomposition and weathering of the joint walls.
3. Deposition by ground water flow containing the products of leaching of calcareous rocks.
4. Filling materials brought from the surface.
5. Cemented fills due to the hydrothermal effects and geological compressions

The perpendicular distance between the adjacent rock walls is termed as the width of filled discontinuities.

Due to the enormous variety of occurrences, filled discontinuities display a wide range of physical behaviour, in particular as regards their shear strength deformability and permeability. Short term and long term behaviour may be quite different such that it is easy to be misled by favourable short term conditions.

Upon loading, rock masses experience early plasticity as accommodated in crack closure for intact rocks or joint closure for jointed rocks. Further deformations are elasto-plastic until the brittle failure takes place in the intact rocks. The rock masses with multiple joints conceal brittle failure largely as the joints tolerate large plastic deformations. Figure [4.1 to 4.8] shows a conceptual model of the stress–strain plots for jointed rocks with increasing joints. As a result, the peak strength goes down and failure occurs at a higher strain.

The wide range of physical behaviour depends on many factors of which the following are very important;-

1. Mineralogy of filling material.
2. Grading of particle size.
3. Over Consolidation ratio.
4. Water content and permeability
5. Previous shear displacement.
6. Wall roughness
7. Width
8. Fracturing or crushing of wall rock.
9. Cementation of fill material

According to Lama [23] mechanical behaviour of joint filled with any material, thickness of filling material depends upon the type of filling material, thickness of filling material and height of asperities. In addition to the above mentioned properties the following factors are also significant which govern strength and deformational characteristics of rocks. They are-

1. The angle made by the joint with the principle stress direction (β).

2. The degree of joint separation.

3. Opening of the joint

4. Number of joints in a given direction

5. Strength along the joint

6. Joint frequency

7. Joint roughness

The present study aims to link between the ratios of intact and joint rock mass strength with factor Jf, Jfg and physical simulation of strength of cemented rock joints.

2.0 LITERATURE REVIEW

The various forces acting in the eco-system make the rock in its geological, a regulated discontinuous because of joints, cracks, cavities, fissures, schistosity. behaviour of network of fissures, bedding planes, joints with or without fill is as important as or even more vital with regard to rock properties than the mineralogical composition itself. They create non-linear load deformation response, especially at low stress level reduces tensile strength: create stress dependency in material properties and produce variability and scatter on the tests results.

A better definition of rock may now be given as granular, allotropic, heterogeneous technical substance which occurs naturally and which is composed of grains, cemented together with glue or by a mechanical bond, but ultimately by atomic, ionic and molecular bond within the grains. Thus by rock an engineer means a firm and coherent substance which normally cannot be excavated by normal methods alone. Thus like any other material a rock is frequently assumed to be homogenous and isotropic but in most cases it is not so .

Rock is a discontinuous medium with fissures, fractures, joints, bedding planes, and faults. These discontinuities may exist with or without gouge material. The strength of rock masses depend on the behavior of these discontinuities or planes of weakness. The frequency of joints, their orientation with respect to the engineering structures, and the roughness of the joint have a significant importance from the stability point of view. The characterization of the strength and deformation behavior of jointed rocks is significant for safe design of civil structures such as arch dams, bridge piers and tunnels. The properties of the intact rock between the discontinuities and the properties of the joints themselves can be determined in the laboratory where as the direct physical measurements of the properties of the rock mass are very expensive. For determining the rock mass properties indirectly, a theory needs to be established and tested in some independent way. A number of experimental studies have been conducted both in field and in the laboratory to understand the behavior of natural as well as artificial

joints. In situ tests have also been carried out to study the effect of size on rock mass compressive strength. Artificial joints have been studied mainly as they have the advantage of being reproducible. The anisotropic strength behavior of shale, slates, and phyllites has been investigated by a large number of investigators. Laboratory studies show that many different failure modes are possible .In jointed rock and that the internal distribution of stresses within a jointed rock mass can be highly complex. Due to large expense and time involved in experimental studies, coupled with the need for highly accurate measurement techniques, a number of investigators attempted to study the behavior of joints using analytical models. To model the highly complex behavior of jointed rock masses, the strength and deformability of jointed rock masses should be expressed as a function of joint orientation, joint size, and frequency. Moreover it is not possible to represent each and every joint individually in a constitutive model. Thus there is a need for a simple technique such as the equivalent continuum method which can capture reasonably the behavior of jointed rock mass using minimum input. The method presented in this paper recognizes that the rock will act both as an elastic material and a discontinuous mass. Considering the inherently inhomogeneous nature of rock masses, this approach attempts to obtain statistical relationships from the analysis of a large set of experimental data of jointed rock mass [41]

The anisotropic behaviour of the rock required analysis whenever the engineers faced the problems concerning.

1. The foundation
2. The Underground work
3. The stability of excavation in rocks
4. Rock indictment to protect the local natural
   Environment.

The Successful solution of the above mentioned problems frequently demand the evaluation of two important design parameters; shear strength and deformability. A reliable estimate of these parameters is much important so that sophisticated design tools can be meaningfully applied to the presence of micro and macro discontinuities their after deposition of fill in most of the rock render them non ideal. Regular cracks and fissures are generally found at shallow depth beneath the surface and some even present at the depth of thousand meters. Sometimes Gouge material entrapped changes it behaviour beyond ordinary comprehension.

2.1 Factors affecting the strength.

An intact rock is considered to be an aggregate of mineral, without any structural defects and also such rocks are treated as isotropic, homogeneous and continuous. Their failures can be classified as brittle which implies a sudden reduction in strength when a limited stress level is exceeded.

Strength of intact rock mass is mainly influenced by the following factors:

Geological	➢ Geological age, weathering and other alternatives
Litho logical	➢ **Cementing material, mineral composition,, texture and fabric, anisotropy**.
Physical	➢ Density/specific gravity, void index, porosity
Mechanical	➢ Specimen preparation, geometry, end contact/ end restraint, type of testing machine, plate of loading
Environmental	➢ Moisture content, nature of pore fluids, temperature, confining pressure.

The shear strength of jointed rock mass depends on the type and nature of origin of the discontinuity, roughness, depth of weathering, and type of filling material. The strength behaviour of rock mass is governed by both intact rocks. Properties and properties of discontinuities. The shear strength of rock mass depends on several factors like:

Table 2.1 Factor affecting shear strength

1	Strength along the joint
2	Degree of joint separation
3	Opening of the joint
4	Number of joints in a given direction
5	Angle made by the joint with the principal stress direction (β)
6	Joint frequency and joint roughness
7	Joint cementing material filling

2.1.1 Effect of moisture

Investigator tested the number of joints with clay fillers. The shear plane is limited along the weakest contact and traverses through clay. The influence of clay bands is very much affected by the presence of humidity. The drop in shear strength is observed with increases in humidity even under the conditions that the clay band is not squeezed out. With further increases in humidity to a stage clay become plastic and starts getting extruded out. The joint slowly closes and the two surface of rock come in contact. The critical shear strength change starts at 25% moisture content of clay and at 52% moisture content of the clay band is completely extruded out[40]

A thin layer of sand as filler between the hard rock's (sand stone lime stone) does not have any significant influence but in case of relatively weak rock like clay and marl its influence is rather to increases the angle of friction. The shear plane is a thick layered of sand is limited in the sand bed itself. It is independent of smoothness or roughness of the joint. The coefficient of friction increases with the increases in

the fragment size from 2mm to 20-30 mm. And further increases do not exert any influence on the friction coefficient.

2.1.2 Effect of size and composition of the filler:

When crushed stone with clay is present as filling material, the shear resistance is mainly determined by the humidity of clay component. With hard dry clay, the crushed stone has almost no influence and the coefficient of friction is that of clay. At semi- hard semi- plastic consistency, the shear strength goes up with increases with in fragment content percentage from 20-30% up to 90% At fully plastic consistency the fragmentary material affects very much the shear strengths only at the fragment content from 60-70% upwards. The range of values of residual angle of friction for a variety of clays and clay mixture is given below in Fig. 2.1 Residual friction angle is determined assuming thickness of joint wall asperities [34].

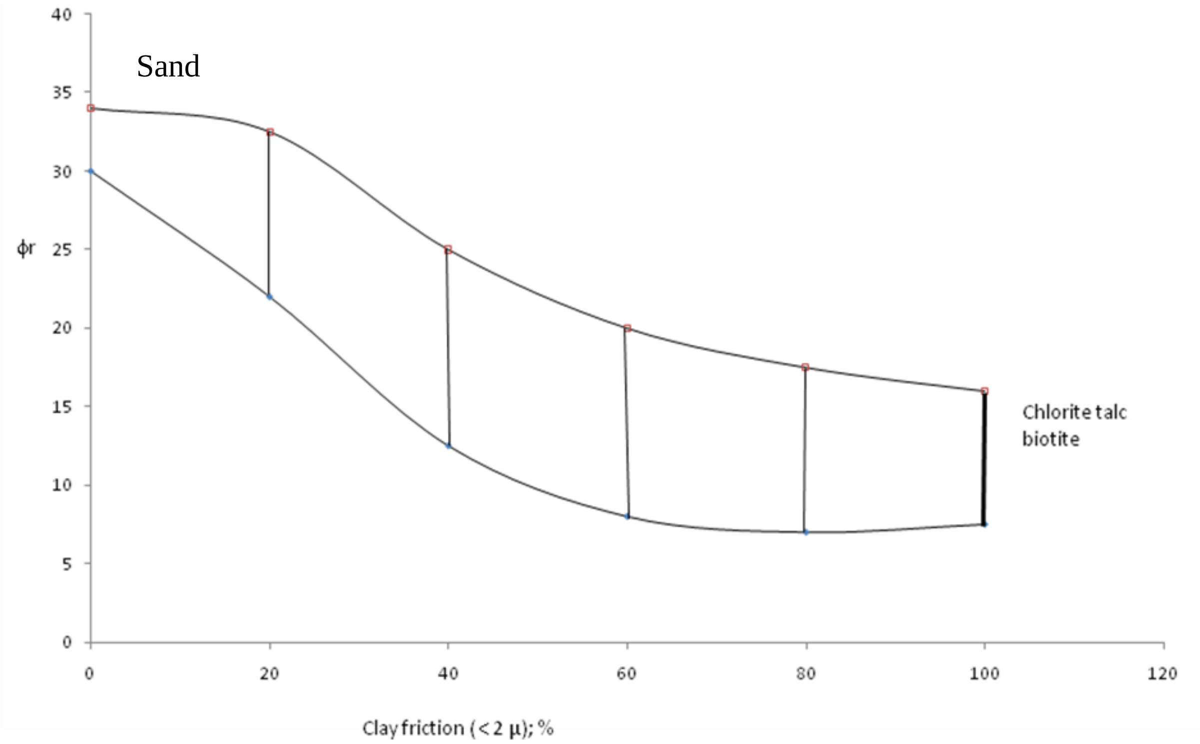


Fig 2.1 Dependence of residual shear strength on clay friction [41].

2.1.3 Effect of confining pressure:

E.H. Rutter [28] studied the behaviour of oven dry kaolinite at various thickness in 30^0 saw cut dolerite at 200 MPa confining pressure. Without clay film it exhibits a violent stick slip.

The decrease in strength with increases in thickness of kaolinite Gouge together with microscopic observation shows that a substantial part of applied displacement is distributed through the clay volume. These data show that slow, stable creep of fault with wide gouge may occur at low stress level (10 MPa) Weakening effect of water and development of pore pressure will accentuate this tendency. Stick-slip as observed in non swelling clays by Summers and Byearlee [31] are probably associated with occasional localization of deformation on to a single slip plane within the gouge zone or at its boundaries.

2.1.4 Rock discontinuities

Faults, joints, bedding planes, fractures, fissures and micro fissures are of wide occurrence in rocks encountered in engineering practice. Characteristics of these discontinuities play a major role in controlling the engineering behavior of rock masses. Following are the discontinuity characteristics for the rock mass. (a) Nature of their occurrence (b) orientation and position in space (c) continuity (d) intensity (e) surface geometry (f) genetic type and (g) nature and thickness of joint-fill. In every engineering situation knowledge of these characteristics is required.

It is also important to obtain data on discontinuity intensity in addition to discontinuity orientation and condition of discontinuity.

2.1.4.1 Discontinuity intensity

Index adopted to describe discontinuity intensity is influenced by nature of exposure and the survey technique. Investigator described discontinuity intensity in London clay in terms of the number of discontinuities per unit volume of material. They used scan-line survey-technique on rock face and expressed discontinuity intensity as number of discontinuities per unit distance normal to the strike of a set of sub-parallel discontinuities. Deere [7] proposed the method of RQD i.e. Rock quality designation. RQD incorporates sound pieces of bore-hole relation between RQD and mean discontinuity frequency shown in Fig [2.2]

$RQD = -3.68\lambda + 110.4$ for $6<\lambda<16$

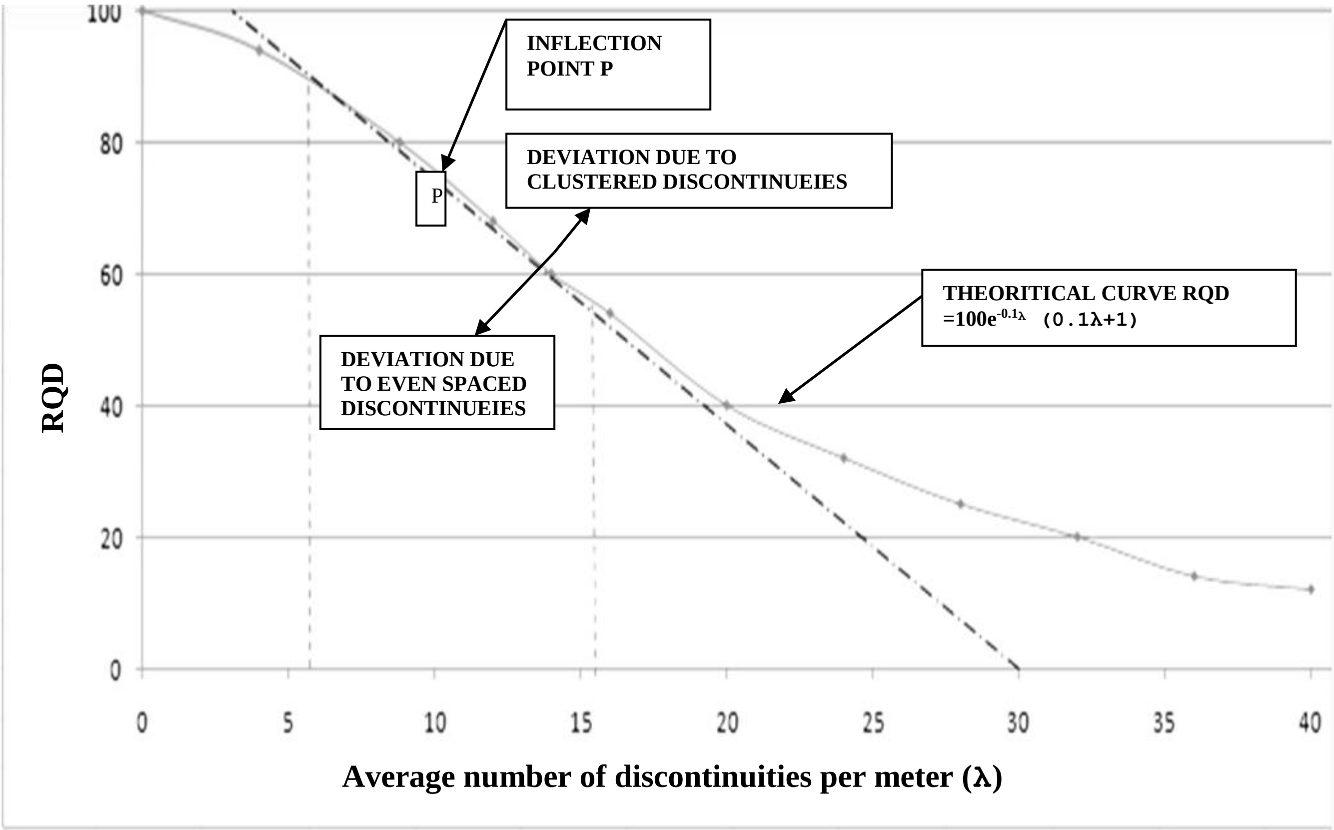


Fig: 2.2: Relation between RQD and discontinuity frequency [26]

Core that are 10cm or greater in length. This is calculated by summing up these intact lengths and is expressed as a percentage of the total length of the run i.e. length drilled.

$$RQD = 100/L \sum_{i}^{n} (c_i) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.1)$$

Where, xi = Length of the 1th length ≥ 10cm,

n= number of intact lengths ≥ 10 cm, and

L = length of bore hole or scan line along which the RQD is required.

Priest and Hudson [26] presented a theoretical approach to the discontinuity spacing and RQD based on statistical distribution of spacing values that could occur along scan line and compared these results with experimental data obtained in the field discontinuity survey. They considered all possible distribution of discontinuity spacing along a straight line through a rock mass. The effect of evenly spaced clustered and randomly positioned discontinuities were examined theoretically and found a negative exponential form of frequency versus spacing curve. They also analyzed the field results and found that negative exponential distribution holds good for true distribution pattern of discontinuities.

On the basic of negative exponential distribution of discontinuity spacing values, Priest and Hudson [26] established. Following relation between the theoretical ROD and the average number of discontinuities per meter.

$$RQD = 100\, e^{(-0.1\lambda)}\,(0.1\lambda+1) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.2)$$

They found a good agreement between the measured and theoretical values of RQD. Fig. 2.2 shows a graph of equation 2.2 relating RQD and average number of discontinuities per meter. Along with the experimental data for the value of λ between λ= 6 per meter and λ = 16 per meter. The relation between and RQD is approximately linear and can be expressed as

$$RQD = -3.68\lambda + 110.4 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.3)$$

Number of sets of joints will decrease the RQD and will increase the joint frequency i.e. number of joints per meter.

2.2 Significant rock jointed properties

The strength of the rock mass is only a fraction of the strength of the intact strength. The reason for this is that failure in the rock mass is a combination of both intact rock strength and separation or sliding along discontinuities. The latter process usually dominates. Sliding on discontinuities occurs against the cohesion and/or frictional resistance along the discontinuity. The cohesion component is only a very small fraction of the cohesion of the intact rock. An important aspect of rock behavior under the uniaxial condition is the change in behavior from brittle to ductile nature at high confining pressure.

2.2.1 Joint intensity:

The joint intensity is the number of joints per unit distance normal to the plane of joints in a set. It influences the stress behavior of rock mass significantly, strength of rock decreases as the number of joints increases this has been well established on the basis of studies by Lama [23]

To understand the strength behavior of jointed rock specimen, Arora [1] introduced a factor (Jf) defined by the expression as

Jf =Jn /n * r

The joint factor (Jf) is defined as a ratio of joint frequency (Jn), to the product of joint orientation parameter (n) and joint strength parameter (r)

Jn = Number of joints in the direction of loading (equal to number of joints per meter length of the sample)

n =Orientation parameter related to inclination of joints (β) with the direction of major principal stress

r = Joint strength parameter depending on the joint condition (ϕ_j), which is equivalent friction angle along the joint plane so that the roughness of the surface is represented through this value (it is obtained by a shear test on the rock joint)

2.2.2 Effect of gouge on joint factor

In the presence of gouge, the strength ratio followed a relationship with joint factor, which needed modification for depth of joints from loading plane and thickness of the gouge material. Trivedi [40] analyzed the results of the uniaxial compression tests conducted on jointed Kota sand stone with and without gouge as shown in Fig. 2.3

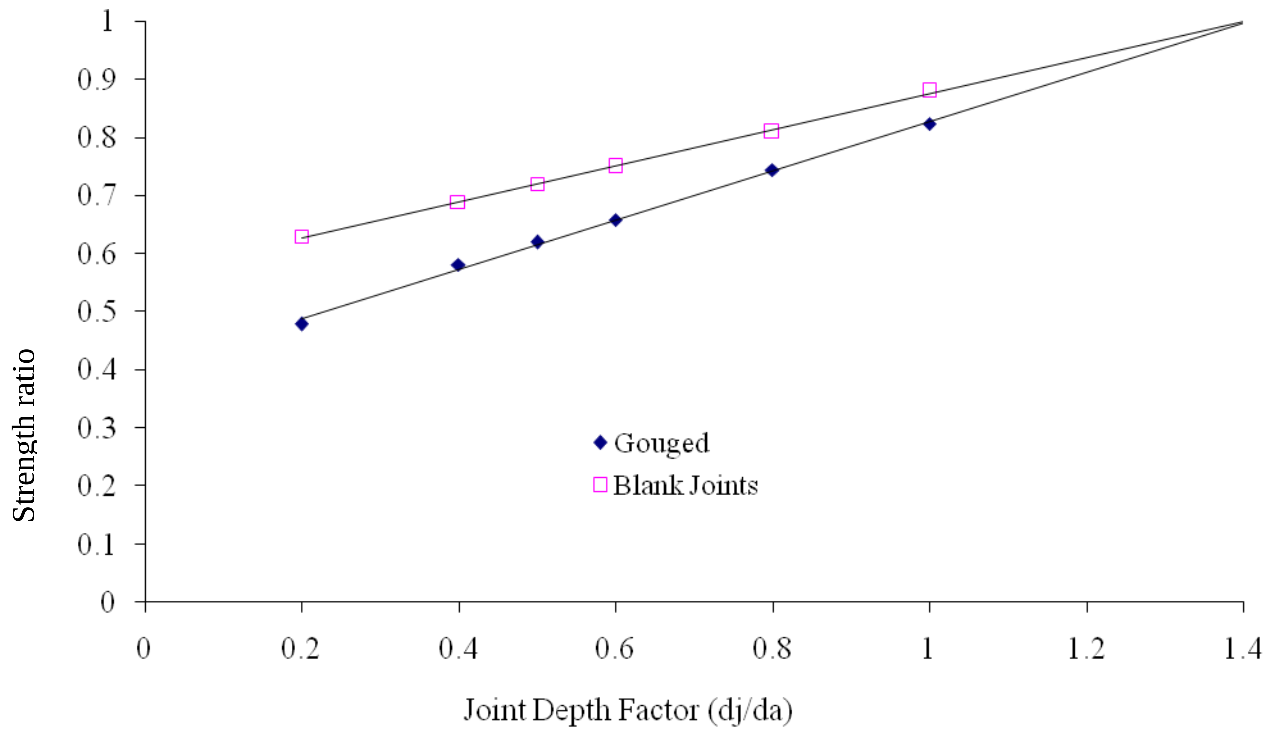


Fig.2.3 Variation of strength ratio with joint depth factor [40].

. It shows that the strength ratio varies according to the joint depth factor (dj/da) with respect to the loading plane, where dj is depth of joint (mm) from the loading plane and da is the reference depth = diameter of the specimen (mm). There is a linear reduction in the strength of jointed rocks with the proximity of joints to the loading surface. The presence of clayey gouge tends to produce further reduction in the strength. However, if the distance of joints is at a depth more than a value of a non-dimensional joint depth parameter (Jdj), it does not affect the strength ratio anymore.

The Trivedi [40] introduced a non-dimensional joint depth parameter (Jdj) as a multiplication factor for the joint factor (Jf).

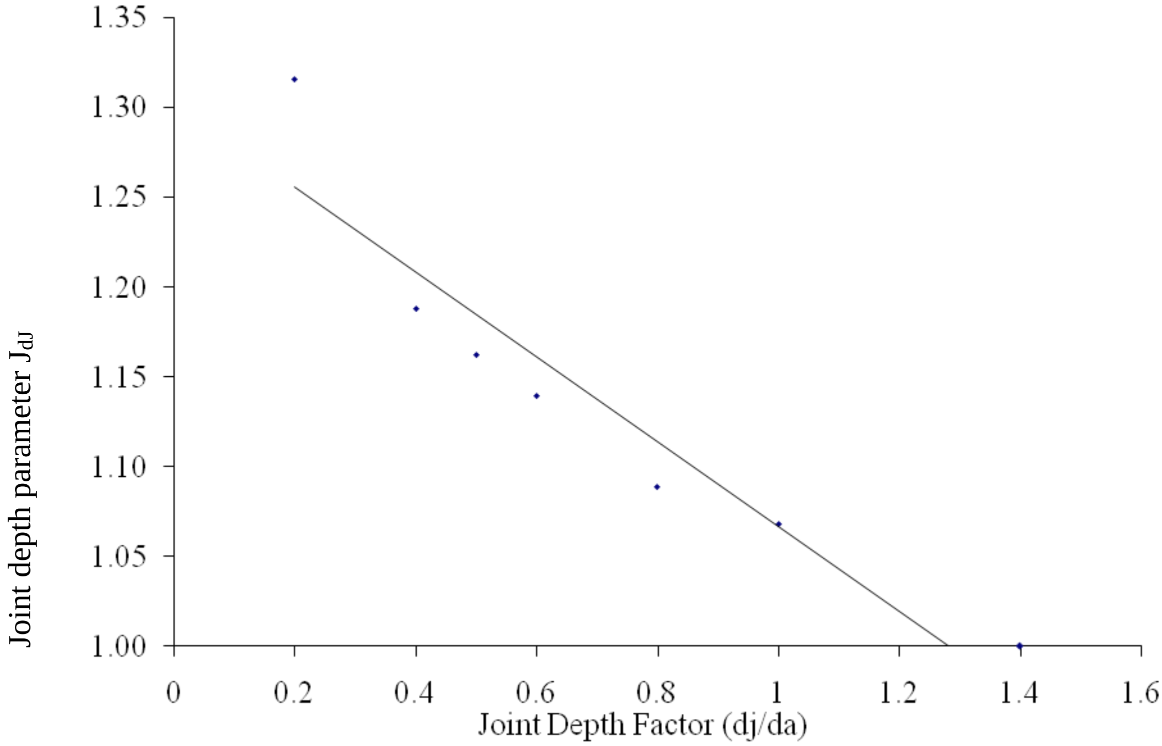


Fig 2.4 Relationship of joint depth parameter with joint depth factor [40]

Figure 2.4 shows that the values of joint depth parameter (Jdj) may always be more than one. For joints Located at a same depth relative to the mid height of the sample, its value is taken as unity. Further, it does not remain a relevant factor for joints located at significant depths as the case may arise frequently in the field. The analysis of the tests results on jointed Kota sand stone with varying gouge thickness (t),which indicated that increasing thickness (up to 3 mm) reduces strength of the jointed rocks. The increase in the thickness of gouge (beyond 3 mm) decreases the strength ratio to an extent when strength of jointed rock reaches the residual strength of multi-fractured rock mass (σ_{cr}<35%).

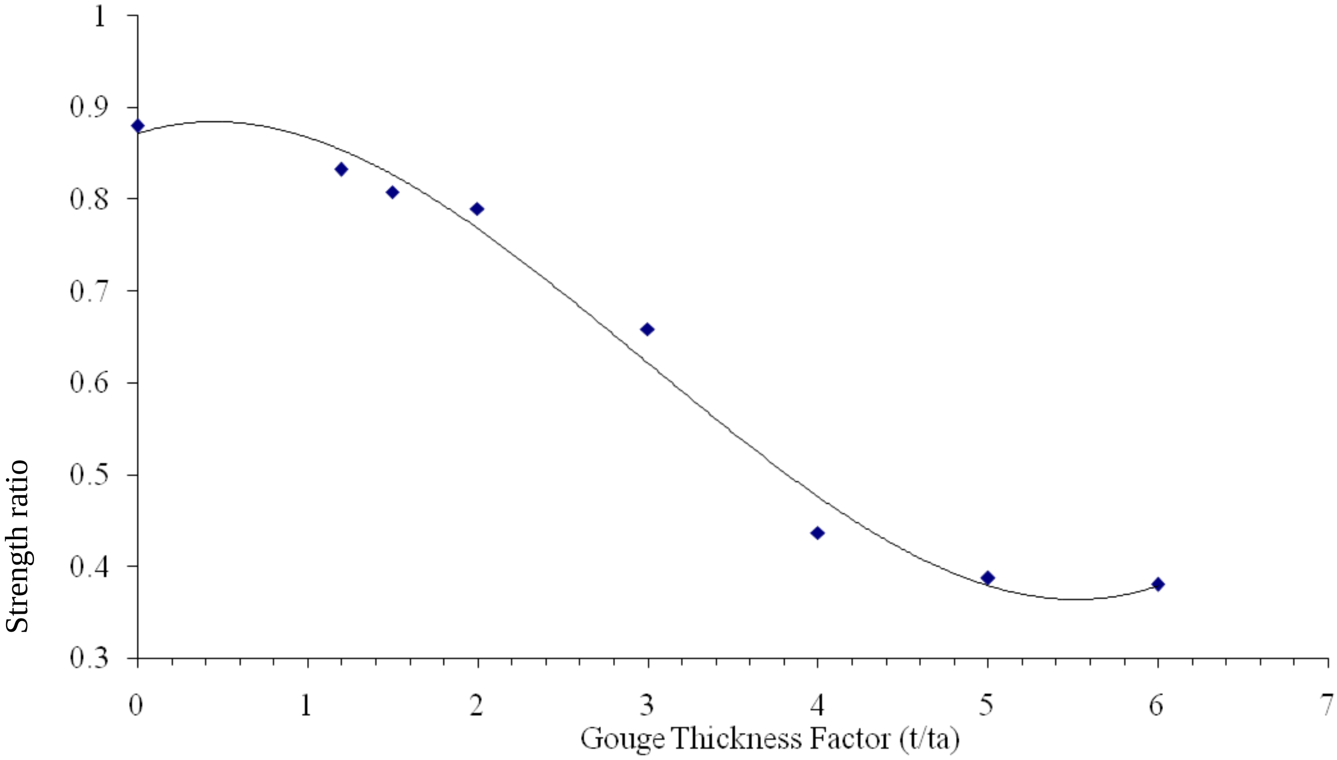


Fig.2.5 Relationship of strength ratio with gouge thickness factor [40]

Figure 2.5 shows the variation of strength ratio with gouge thickness factor (t/ta). It was observed that initial increase in the gouge thickness factor (<2) the strength reduction in horizontally jointed samples is insignificant. This thickness uses packing of gaps in asperities on compression accompanied

by initial plastic deformations. However, further increase in thickness of gouge material (t/ta > 5), the strength drop is exponential as long as any further increase in thickness (t/ta >5), the strength of jointed rock reaches

Residual value. Trivedi [40] introduces a gouge thickness parameter (Jt) to incorporate the effect of thickness in non dimensional form as shown in Fig. 2.6 below.

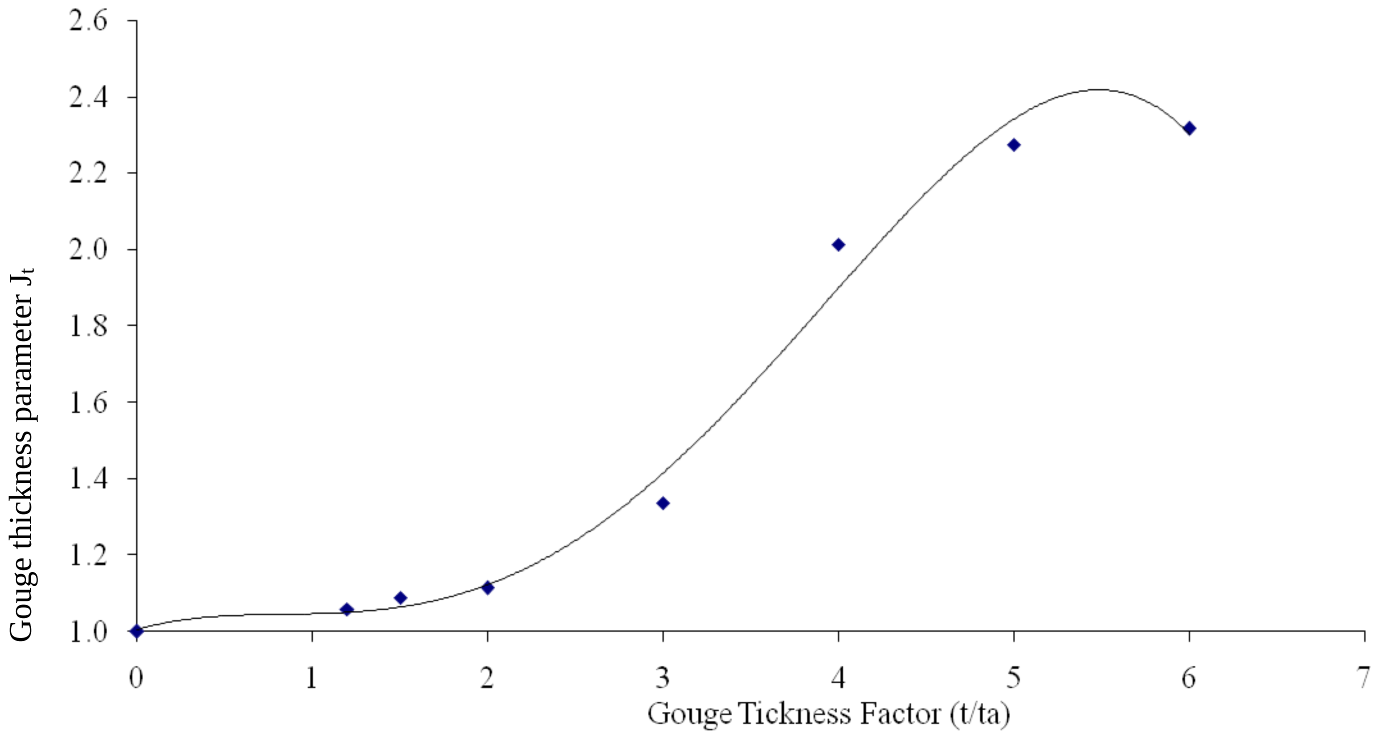


Fig.2.6 Relationship of gouge thickness parameter with gouge thickness factor [40].

The gouge thickness parameter (J_t) varies as a function of gouge thickness factor (t/ta). Beyond a value of gouge thickness parameter (J_t), the strength drop is observed at a lower rate compared to an initial drop.

Based on the results [2, 34, 35], Trivedi [40] modified the joint factor (J_f) in terms of non-dimensional quantities as (Jfg) given in Eq. 2.4

$J_{fg}=c_g(J_n.L_{na}/nr)$…………………………………………………..(2.4)

where cg is a modification factor for gouge,
$c_g = J_{df}.J_t/g_d$ …………………………………………………….(2.5)

J_{dj} Correction for the depth of joint (joint depth parameter)

J_t Correction for the thickness of gouge in joint (gouge thickness parameter)

g_d Correction factor depending upon the compactness or relative density of gouge in joint, equal to unity for fully compacted joint fill. For clean compact joints, when no gouge is present cg is equal to unity,

J_n Number of joints per unit length in the direction of loading (joints per metre length of the sample),

L_{na} reference length = 1 m

The analysis of the experimental observation of strength ratio of jointed rocks with and without gouge with a large number of horizontal joints indicate that a scheme of corrections may converge the effect of gouge of parent rock Material with blank joints upon mutual closure of the joints on compression

2.2.3 Orientation of joints: The orientation of joints is one of the most important parameters which influence the resultant shear stress distribution along with nature and extent of failure zones.

On the basis of Mohr Coulomb equation, Jaegar and Cook [19] reported the criteria for slip in the single weak plane. They developed the following expression to show the variation of deviator stress ($\sigma_1 - \sigma_3$) necessary to cause the failure with the variation of joint β with σ_3 and ϕ kept fixed.

$(\sigma_1 - \sigma_3) = (2c + 2\tan\phi) / \{(1 - \tan\phi.\cot\beta) \sin 2\beta\}$………………… (2.6)

Table 2.2 the value of inclination parameter [40]

Orientation of joint $\beta°$	Inclination parameter n
0	0.810
10	0.460
20	0.105
30	0.046
40	0.071
50	0.306
60	0.465
70	0.634
80	0.814
90	1.00

2.2.4 Joint roughness:

Joint roughness is of paramount importance to the shear behavior of rock joints .this is because joint roughness has a fundamental influence on the development of dilation and as a consequence the strength of joint during relative shears displacement. When a fractured rock surface is viewed under a magnification the profile exhibits a random arrangement of peaks and valleys called asperities forming a rough surface. The surface roughness is owing to asperities with short spacing and height.

Following is the equation for friction angle (ϕ_e) along the joints

$\phi_e = \phi_u + i$ ………………………………………………………...... (2.7)

Where

ϕ_u is the friction angle of smooth joint

i is the inclination of asperity

Joint roughness has been considered as a parameter that effectively increases the friction angle Φr which is given by the relation below

$\tau = \sigma_n \tan(\phi_r + i)$ for small values σ_n ……………………………........(2.8)

$\tau = c + \sigma_n \tan\phi_r$ for large values of σ_n …………………… …. ....(2.9)

Where τ = Peak shear strength of the joint.

σ_n=normal stress on the joint

ϕ_r = Residual friction angle

Typically for rock joints the value of I is not but gradually decreases with increasing shear displacement. The variation in I is due to the random and irregular surface geometry of natural rock joints the finite strength of the rock and the interplay between surface sliding and asperity shear mechanism.

For computing shear strength along the sliding joint Barton [4] suggested the following relationship

$\tau/\sigma_n = \tan[(90 - \phi_u(d_n/\phi_u) + \phi_u]$ ……………………...........................(2.10)

Where

d_n is the peak dilation angle which is almost equal to 10 log10(σ_c/σ_n)

σ_c is the uniaxial compressive strength

2.2.5 Joint roughness coefficient

The empirical approach proposed by Barton [4] is most widely used. They expressed roughness in terms of a joint roughness coefficient that could be determined either by tilt, push or pull test on rock samples or by visual comparison with a set of roughness profile.

The joint roughness coefficient (JRC) represents a sliding scale of roughness which varies from approximately 20 to zero from roughest to smoothest surface respectively.

Table 2.3: Suggested values for r based on uniaxial compressive strength

Uniaxial-Compressive strength of Intact Rock* (σ_{ci}) MPa	Joint strength parameter *(r)	Uniaxial-Compressive strength of Intact Rock** (σ_{cj}) MPa
2.5	0.30	1.77
5	0.45	3.97
15	0.60	12.61
25	0.70	21.55
45	0.80	39.51
65	0.90	57.91
100	1.00	90.12

* Arora[1], Ramamurti and Arora[27], ** Trivedi [40]

Table 2.4: Suggested values for fitting parameter for r based on uniaxial Compressive strength

Fitting parameter	Intact rock*		Jointed rock**	
	a_{ci}	b_{ci}	a_{cj}	b_{cj}
Empirical Values	0.182	0.130	0.171	0.192
R^2	0.990		0.991	

* Arora[1], Ramamurti and Arora[27], ** Trivedi [40]

$r_{ci} = a_{ci} \ln(\sigma_{ci}/\sigma_a) + b_{ci}$……………………………………..(2.11)

$r_{cj} = a_{cj} \ln(\sigma_{cj}/\sigma_a) + b_{cj}$……………………………………..(2.12)

Where

σ_{cj} = Uniaxial compressive strength of jointed rock in kPa

σ_a =Reference Pressure, a and b are fitting constants given in Table (2.4)

The Eq. 2.12 calls for a necessity of adjustments for joint strength parameter with a consideration of logarithmic of pressure on the joint material. Keeping the same values of joint number and orientation, if the joint strength parameter

is adopted as per Eq. 2.12 RAC [27] and a fixed r ($\phi_j = \pi/4; \tan \phi_j = 1$), respectively.

This observation supports the effect of reduction of joint strength parameter leading to increasing joint factor and consequent strength reduction. In Hoek and Brown criterion [10, 11], the increasing confinements have similar effect of a non-linear strength reduction. There had been varied interpretation of joint factor particularly in relation to joint strength parameter. Various investigators [1, 18, 27, 32, and 34] considered it as a constant friction factor independent of dilatancy. In the present framework, we correlated the resultant friction due to the joints with the dilatancy of the joint material.

2.2.6 Scale effects

The strength of the rock material decreases with increase of the volume of test specimen. This property is called scale effect can also be observed in soft rock.

Investigator did experimental studies of scale effects on the shear behavior of rock joints by performing direct shear test on different sized replicas cast from various natural joint surfaces. Their results show significant scale effects on shear strength and deformation characteristic. Scale effects are more pronounced in case of rough, undulating joint types, where they are virtually seen absent for plane joints. There result showed that both the JRC and JCS reduced to the changing stiffness of rock mass the block size or joint spacing increases or decreases to overcome the effects of size they suggested tilt or pull tests on singly jointed naturally occurring blocks of length equal to mean joint spacing to derive almost scale free estimates of JRC as

$JRC = \alpha - \phi_r / \log(JCS/\sigma_{no})$

Where

α=tilt angle

σ_{no}=Normal stress when sliding occurs

2.2.7 Dilation

Dilation is the relative moment between two joint faces along the profiles.

Peak dilation angle of joints was predicted by Barton [4] based on the roughness component which includes mobilized angle of internal friction and JRC, residual friction angle and normal stress.

Barton [4] predicted that dilation begins when roughness is mobilized and dilation declines as roughness reduces.

2.2.8 Influence of single plan weakness:

In a laboratory test the orientation of plane of weakness with respect to Principal stress direction remains unaltered. Variation of the orientation of this plane can be achieved by obtaining cores in different directions. In a field situation either in foundations of dam around underground or open excavation the orientation of joint system remains stationary but the direction of principal stress rotates resulting in a change in the strength of rock mass.

2.2.9 Studies on planer joints:

In the present study, plaster of Paris specimen will be prepared to have the Joint plane `at desired orientation using matching metal casting to obtain joint plane with possible limit of tolerance. For sand stone and granite, the specimen should cut along the desired direction.

2.3 Strength criteria for rock

Unlike isotropic rocks, the strength criterion for anisotropic rocks is more complicated because of the variation in the orientation angle β. A number of empirical strength criteria have been proposed in the past by Griffith[9].An idealized cylindrical specimen of anisotropic rock with an oblique plane of weakness makes an angle β. The angle β is designated as the orientation angle. Hock and Brown [10] showed clearly the strength of all rocks is maximum at $\beta = 0^o$ to 90^o and is minimum for $\beta = 20^o$ to 30^o.

Using the non linear failure envelopes predicted by classical Griffith [9] theory for plane compression and through a process of trial and error, Hock and Brown [10] presented an empirical failure criterion applicable for both isotropic and anisotropic rock.

$s_1 = s_3 + m\ s_c.\ s_3 + s.\ s_c$ ……………………………..............(2.13)

Where

s= 1 for intact rock and, 0 for crushed rock

m varies widely as a function of rock quality and type.

Ramamurthy [27] proposed an empirical strength criterion to account for the non-linear strength response of isotropic intact rocks in the following form:

$(s_1 - s_3)/\ \alpha 3 = Bi\ (s_{ci}/\ s_3)^{\alpha i}$ …………………………………....(2.14)

Where

s_{ci} is the uniaxial compressive strength of intact rock without a weak plane, s_1 and $s3$ are plane stresses, si is the slope of plot between $(s_1 - s_3)/\ s_3$ and $(s_{ci}/\ s_3)$ on the log-log scale and Bi $=(s_1 - s_3)/\ s_3$ and $(s_{ci}/\ s_3) = 1$, s_i and Bi are considered as strength parameters. He had suggested a constant value of 0.8 for s_i at all orientations even for intact anisotropic rocks. Owing to the fact that Bi parameter did not very much in their analysis, a constant value for Bi as well could have assumed. The variation in the value of Bi was calculated corresponding to a constant average value of $s_1 = 0.8$. Ramamurthy et al. on the basis of the results obtained from the triaxial compressive strength on three anisotropic rocks viz. quartzite, carbonaceous and micaceous hyalites and plots between log and log for different orientations, have concluded that even for intact anisotropic rocks, the strength parameters denoted by s_j and B_i cannot be taken as constants and these parameters showed systematic variation with of anisotropic rock and orientation angle refers to intact rock without weak plane and j with weak planes.

In order to predict the strength of anisotropic or jointed rock from the

Proposed criterion as $(s_1 - s_3)/\ s_3 = B_i\ (s_{ci}/\ s_3)^{\alpha i}$

2.4 Uniaxial compressive strength ratio

The uniaxial compressive strength ratio of a rock mass is represented in a non

Dimensional form as the ratio of the compressive strength of jointed rock to that of intact rock. The uniaxial compressive strength ratio is expressed as

$s_{cr} = s_{cj} / s_{ci}$ ……………………………………………….................(2.15)

Where

s_{cj} = uniaxial compressive strength of jointed rock and s_{ci} uniaxial strength of intact rock.

The uniaxial compressive strength ratio of the experimental data is plotted against the joint factor. The joint factor for the experimental specimens is estimated based on the joint orientation, joint strength. Based on the statistical analysis of the data, empirical relationships for the uniaxial compressive strength ratio as a function of joint factor (J_f) are derived.

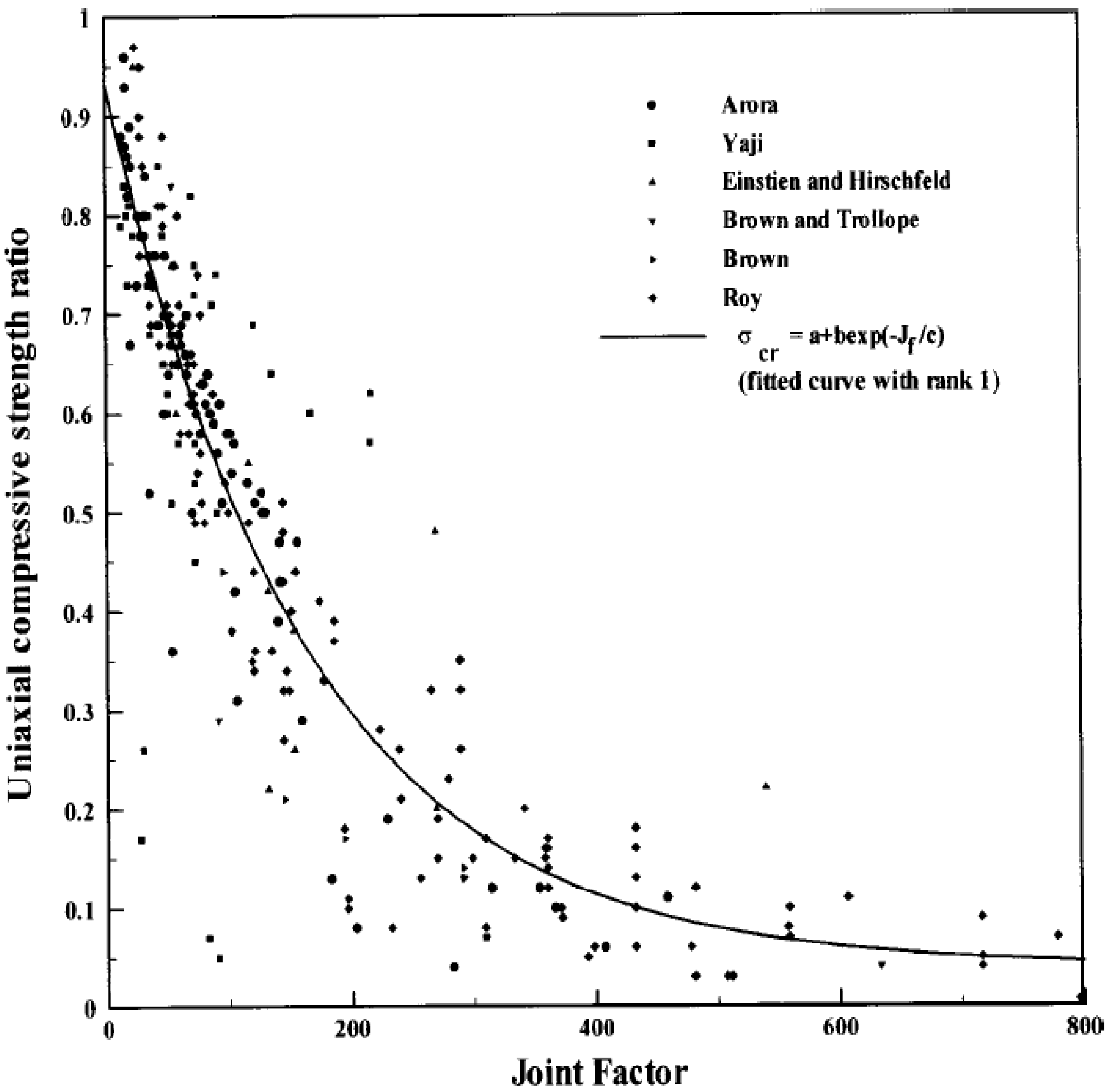


Fig: 2.7 Uniaxial compressive strength vs joint factor [18]

2.5. Elastic modulus

Elastic modulus expressed as tangent modulus at 50% of the failure stress is Considered in this analysis. The elastic modulus ratio is expressed as:

Er= Ej/ Ei…………………………………………………….(2.16)

Where

Ej=tangent modulus of the jointed rock

Ei=tangent modulus of the intact rock

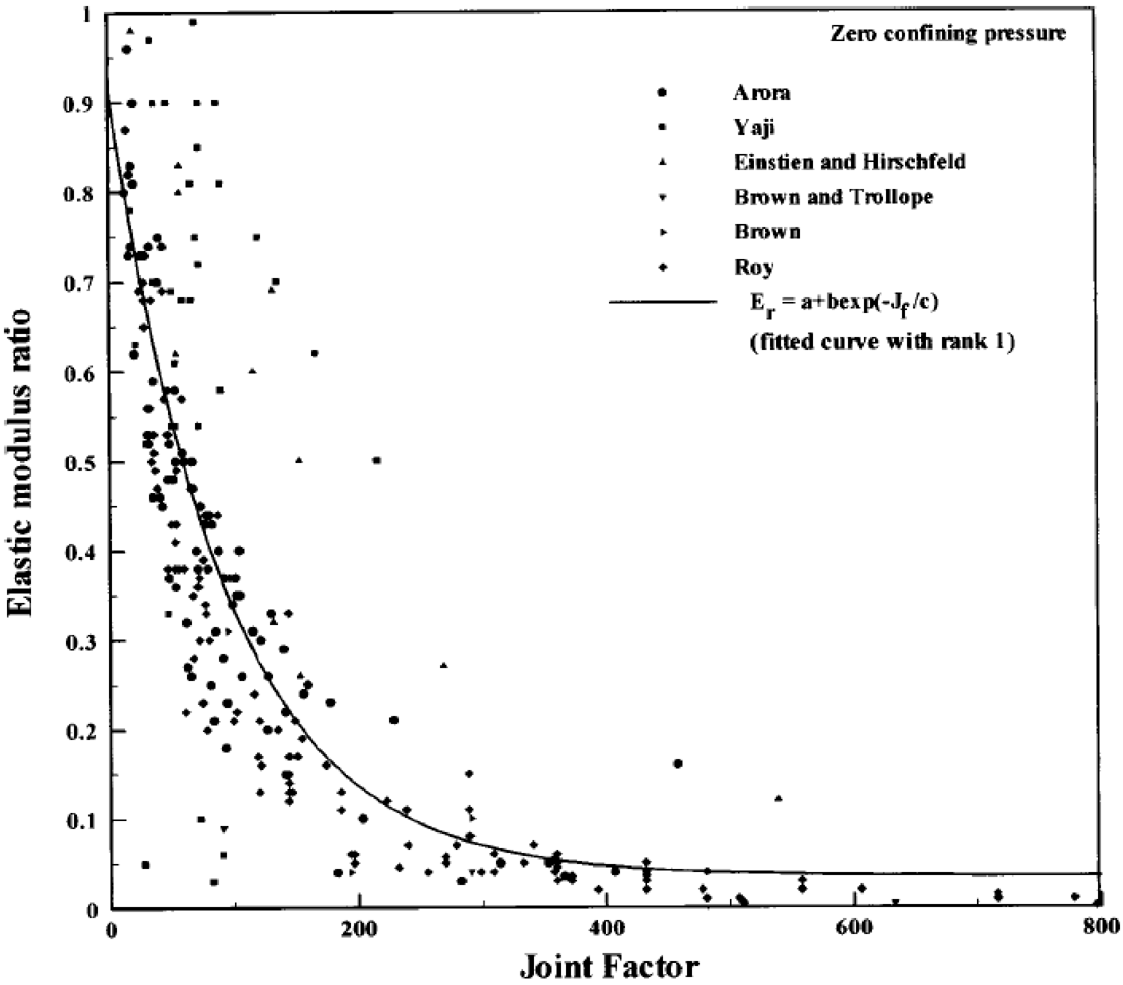


Fig: 2.8 Elastic modulus ratio vs joint factor [18]

2.6 Failure modes in rocks:

The failure modes were identified based on the visual observations at the time of failure. The failure modes obtained are:

- Splitting of intact material of the elemental blocks,
- Shearing of intact block material,
- Rotation of the blocks, and
- Sliding along the critical joints.

These modes were observed to depend on the combination of orientation h and the stepping. The angle β in this study represents the angle between the normal to the joint plane and the loading direction, whereas the stepping represents the level/extent of interlocking of the mass. The following observations were made on the effect of the orientation of the joints and their interlocking on the failure modes. These observations may be used as rough guidelines to assess the probable modes of failure under a uniaxial loading condition in the field.

2.6.1 Splitting

Material fails due to tensile stresses developed inside the elemental blocks. The cracks are roughly vertical with no sign of shearing. The specimen fails in this mode when joints are either horizontal or vertical and are tightly interlocked due to stepping.

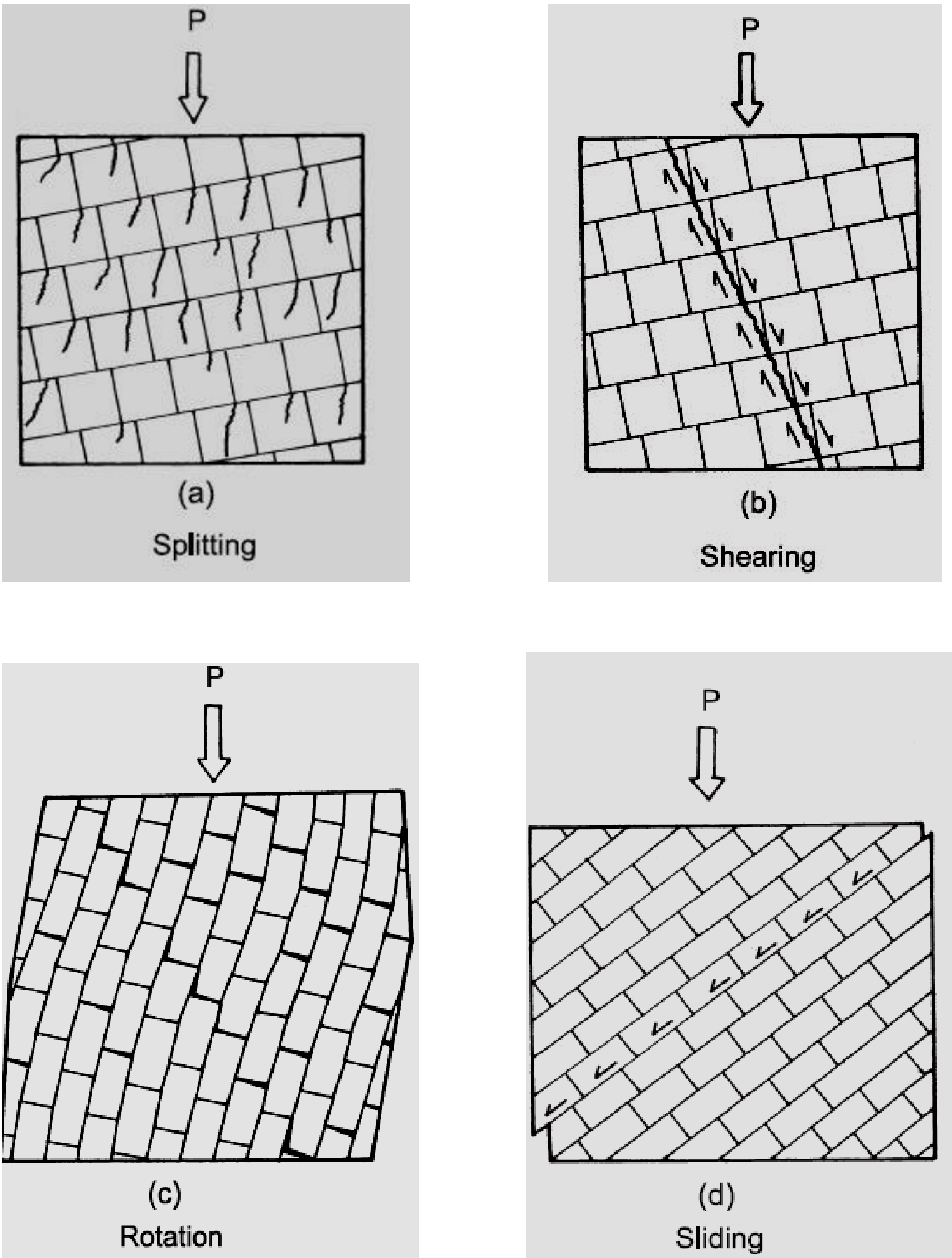


Fig 2.9 Failure modes in rocks [32]

2.6.2 Shearing

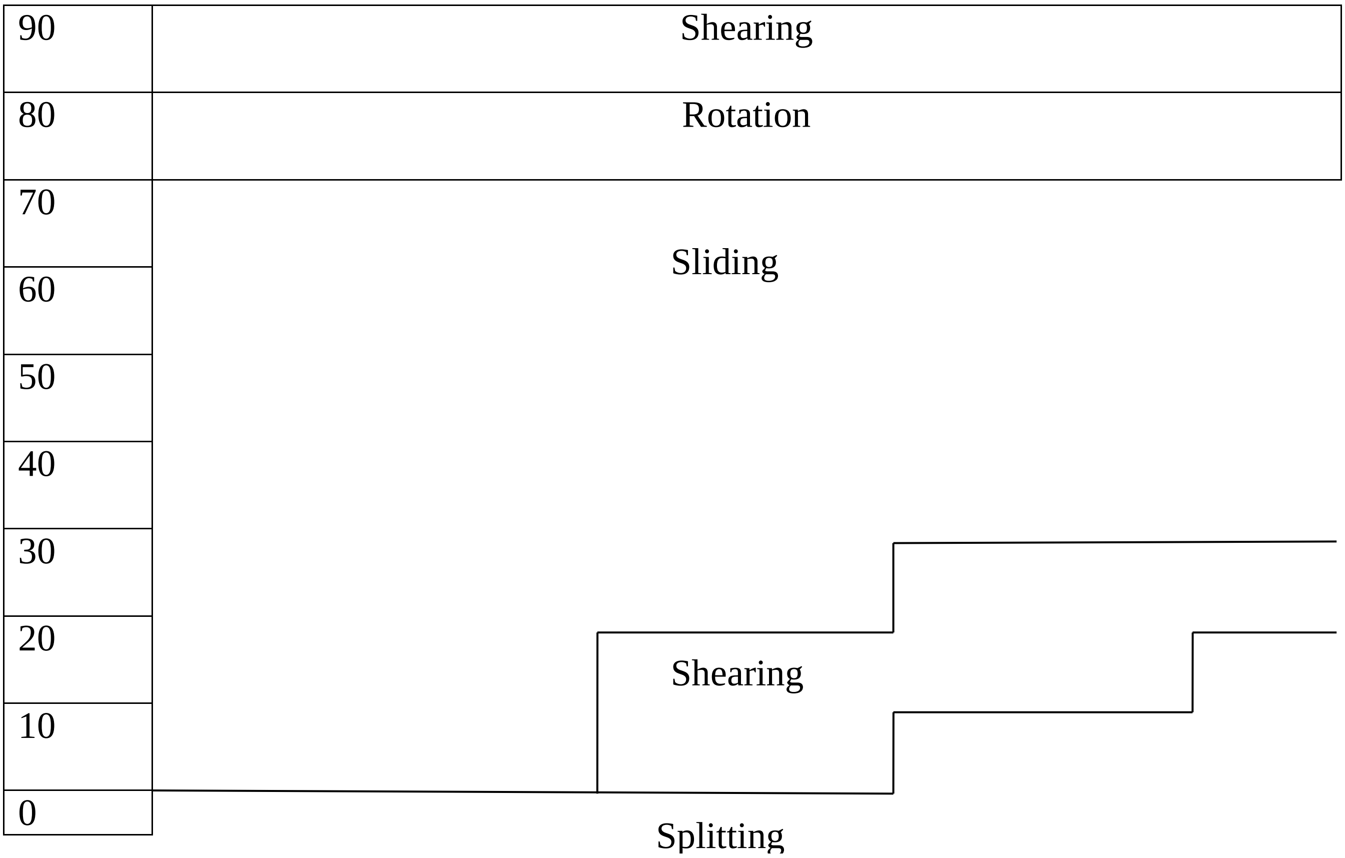
In this category, the specimen fails due to shearing of the elemental block material. Failure planes are inclined and are marked with signs of placements and formation of fractured material along the sheared zones. This failure mode occurs when the continuous joints are close to horizontal (i.e., θ<= 10) and the mass is moderately interlocked. As the angle h increases, the tendency to fail in shearing reduces, and sliding takes place. For θ≈ 30. Shearing occurs only if the mass is highly interlocked due to stepping.

2.6.3 Sliding

The specimen fails due to sliding on the continuous joints. The mode is Associated with large deformations, stick–slip phenomenon, and poorly defined peak in stress–strain curves. This mode occurs in the specimen with joints inclined between θ≈20 – 30. if the interlocking is nil or low. For orientations, θ = 35 – 65 sliding occurs invariably for all the interlocking conditions.

2.6.4 Rotation

The mass fails due to rotation of the elemental blocks. It occurs for all interlocking conditions if the continuous joints have 0> 70, except for θ equal to 90 when splitting is the most probable failure mode.



θ ↑ s →	0	1/8	2/8	3/8	4/8	5/8	6/8	7/8
	Nil	Low		Medium		High		V.High
	Extent of interlocking →							

Fig- 2.10 Modes of failure in jointed rock mass (32)

2.7 Research till date.

Einstein and Hirschfeld [8] conducted triaxial tests to study the effect of joint orientation. Spacing and number of joint sets on the artificially made jointed Specimens of gypsum plaster. They have found that the upper limit of the relation between shear strength and normal stress of the jointed mass with parallel/perpendicular joints as well as inclined joints is defined by the Mohr envelope for the intact material and the lower limit is defined by the Mohr envelope for sliding along a smooth joint surface. The strength of jointed rock masses is minimum if the joints are favorably inclined and increases if the joints are unfavorably inclined. The strength of a jointed specimen is the same as the intact specimen regardless of joint orientation/spacing of joints at very high confining pressures. At low confining pressures, the specimen fails in a brittle mode, and at high confining pressures it exhibits ductile behavior.

Yaji [39] conducted triaxial tests on intact and single jointed specimens of plaster of Paris, sandstone, and granite. He has also conducted tests on step-shaped and berm-shaped joints is plaster of Paris. He presented the results in the form of stress strain curves and failure envelopes for different confining pressures. The modulus number K and modulus exponent n is determined from the plots of modulus of elasticity versus confining pressure The results of these experiments were analyzed for strength and deformation purposes. It was found that the mode of failure is dependent on the confining stress and orientation of the joint. Joint specimens with rough joint surface failed by shearing across the joint, by tensile splitting, or by a combination of thereof.

Arora [1] conducted tests on intact and jointed specimens of plaster of Paris, Jamarani sandstone, and Agra sandstone. Extensive laboratory testing of intact and jointed specimens in uniaxial and triaxial compression revealed that the important factors which influence the strength and modulus values of the jointed

rock are joint frequency J joint orientation with respect to major principal stress direction, and joint strength. Based on the results he defined a joint factor as

Jf = Jn/n x r

Where

Jn = number of joints per meter depth;

n = inclination parameter depending on the orientation of the joint ;

r = roughness parameter depending on the joint condition. The value of "n" is obtained by taking the ratio of log (strength reduction) at = 90° to log (strength reduction) at the desired value of .

According to Terzaghi [33], an intact rock has no joints orhair cracks. Normally joints are recognized as discontinuities at the boundary of the intact rock [1, 4, 8, 10, 11, 21, 22, 24 And 27]. The discontinuities may exist with or without fragments of parent rock material deposited in the joints [2, 3, 18, 25, 32, 34, 35]. Hoek and Brown [10] and Barton [4] measured scale effects in uniaxial compressive strength of intact rock. Their criterion covers the sizes in the range of laboratory scale (50 mm) to field sample of certain size at which intact strength offsets the effect of network of micro cracks. The strength ratio drops to nearly half as the sample size increases to a certain size [35]. Hoek and Brown criterion [10, 11] considered uniaxial compressive strength of intact rock, and proposed a relation for rock mass rating (RMR) and geological strength index (GSI). This system does not directly consider the joint orientation. Further, the joint size is not included directly as a parameter in estimating either the RMR or GSI. However, the effect of joint

Size is indirectly considered in rock mass strength in terms of scale effects. The RMR includes the joint spacing and rock quality designation (RQD). Furthermore, RMR and GSI provide measure of qualitative assessment of rock .unifying the scale dependence, anisotropy, and the effect of discontinuities.

Table 2.5 Strength of jointed and intact rock mass

Ramamurthy and Arora, [27]

Class	Description	UCS Mpa
A	Very high strength	>250
B	High strength	100-250
C	Moderate strength	50-100
D	Medium strength	25-50
E	Low strength	5-25
F	Very low strength	<5

Table 2.6 Modulus ratio of jointed and intact rock mass

Ramamurthy and Arora, [27]

Class	Description	Modulus Ratio
A	Very high modulus ratio	>500
B	High modulus ratio	200-500
C	Medium ratio	100-200
D	Low modulus	50-100
E	Very low modulus ratio	<50

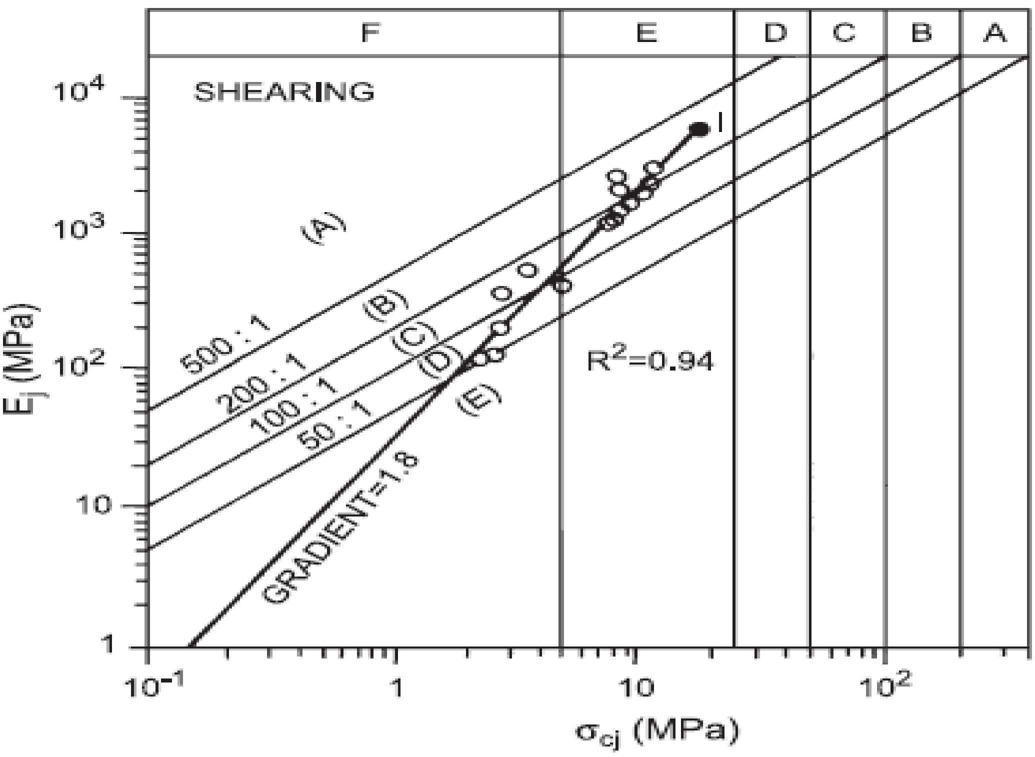


Fig 2.11 Strength and tangent modulus values for shearing mode of failure

Ramamurty amd Arora, [27]

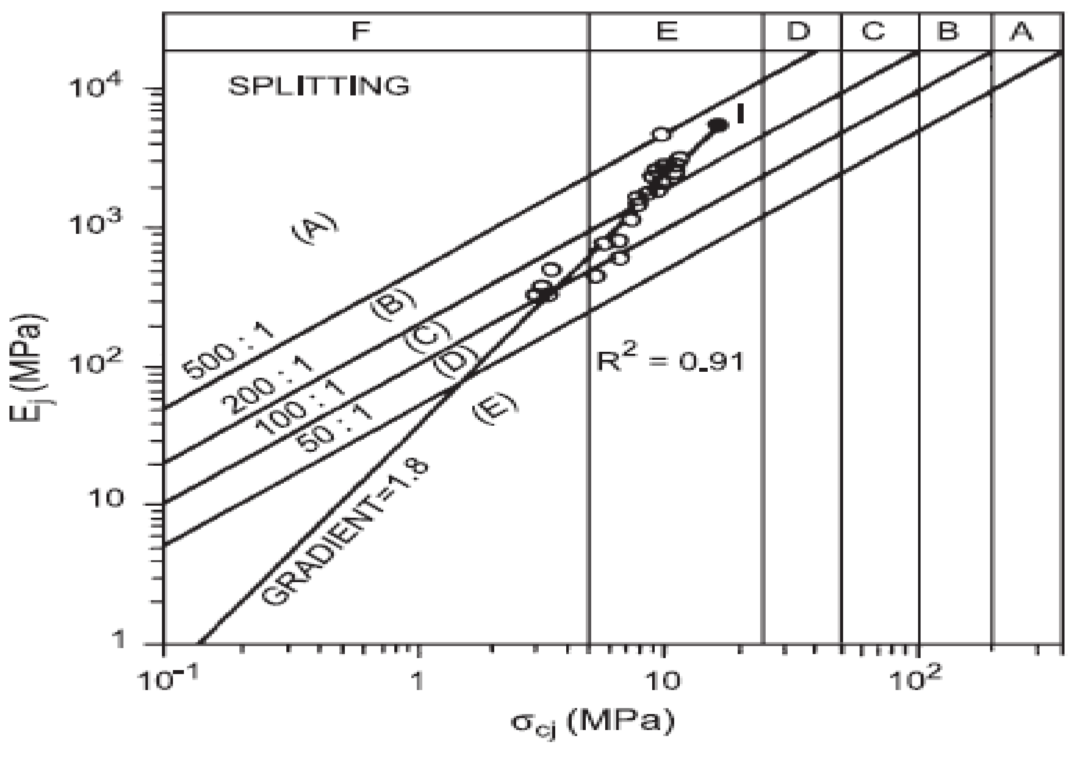


Fig 2.12: Strength and tangent modulus values for splitting mode of failure

Ramamurty and Arora, [27]

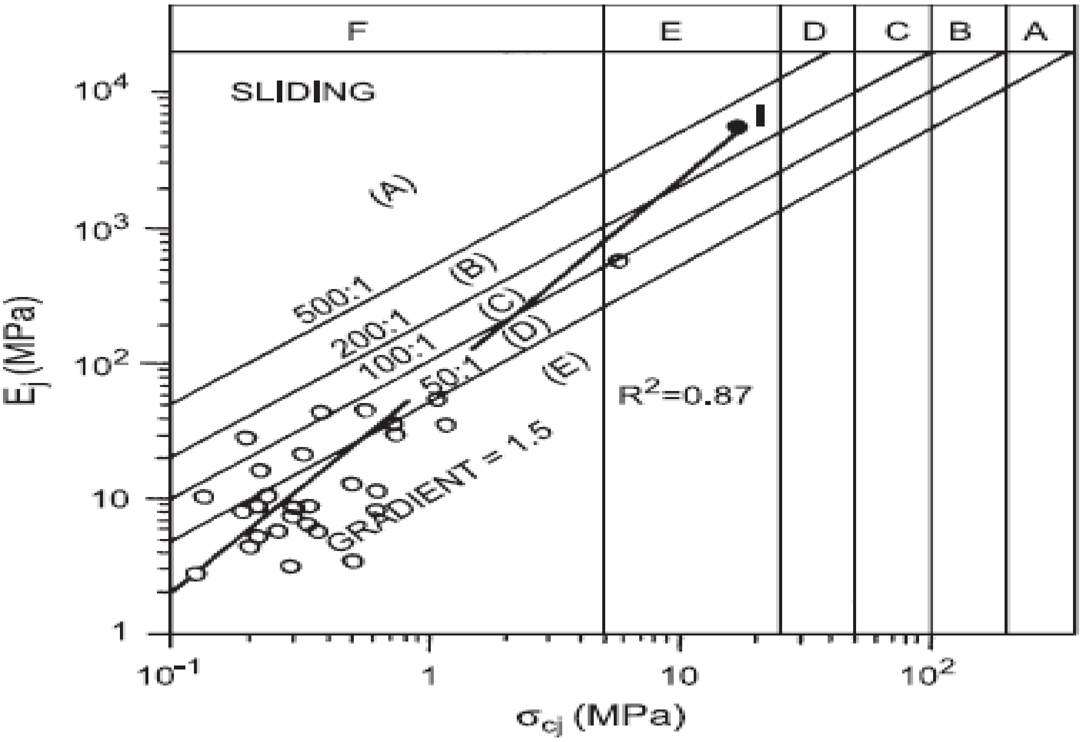


Fig 2.13: Strength and tangent modulus values for sliding mode of failure Ramamurty and Arora [27].

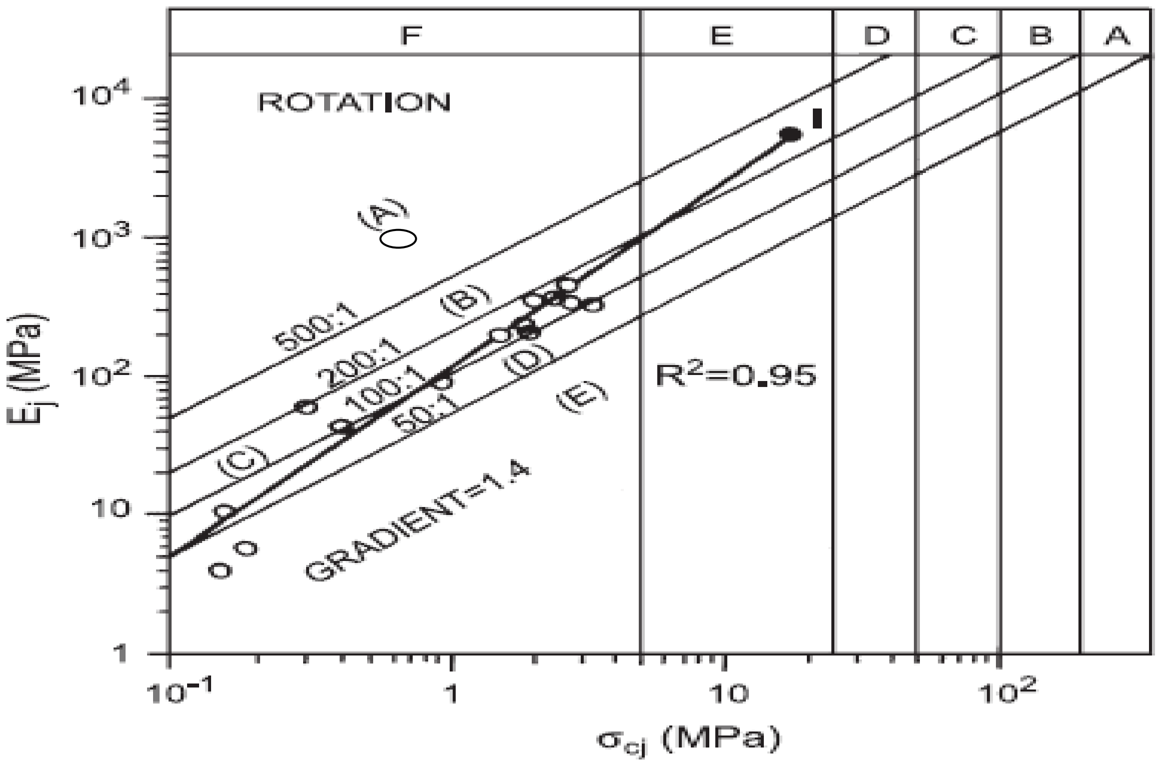


Fig 2.14: Strength and tangent modulus values for rotation mode of failure Ramamurty and Arora [27].

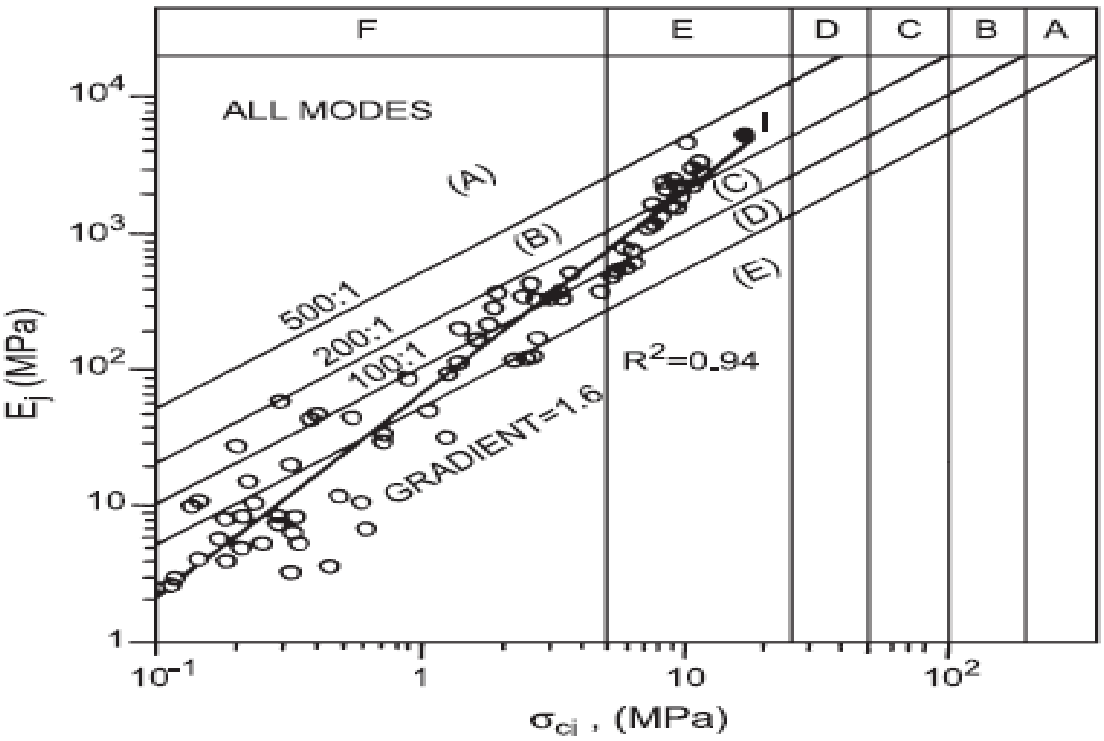


Fig 2.15: Strength and tangent modulus values for all modes of failure

Ramamurty and Arora, [27]

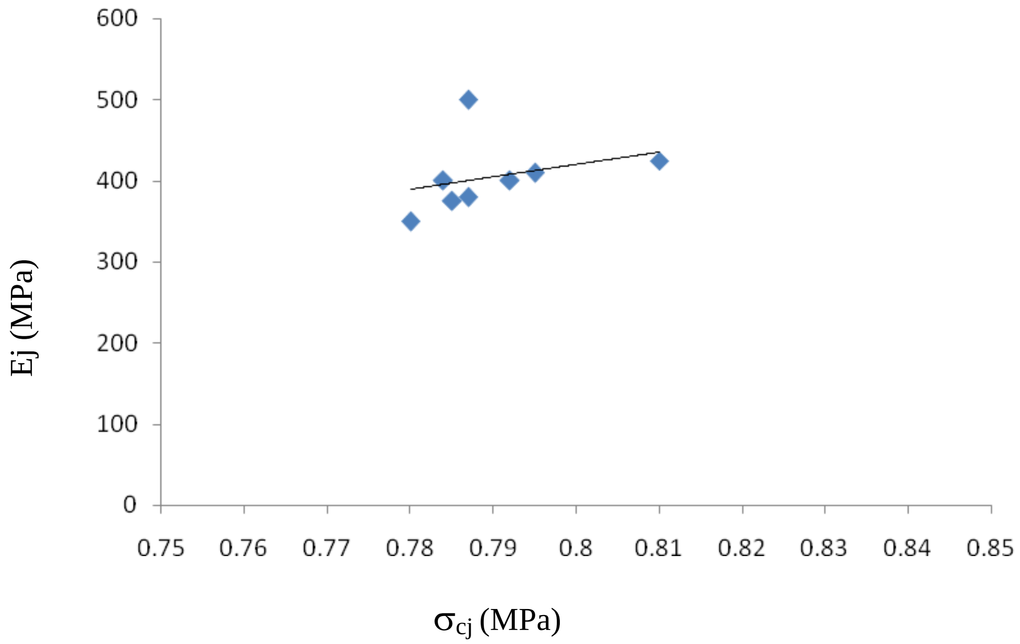


Fig: 2.16 Strength and tangent modulus values for all modes of failure at 7 days[Present Study]

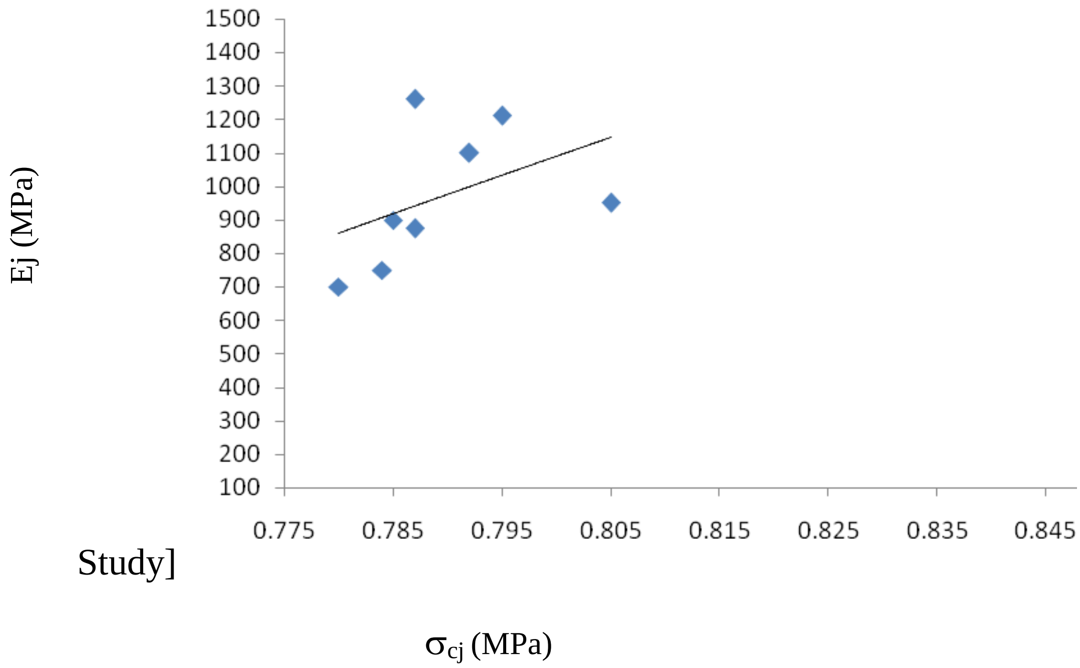


Fig: 2.17 Strength and tangent modulus values for all modes of failure at 28 days.[Present Study]

3.0 LABORATARY EXPERIMENT AND INVESTIGATION

In the present experimental work, strength properties of jointed rock with P.O.P cemented jointed has been studied by means of a systematic and controlled laboratory experiments. Main objectives of the study are –

I. To study the effect of thickness of PoP cemented jointed filled in the horizontal , vertical & inclined joints on the uniaxial compressive strength of rocks

II. To study the effect of orientation of joints on the strength of rock with PoP cemented joints

III. To study the effect of the frequency of horizontal joints with PoP cemented jointed on the strength of rocks

IV. To study the effect of location of horizontal, vertical & inclined joints with PoP cemented jointed on the strength of rock.

V. A large number of uniaxial compressive tests are conducted on the prepared specimens of jointed block mass having various combinations of orientations and different levels of interlocking of joints for obtaining the ultimate strength of jointed rock mass.

3.1 Material of specimen

Various materials like plaster of Paris, Kota sandstone, Jamarani, sandstone, Agra sandstone, Granite, Gypsum plaster can be used for preparing replicas of jointed rock mass in a laboratory.

Research is still being conducted on getting a model material to reproduce the natural rock mass and get satisfactory results in understanding the failure mechanism and strength behaviour.

3.2 Material used:

Experiments have been conducted on model materials so as to get uniform, identical or homogenous specimen in order to understand the failure mechanism, strength and deformation behavior.

It is observer that Standard sand and cement mortar has been used as model material to simulate weak rock mass in the field. Many researchers have used plaster of Paris because of its ease of casting, flexibility, instant hardening, low cost and easy availability.

Any type of joint can be modeled by plaster of Paris. The reduced strength and deformed abilities in relation to actual rocks has made plaster of Paris one of the ideal materials for modeling in Geotechnical engineering. But author cement and standards sand mortar as a specimen to simulate the rock mass and plaster of Paris as cemented joint.

Table 3.1 Standard Sand: [IS: 650-1991]

S.No.	Properties	Values
1	Fineness Modulus	2.02
2	Specific Gravity	2.60
3	Water absorption (%)	1.52
4	Bulk Density	18.28 kN m^{-3}

Table 3.2 Plaster of Paris: [IS: 2592-1978]

S.No	Properties	Results	Specified values
1	Setting time plaster sand mixture	54	45-120
2	Setting time neat plaster	26	20-40
3	Transverse-strength	10.2	5.0 min
4	Residue on 90 Micron	2.2	5.0 min
5	Soundness	O.K	Set plaster pest shall not

			show any sign of pitting.
6	Mass density (Kn/m^3)	13.94	
7	Specific gravity	2.81	
8	Uniaxial-compressive strength(sci in MPa)	11.00	
Table 3.3 Cement 43 Grade OPC:[IS:8112]			
S.No.	Properties	Results	Specified values
1	Specific gravity	3.14	--
2	Blain air Permeability Fineness	300	225 min
3	Normal consistency	30.2	--
4	Initial Setting time in min.	145	30 min
5	Final Setting time min.	225	600 max
6	Compressive-strength (MPa)		
a	7 days compressive Strength	34.6	93 min
b	28days compressive strength.	47.2	43 min
c	Autoclave % exp.	0.24	0.80 max
d	L/C % exp.	1.92	10 max

[D] Water:

To make a specimen, distilled were used which is free from impurities.

3.3 Apparatus used

For mixing standard sand and cement.

(a) 1.5 mm ms plate

(b) Gassing towel.

(c) Measuring cylinder

For compaction $\longrightarrow$ vibration machine.

For curing: (a) Curing tank having temperature 27±2 degree centigrade

(b) Humidity chamber & humidifier

For measuring specimen : scale and Verneer calipers

For compressive strength: Universal testing machine

3.4 Curing:

Mixing and cement and standard sand carried out at the temperature 25 % .After preparation, the samples were laid in curing tank at temperature 27ºC ± 2ºC (humidity 65 %) for 7day and 28 days and then kept in air tight desiccators.

3.5 Induction of gouge

A stiff paste of PoP powder Birla Make obtained. was prepared containing 27 % water and this was used to coat the surface of joints prepared as above of the cylindrical samples:

(a) With constant thickness at varying orientations,

(b) With constant thickness at different location of joints

(c) Constant thickness at different number of joints.

3.6 Uniaxial compressive strength test:

In Uniaxial test the cylinder specimen is subjected to major principal stress till the specimen fails due to shearing along a critical plane of failure. In this test the core should be circular in shape, length 2.5 to 3 times the diameter; end shall be flat within 0.02mm. Perpendicularity of the axis shall not be deviated by 0.001radian and the specimen shall be tested within 30days. The applied load on the specimen shall be at the rate of 5.1to 10.2 kgf/cm^2/sec. The diameter of the specimen shall be either 25mm or 50mm. After measuring the load bearing surface areas the well prepared specimen is put in between the two steel plates of the testing machine and load applied at the predetermined rate along the axis of the sample till the sample fails. The deformation of the sample is measured with the help of a separate dial gauge.

The ends of the cylindrical specimen are hollowed in the form of cone. The cone seating reduce the tendency of the specimen to become barrel shaped by reducing end straits. During the test, load versus deformation readings are taken and a graph is plotted. When a brittle failure occurs, the proving ring dial indicates a definite maximum load which drops rapidly with the further increase if strain. The

applied load at the point of failure should be noted. The load is divided by load borne by the bearing surface of the specimen will give the Uniaxial compressive strength of the same. Generally 7 to 10 tests are to be done for a particular rock type to establish the average values of its compressive strength. For irregular specimen more tests are recommended.

3.7 Testing Procedure

Specimen (intact or jointed) were kept between the loading platens of the compression testing machine and axial compressive .Load was applied through the loading frame till the sample failed. Load deformation measurement was taken at regular interval of axial deformation until failure accrued.

For each case 2-specimen were tested as per ISRM recommendations and failure pattern was observed closely. Few failure patterns are shown in the photograph section.

3.8 Parameters:

The main objective of the experimental investigation is to study the following
Aspects:

- The effect of joint factor (Jf) on the strength characteristics of different nos.of jointed specimens.
- The compressive strength behaviour of intact and jointed specimens
- The deformation behavior of specimens.
- Variation of modulus ratio for intact and jointed specimens.

In view of the above, the experiments have been conducted and the different parameters evaluated are given below.

Uniaxial compression test were done for single joint at various inclinations i.e. $\beta = 0°$, 45° & 90 º and $2J\beta = 90°$, $3J\beta = 90°$, $4J\beta = 90°$ and $5J\beta = 90°$

For each orientation of joints, two U.C.S tests were conducted as shown in the table. These are shown in the figures, 3.3

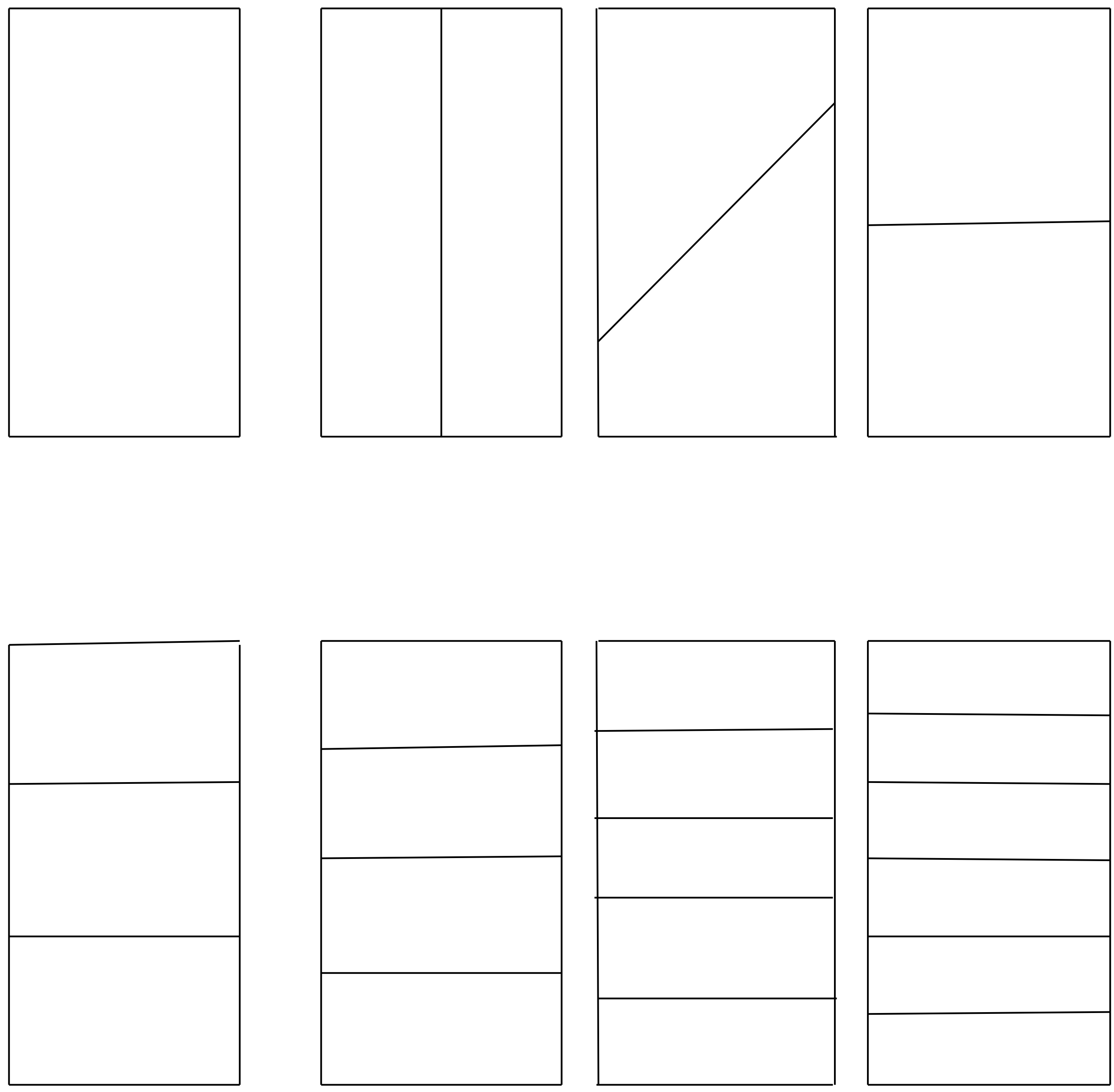


Fig 3.1 The uniaxial compressive strength (σ_{ci}) of intact & jointed specimen tested

3.9 **Strength properties**

The average uniaxial compressive strength (σ_{ci}) of intact specimen of cement standards and mortar tested were 30.1 MN/m^2 after 7 days and 41.15 after 28 days respectively .Rocks exhibits wide range of strength due to variation in the geological process in their formation.

Indian standard code IS: (1123 – 1975) reports that compressive strength of rocks occurs between 20 to 170 MN/m^2. The stress strain curve in the present study shows plastic elastic nature. In the following sections, effect of PoP cemented joints. Joint properties namely joint thickness and nos of joints on the uniaxial compressive strength of rock mass are studied.

3.10 Joint Orientation:

The effect of orientation of gouged joints on uniaxial compressive strength was studied by developing joints at the orientation of joints; 0º, 45º, & 90º.

3.11 Joint Frequency:

It measures number of joints per unit length of the rock sample. Various PoP joints studied in this case were 1J = 90º, 2J = 90º, 3J = 90º, 4J = 90º and 5J = 90º. Fig. 3.4 shows a plot of σ_{cjg} with the increase in the number of joints.

Using this factor three significant aspects concerning joints viz. (i) joint orientation, (ii) joint frequency, (iii) joint roughness and joint strength can be considered together for a limited thickness of gouge

Relation between σ_{cr} and J_f : It has been found from the plot and relationships are as under:-

σ_{cr} = Exp (- αJ_f) for jointed rocks due to Trivedi [34]

Where α value is -.005

σ_{cr} = Exp (- K J_f + Z)

K = 0.047 & Z = 2.2 for $75^\circ \geq B \geq 60^\circ$

K = 0.014 & Z = 0.34 for $90^\circ \geq B \geq 75^\circ$

K = 0.01 & Z = 0.1 for n – number of joints

$\boldsymbol{\sigma_{cr}=\sigma_{ig}/\sigma_{cg}}$

σ_{cg} is the uniaxial compressive strength of jointed rock with gouge. The strength of jointed rock is reduced by factor Z and its nature is varied by the factor.

4.0 RESULTS

Following are the stress- strain % graph of obsevered value of load and deformation for specimen of cement and standard sand with different orientation and number of PoP cemented joints .

- Length of specimen = 76 mm,
- diameter of specimen =38mm.
- Cross section area specification=1134 mm^2.
- Strain rate = 0.5mm/minute.

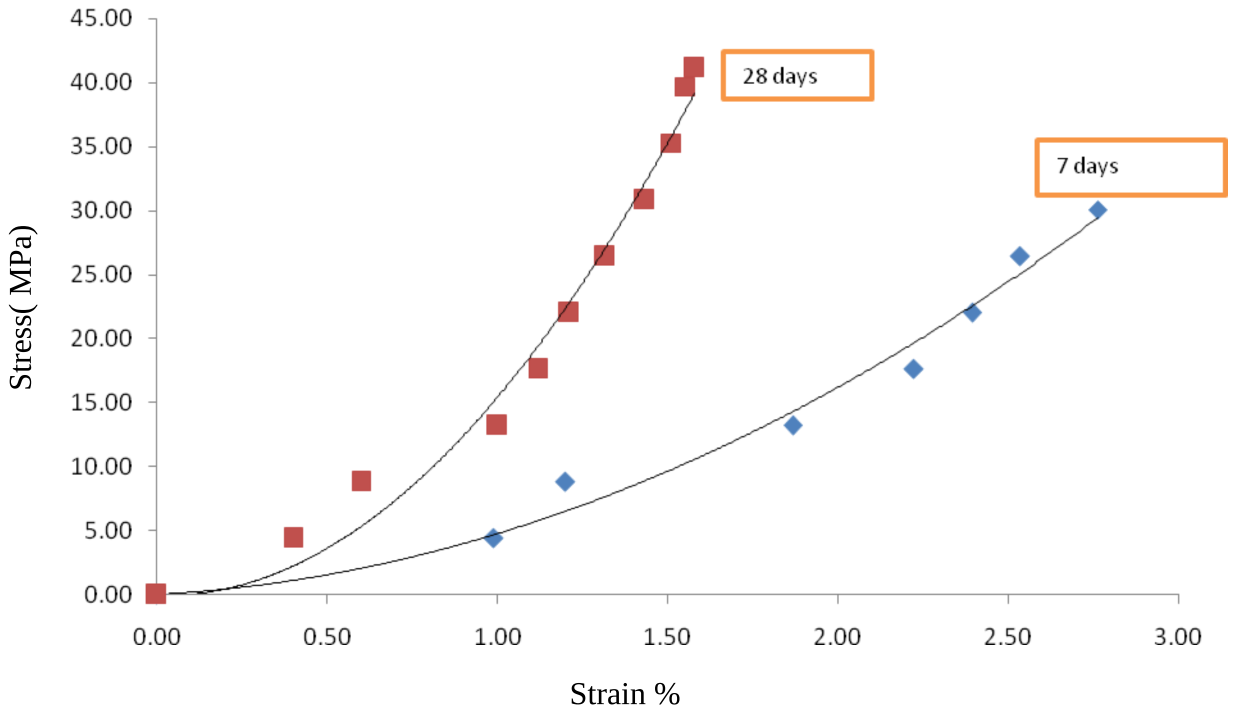


Fig: 4.1 Stress-strain curve after 7 and 28 days of intact specimen

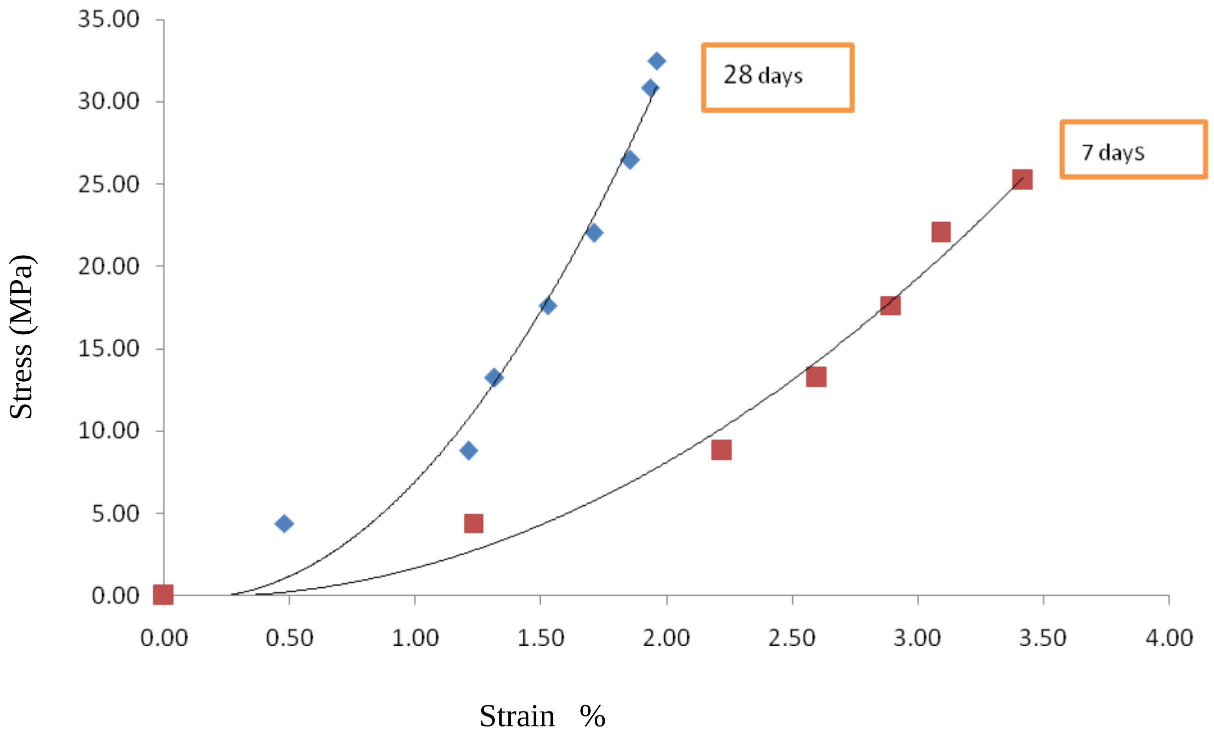


Fig: 4.2 Stress-strain curve after 7 and 28 days of 1J β=0° specimen

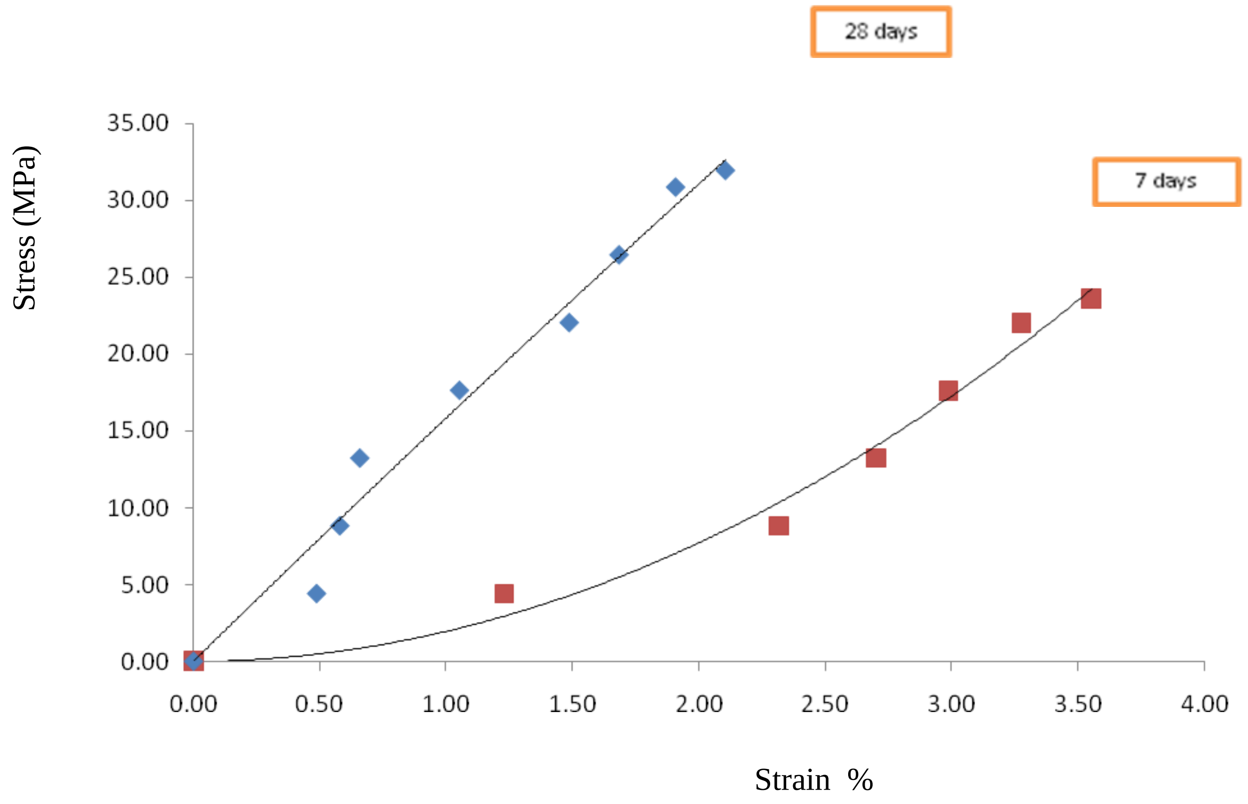


Fig: 4.3 Stress-strain curve after 7 and 28 days of 1 J β=45^{0} specimen

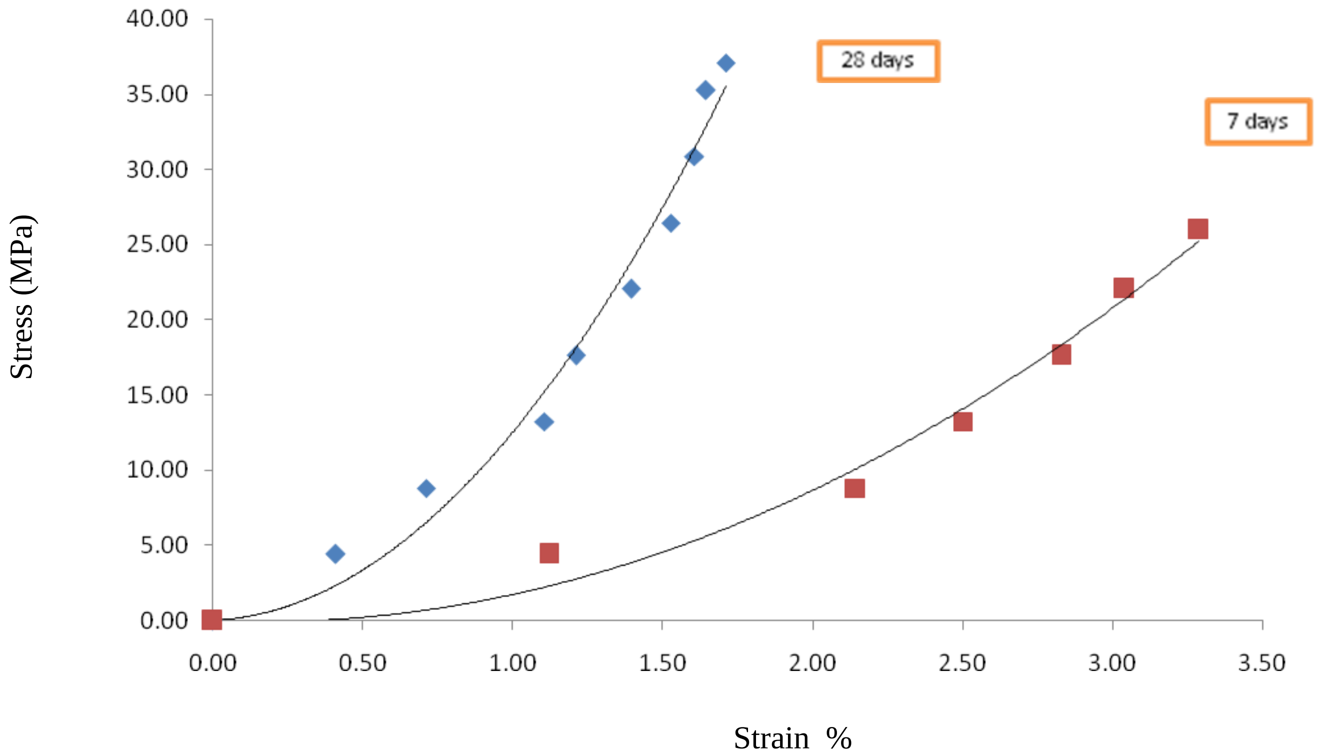


Fig: 4.4 Stress-strain curve after 7 and 28 days of 1 J β=90^{0} specimen

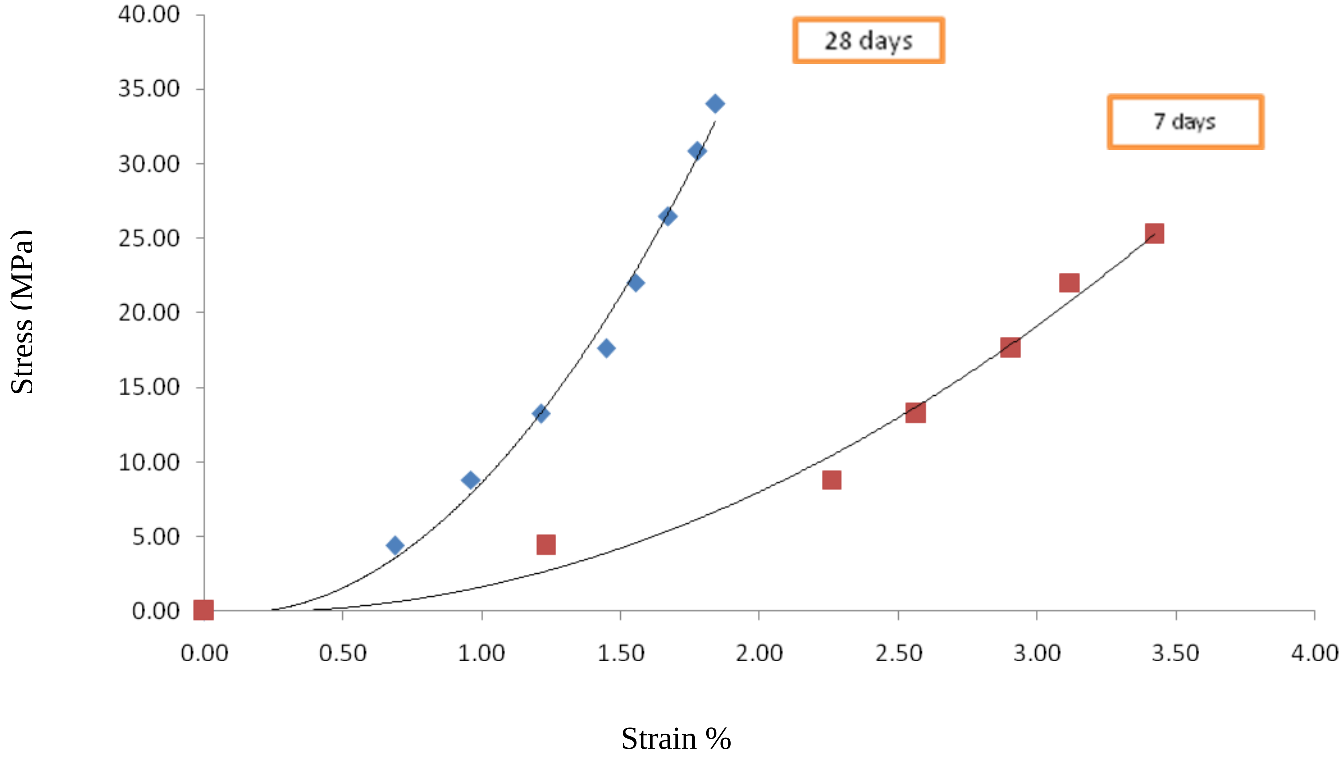


Fig: 4.5 Stress-strain curve after 7 and 28 days of 2 J β=90$^{\theta}$ specimen

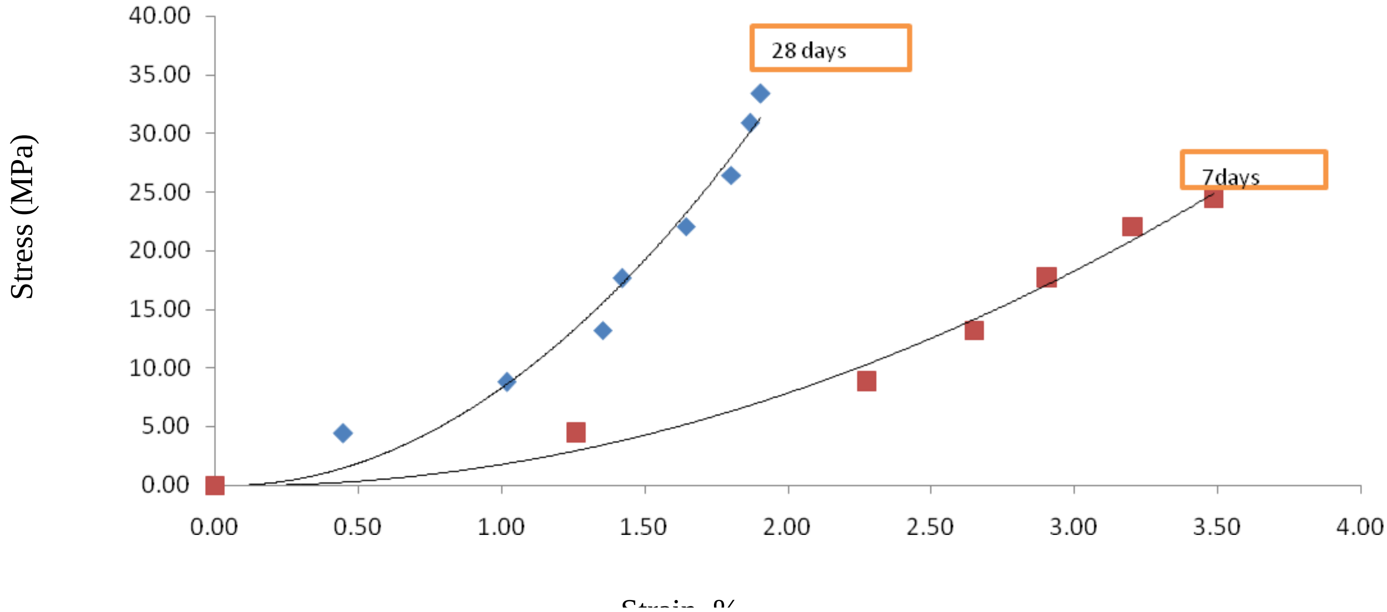


Fig: 4.6 Stress-strain curve after 7 and 28 days of 3β=90$^{\theta}$ specimen

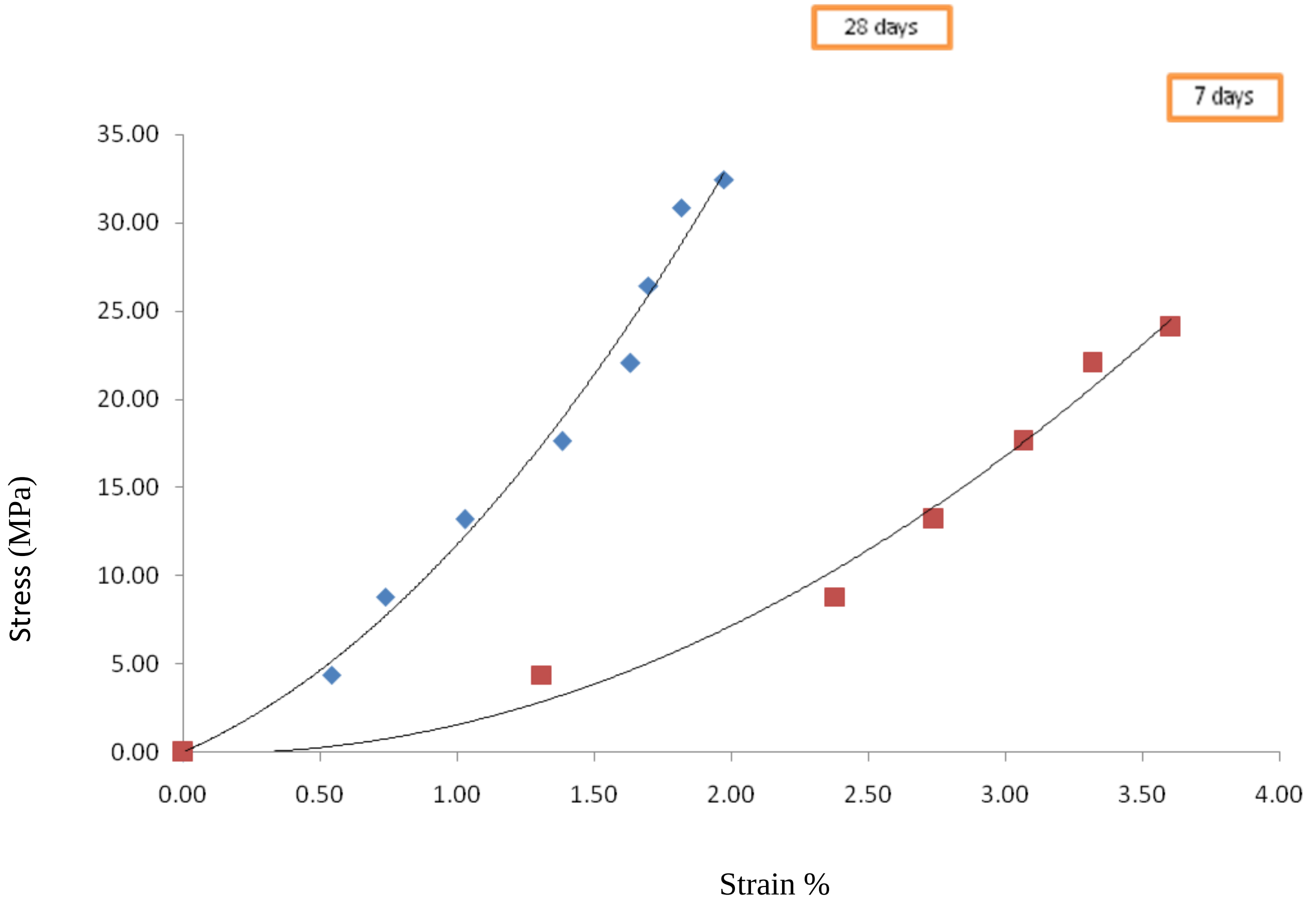


Fig: 4.7 Stress-strain curve after 7 and 28 days of 4 J β=90° specimen

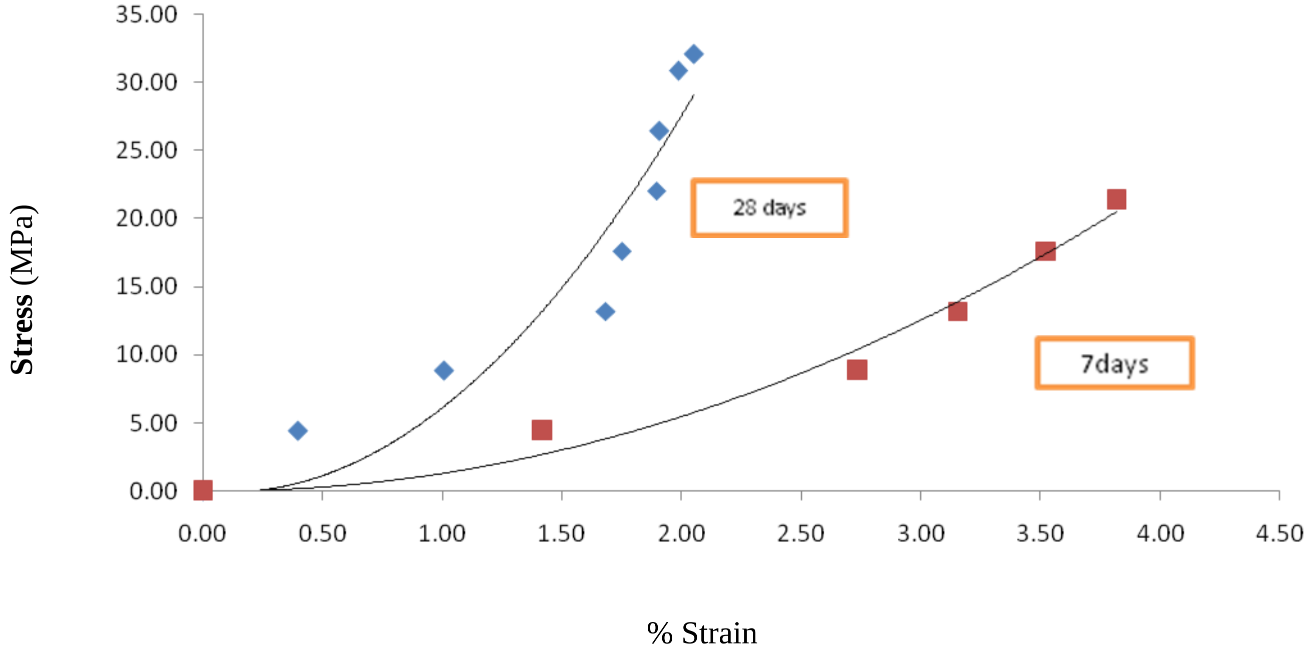


Fig: 4.8 Stress-strain curve after 7 and 28 days of 5 J β=90° specimen

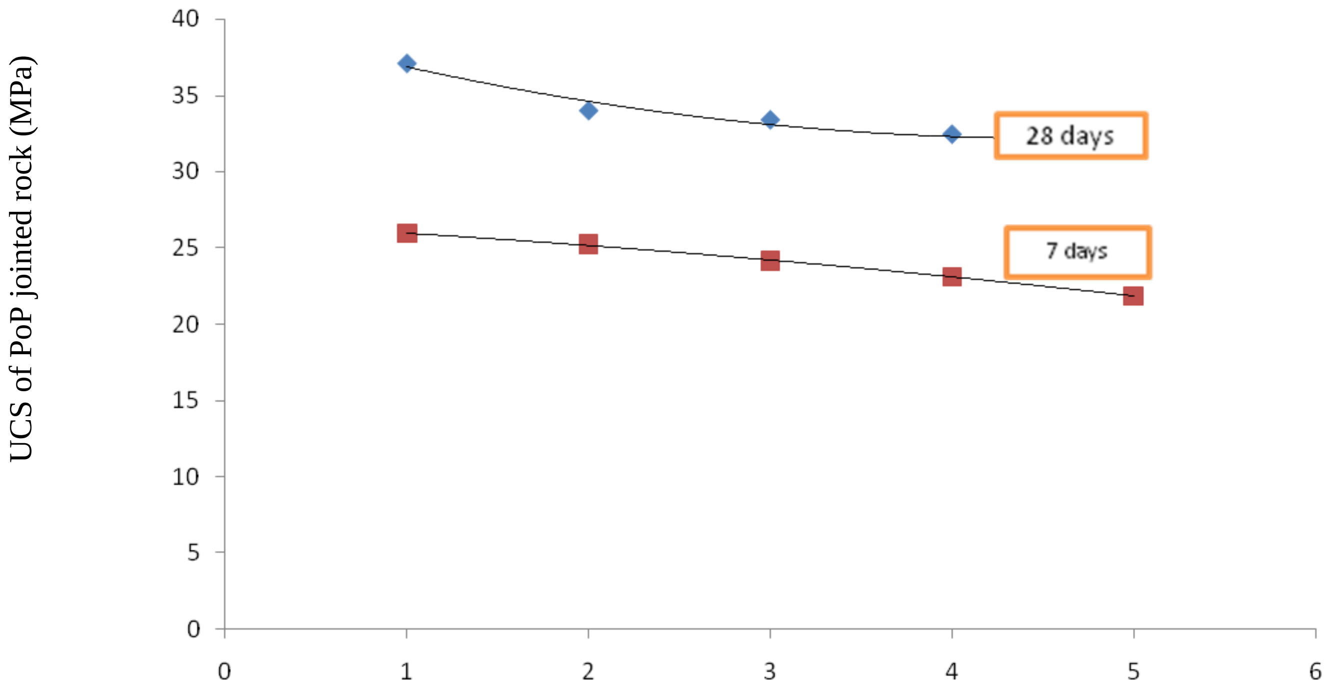


Fig: 4.9 Number of filled joints β=90°

A plot between the UCS of the PoP filled jointed specimen at β=90° and the frequency of filled joints

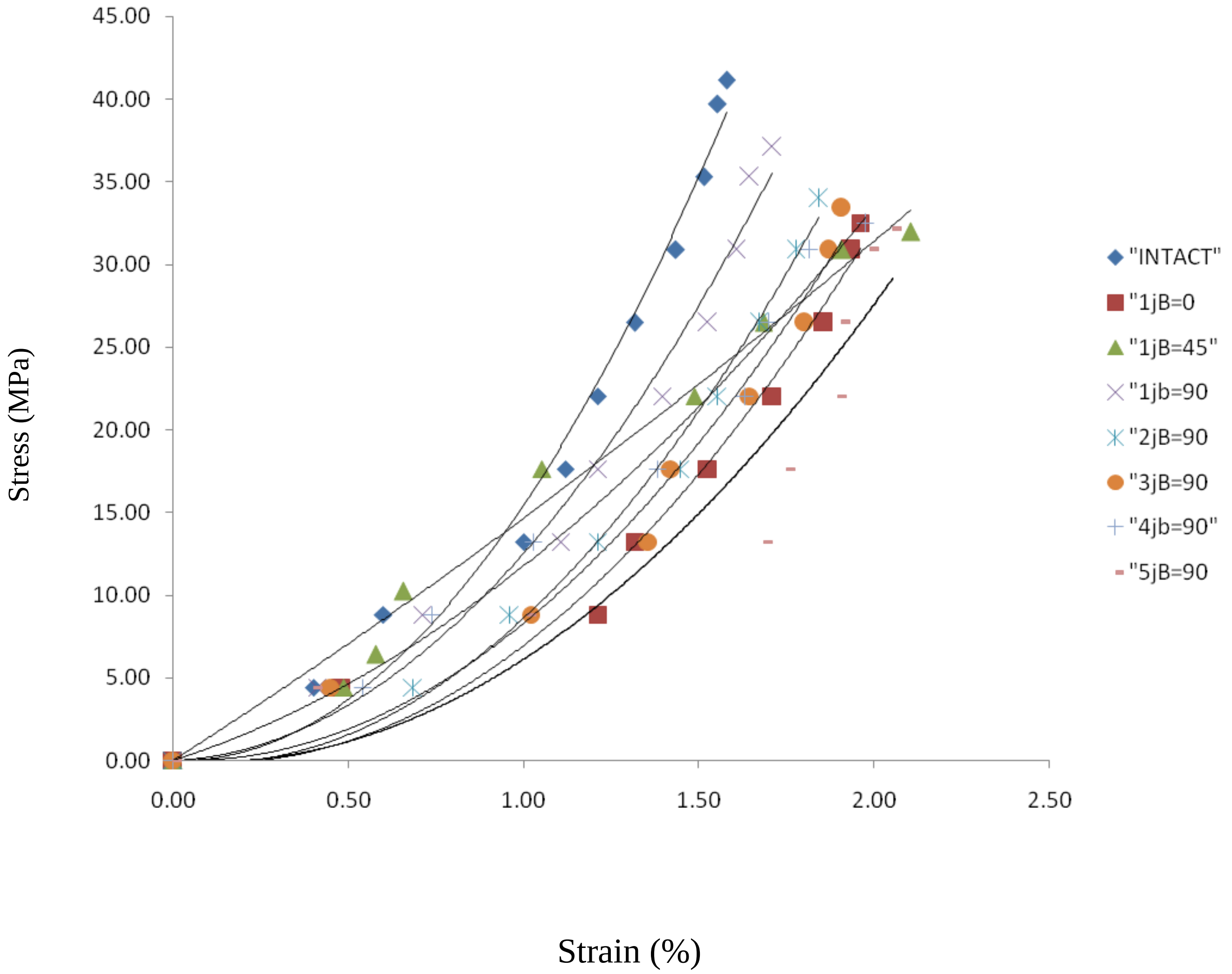


Fig: 4.10 Comparison of intact and jointed specimens 28 days

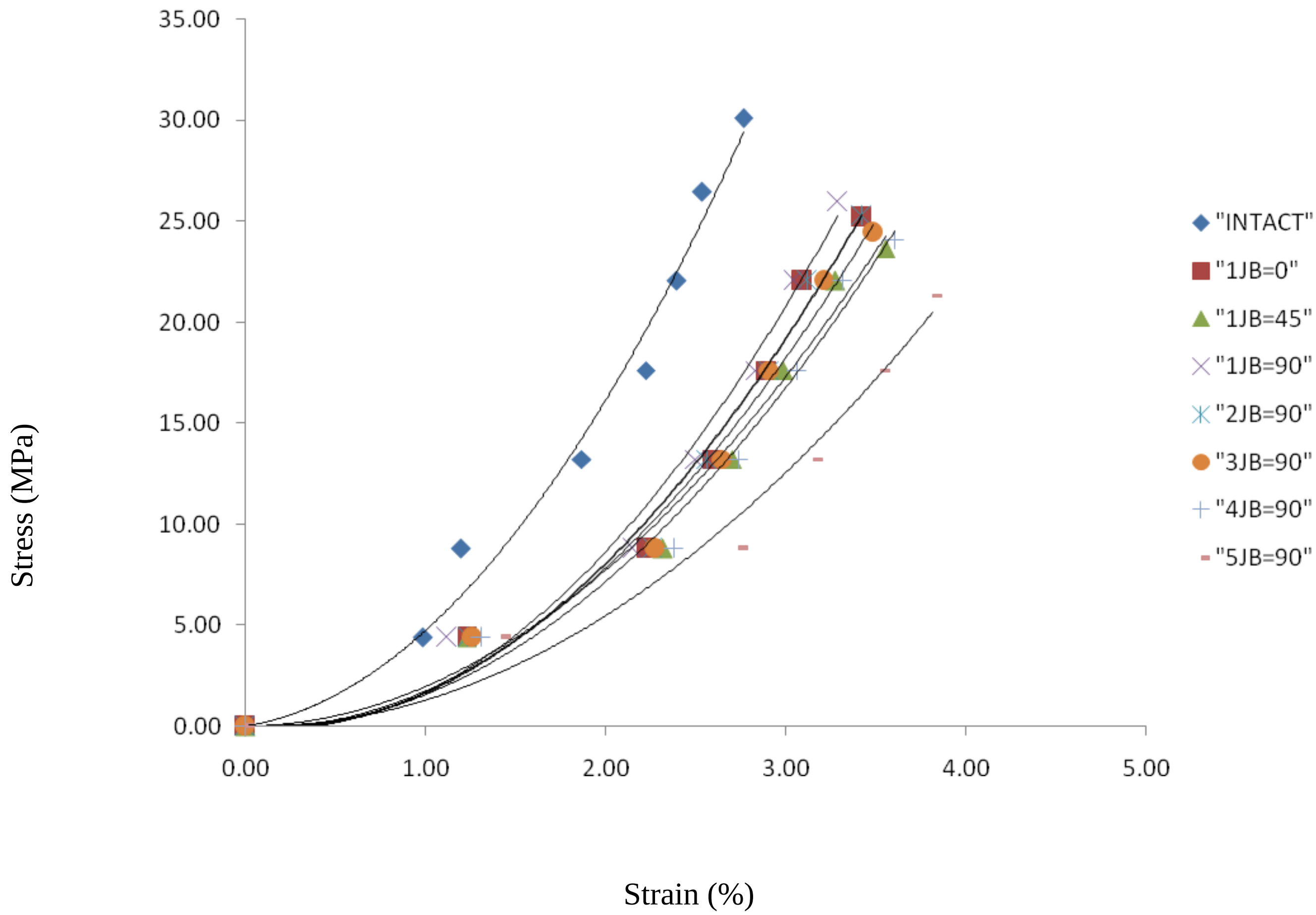


Fig: 4.11 Comparison of intact and jointed specimens 7 days

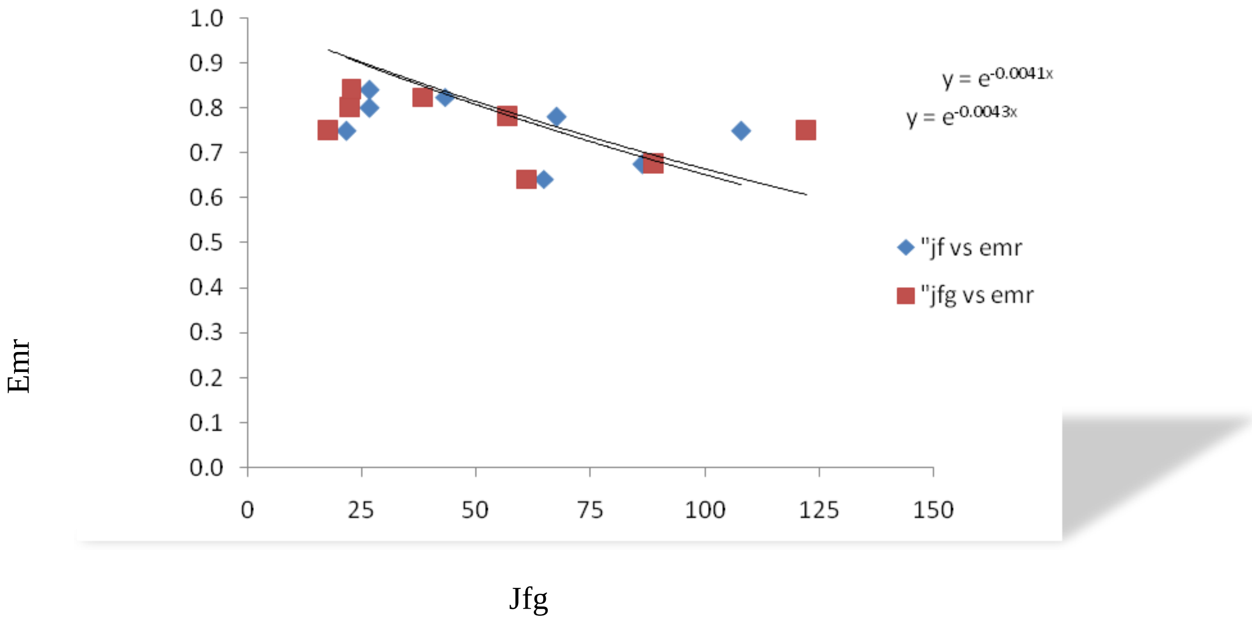


Fig: 4.12 Graph between between modulus ratio-J_{fg} on 28 days

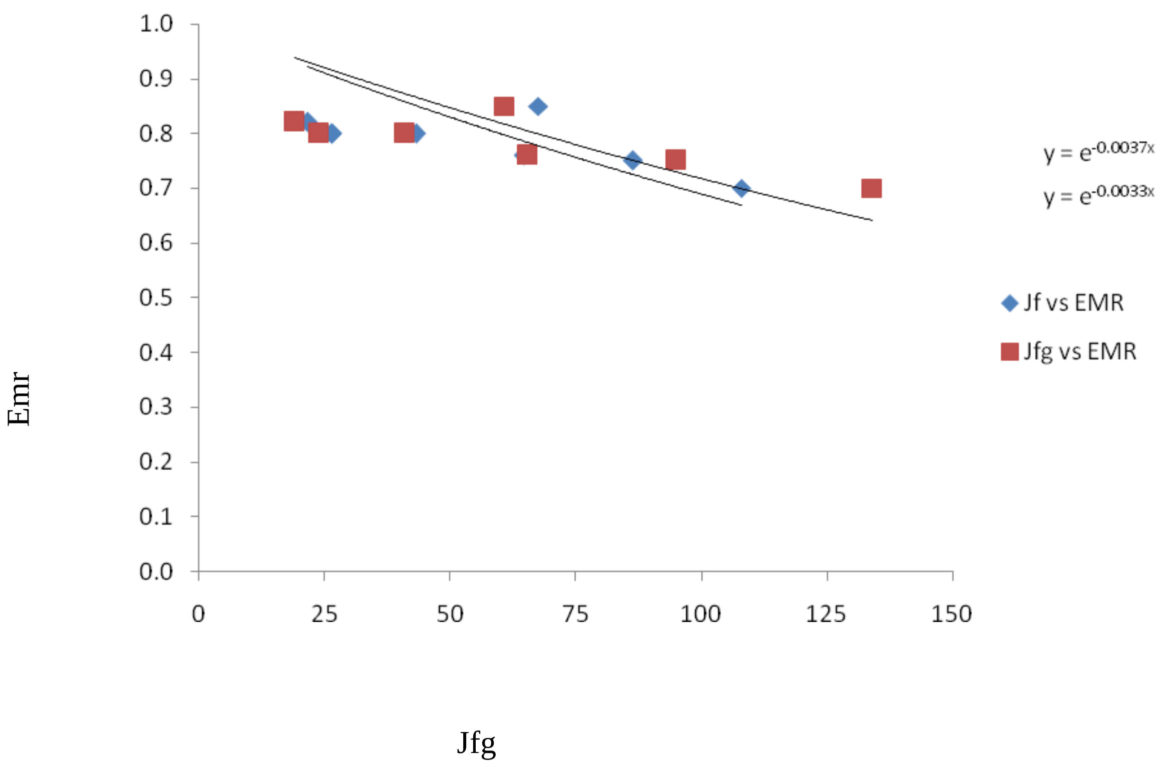


Fig. 4.13 Modulus ratio vs &J_{fg} on 7 days

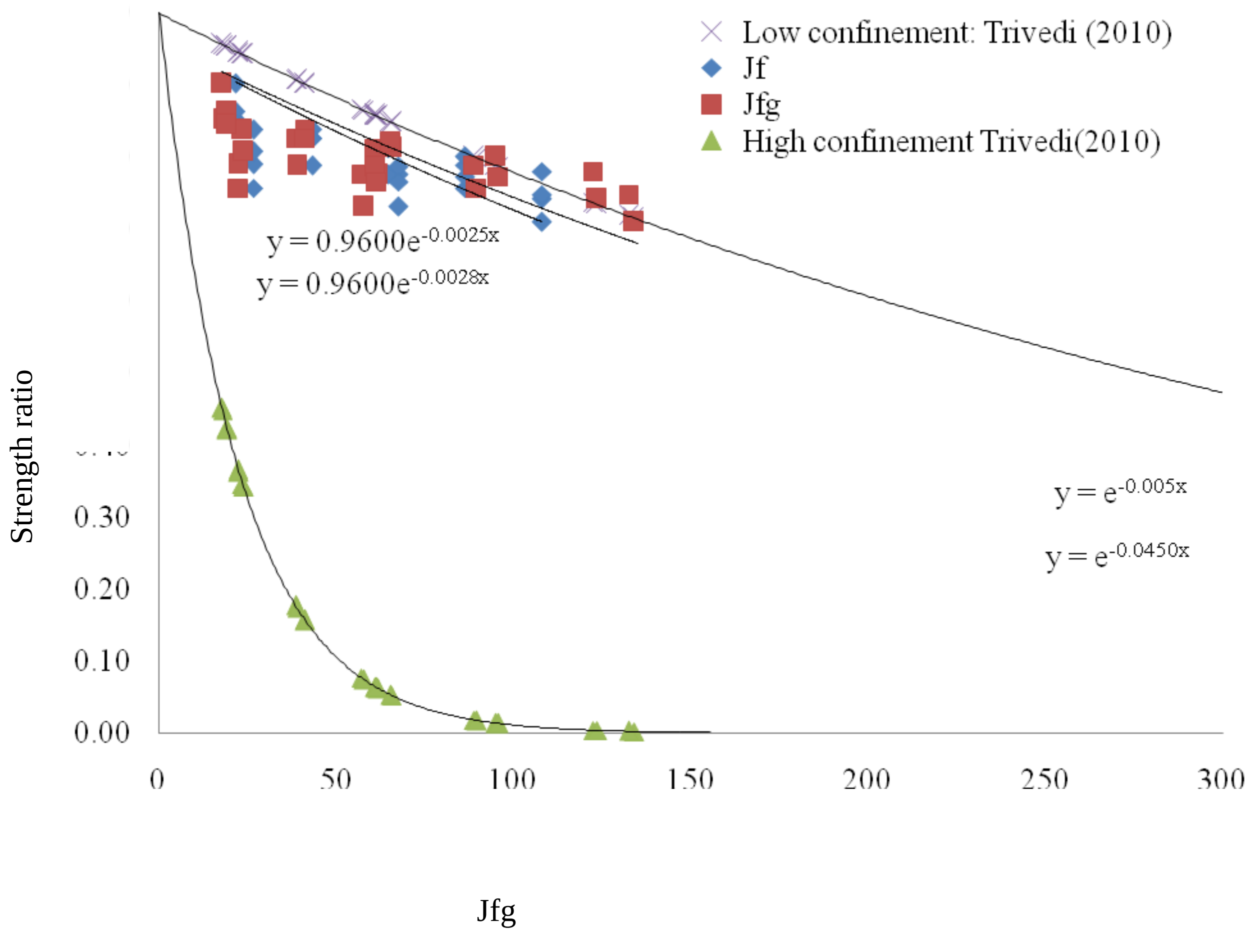


Fig. 4.14 Strength ratio vs Jfg on 7 days

5.0 CONCLUSIONS

This chapter includes briefly, the summary of work program, discussion on the results, conclusions and outlines hinting at the possible future scope of the work.

Various parameters such as the strength of PoP, composition, curing the specimen and loading rates were controlled in order to obtain reproducible results, throughout the experimental program. The effect of changes in the four of the parameters, thickness of the gouge, orientation of the gouged joints, number of horizontally gouged joints, and the location of horizontally gouged joint is studied. Effect of gouge is always that of reduction of strength. The general equation of compressive strength ratio of the jointed rocks is

σ_{cr}=exp(-k J_f+z) where k and z are constants.

This thesis describes an approach to find strength of jointed rocks with PoP cemented joints in terms of empirically established modified joint factor (J_f). Historically, the joint factor is adopted in relation to a constant joint strength parameter (r), constant joint orientation parameter (n), and constant. Number of joints (J_n) and modification factor for gouge (c_g) in terms of gouge thickness (t), compactness of fill material (g_d) and distance of joints from loading plane (dj).These consideration bring forth multiplicity in interpretation of empirical joint factor and hence strength ratio. The joint strength parameter, joint orientation parameter, number of joints and modification factor for gouge c_g

On the basis of current experimental study on the intact and jointed specimen of cement and standard sand with PoP cemented joints, the following conclusions are drawn:

1. The uniaxial compressive strength of intact specimens of cement and standard sand is found to be 41.15 MPa and 30.85 MPa after 28 days and 7 days respectively.

2. As the number of joints increase the uniaxial compressive strength of cemented joints decreases, however at a decreasing rate.
3. The strength of jointed specimen depends on the joint orientation β with respect to the direction of major principal stress. The strength at β=45º is found to be minimum and the strength at β = 90º is found to be maximum.
4. The values of modulus ratio (Er) also depend on the joint orientation β. The modulus ratio is least at 45º
5. With increase in joint factor (J_{fg}) the strength decreases.
6. There is a slight variation between the present experimental results and those obtained from the empirical formula given by Arora and Ramamurthy[25]
7. The cemented joints exhibit relatively less decrease compressive strength in comparison of open joint and clay joints.

The cemented joint indicate that the values for cemented joint are obtained close to the upper bound predicted by Trivedi [37] if the curves are plotted as best fit due to the unavailable of data for the strength ratio between 0-0.9,the best fit curve was obtained with a coefficient of $0.96\ e^{\alpha Jfg}$ where α value goes above the predicted upper bound of the strength ratio.

It may also be stated that the statistical data considered over here a significantly smaller and therefore varied interpretation are possible. The α values obtained by Trivedi [37] was between -0.045 to -0.005.The value α obtained in our study varied joints factors for the cemented gouge discontinuities inserted amid the sand mortar jointed specimen was found to be between these two limits.

The statistical average of strength ratio was obtained to be $0.096\ e^{-0.0025fg}$ and $0.096\ e^{-00285fg}$ at 7 days and 28 days aging if we try to find out the correlation for joints factor for 7 days and 28 days curing.

All these finding indicate that cemented gouge amid the joint may not exactly follow the pattern as per the blank joint predicted by Arora and Ramamurthy [1, 25] and also at a difference from the observation of Trivedi [37, 40]

6.0 SCOPE OF FUTURE STUDY

The value of αin the eqn. s_{cr}=Exp ($-\alpha J_f$) vary with the aging properties of the joint. This can be verified by further researches in the field. The effect of mineralogical composition of clays, moisture content, the rock type and the degree of confinement of the gouge within the joints may also become critical parameters in the determination of the factors.

Various forces of post structural origin may become active if the partly cemented seam (with possibility of seepage water) is entrapped within the fault zone of the lower rocky strata. A violent slip, in case, may result due to impact dynamic loading. The effect of confinement becomes pronounced in such cases. This can also be verified.

The effect of gouge on the strength of jointed rocks leads to one more conclusions that such rocks fail at a very high strain. Obviously this can be attributed to the ductility of the gouge.

The problem related to gouge may be encountered elsewhere when the high altitude dams are constructed and fragile rock joints are intruded by gouging action in the lower strata. The cemented fill simulated the condition of a grout in the fragile jointed system. This field needs a lot of research and understanding backed by a deep engineering judgment.

1. Strength and deformation behaviour of cement jointed specimens can be studied under triaxial conditions.
2. The effect of temperature, confining pressure and rate of loading on the strength characteristics of cemented joint can be studied.
3. Studies can be made by introducing multiple cemented joints in varying orientation.
4. Investigation can be done on various cementing materials in specimen with joints at different angles.

5. Similar study can be carried out with different thickness. And different material filled joints.
6. Numerical model can be developed

The Present work is aimed to visualize the strength and deformational behaviour of jointed rock in which joints are filled with PoP. This is done by inducing artificial joints and then introducing PoP paste in the joints. Such a sample is dried at room temperature.

Photo: 1 Mixing of standard sand with 43 grade OPC cement.

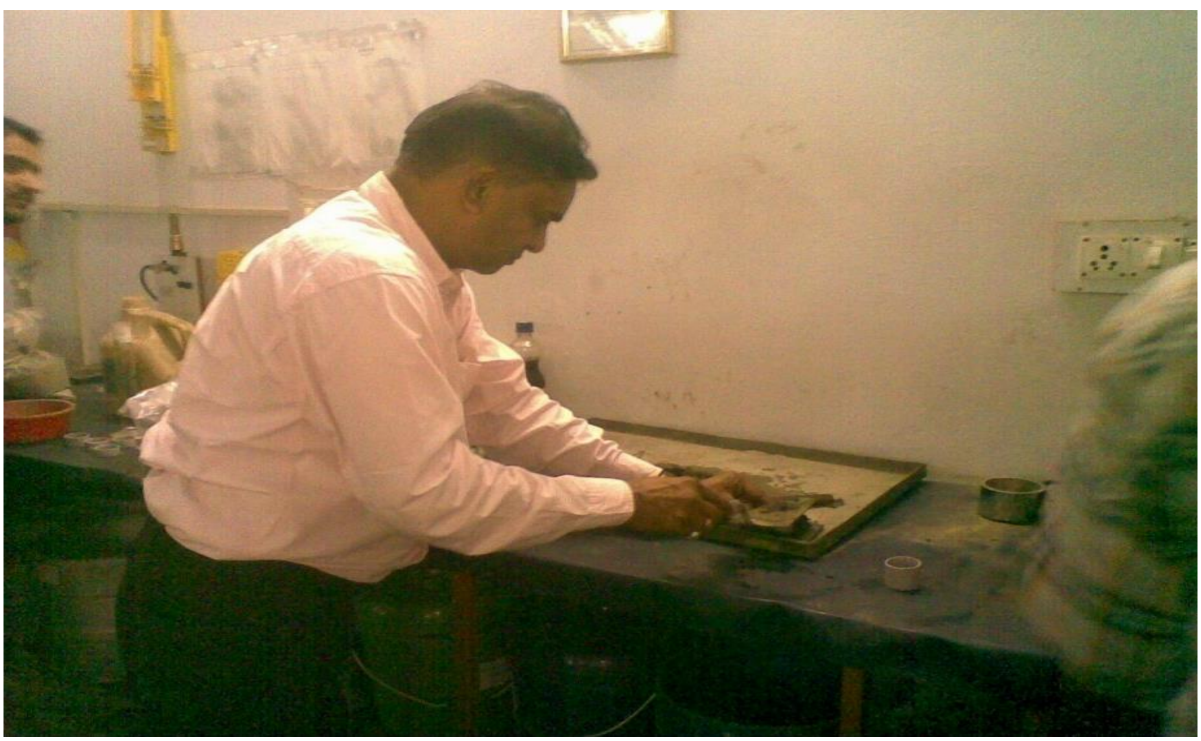


Photo: 2 filling of mortar into the mould.

Photo: 3 Vibration machine to shaking the mortar filled mould.

Photo:4 Photograph of curing tank in which specimen were laid for curing.

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Photo: 5 Making plane the surface of specimen with help of grinder

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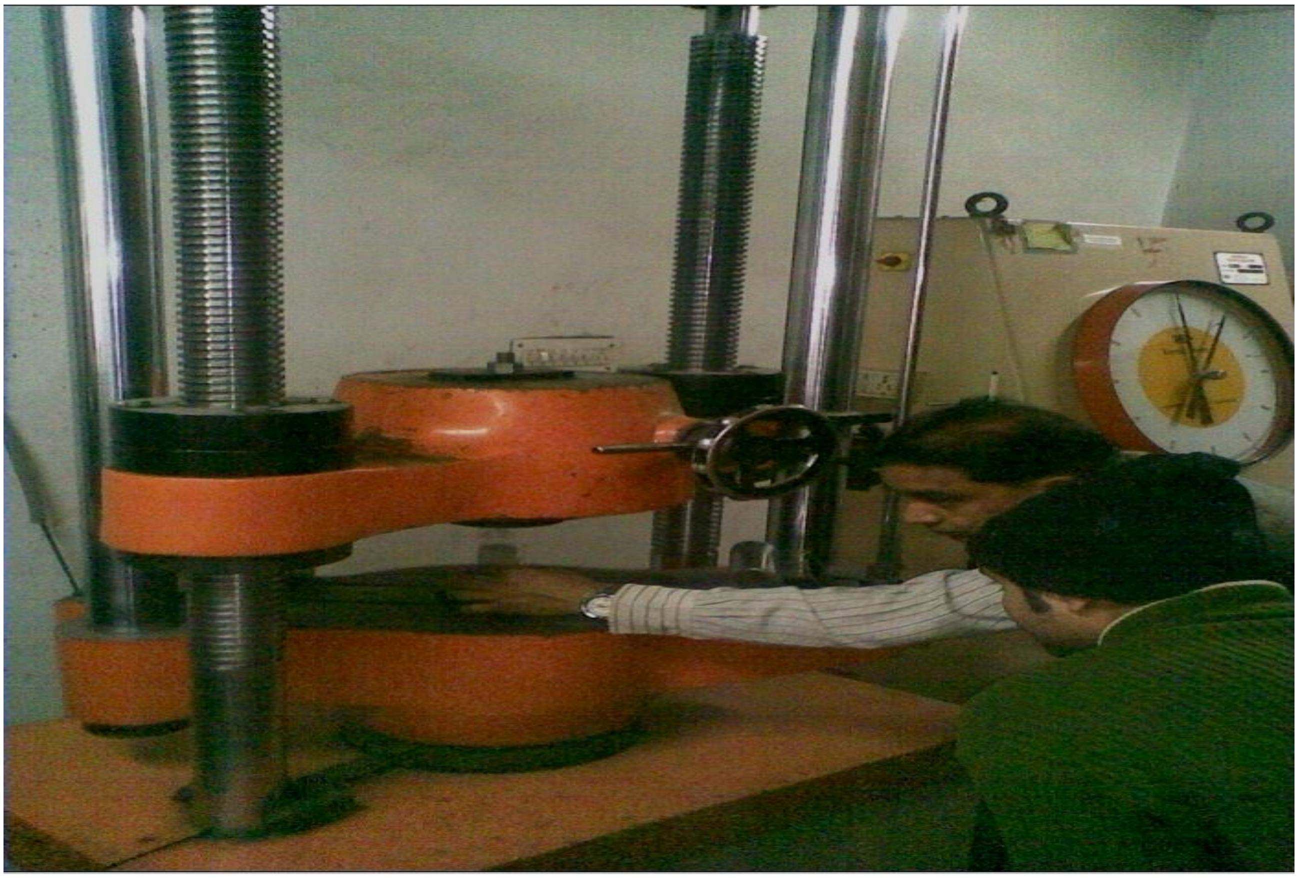

Photo: 6 Placing of specimen on uniaxial compression testing

Photo: 7 Specimen during testing on uniaxial compression testing machine

Photo: 8 Observation of uniaxial compression machine at failure of specimen .

Photo: 9 Specimen at failure.

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