Study on Strength and Deformation Characteristic of Jointed Rock Mass

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE

REQUIREMENTS FOR THE DEGREE OF

Bachelor of Technology

in

Civil Engineering

By

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Department of Civil Engineering

National Institute of Technology

Rourkela

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CERTIFICATE

This is to certify that the thesis entitled "**Strength and Deformation Characteristic of Jointed Rock Mass"** submitted by **ASHISH AGRAWAL (Roll No 10501001) in** partial fulfillment of the requirements for the award of Bachelor of Technology degree in Civil Engineering at the NATIONAL INSTITUTE OF TECHNOLOGY, Rourkela, deemed university is an automatic work carried by him under my supervision and guidance.

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ABSTRACT

Several massive, complicated and difficult to design structure are under construction or planning stages under very complex geological condition in India and around the world. Even small variation in appraisal and design can cost millions. Hence initial development of understanding under control condition is very important and desirable for characteristic and prediction of behavior. It is essential to have a clear understanding of strength and deformation behavior of jointed mass for realistic analysis and rational designing of engineering structure .The most important factors which govern the strength of rock mass type are type of rocks, bedding planes, stress condition, presence of cracks and fissures and nature of joint surfaces, presence of minerals in bedding planes also play an important role in the strength and deformation behavior of jointed rock mass. 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

Considering the importance of this study the experimental study has been under taken to determine the strength and deformation behavior of jointed rock mass. Models will be prepared using Plaster of paris, cement mad mica and different degrees of anisotropy have been induced by making joints in them varying from 0 to 90 degree.

The specimen will be tested under direct shear, uniaxial compression to determine the various parameters.

Chapter 1

INTRODUCTION

INTRODUCTION

Natural geological conditions are usually complex . In India the topography is varied and complex. Rocks are taken as separate field of engineering and efficient from engineering geology. It not only deals with rocks as engineering materials but it also deals with changes is mechanical behavior in rocks such as stress, strain and movement in rocks brought in due to engineering activities .it is also associated with design and stability of underground structure in rock. Rock itself may be homogenous but when we consider rock mass over which we plan our construction ,may behave altogether in different manner due to its defects in the masses such as jointing , bedding planes, fissiures ,cavities and other discontinuities .

To predict the behavior of rock mass to nearest value, ”in-situ” tests are done but these tests are very very expensive . in such cases modeling is fixed and this is very important .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 un predictable nature of modes of failure

Nowhere in the world does rock exist in complete intact state they all contain discontinuities.

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 shear strength of jointed rock mass depends on the type and origin of discontinuity, roughness, depth of weathering and type of filled material . The strength behavior of rock mass is governed by both intact rock properties and properties of discontinuities. The strength of rock mass depends on several factors as follows.

1. The angle made by the joint with the principle stress direction ( ).
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 ant joint rock mass strength with joint factor j_f and other factors.

Chapter 2

REVIEW OF LITERATURE

REVIEW OF LITERATURE

2.1 Rock for engineers

A better definition of rock may now be given as granular, aelotropic , 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. Reliable characterization of the strength and deformation behavior of jointed rocks is very important 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.

2.2Research till date.

Einstein and Hirschfeld (1973) and Einstein et al. (1970) 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 forsliding 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 (1984) 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 depende.nt 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 (1987) 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

$J_f = J_n/n \times r$

Where

J_n = 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 .

2.3 Uniaxial compressive strength

The uniaxial compressive strength of rock mass is represented is a non dimensional form as the ratio of compressive strength of jointed rock and that of intact rock .The uniaxial compressive strength ratio is expressed as

1 $\sigma_{cr}=\sigma_{cj}/\sigma_{ci}$

Where

σ_{cj} = uniaxial compressive strength of jointed rock

σ_{ci} = uniaxial strength of intact rock

The uniaxial compressive strength of the experimental data should be plotted against the joint factor .The joint factor for the experimental specimen should be estimated based on the joint orientation ,strength and spacing . Based on the statistical analysis of the data empherical relationship for uniaxial compressive strength ratio as function joint factor( ) are derived.

2.4 Elastic Modulus

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

$E_r = E_j / E_i$

Where

E_j =is the tangent modulus of jointed rock

E_i =is the tangent modulus of intact rock

2.5 Rock and rock mass

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

2.6 Intact rock mass

Strength of intact rock mass

Strength of intact rock mass mainly by following factors

(1)Geological

(2)Lithiological

(3)Physical

(4)Mechanical

(5)Environmental factors

When a rock is on the earth surface there is no confining pressure. If the rock mass is present below the earth surface, confine pressures on the strength of the rock has been investigated extensively, various investigation are conducted to study the influence of confining pressure show a non linear variation of strength with confining pressure .An important behavior under uniaxial condition is the change in behavior brittle to ductile nature at confining pressure.

Factors affecting intact rock strength

Table 2.1

Geological	Lithological	Physical	Mechanical	Environmental
Geological age	Mineral composition	Density	Specimen preparation	Moisture content
Weathering and other alternatives	Cementing material	Specific gravity	Specimen geometry	Nature of pore fluids
	Texture and fabric	Porosity	End contact	Temperature
	Anisotropy		Type of testing machine	Confining pressure

Effect of confining pressure, temperature, rate of loading

Other than the situ condition there are so many factors which effect the strength of intact rocks. The final summary of these factors are

1.Confining pressure increases the strength of the rock and the degree of post yield axial strain hardening these effects diminishes with increasing pressures.

2. At low confining pressure there is increasing dilation which reduces at higher confining pressure until a highest of 400 MN/m^2 .

3. The strength of the rock decreases with the increase of temperature the effect being different on different rocks.

4. The effect of pore water pressure depends on the porosity of rocks, viscosity of the pore fluid , specimen size and rate of straining, usually increase of pressure decreases strength .

5. Usually the strength increase with the rate of loading ,but here opposite cases hav been observed .

Rock discontinuity

Faults, joints, bedding planes, fractures, fissures are widespread occurance in rocks encounted in engineering practice. Discontinuities play a major role in controlling the engineering behavior of rock mass.

The earthquake takes a major part in discontinuity. The engineering behavior of rock mass as per Piteau (1970) depends upon the following.

1.Nature of occurance

2.Orientation and position in space

3.Continuity

4.Intensity

5.Surface geometry

The form of index adopted to describe discontinuity intensity is of the following type

1) Measurement of discontinuities per unit volume of rock mass(Skerpton 1969)

2) Rock quality design (RQD) technique(Deere 1964)

3) Sacn line survey technique(piteau1979)

4) A linear relationship between RQD and average number of discontinuities per meter ()

was suggested by Bieniawaki(1973)

2.7 Jointed rock properties

Joint rock 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 stuies by (Goldstien 1966, Walker1971, Lma1971).

To understand the strength behavior of jointed rock specimen, arora 1987 introduced a factor (Jf) defined by the expression as

$$J_f = J_n / n \times r$$

Where Jn = no. of joints per meter length

n = joint inclination parameter which is a function of joint orientation

r = roughness parameter(depends on joint condition)

Table2.2

The value of inclination parameter (Ramamurty,1993)

Orientation of joint β^o	Inclination parameter n
0	0.810
10	0.46
20	0.105
30	0.046
40	0.071
50	0.306
60	0.465
70	0.634
80	0.814
90	1.00

2.8 Joint roughness

Joint roughness is of paramount importance to the shera 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 shear 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.

Patton 1966 suggested the following equation for friction angle(ϕ_e)along the joints

$\Phi_e = \Phi_u + i$

Where

Φu is the friction angle of smooth joint

i is the inclination of asperity

according to Patton joint roughness has been considered as a parameter that effectively increases the friction angle Φ_r which is given by the relation below

$\tau = \sigma_n \tan(\Phi_r + i)$ for small values σ_n

$\tau = c + \sigma_n \tan\Phi_r$ for large values of σ_n

where

τ = Peak shear strength of the joint.

σn=normal stress on the joint

Φr = 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 alonge the sliding joint Barton (1971)suggested the following relationship

$\tau/\sigma_n = \tan[(90 - \Phi_u(d_n/\Phi_u) + \Phi_u]$

where

d_n is the peak dilation angle which is almost equal to $10 \log_{10}(\sigma_c/\sigma_n)$

σ_c is the uniaxial compressive strength

Joint roughness coefficient

The empirical approach proposed by Barton and Choobey (1977) 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 0 from roughest to smoothest surface respectively.

Scale Effects

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

Bandis etel (1981) 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 result showed that both the JRC and JCS reduced to the changing stiffness of rock massas 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

$$JRC = \alpha - \phi_r / \log(JCS/\sigma_{n0})$$

Where

α=tilt angle

σ_{n0}=Normal stress when sliding occures

Dilation

Dilation is the relative moment between two joint faces along the profiles. For rocks, Fecker and Engers (1971) indicated that if all the asperities are over- overdden and there is shearing off, the dilation (hn) for any displacement can be given as

$h_n = n_i \tan d_n$

where

n_i is the displacements (in steps of length)

dn is the max angle between the reference plane and profile for base length

Dilation can be represented in form of dilation angle as follows

$\Delta d = \Delta v / \Delta h$

Where Δv is the vertical displacement perpendicular to the direction of the shear force, Δh is the horizontal displacement in the direction of the applied shear force

Peak dilation angle of joints was predicted by Barton and choobey(1977) based on the roughness component which includes mobilized angle of internal friction and JRC,residual friction angle and normal stress.

Barton (1986) predicted that dilation begins when roughness is mobilized and dilation declines as roughness reduces.

2.9 Strength criterion for anisotropic rocks

Strength criterion

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 formulae have been proposed like by Navier –coulomb and Griffith criteria.it is clearly shown that the strength for all rocks is maximum at $\beta=0^o$ or90^o and is minimum at $\beta=20^o$ or 30°.

Influence of single plane of weakness

In a laboratory test the orientation of the plane of weakness w.r.t. principal stress directions remains unaltered. Variation of the orientation of this plane can only be achieved by obtaining cores in different directions. In field situation either in foundation of dams around underground or open excavation the orientation of joint system remainsstationary but the directions of principal stress rotate resulting in a change in the strength of rock mass. Jaegar and Cook (1979) developed a theory to predict the strength of rock containing a single plane of weakness.

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

where ϕ = friction angle

β = Angle of inclination of plane of weakness with vertical failure by sliding will occur for angles 0 to 90

2.10 INFLUENCE OF NUMBER AND LOCATION OF JOINTS

For plaster of paris representing weak rock, the variation of number of joints per meter length (jn, joint frequency)with the ratio of uniaxial strength of joint and intact specimens under unconfined compression has been presented in fig . The ratio of module of jointed specimen to that of intact specimen is also included . the reduction of strength is observed to be lower than the modulus values with joint frequency.

The location of a single joint w.r.t. loading surface defined by dj= Dj/B(ratio of depth of joint Dj to the width or dia of loaded area) greatly influences the strength of rock when the joint is placed very close the strength of joint away from the loading face the strength of jointed rock increases and attain a value the same as that of the intact rock when the joint is located at about 1.2 B below the loading surface. The modulus of jointed rock is higher than that of intact rock so long as the joints within the depth equal to width of loaded arrears. The stiffness of the rock is highest when the joint is close to loading face contrary to the strength influence of location of a joint on the stiffness continuous to decrease even up to depth twice the width of loaded area.

2.11 PARAMETERS CHARACTERIZING TYPE OF ANISOTROPY

Broadly three possible parameters define the concept of strength anisotropy of rocks. These are

1) Location of maximum and minimum compressive strength ( ) in the anisotropic curve in terms of the orientation angle ().

2) The value of uniaxial compressive strength at these orientation

3) General shape of anisotropy curve.

Rock exhibit maximium strength at 0o or 90oand minimum strength between 20o to 40o(Arora and Ramamurty 1987) has introduced an inclination parameter (n) to predict the behavior of different orientation of joints in rock behavior. The relationship between n and β is given on the experiment on plaster of paris specimen. The variation n and β was observed to be similar to the variation of uniaxial compressive strength ratio σ_{cr} with the value for the corresponding β values.

2.12 DEFORMATION BEHAVIOUR OF ROCK MASS

Deformation behavior of jointed rock is greatly influenced by deformability along the joints. In addition to significant influence on strength of the rocks joints will generally lead to marked reduction in the deformation modulus which is another parameter of interest to the designer. In situ testing such as plate load and radial jacking have been generally paerformed in practice for determining the rock mass module values. The deformation characteristic of a rock mass depends on the orientation of joint with respect to the loading direction the insitu stress condition the spacing of joints and the size of loading region .

Equation given by Konder (1963)

$$(\varepsilon_1)/(\sigma_1 - \sigma_3) = a + b\varepsilon_1$$

Where ε1 = axial strain

a= reciprocal strain modulus

b= reciprocal of asymptotic value of deviation stress

Chapter 3

LABORATORY INVESTIGATION

LABORATORY INVESTIGATION

3.1 MATERIALS 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 plaster of paris 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 avalibility. 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 material for modeling in Geotechnical engineering.

3.2 PREPARATION OF SPECIMEN

Plaster of paris is procured. This plaster of paris powder produced by pulverizing partially burnt gypsum is dully white in colour with smooth feel of cement. the water content at which maximum density is to be arrived is found out by conducting number of trial test with different percent of distilled water. The optimum moisture content was found out to be 30% by weight.

For prepratin of specimen 200 gm of plaster of paris is mixed thoroughly with 60.0 cc(30% by weight)water to form a uniform paste. The plaster of paris specimens prepared by pouring the plaster mix in the mould is vibrated in the vibrating table machine for approximately 2 min for proper compaction and to avoid presence of air gaps. After it is allowed to set for 5 min. after hardening the specimen was extruded manually from the mould by using an extruder. The polished specimens are then kept at room temperature for 48 hours.

3.3 CURING

After keeping the specimens in oven they are placed inside desiccators in concentrated sulphuric acid. This is done mainly to maintain the relative humidity in range of 40% to 60%. This

humidity is maintened constant in desiccators by keeping a solution of concentrated sulphuric

acid of 47.7cc with distilled water 52.3cc. specimens are allowed to cure inside the desiccators till constant weight is obtained (about 15 days). Before testing each specimen of plaster of paris obtaining constant weight dimensioned to L/D = 2:1, D = 38 mm at L = 76 mm.

3.4 MAKING JOINTS IN SPECIMEN

The following instruments are used in making joints in specimen

1) 'V' block

2) Light weight hammer

3) Chisel

4) Scale

5) Pencil

6) Protractor

Two longitudinal lines are drawn on the specimen just opposite to each other. At the centre of the line the desired orientation angle is marked with the help of a protractor. Then this marked specimen is placed on the 'V' block and with the help of chisel keeping its edge along the drawn line, hammered continuously to break along the line. It is observed that the joints such formed comes under a category of joint. The uniaxial compressive strength test are conducted on intact specimen, jointed specimen with single and double joints to know the strength as well as deformation behavior of intact and jointed rocks.

3.5 EXPERIMENTAL SETUP AND TEST PROCEDURE

In study specimen are tested to obtain their uniaxial compressive strength, deformation behavior and shear parameter. The tests conducted to obtain these parameter are uniaxial compression test and direct shear test. These test are carried as per ISRM and IS code. The uniaxial compressive strength were conducted on conventional strain controlled machine at a strain rate of 1.25 mm/

min.

The direct shear test is conducted to determine (roughness factor) joint strength r = tan ϕ_j in order to predict the joint factor J_f(Arora 1987) these test were carried out on conventional direct shear test apparatus with modification in direct shear boxas the specimen is cylindrical and the box is cuboidal.two identical wooden blocks of sizes 59X59X12 mm each having circular hole dia of 39 mm at the centre were inserted into two halves of shear box the specimen is then place inside the shear box.

3.6PARAMETER STUDIED

The main objective o the experimental investigation is to study the following aspects.

1) The effect of joint factor in the strength characteristic of jointed specimen.

2) The deformation behavior of jointed specimen.

3) The shear strength behavior of plaster of paris.

Uniaxial compressive strength test were conducted on intact specimen, jointed specimen with single and double joints to know the strength as weel as the deformation behavior of intact and jointed rocks.

The specimens are tested at different confining pressure at 0.1,0.2,0.3,0.4 Mpa respectively for different orientation angle such as 10,20,30,40,50,60,70,80,90 degree and for intact specimen

The jointed specimen were placed inside a rubber membrane before testing U.C.S to avoid slippage alonge the jointed just after the application of load. Direct shear test were conducted in jointed specimen for plaster of paris to know C_j and ϕ_j at 0.1,0.2,0.3 Mpa

TYPES OF JOINTS STUDIED

Table 3.1

Uniaxial compression test

Types of joints	1j-0	1j-10	1j-20	1j-30	1j-40	1j-50	1j-60	1j-70	1j-80	1j-90
Single joint	3	3	3	3	3	3	3	3	3	3
Double joint	0	3	3	3	3	3	3	3	3	3

STANDARD TABLE FOR REFERENCES

Direct shear test calibration chart

Proving ring No-66111

Least count = 0.0001 inch

Table 3.2

Load in pounds	Deflection in inches
0	0
50	0.0059
100	0.01165
150	0.01745
200	0.0.0236
250	0.0284
300	0.0352
350	0.04115
400	0.04715
450	0.05325
500	0.0594

Uniaxial compression test

Calibration chart

Proving ring No-PR20 KN. 01002

Value of each smallest division – 0.02485 KN (24.844N)

Dial gauge least count = .002mm

Table 3.3

Force applied in KN	Deflection of dial gauge
0	0
2	80.5
4	161.0
6	241.5
8	322.0
10	402.5
12	483.0
14	564.0
16	644.0
18	725.0
20	805.0


Contass
LOAD
STOP
UNLOAD


TEQIP

CHAPTER 4

RESULTS

RESULTS

The experimental findings have been compared with the empirical relationship given by Arora(1987)

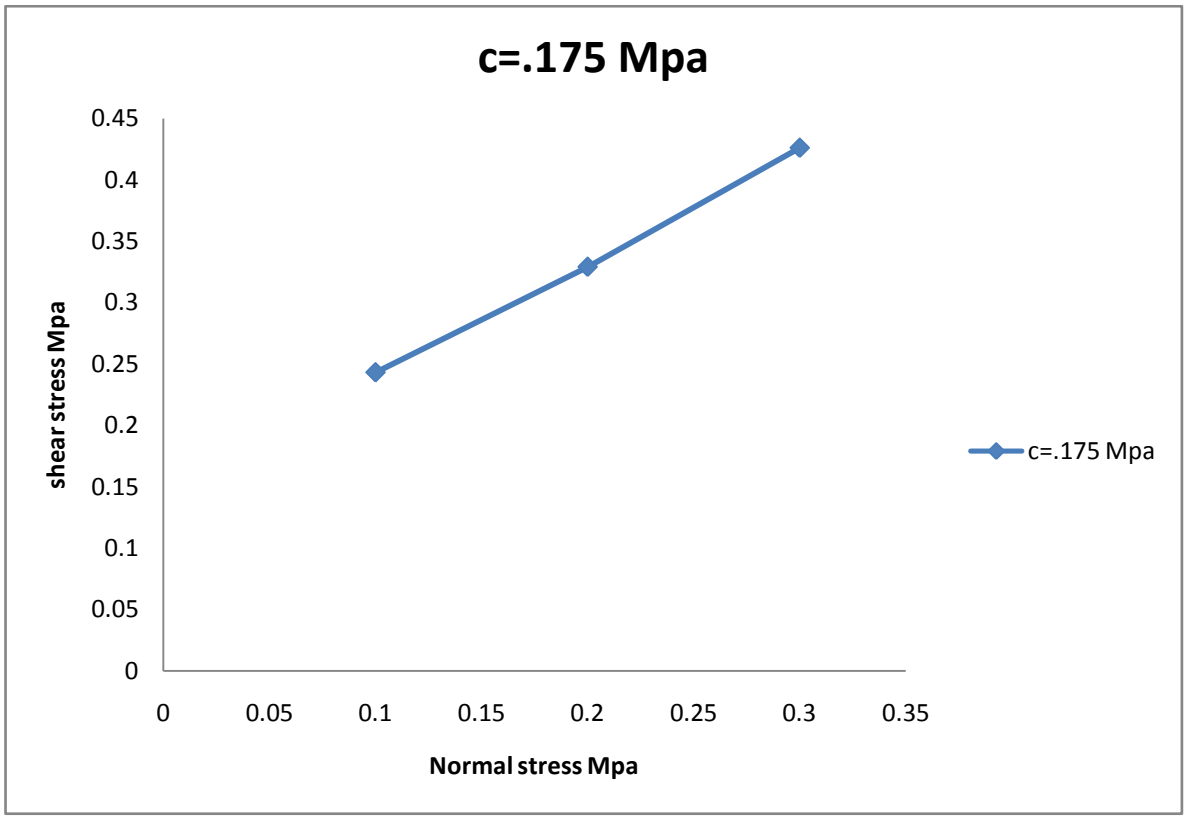
4.1Roughness parameter

The roughness parameter(r) is the tangent value of the friction angle ϕ_j of the joint was obtained from the direct shear test conducted at different normal stresses. The variation of shear for different normal stress is

Table 4.1

Normal stress Mpa	0.1	0.2	0.3
Shear stress Mpa	.243	.329	.426

Fig 4.1



Normal stress vs Shear stress($\Phi=40^o$)

Values of stress and strain for intact specimens

Length of specimen =76 mm

Diameter of specimen =38mm

Cross section area of specimen =1134mm^2

Strain rate = 0.5 mm/minute

Table 4.2

Axial strain(Ca %)	Uniaxial compressive strength Mpa
0	0
.62	3.354
1.19	5.417
1.35	5.782
2.02	6.763
2.67	8.467
3.41	10.974

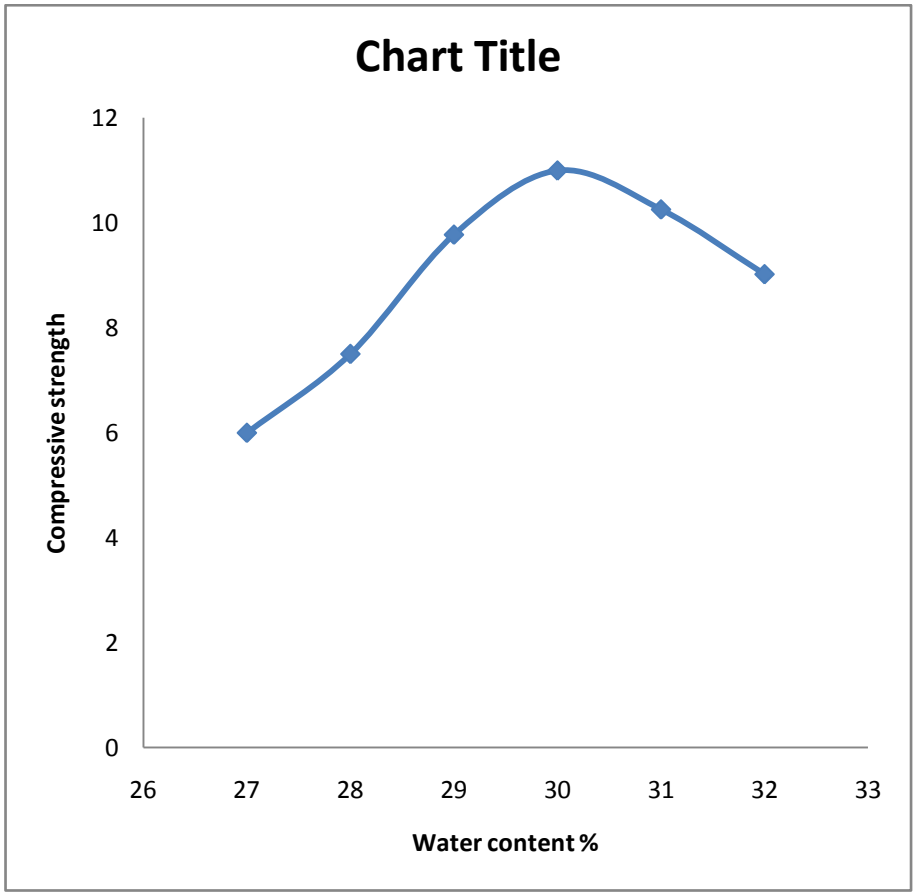
The modulus of elasticity for intact specimen (Eti) has been calculated at50% of σ_{ci} value to account the tangent modulus. The value of Mn for the intact specimens was found to be 0.571 x 10^3 Mpa.

The variation of stress as obtained in uniaxial compression test for the intact specimen of Plaster of Paris for different values of water content is illustrated below:

Table 4.3

Water content %	27	28	29	30	31	32
Uniaxial compressive strength	5.989	7.492	9.764	10.986	10.245	9.011

Fig4.2



Water content vs compressive strength

The optimum value of uniaxial compressive strength (σ_{ci}) evaluated from the above test was found to be 10.974 MPa.

4.2Jointed Specimen

The uniaxial compressive strength for jointed specimen (σ_{cj}) is evaluated .After obtaining the valueof σ cj it was observed that the strength of Plaster of Paris was minimum for orientation angle β=30°.

The values of σ cr with different joint orientation angles (β) were obtained by using the relationship:

$$\sigma_{cr} = \sigma_{cj} / \sigma_{ci}$$

The value of joint factor (Jf) has been evaluated by using the relationship:

$$J_f = J_n/(n.r)$$

Arora(1987) has suggested the following empirical relationship between J_f and σ_{cr} :

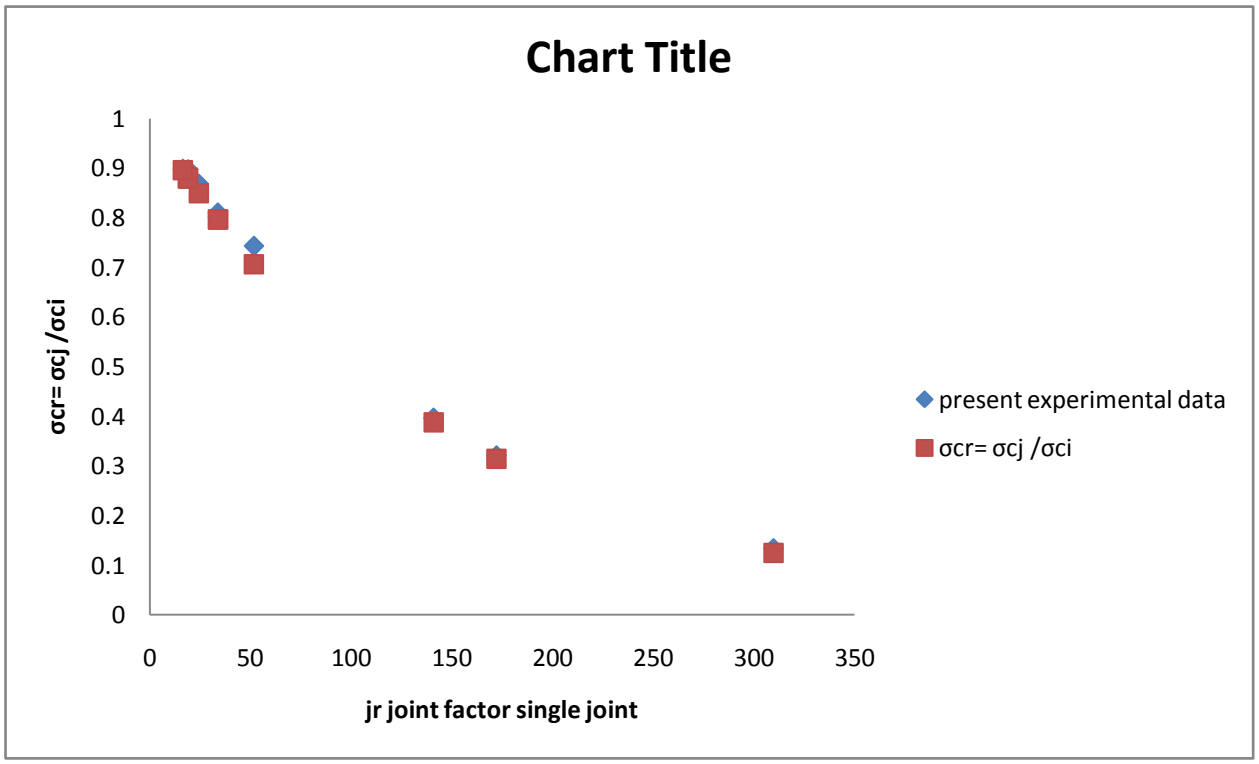
$$\sigma_{cr} = e^{-.008Jf}$$

The variation between σ_{cr} and J_f is illustrated for single joints. Also a comparative study of the Experimental tests and the empirical relationship given by Arora is provided.

Table 4.4

Joint in degree	Jn	n	$J_f=j_n/nxr$	(σ_{cj})Mpa	$\Sigma_{cr}=\sigma_{cj}/\sigma_{ci}$	Predicted Arora(1987)
0	13	0.82	18.89	9.854	0.897	0.88
10	13	0.46	33.68	8.898	0.81	0.797
20	13	0.11	140.86	4.361	0.397	0.388
30	13	0.05	309.8	1.472	0.134	0.125
40	13	0.09	172.16	3.526	0.321	0.315
50	13	0.3	51.64	8.163	0.743	0.707
60	13	0.46	33.68	8.799	0.801	0.797
70	13	0.64	24.21	9.535	0.868	0.850
80	13	0.82	18.89	9.776	0.890	0.880
90	13	0.95	16.31	9.931	0.904	0.896

Fig 4.3



Variation of σ_{cr} vs j_f single joint

Table 4.5

Joint in degree	Jn	n	$J_f=j_n/nxr$	(σ_{cj})Mpa	$\Sigma_{cr}=\sigma_{cj}/\sigma_{ci}$	Predicted Arora(1987)
30-30	26	0.05	619.7	0.406	0.037	0.015
40-40	26	0.09	344.32	1.527	0.139	0.099
50-50	26	0.3	103.29	6.251	0.569	0.499
60-60	26	0.46	67.36	7.382	0.672	0.616
70-70	26	0.64	48.42	8.184	0.745	0.722
80-80	26	0.82	37.79	8.887	0.809	0.776
90-90	26	0.95	32.62	9.129	0.831	0.803

Maximum value of σ_{cr} is observed at 1j-90°and minimum at 1j-30°.

Fig 4.3

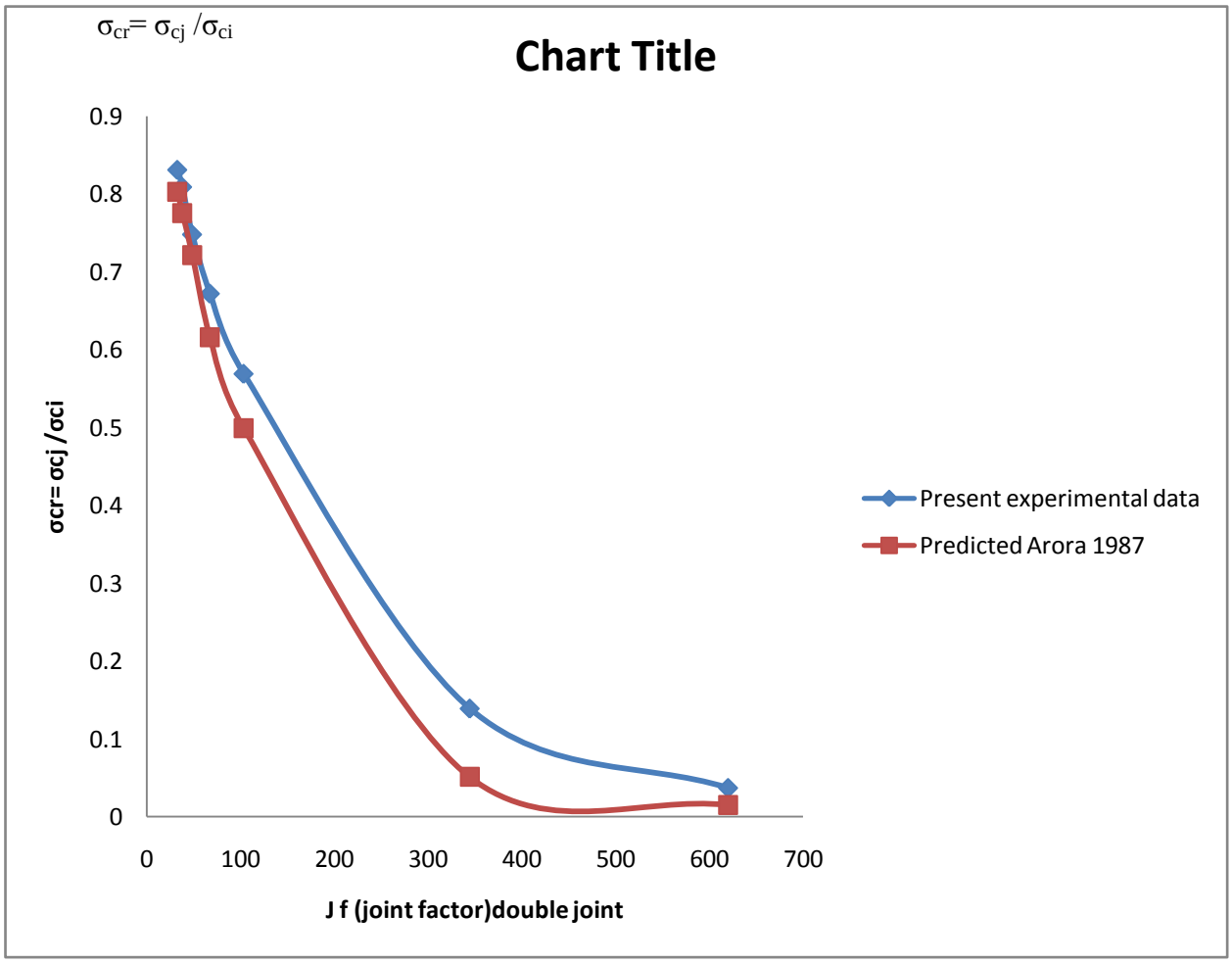
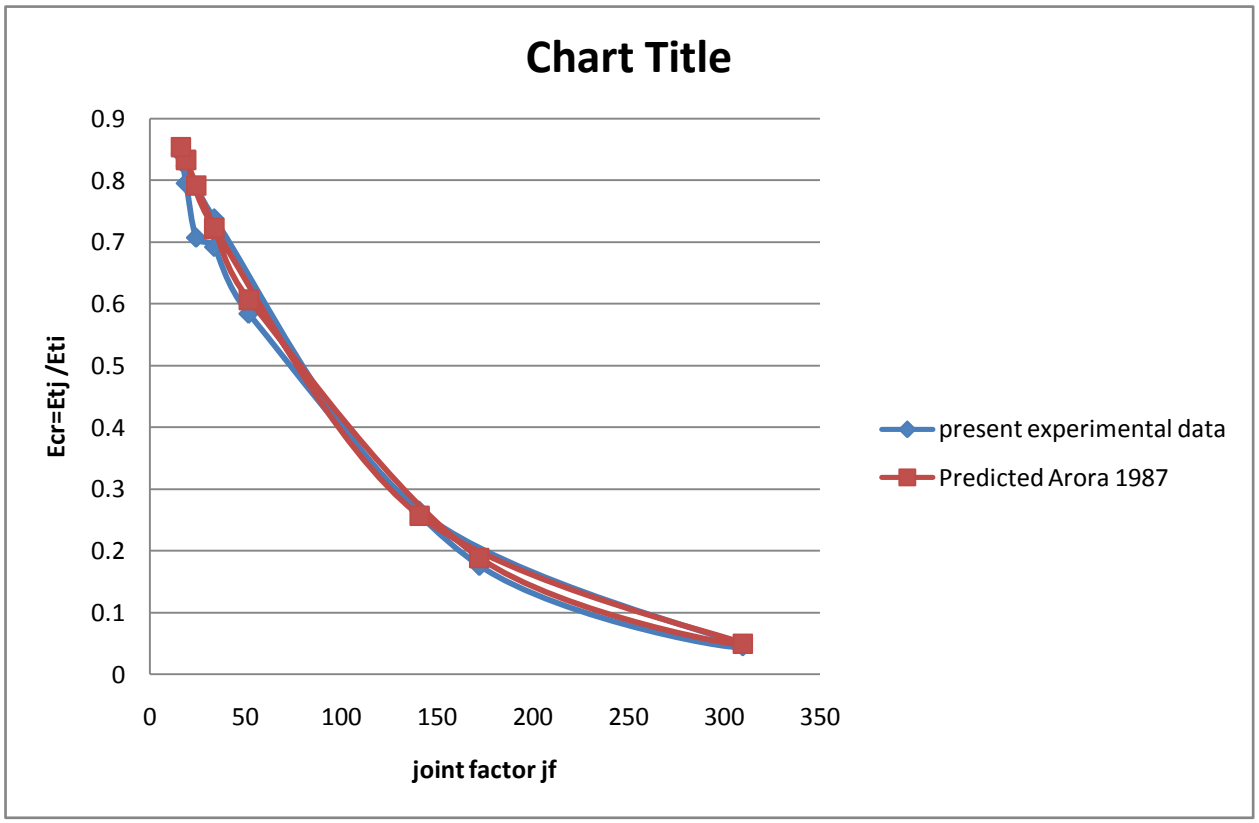

σcr= σcj /σci
Chart Title
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
σcr= σcj /σci
0
100
200
300
400
500
600
700
J f (joint factor)double joint
Present experimental data
Predicted Arora 1987

Table 4.6

Joint in degree	Jn	n	$J_f=j_n/nxr$	(E_{tj})Mpa	$E_{cr}=E_{tj}/E_{ti}$	Predicted Arora(1987)
0	13	0.82	18.89	0.469	0.823	0.833
10	13	0.46	33.68	0.421	0.738	0.722
20	13	0.11	140.86	0.151	0.265	0.257
30	13	0.05	309.8	0.026	0.046	0.050
40	13	0.09	172.16	0.100	0.176	0.189
50	13	0.3	51.64	0.333	0.584	0.607
60	13	0.46	33.68	0.395	0.692	0.723
70	13	0.64	24.21	0.403	0.707	0.792
80	13	0.82	18.89	0.453	0.795	0.833
90	13	0.95	16.31	0.481	0.843	0.854

Fig 4.4



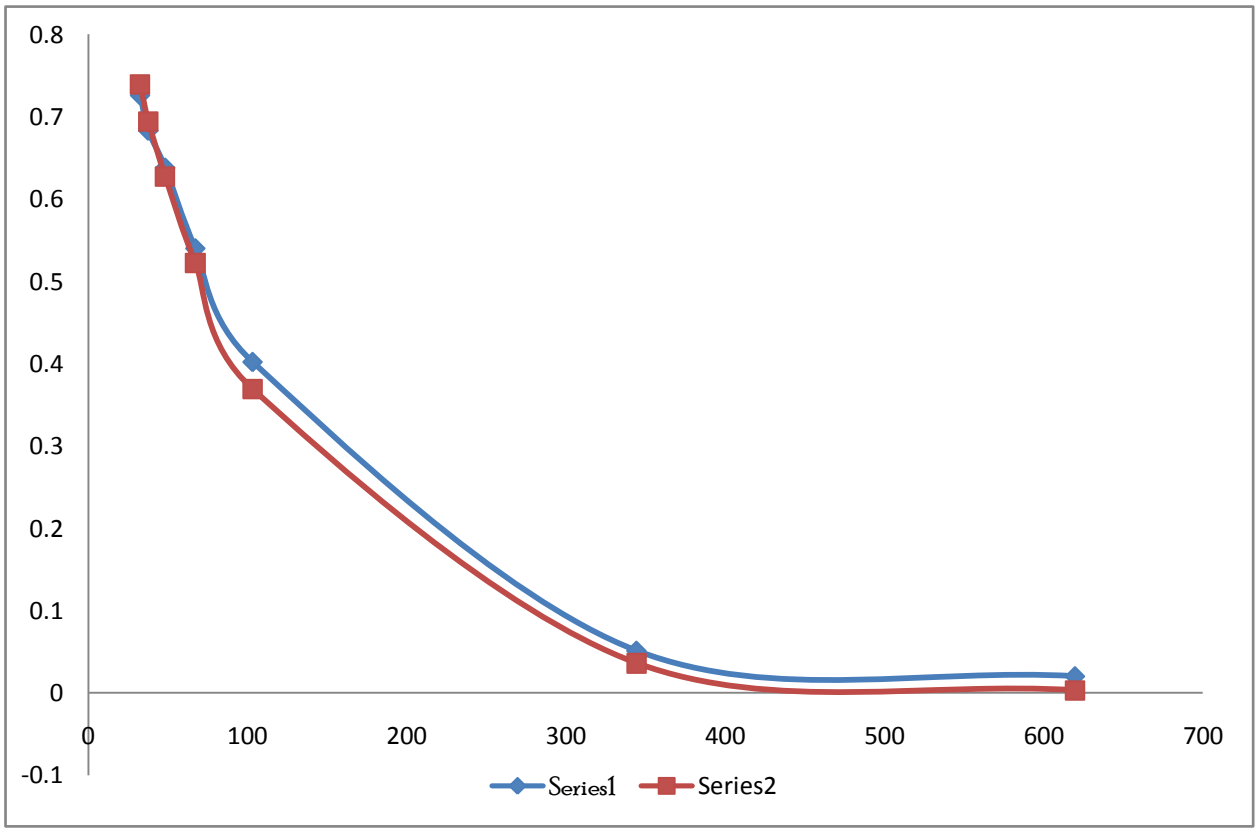
Variation of E_r vs J_f for intact joint (single joint)

Values of Er experimental and predicted

Table 4.7

Joint in degree	Jn	n	$J_f=j_n/nxr$	(E_{tj})Mpa	$E_{cr}=E_{tj}/E_{ti}$	Predicted Arora(1987)
30-30	26	0.05	619.7	0.011	0.020	0.003
40-40	26	0.09	344.32	0.029	0.051	0.036
50-50	26	0.3	103.29	0.229	0.402	0.369
60-60	26	0.46	67.36	0.308	0.54	0.522
70-70	26	0.64	48.42	0.364	0.638	0.627
80-80	26	0.82	37.79	0.389	0.683	0.694
90-90	26	0.95	32.62	0.414	0.726	0.739

Fig 4.5


0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
0
100
200
300
400
500
600
700
Series1
Series2

CHAPTER 5

CONCLUSION

5.1 CONCLUSION

On the basis of current experimental study on the intact and jointed specimen of plaster of Paris the following conclusions are drawn:

1. The uniaxial compressive strength of intact specimen of plaster of Paris is found to be 10.986Mpa.
2. The strength of jointed specimen depends on the joint orientation β with respect to the direction of major principal stress. The strength at β=30º is found to be minimum and the strength at β = 90º is found to be maximum.
6. There is not much variation between the present experimental results and those obtained from the empirical formula given by Arora and Ramamurthy.
3. As the number of joints increase the uniaxial compressive strength decreases.
4. The compressive strength is more when the double joints are made at angle of orientation at 60º – 60º to than at 90 º - 90º.
5. The values of Modulus ratio (Er) also depends on the joint orientation β. The modulus ratio is least at 30º.

6.With increase in joint factor(j_f) the strength decreases.

5.2 SCOPE OF FUTURE WORK:

1. The effect of temperature, confining pressure and rate of loading on the strength characteristics can be studied.
2. Studies can be made by introducing multiple joints in varying orientation.
3. Strength and deformation behaviour of jointed specimens can be studied under triaxial conditions.
4. Similar study can be carried with gouge filled joint.
5. Investigation can be done on same specimen with joints at different angles.
6. Numerical model can be developed.

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