CHAPTER - 1
INTRODUCTION

1.1 GENERAL

Engineers are actively engaged in search for solutions to complex problems involving the behaviour of rock masses. Stability analysis of a rock mass is an important and complicated problem related to the safety of engineering buildings. One of the major tasks of engineering geomechanics is to evaluate the rock mass stability both qualitatively and quantitatively. The ultimate strength and deformation of jointed rock mass are important parameters that designers look for in selecting sites for foundations of civil structures in rocks. In nature rock is exists as a rock mass. Rocks are not as closely homogeneous and isotropic as many other engineering materials. Rock masses are heterogeneous and discontinuous medium with fissures, fractures, joints, bedding planes, and faults. These discontinuities may exist with or without gouge material. The strength of rock masses depends on the behaviour 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 has a significant importance from the stability point of view. Reliable characterization of the strength and deformation behaviour of jointed rocks is very important for safe and economical design of various types of civil structures such as arch dams, bridge piers, and tunnels.

The relation between joint factor (J_f), Uniaxial compressive strength ratio (σ_{cr}) and Uniaxial compressive strength of jointed rock (σ_{cj}) is of paramount importance for such study. The best estimate of the design parameters can only be made through large size field testing of the mass and loading it up to failure. It is, however, extremely difficult, if not impossible, to stress a large volume of jointed mass in the field up to ultimate failure. A better alternative is to get the deformability characteristics by stressing a limited area of the mass up to a certain stress level and then relate the ultimate strength of the mass of the laboratory uniaxial compressive strength (UCS) of the rock material.

Laboratory rock testing is performed to determine the strength and elastic properties of intact specimens and the potential for degradation and disintegration of the rock material. The derived parameters are used in part for the design of rock fills, cut slopes, shallow and deep foundations, tunnels, and the assessment of shore protection materials (rip-rap). Deformation and strength properties of intact specimens aid in evaluating the larger-scale rock mass that is significantly controlled by joints, fissures, and discontinuity features (spacing, roughness, orientation, infilling), water pressure and ambient geostatic stress state.

1.2 AIMS AND OBJECTIVES OF STUDY

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

(1.) The effect of different size of intact specimens of plaster of paris on the uniaxial compressive strength.

(2.) The effect of different gouge material on the uniaxial compressive strength of jointed specimens.

(3.) The effect of plane of orientation of joint on the uniaxial compressive strength of jointed specimens.

In view of the above, uniaxial compressive test were done for single joint specimens with different gouge material at various inclination i.e. $\alpha = 0^0$, 30^0,45^0 and 60^0.

1.3 In this present chapter we discussed about the general introduction. Why this study is important and aims and objectives of the study. In the next chapter we discuss about the literature important for this study.

CHAPTER - 2
LITERATURE REVIEW

In order to fulfill the aims and objectives of the present study following literatures have been reviewed.

2.1 ROCK

Rock is a naturally occurring solid aggregate of minerals. Rock may be defined as a granular, allotropic, heterogeneous technical substance which occurs naturally and which is composed of grains cemented together by mechanical bond but ultimately by atomic, ionic and molecularly within the grains. Rocks are generally classified by mineral and chemical composition, by the texture of the constituent particles and by the processes that formed them. Thus by an engineer rock is a firm and coherent substance which normally cannot be excavated by general 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. Although civil and mining engineers have worked with rock since pre-historic times engineering knowledge in this area has, until recently, been largely uncoordinated with each individual or group of engineers developing their own methods and experience outside the framework of an established academic and professional discipline. This state of affairs existed until approximately the time of the first congress of the then newly formed International Society for Rock Mechanics (ISRM) held in Lisbon, Portugal, in 1966. The majority of rock masses, in particular those within a few hundred meters from the surface, behave as dis-continua, with the dis-continuities largely determining the mechanical behaviour. It is therefore essential that both the structure of a rock mass and the nature of its discontinuities are carefully described in addition to the lithological description of the rock type.

(i) JOINT: A break of geological origin in the continuity of a body of rock along which there has been no visible displacement. A group of parallel joints is called a set and joint sets intersect to form a joint system. Joints can be open, filled or healed. Joints frequently formed parallel to bedding planes, foliation and cleavage and may be termed bedding joints, foliation joints and cleavage joints accordingly.

(ii) FAULT: A fracture or fracture zone along which there has been recognizable displacement, from a few centimeters to a few kilometers in scale. The walls are often striated and polished (slickensided) resulting from the shear displacement. Frequently rock on both sides of a fault is shattered and altered or weathered, resulting in fillings such as breccia and gauge. Fault widths may vary from millimeters to hundreds of meters.

(iii) DISCONTINUITY: It is the collective term for most types of joints, weak bedding planes, weak schistocity planes, weakness zones and faults. The ten parameters selected to describe discontinuities in rock masses are defined below:

(a) **Orientation:** Attitude of discontinuity in space described by the dip direction (azimuth) and dip of the line of steepest declination in the plane of the discontinuity.

(b) **Spacing:** Perpendicular distance between adjacent discontinuities. Normally refers to the mean or modal spacing of a set of joints.

(c) **Persistence:** Discontinuity trace length as observed in an exposure may give a crude measure of the areal extent or penetration length of a discontinuity. Termination in solid rock or against other discontinuities reduces the persistence.

(d) **Roughness:** Inherent surface roughness and waviness relative to the mean plane of a discontinuity. Both roughness and waviness contribute to the shear strength. Large scale waviness may also alter the dip locally.

(e) **Wall strength:** Equivalent compression strength of the adjacent rock walls of a discontinuity may be lower than rock block strength due to weathering or alteration of the walls. An important component of shear strength is resulted if rock walls are in contact.

(f) **Aperture:** Perpendicular distance between adjacent rock walls of a discontinuity, in which the intervening space is air or water filled.

(g) **Filling:** Material that separates the adjacent rock walls of a discontinuity and that is usually weaker than the parent tock. Typical filling materials are sand, silt, clay, breccia, gauge, mylonite. Also include thin mineral coatings and healed discontinuities, e. g. quartz and calcite veins.

(h) **Seepage:** Water flow and free moisture visible in individual discontinuities or in the rock mass as a whole.

(i) **Number of sets:** The number of joint sets comprising the intersecting joint system. The rock mass may be further divided by individual discontinuities.

(j) **Block size:** Rock block dimensions resulting from the mutual orientation of intersecting joint sets, and resulting from the spacing of the individual sets. Individual discontinuities may further influence the block size and shape.

2.2 INTACT 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, 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 factor.

TABLE-1

FACTORS AFFECTING THE STRENGTH OF ROCK

Geological	Geological age, weathering and other alternatives
Lithological	Mineral composition, cementing material, 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 factors	Moisture content, nature of pore fluids, temperature, Confining pressure.

Goldstein et al. (1966) Uniaxial compression tests were conducted on composite specimens made from cubes of plaster of Paris and the following relationship is suggested by him,

$$\sigma_{cm} / \sigma_{ce} = a + b\ (I/ L)^{e}$$

Where, σ_{cm} = compressive strength of the composite specimen; σ_{ce} = compressive strength of the element constituting the block; L= length of the specimen; I = length of rock element; and a, b, and e=constants, where $e<1$ and $b = (1-a)$.

2.3 JOINTED ROCK MASS

Faults, joints, bedding planes, fractures, and fissures are widespread occurrence in rocks encounter 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.

- Nature of occurrence
- Orientation and position in space
- Continuity
- Intensity
- 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) Scan line survey technique (**Piteau, 1979**)

4) A linear relationship between RQD and average number of discontinuities per meter (**Bieniawaki, 1973)**

2.4 JOINT ROCK PROPERTIES

2.4.1 JOINT ROCK INTENSITY

The joint intensity is the number of joints per unit distance normal to the plane in a set. It influences the strength and deformation behaviour of the rock mass significantly.

Hayashi (1966) conducted uniaxial compression tests on the jointed specimens of plaster of Paris and found that the strength decreased with increasing number of joints.

Brown and Trollope (1970) carried out a series of triaxial compression tests on a block jointed systems using cubic blocks with different joint orientations and unjointed joint material. The mechanical behavior of the simplest block-jointed system was markedly different from the unjointed specimen; a power law was fitted to the test results. The difficulty involved in the application of the power law to practical problems is that it requires appropriate strength parameters for each rock mass to be determined experimentally. **Brown (1970)** reported triaxial compression tests on prismatic samples in which parallelopiped and hexagonal blocks were used to produce intermittent joint planes and simulate more complex and real practical behavior.

Lama (1974) conducted extensive tests by using model materials of different strengths to determine the influence of the number of horizontal and vertical joints on both deformation moduli and strength. He proposed the following equation based on his results:

$$\sigma_c \text{ or } E_d = k + (L/I)^V$$

Where σ_c = compressive strength, E_d = deformation modulus, k= strength of the specimen containing more than 150 joints, v = constant, L = length of specimen, I = lenth of intact rock element.

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 in 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 joints. Joint specimens with rough joint surface failed by shearing across the joint, by tensile splitting, or by a combination of thereof.

To understand the strength characteristics of jointed rock mass specimen, **Arora (1987)** introduced a factor (J_f) defined by the expression as:

$$\mathbf{J_f = J_n / n \times r}$$

Where J_n = no. of joints per meter length.

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

r = roughness parameter i.e. tan φ_j which depends on the joint condition.

The value of 'n' is obtained by taking the ration of log (strength reduction) at $\beta = 90^o$ to log (strength reduction) at the desired value of β. This inclination factor is independent of joint frequency. The joint strength parameter 'r' is obtained from a shear test along the joint. and is given as $r = \tau_j / \sigma_{nj}$, where τ_j is the shear strength along the joint and σ_{nj} is the normal stress on the joint. The values of 'n' and 'r' are given by **{Ramamurthy (1994) and Arora (1987)}** based on extensive laboratory testing.

2.4.2 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. **Einstein and Hirschfeld (1973)** 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 favourably inclined and increases if the joints are unfavourably inclined. The strength of a jointed specimen is the same as the intact specimen regardless of joint orientation or 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. On the basis of Mohr Coulomb equation, **Jaegar and Cook (1979)** 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.

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

TABLE-2
INCLINATION PARAMETERS ACCORDING TO ORIENTATION OF THE JOINT (Arora, 1992)

Inclination parameter,(n)	Orientation of joint, β in degrees from vertical
0.814	0
0.105	20
0.046	30
0.071	40
0.306	50
0.465	60
0.634	70
0.814	80
1.000	90

2.4.3 JOINT ROUGHNESS

Joint roughness is of paramount importance to the shear behaviour of 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 due 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,

$$\tau = \sigma_n.\tan(\Phi_r+i) \qquad \text{for small values of } \sigma_n$$

$$\tau = c+\sigma_n.\tan\Phi_r \qquad \text{for large values of } \sigma_n$$

Where τ = Peak shear strength of the joint; σ_n =normal stress on the joint; Φ_r = Residual friction angle.

2.4.4 JOINT ROUGHNESS COEFFICIENT

Roughness is an important controlling factor for the shear behaviour of rock joints. The roughness is expressed in terms of a joint roughness coefficient that can be either determined 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 the roughest to smoothest surface respectively.

2.4.5 SCALE EFFECT

The scale effect is overwhelming in rocks. Rock strength varies widely with sample size. The strength of rock materials decreases with increase of the volume of the test specimen. This property is called scale effect which can also be observed in soft rocks. **Bandis (1981)** did experimental studies of scale effects on the shear behaviour of rock joints by performing direct shear test on different sized specimens with various natural joint surfaces. Their results show significant scale effects on shear strength and deformation characteristics. Scale effects are more pronounced in case of rough, undulating joint type where as they are virtually absent in case of planar joints. The key factor seems to be the involvement of different length of joints. The results showed that both joint roughness coefficient (JRC) and compressive strength of rock at the fracture surface (JCS) reduced to the changing stiffness of a rock mass as the block size or joint spacing increases or decrease to overcome the effects of size 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 :

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

Where α = tilt angle, σ_{n0} = normal stress when sliding occurs

2.4.6 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- ridden and there is shearing off, the dilation (h_n) for any displacement can be given as,

$$\mathbf{h_n = n_i . \tan d_n}$$

where, n_i is the displacements (in steps of length); d_n is the max. angle between the reference plane and profile for base length.

Dilation can be represented in form of dilation angle as follows,

$$\mathbf{\Delta d = \Delta v / \Delta h}$$

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 choubey (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.

2.5 STRENGTH CRITEREON FOR JOINTED ROCKS

The strength of jointed rock mass is only a fraction of the strength of intact rock mass. The reason for this is that failure in that failure in the jointed rock mass is a combination of both intact rock strength and separation or sliding along discontinuities. Unlike isotropic rocks, the strength criterion for jointed 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 Navier – Couloumb and Griffith.

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. **Hoek and Brown (1980)** showed clearly the strength of all rocks is maximum at $\beta = 0°$ and is minimum for $\beta = 20°$ to $30°$.

Using the non linear failure envelopes predicted by classical Griffith's theory for plane compression and through a process of trial and error, **Hoek and Brown (1980)** presented an empirical failure criterion applicable for both isotropic and anisotropic rock.

$$\boldsymbol{\sigma_1 = \sigma_3 + (m\, \sigma_c.\, \sigma_3 + s.\, \sigma_c 2_) 1/2}$$

Where s= 1 for intact rock

s = 0 for crushed rock

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

Ramamurthy (1994) and Rao (1993) proposed an empirical strength criterion to account for the non-linear strength response of isotropic intact rocks in the following form:

$$\boldsymbol{(\sigma_1 - \sigma_3)/\, \sigma_3 = Bi\, (\sigma_{ci}/\, \sigma_3)^{\alpha i}}$$

Where σ_{ci} is the uniaxial compressive strength of intact rock without a weak plane, σ_1 and σ_3 are plane stresses, αi is the slope of plot between $(\sigma_1 - \sigma_3)/\sigma_3$ and (σ_{ci}/σ_3) on the log-log scale and Bi $=(\sigma_1 - \sigma_3)/\sigma_3$ and $(\sigma_{ci}/\sigma_3) = 1$, αi and Bi are considered as strength parameters. The authors had suggested a constant value of 0.8 for αi at all orientations even for intact anisotropic rocks. Owing to the fact that Bi parameter did not vary 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 $\alpha i = 0.8$.

2.5.1 INFLUENCE OF SINGLE PLANE OF 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.

2.5.2 INFLUENCE OF SAMPLE SIZE

It is generally assumed that there is a significant reduction in strength with increasing sample size. Based upon an analysis of published data, **Hoek and Brown (1980)** have suggested that the uniaxial compressive strength σcd of a rock specimen with a diameter of d mm is related to the uniaxial compressive strength σ_{c50} of a 50mm diameter sample by the following relationship:

$$\sigma_{cd} = \sigma_{c50}\,(50/d)^{0.18}$$

2.5.3 INFLUENCE OF NUMBER AND LOCATION OF JOINTS

For plaster of Paris representing weak rock, the variation of number of horizontal joints per meter length (J_n, joint frequency) with the ratio of uniaxial strength of joint and intact specimens under unconfined compression. The location of a single joint with respect to the loading surface defined by df = Dj/B (ratio of depth of joint Dj to the width or diameter B of the loaded area) greatly influences the strength of rock when the joint is placed very close to the loading face the strength of joint away from the loading face the strength of jointed rock mass increases and attain a value the same as that of intact rock so long as the joints within the depth equal to the width of loaded arrears. The stiffness of the rock is the highest when the joint is very close to the loading face contrary to what has been observed for strength influence of the location of a joint on the stiffness continuous to decrease even up to a depth twice the width if the loaded area.

2.6 UNIAXIAL COMPRESSIVE STRENGTH

The uniaxial compressive strength 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

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

Where σ_{cj} = uniaxial compressive strength of jointed rock and σ_{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.

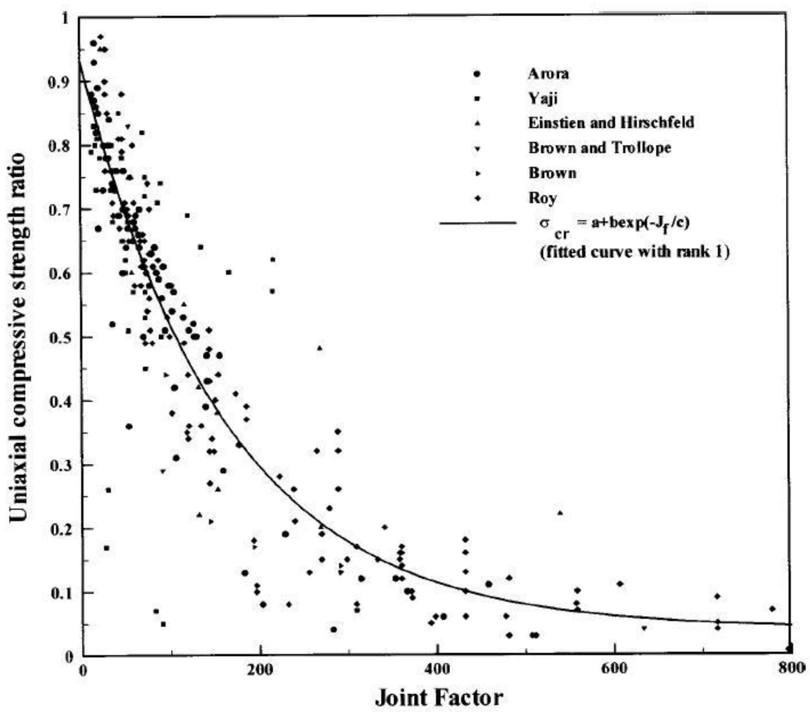

FIG 1- UNIAXIAL COMPRESSIVE STRENGTH Vs JOINT FACTOR

2.7 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:

$$\mathbf{Er = Ej/ Ei}$$

Where Ej=tangent modulus of the jointed rock and *Ei*=tangent modulus of the intact rock.

Ramamurthy (1994) conducted tests on intact and jointed specimens of plaster of Paris, Jamrani sandstone, and Agra sandstone. By Extensive laboratory testing of intact and jointed specimens in uniaxial compression in both confined and unconfined states he then predict the relation between modulus ratio and joint factor as

$$\mathbf{M_{rj} / M_{ri} = exp\ [- 3.72 \times 10^{-3}\ J_f\ ]}$$

Where**,** M_{rj} = modulus ratio of jointed rock, M_{ri} = modulus ratio of intact rock, and J_f = joint factor.

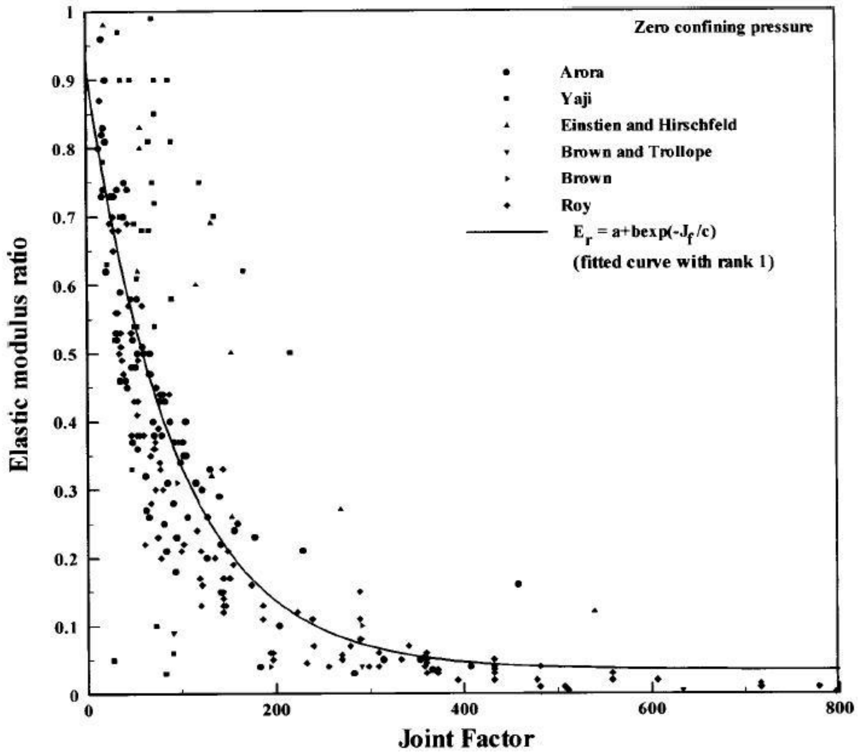

FIG 2 **- ELASTIC MODULUS RATIO Vs JOINT FACTOR**

2.8 FAILURE MODES IN ROCK

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

(i) Splitting of intact material of the elemental blocks,

(ii) Shearing of intact block material,

(iii) Rotation of the blocks, and

(iv) Sliding along the critical joints.

These modes were observed to depend on the combination of orientation h and the stepping. The angle θ 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.

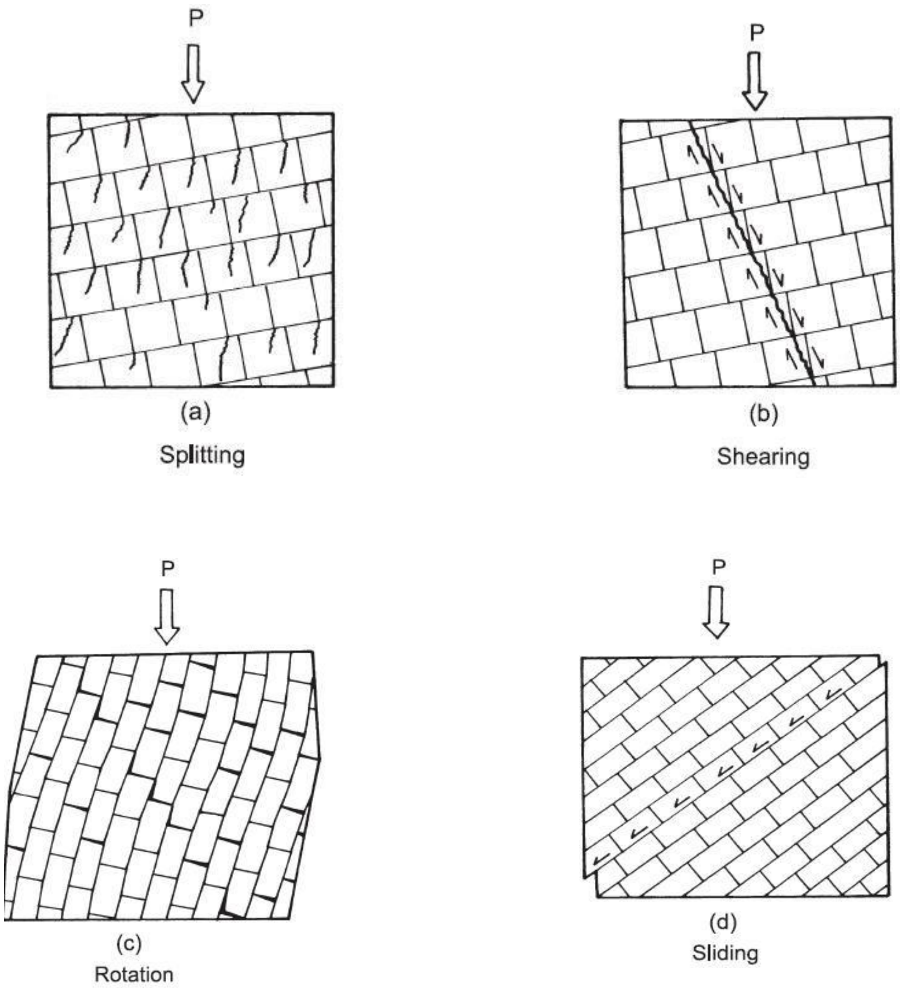

FIG 3 - MODES OF FAILURE IN ROCKS

2.8.1 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.

2.8.2 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 displacements and formation of fractured material along the sheared zones. This failure mode occurs when the continuous joints are close to horizontal (i.e., $\theta <= 10$) and the mass is moderately interlocked. As the angle h increases, the tendency to fail in shearing reduces, and sliding takes place. For $\theta \approx 30$, shearing occurs only if the mass is highly interlocked due to stepping.

2.8.3 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 $\theta \approx 20 - 30$ if the interlocking is nil or low. For orientations, θ = 35 – 65 sliding occurs invariably for all the interlocking conditions.

2.8.4 ROTATION

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

TABLE 3 - STRENGTH CLASSIFICATION OF JOINTED AND INTACT ROCK MASS (Ramamurthy and Arora, 1993)

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 4 - MODULUS RATIO CLASSIFICATION OF JOINTED AND INTACT ROCKS (Ramamurthy and Arora, 1993)

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

2.9 In this present chapter we discussed about what is rock, intact rock mass, jointed rock mass, strength of rock mass, factors affecting the strength of rock and failure modes in rocks. Now at this stage we will be able to understand the basic concept about rock. In the next chapter we discussed about the materials and methods required for experimental investigations.

CHAPTER – 3

MATERIALS AND METHODS

Experimental investigations have been carried out on specimen made with plaster of paris. The specimen with three different diameters (63.5mm, 90mm, and 114mm) were made. The height to diameter ratio was kept two to eliminate any possible buckling effect. The anisotropy was introduced into the intact specimen by developing a number of rough joints at various inclinations. Now cement and Araldite were used as gouge material to join the samples. Now number of uniaxial compressive tests were 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 THE 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 behavior.

Usually plaster of Paris is used as a model material in simulation of model material to simulate the weak rock mass in the field. Because of its ease in casting, flexible behaviour, low cost, instant hardening and commercial availability Plaster of Paris is the most frequently used material in these kinds of tests. Moreover hardened plaster of Paris replicates the behaviour of soft natural rocks. In addition to the above advantages when foreign materials such as mica, sand, calcite etc are mixed to plaster of Paris its strength can be varied from low to reasonably high values. Any type of joint can be made using plaster of Paris. Thus it's reduced strength and greater deformability in relation to actual rocks has made it the ideal material for modeling.

3.1.1 Plaster of Paris:

Plaster of Paris is a type of building material based on calcium sulphate hemihydrates, nominally $CaSO4.1/2H_2O$. It is created by heating Gypsum to about 300ºF (150ºC).

$$CaSO_4.H_2O \longrightarrow 2CaSO_4.1/2H_2O + 3H_2O$$

A large Gypsum deposit at Montmartre in Paris is the source of the name. When the dry plaster powder is mixed with water, it reforms into Gypsum. Plaster is used as a building material similar to mortar or cement. Like those material Plaster starts as a dry powder that is mixed with water to form a paste which liberates heat and then hardens. Unlike mortar and cement, plaster remains quite soft after setting and can be easily manipulated with metal tools or even sand paper. These characteristics make Plaster suitable for a finishing, rather than a load bearing material or even sand paper. These characteristics make Plaster suitable for a finishing, rather than a load bearing material.

TABLE 5 - PROPERTIES OF PLASTER OF PARIS

S.No.	PROPERTIES	VALUE
1.	Dry density (KN/M^3)	8.81
2.	Specific gravity	2.60
3.	Porosity (%)	58.00
4.	Cohesion intercept (MPa)	2.87
5.	Angle of friction(^{0})	37.0
6.	Deere and Miller (1966) classification	EL
7.	ISRM(1975) Classification	Low srength

3.2 GOUGE MATERIALS

Cement and Araldite were used as a gouge material. First the specimens were cut and then joined with the gouge material to see the effect of gouge material on the uniaxial compressive strength of jointed rock.

TABLE 6 - PROPERTIES OF ARALDITE

S.No.	PROPERTIES (MIXED)	VALUE
1.	Colour (visual)	Beige
2.	Specific gravity	1.00
3.	Viscosity at 25^{0}C (pas)	Ca.60
4.	Roller peel test (ISO 4578)	4N/mm
5.	Shore hardness	D 75
6.	Elongation at break	50-75%
7.	Flexure strength(ISO 178) cure 1 day/23^{0}C tested at 23^{0}C	43 MPa
8.	Flexure modulus(ISO 178) cure 1 day/23^{0}Ctested at 23^{0}C	1642 MPa
9.	Cohesion intercept(MPa)	O.6894-1.2065
10.	Angle of friction(^{0})	17-24

TABLE 7 - PROPERTIES OF CEMENT (OPC 43 GRADE)

S.No.	PROPERTIES	OBSERVED VALUES	VALUES SPECIFIED BY IS:8112-1989
1.	Normal Consistency (%)	31.5	……..
2.	Soundness (mm)	1.8	Not more than 10
3.	Fineness %	4	Not more than 10
4.	Initial Setting Time (minutes)	110	>=30
5.	Final Setting Time (minutes)	230	<=600
6.	Compressive Strength (MPa) i) 3 days ii) 7 days	 26.07 34.40	 >23 >33
7	Specific gravity	3.15	………
8	Cohesion(MPa)	5.6	………..
9	Friction angle(^{0})	28.3	………..

3.3 PREPARATION OF SPECIMENS

Plaster of Paris (B.C.C made) was procured from the local market. This plaster of Paris powder was produced by pulverising burnt gypsum, is dull white in colour, with a smooth feel of cement. The entire quantity for experimentation was bought in a single lot and stored air tight to avoid the entry of atmospheric moisture. This is highly essential in order to keep the different parameters such as density, moisture content, liquid limit, plastic limit etc to remain same for all the specimens that will be prepared. For preparation of specimens, plaster of Paris was mixed with required quantity of distilled water to form a uniform paste. This uniform paste was then poured into a cylindrical mould, which was smeared with grease/oil to avoid any kind of void formations in the specimen and for easy extrudation. The uniform paste inside the mould was kept over the vibrating machine and was allowed to vibrate for about 3 – 4 minutes. After hardening the specimen was extracted manually from the mould.

3.4 INTRODUCTION OF ANISOTROPY

In the rock mass joint planes may be oriented in different directions with respect to the stress field and this may vary from place to place. Also, during externally applied loading the principal stress direction may change. To investigate these aspects in this study, single plane of weakness and its inclination with respect to major principal stress direction should be considered.

3.5 UNIAXIAL COMPRESSIVE STRENGTH TEST:

In Uniaxial test the cylinder specimen of the soil 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 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 5.1 to 10.2 $kgf/cm^2/sec$. 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 ends of the cylindrical specimen are hollowed in the form of cone. The cone seatings reduce the tendency of the specimen to become barrel shaped by reducing end straits. When a brittle failure occurs, the proving ring dial indicates a definite maximum load which drops rapidly with the further increase in 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.

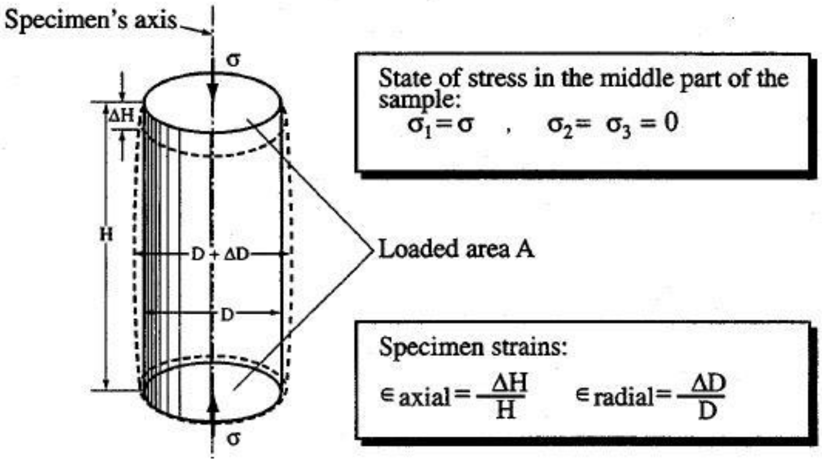

FIG 4 - STRESSES IN A UCS SAMPLE

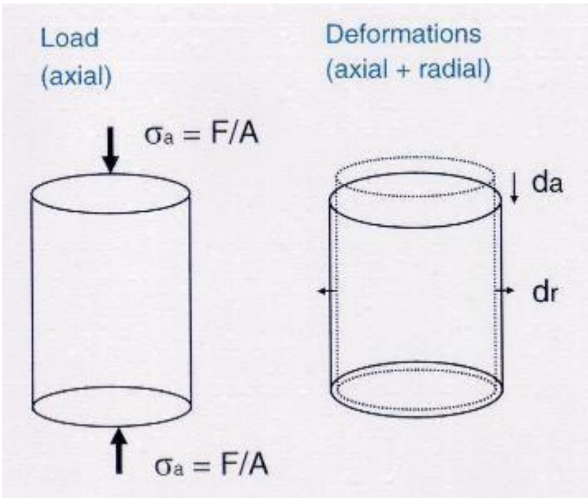

FIG 5 - LOADING ON THE SPECIMEN

3.6 In this present chapter we discussed about the material for specimen preparation, properties of this material, gouge materials, properties of gouge materials, method of preparation of specimens and method of testing of specimens. In the next chapter we discussed about the results obtained from the experiment conducted on these specimen.

CHAPTER -4

RESULTS AND DISCUSSION

4.1 GENERAL

The specimen with three different diameters (63.5mm, 90mm, and 114mm) were test under uniaxial compression and results of these tests are listed below.

TABLE 8- UNIAXIAL COMPRESSION TEST RESULTS FOR INTACT SPECIMENS

S.NO.	SIZE OF SAMPLE(MM)	STRENGTH(MPa)
1.	63.5	3.94
2.	90	7.33
3.	114	3.06

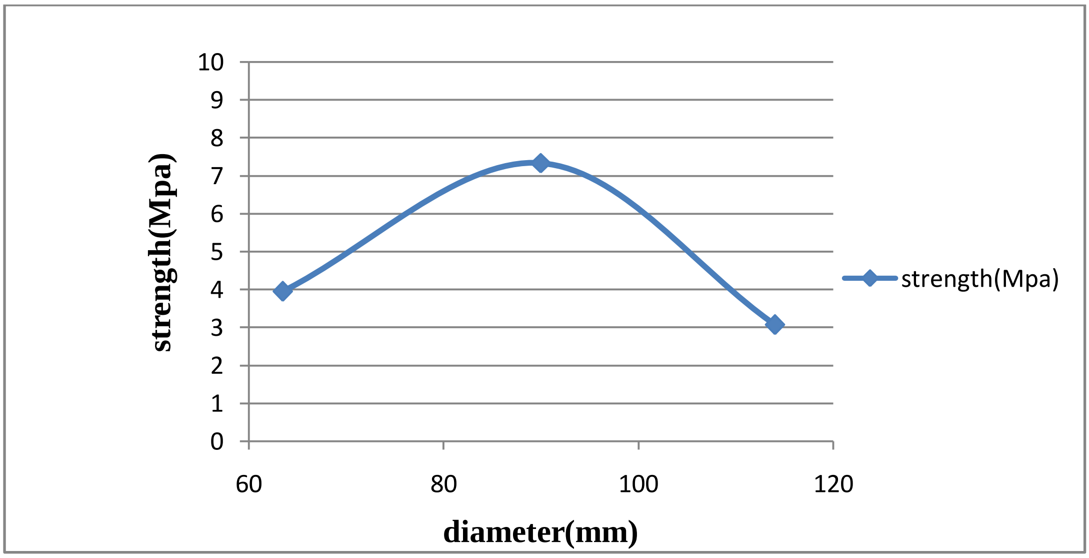

FIG 6 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF INTACT SPECIMENS OF DIFFERENT SIZES.

DISCUSSION

When we increased the size of sample from 63.5 mm to 90 mm the strength of the sample increased by 86% but when we increased the sample size from 90 mm to 114 mm the strength of the sample decreased by 58.25% when compared to 90mm sample and decreased by 22.33% when compared to 63.5 mm sample

TABLE 9 - UNIAXIAL COMPRESSION TEST RESULTS FOR JOINTED SPECIMENS WITH ALARDITE $\alpha=0^0$

S.NO.	SIZE OF SAMPLE(MM)	STRENGTH(MPa)
1.	63.5	4.42
2.	90	7.25
3.	114	3.41

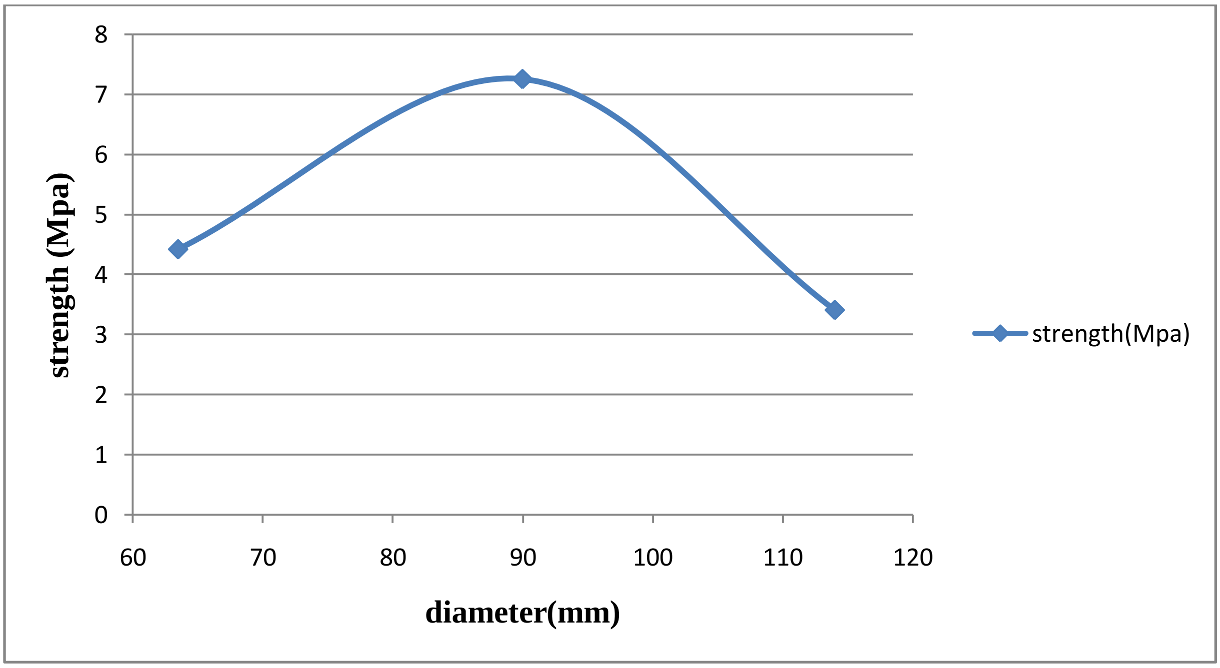

FIG 7 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH ALARDITE AT $\alpha=0^0$.

DISCUSSION

When we increased the size of sample from 63.5 mm to 90 mm the strength of the sample increased by 64.03% but when we increased the sample size from 90 mm to 114 mm the strength of the sample decreased by 52.96% when compared to 90mm sample and decreased by 22.85% when compared to 63.5 mm sample.

TABLE 10 - UNIAXIAL COMPRESSION TEST RESULTS FOR JOINTED SPECIMENS WITH ALARDITE $\alpha=30^0$

S.NO.	SIZE OF SAMPLE(MM)	STRENGTH(MPa)
1.	63.5	3.94
2.	90	7.17
3.	114	3.02

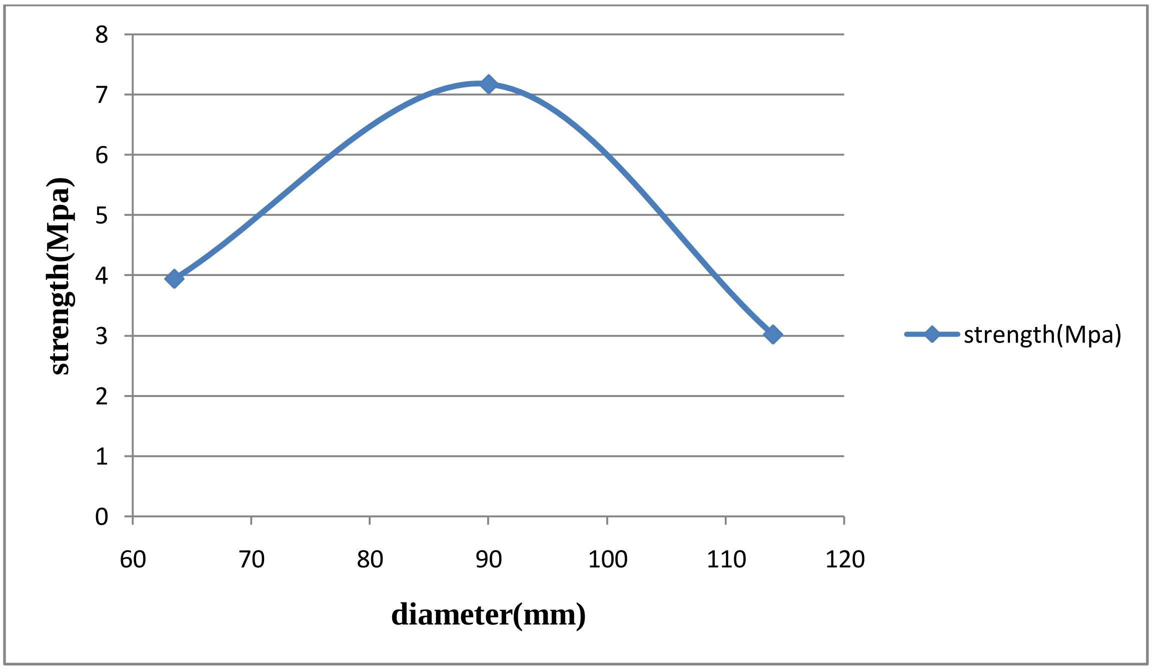

FIG 8 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH ALARDITE AT $\alpha=30^0$.

DISCUSSION

When we increased the size of sample from 63.5 mm to 90 mm the strength of the sample increased by 81.97% but when we increased the sample size from 90 mm to 114 mm the strength of the sample decreased by 57.88% when compared to 90mm sample and decreased by 22.33% when compared to 63.5 mm sample.

TABLE 11 - UNIAXIAL COMPRESSION TEST RESULTS FOR JOINTED SPECIMENS WITH ALARDITE α=45^0

S.NO.	SIZE OF SAMPLE(MM)	STRENGTH(MPa)
1.	63.5	3.94
2.	90	7.17
3.	114	3.02

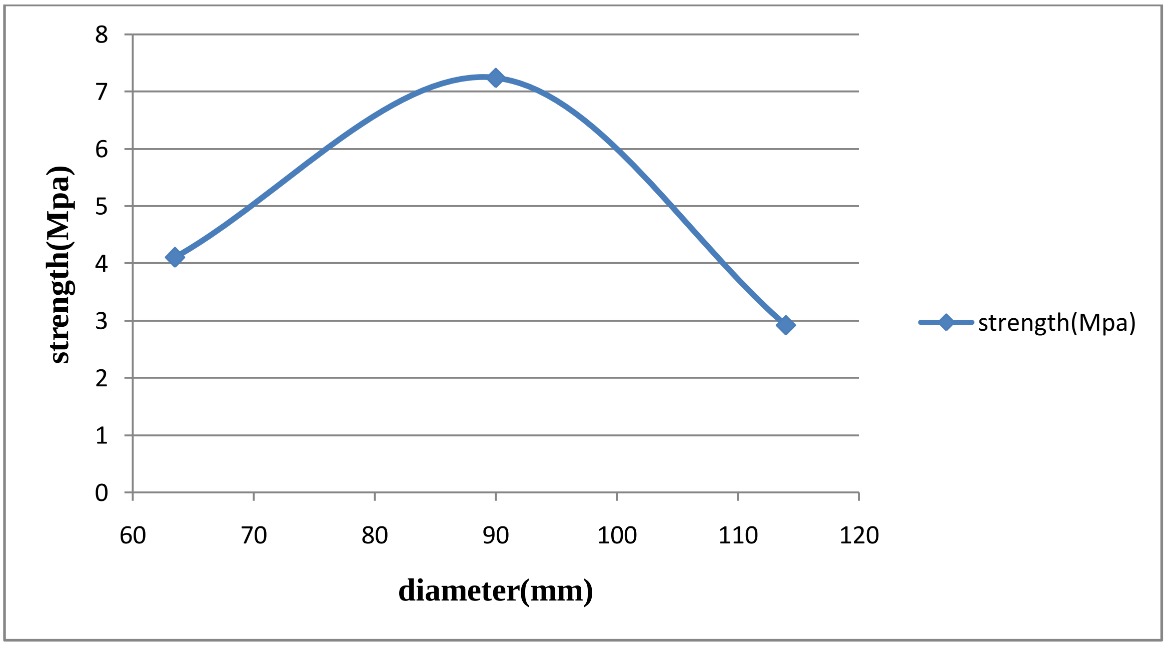

FIG 9 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH ALARDITE AT α=45^0.

DISCUSSION

When we increased the size of sample from 63.5 mm to 90 mm the strength of the sample increased by 81.97% but when we increased the sample size from 90 mm to 114 mm the strength of the sample decreased by 57.88% when compared to 90mm sample and decreased by 22.33% when compared to 63.5 mm sample.

TABLE 12 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH ALARDITE AT α=60^0.

S.NO.	SIZE OF SAMPLE(MM)	STRENGTH(MPa)
1.	63.5	3.79
2.	90	6.92
3.	114	2.92

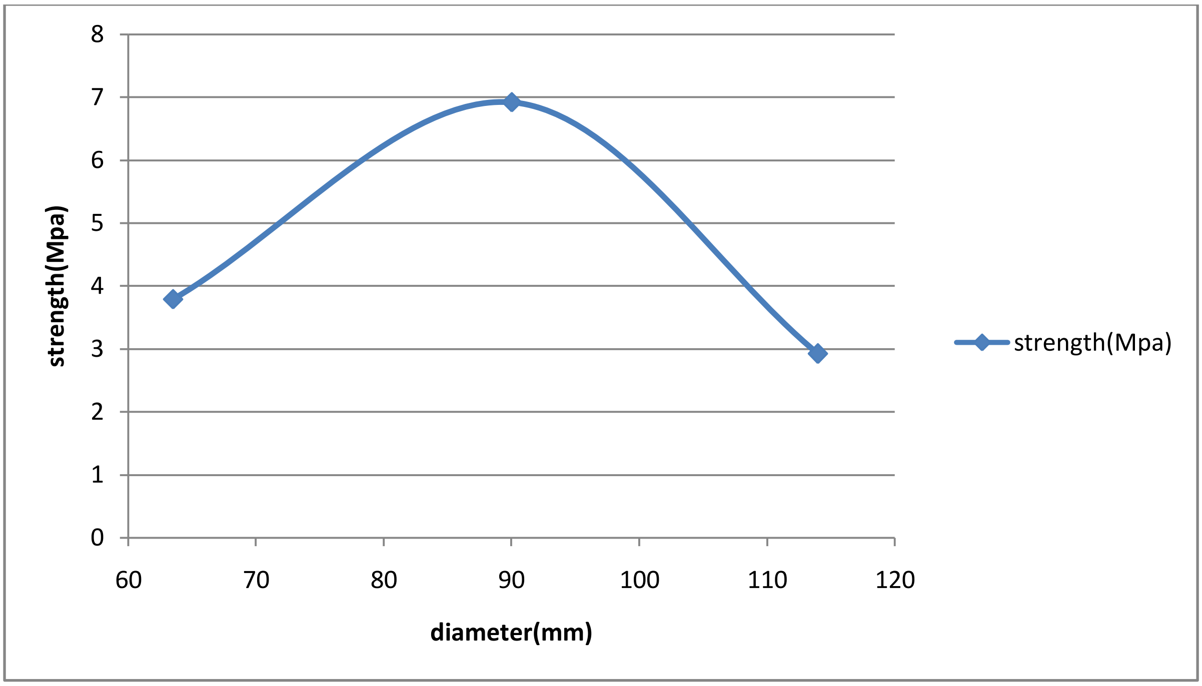

FIG 10 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH ALARDITE AT α=60^0.

DISCUSSION

When we increased the size of sample from 63.5 mm to 90 mm the strength of the sample increased by 82.79% but when we increased the sample size from 90 mm to 114 mm the strength of the sample decreased by 57.80% when compared to 90mm sample and decreased by 22.95% when compared to 63.5 mm sample.

TABLE 13 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH CEMENT AT $\alpha=0^0$.

S.NO.	SIZE OF SAMPLE(MM)	STRENGTH(MPa)
1.	63.5	1.58
2.	90	2.17
3.	114	1.50

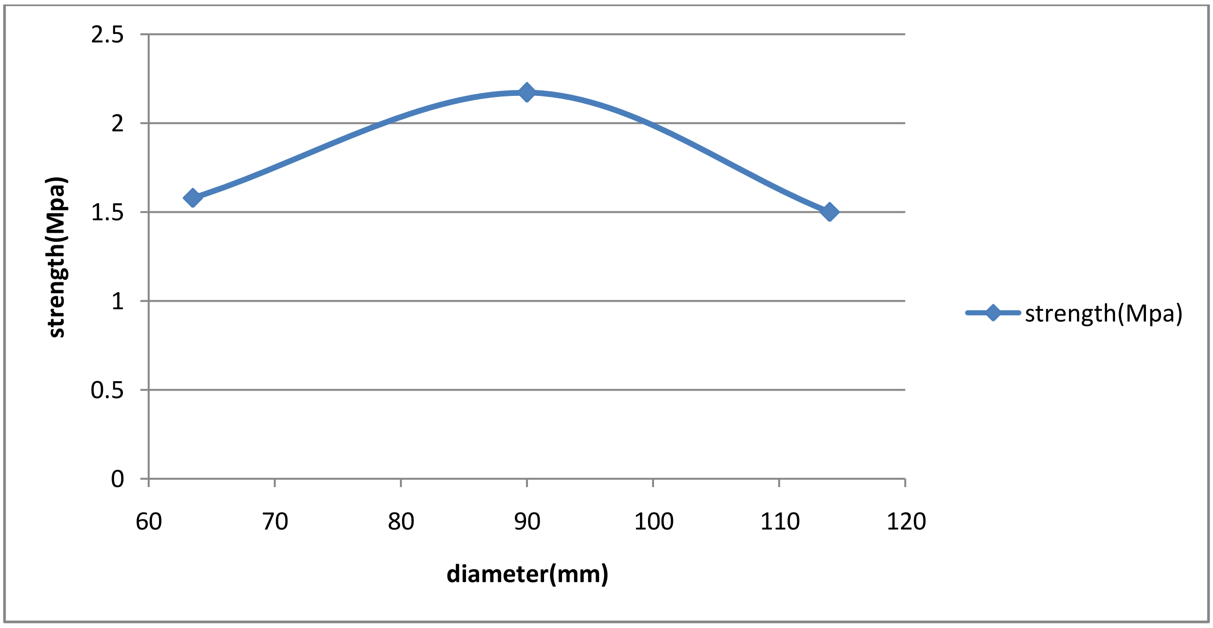

FIG 11 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH CEMENT AT $\alpha=0^0$.

DISCUSSION

When we increased the size of sample from 63.5 mm to 90 mm the strength of the sample increased by 37.34% but when we increased the sample size from 90 mm to 114 mm the strength of the sample decreased by 30.87% when compared to 90mm sample and decreased by 5.06% when compared to 63.5 mm sample.

TABLE 14 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH CEMENT AT α=30^0.

S.NO.	SIZE OF SAMPLE(MM)	STRENGTH(MPa)
1.	63.5	1.58
2.	90	2.01
3.	114	1.46

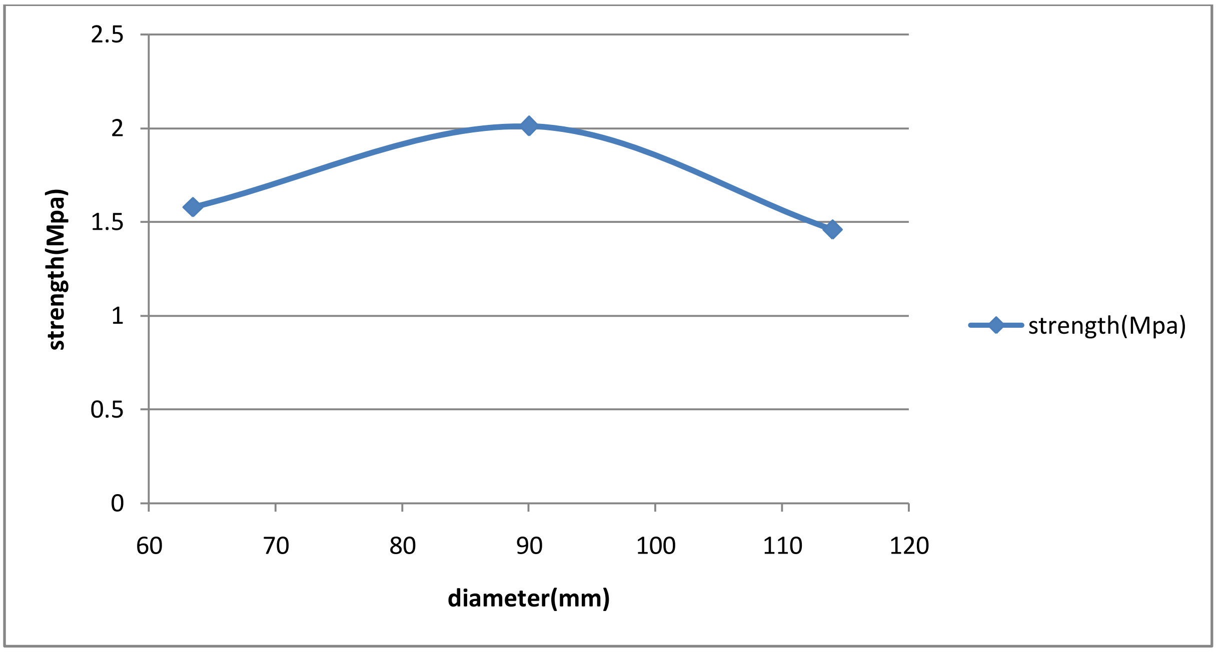

FIG 12 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH CEMENT AT α=30^0.

DISCUSSION

When we increased the size of sample from 63.5 mm to 90 mm the strength of the sample increased by 21.39% but when we increased the sample size from 90 mm to 114 mm the strength of the sample decreased by 27.36% when compared to 90mm sample and decreased by 7.59% when compared to 63.5 mm sample.

TABLE 15 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH CEMENT AT $\alpha=45^0$.

S.NO.	SIZE OF SAMPLE(MM)	STRENGTH(MPa)
1.	63.5	0.95
2.	90	1.14
3.	114	0.85

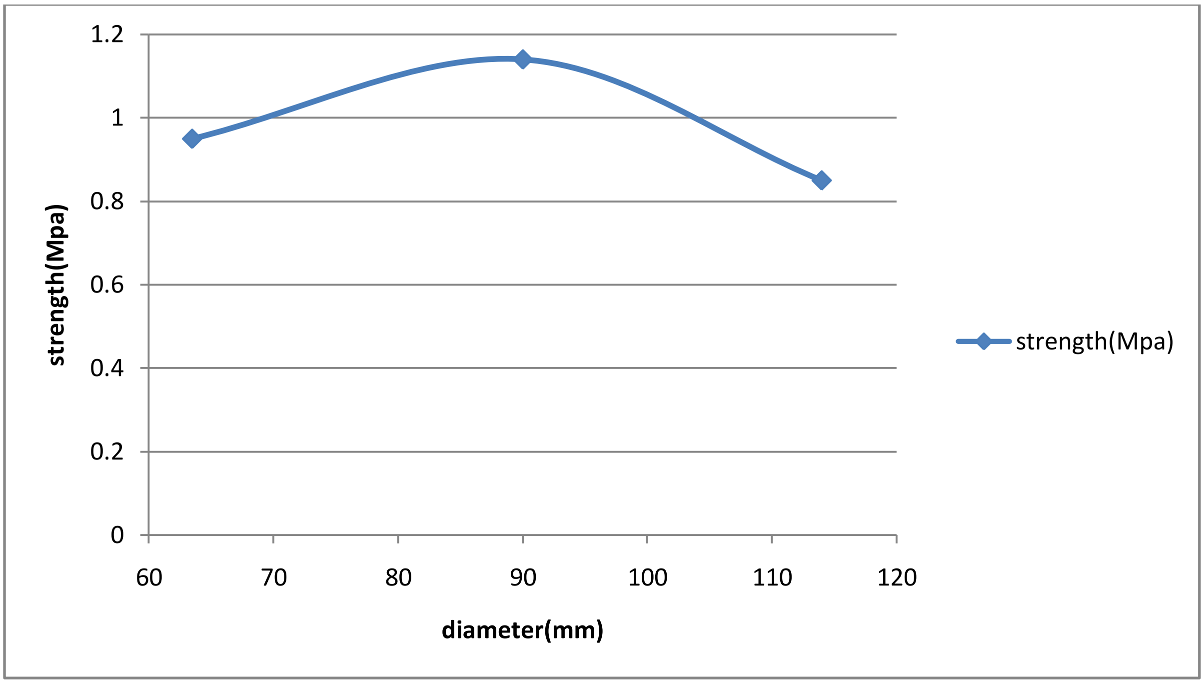

FIG 13 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH CEMENT AT $\alpha=45^0$.

DISCUSSION

When we increased the size of sample from 63.5 mm to 90 mm the strength of the sample increased by 20% but when we increased the sample size from 90 mm to 114 mm the strength of the sample decreased by 25.43% when compared to 90mm sample and decreased by 10.53% when compared to 63.5 mm sample.

TABLE 16 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH CEMENT AT α=60^0.

S.NO.	SIZE OF SAMPLE(MM)	STRENGTH(MPa)
1.	63.5	0.79
2.	90	0.91
3.	114	0.68

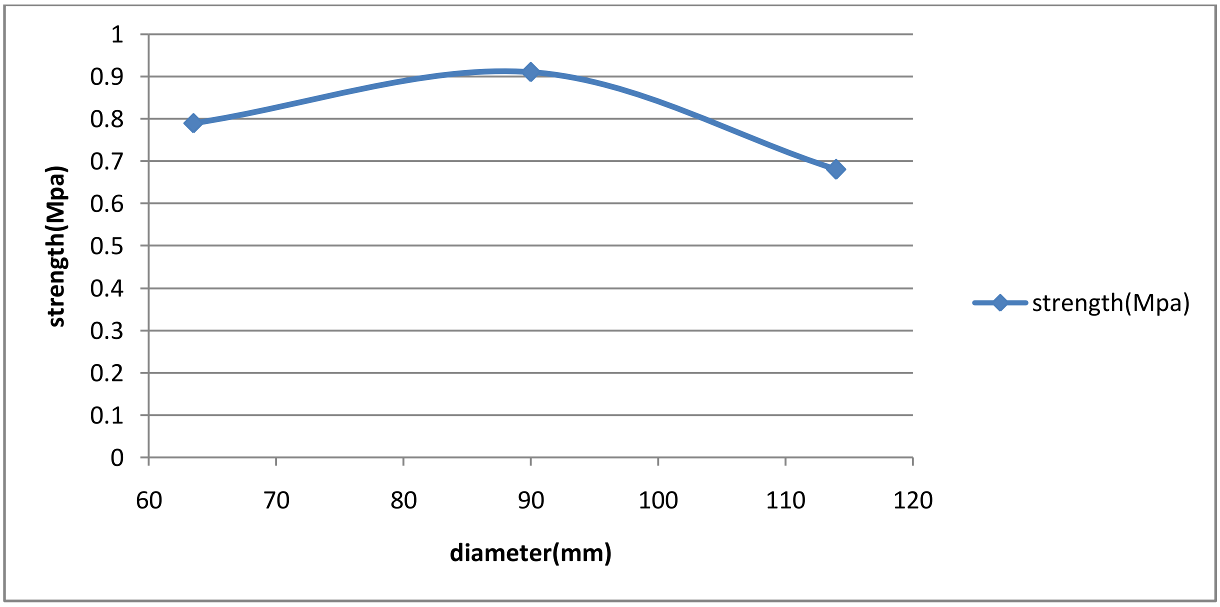

FIG 14 - VARIATION OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS WITH CEMENT AT α=60^0.

DISCUSSION

When we increased the size of sample from 63.5 mm to 90 mm the strength of the sample increased by 15.18% but when we increased the sample size from 90 mm to 114 mm the strength of the sample decreased by 25.27% when compared to 90mm sample and decreased by 13.92% when compared to 63.5 mm sample.

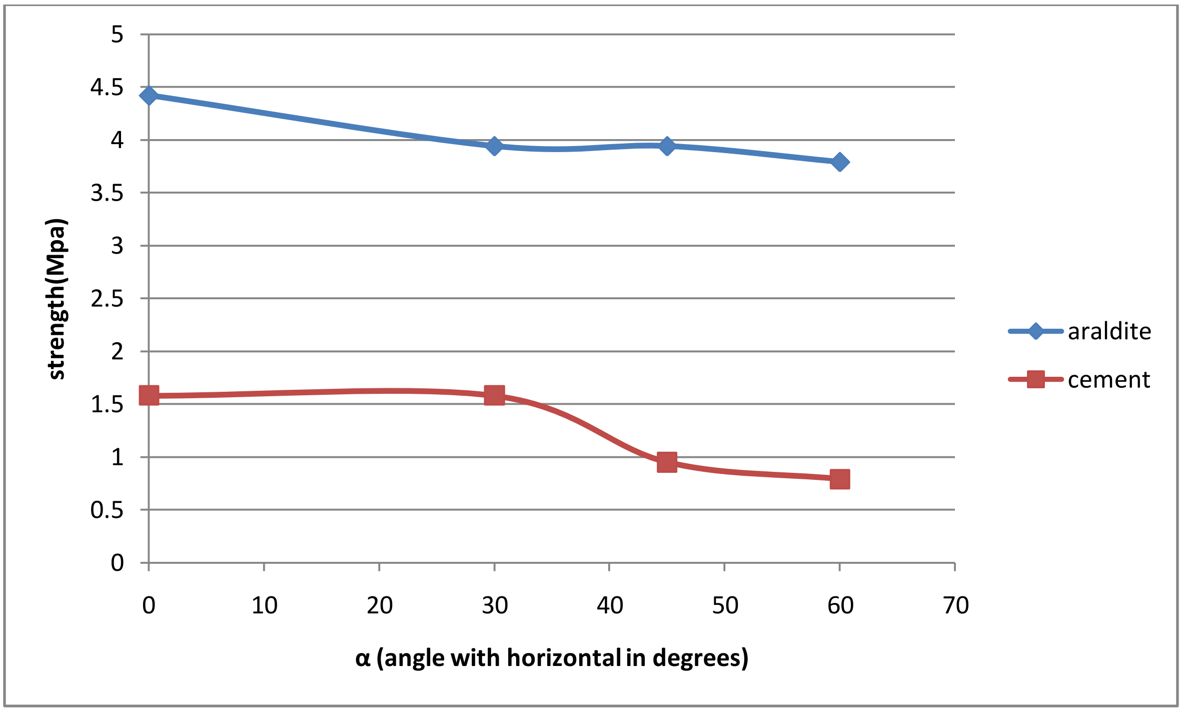

FIG 15 - COMPARISON OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS (63.5 MM DIA.).

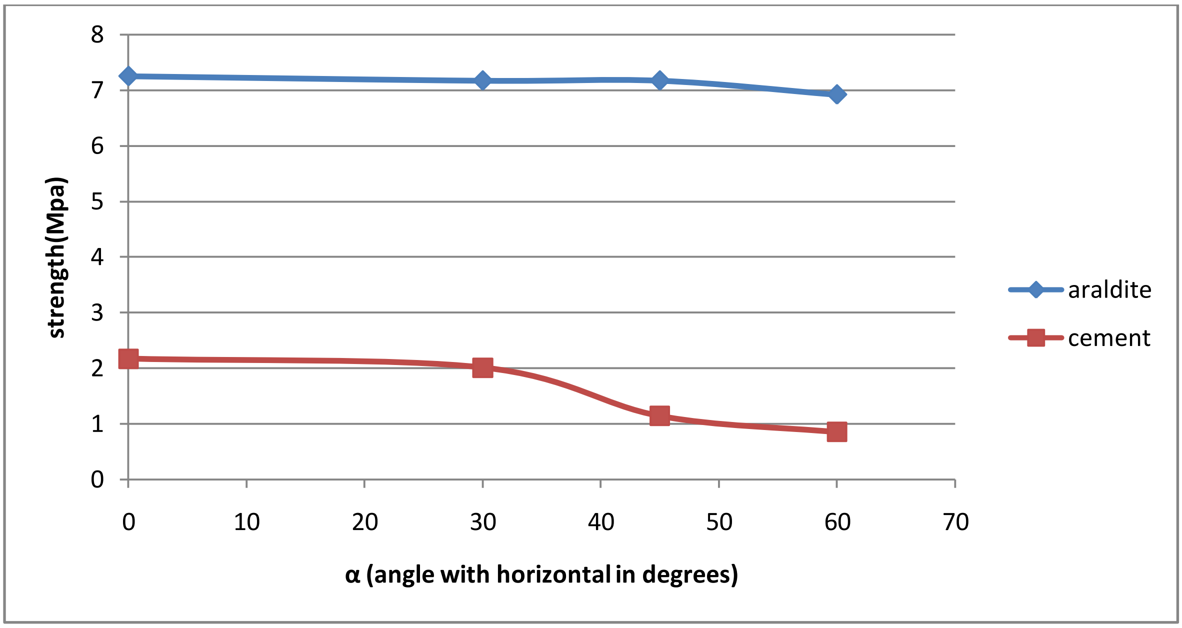

FIG 16 - COMPARISON OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS (90 MM DIA.).

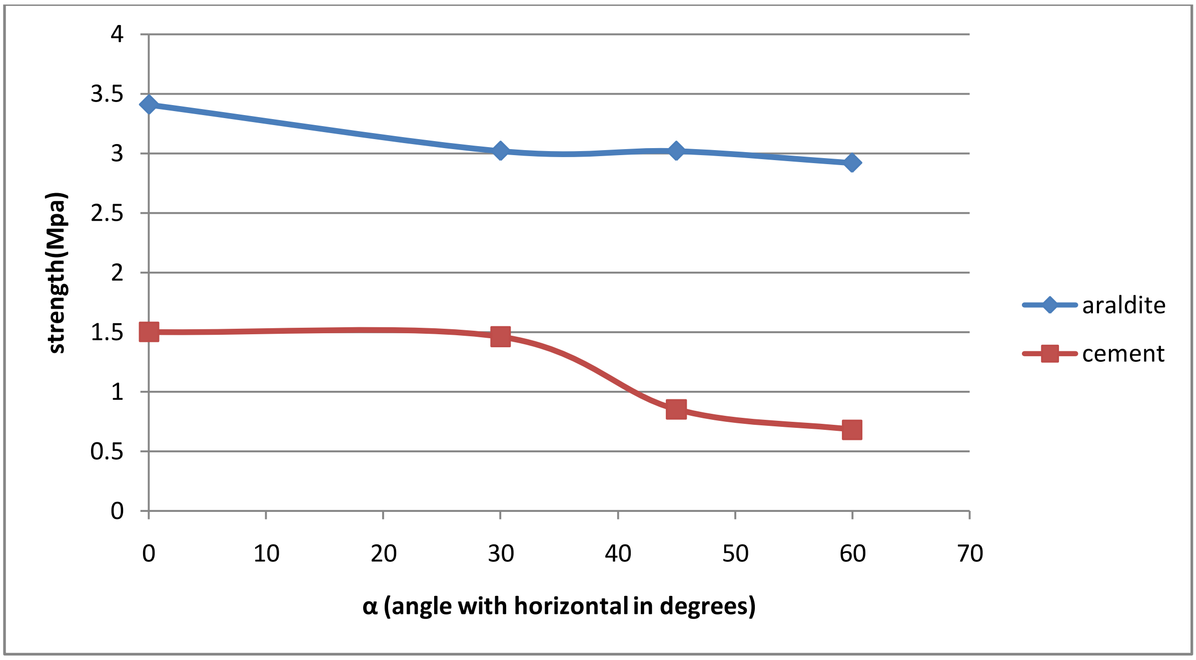

FIG 17 - COMPARISON OF UNIAXIAL COMPRESSIVE STRENGTH OF JOINTED SPECIMENS (114 MM DIA.)

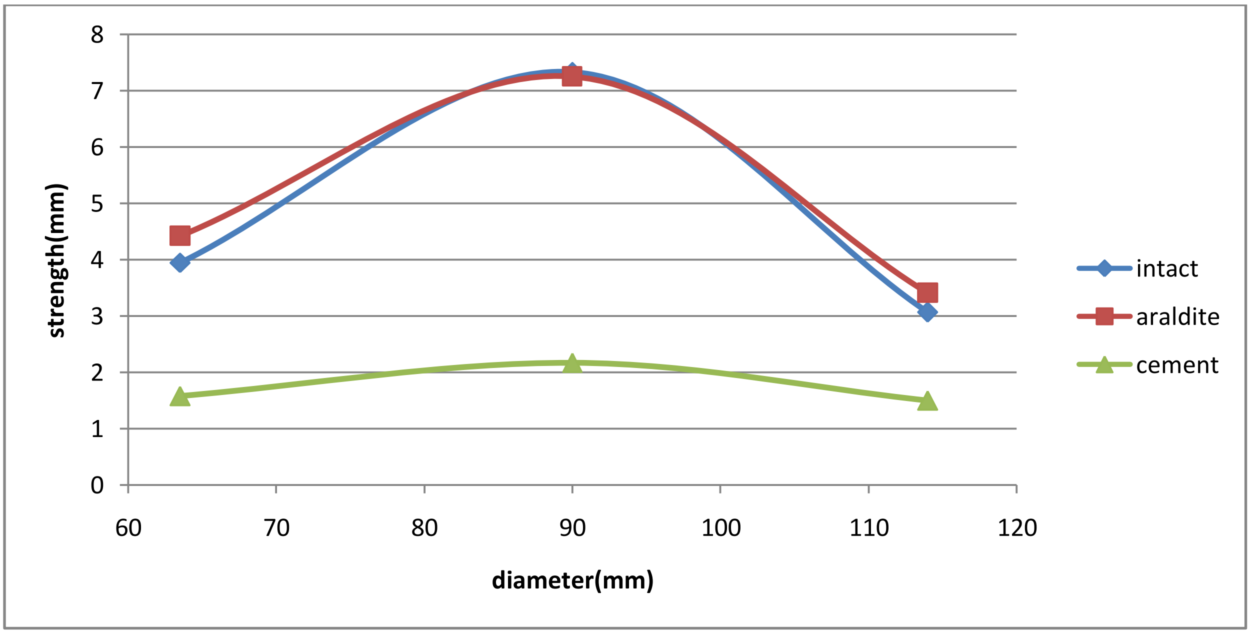

FIG 18 - COMPARISON OF UNIAXIAL COMPRESSIVE STRENGTH OF INTACT AND JOINTED SPECIMENS AT $\alpha = 0^0$.

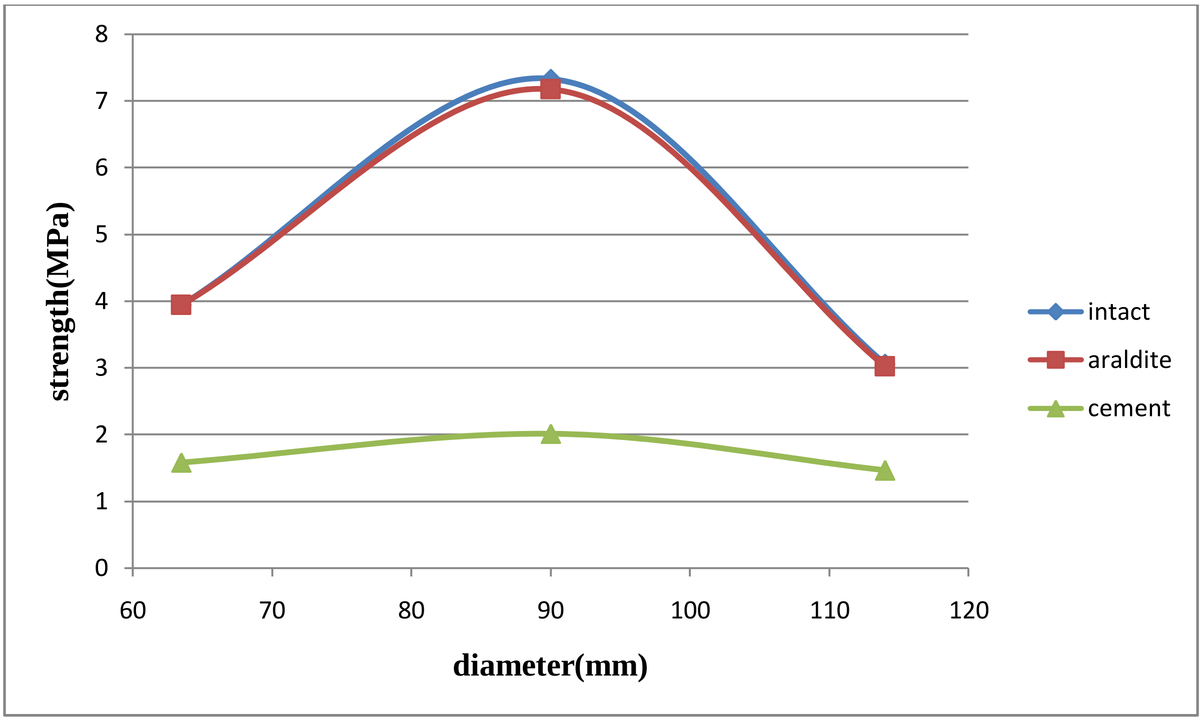

FIG 19 - COMPARISON OF UNIAXIAL COMPRESSIVE STRENGTH OF INTACT AND JOINTED SPECIMENS AT α =30^{0}.

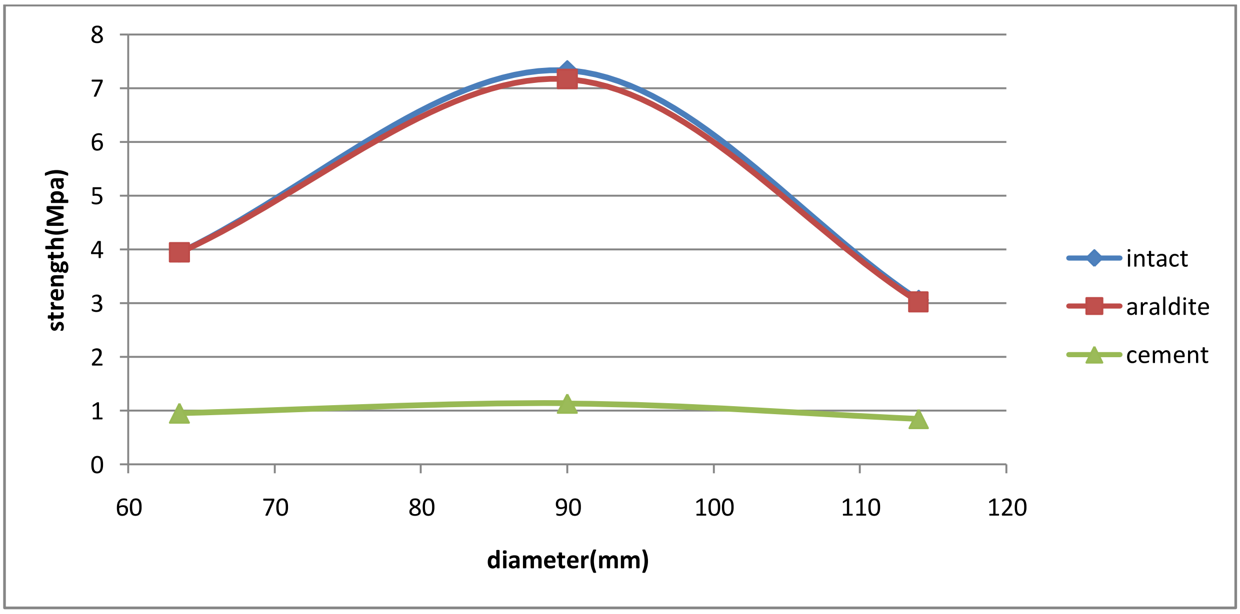

FIG 20 - COMPARISON OF UNIAXIAL COMPRESSIVE STRENGTH OF INTACT AND JOINTED SPECIMENS AT α =45^{0}.

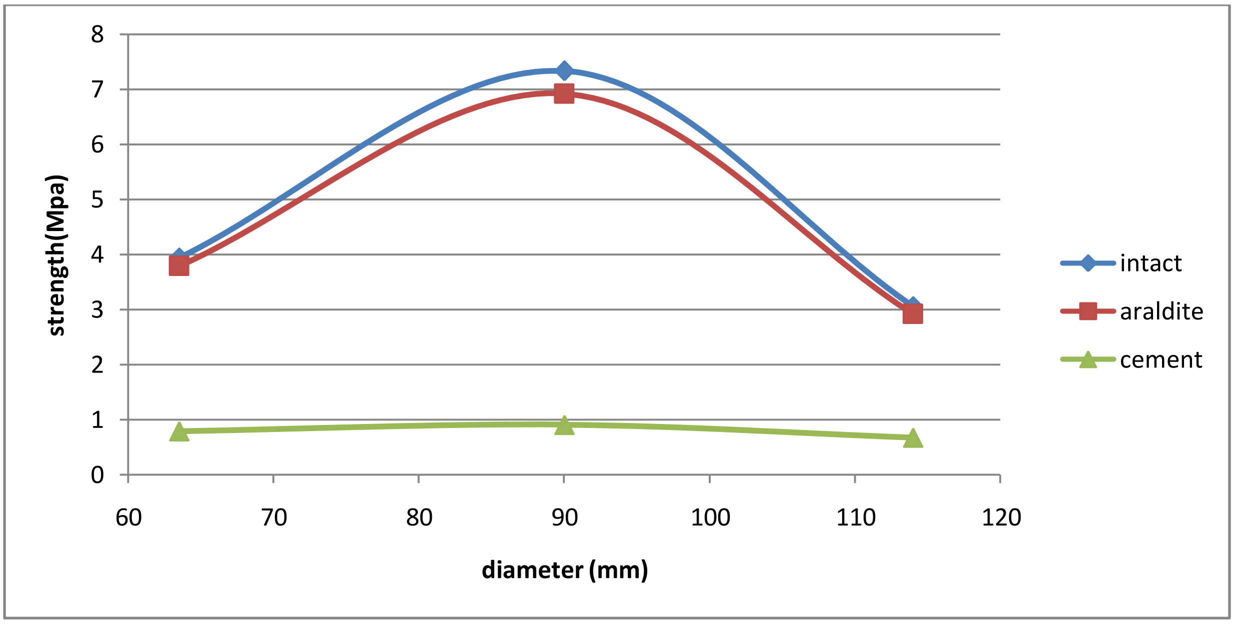

FIG 21 - COMPARISON OF UNIAXIAL COMPRESSIVE STRENGTH OF INTACT AND JOINTED SPECIMENS AT $\alpha = 60^0$.

4.2 CALCULATION OF JOINT FACTOR

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

Where J_n = no. of joints per meter length.

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

r = roughness parameter i.e. tan φ_j which depends on the joint condition.

J_n = 8 for 63.5 mm,

6 for 90 mm,

4 for 114 mm.

n = 1.0000 for $\alpha = 0^0$

0.4650 for $\alpha = 30^0$

0.1885 for $\alpha = 45^0$

0.0450 for $\alpha = 60^0$

r = tan φ_j

0.5384 for Cement

0.4452 for Araldite

TABLE 17 – VALUES OF JOINT FACTOR (J_f) AND STRENGTH RATIO (σ_{cr})

SIZE OF SAMPLE(mm)	CONDITION OF JOINT	J_f	σ_{cr}
2.5	0^0 cut with araldite	18	1.12
2.5	30^0 cut with araldite	38.64	1
2.5	45^0 cut with araldite	95.32	1
2.5	60^0 cut with araldite	390.64	0.96
2.5	0^0 cut with cement	14.85	0.40
2.5	30^0 cut with cement	31.95	0.40
2.5	45^0 cut with cement	78.83	0.24
2.5	60^0 cut with cement	323.02	0.20
3.5	0^0 cut with araldite	13.47	0.99
3.5	30^0 cut with araldite	28.98	0.98
3.5	45^0 cut with araldite	71.50	0.98
3.5	60^0 cut with araldite	292.98	0.94
3.5	0^0 cut with cement	11.14	0.29
3.5	30^0 cut with cement	23.96	0.27
3.5	45^0 cut with cement	59.12	0.16
3.5	60^0 cut with cement	242.26	0.12
4.5	0^0 cut with araldite	8.98	1.11
4.5	30^0 cut with araldite	19.32	0.99
4.5	45^0 cut with araldite	47.66	0.99
4.5	60^0 cut with araldite	195.32	0.95
4.5	0^0 cut with cement	7.43	0.49
4.5	30^0 cut with cement	15.97	0.48
4.5	45^0 cut with cement	39.41	0.28
4.5	60^0 cut with cement	161.51	0.22

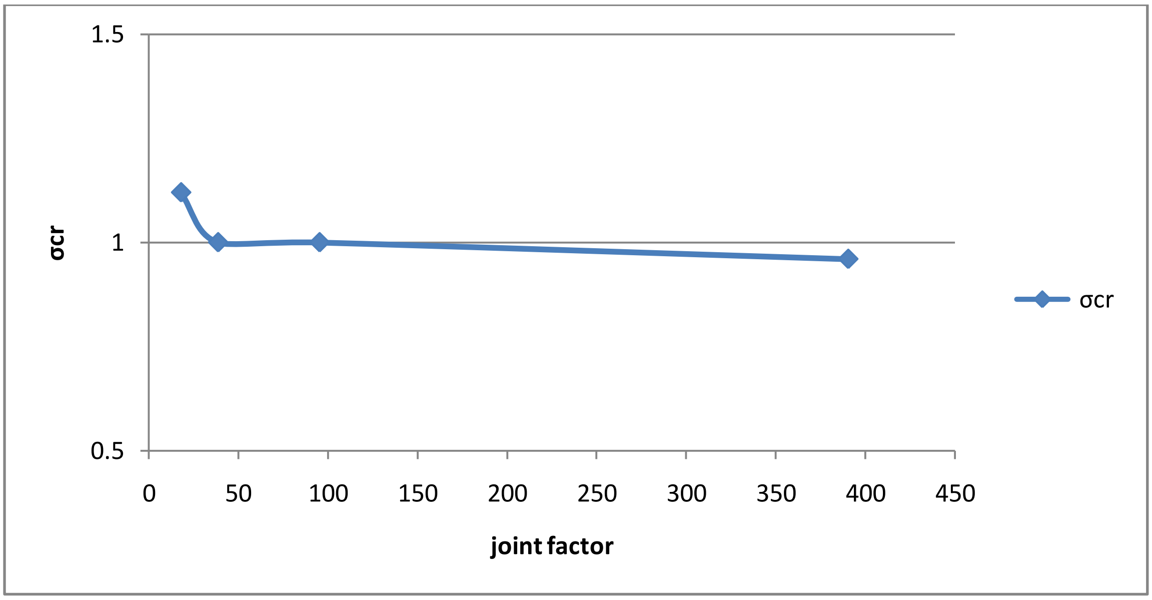

FIG 22 - VARIATION OF STRENGTH RATIO OF JOINTED SPECIMENS TO INTACT SPECIMENS WITH JOINT FACTOR IN CASE OF ARALDITE (DIA. 63.5 MM).

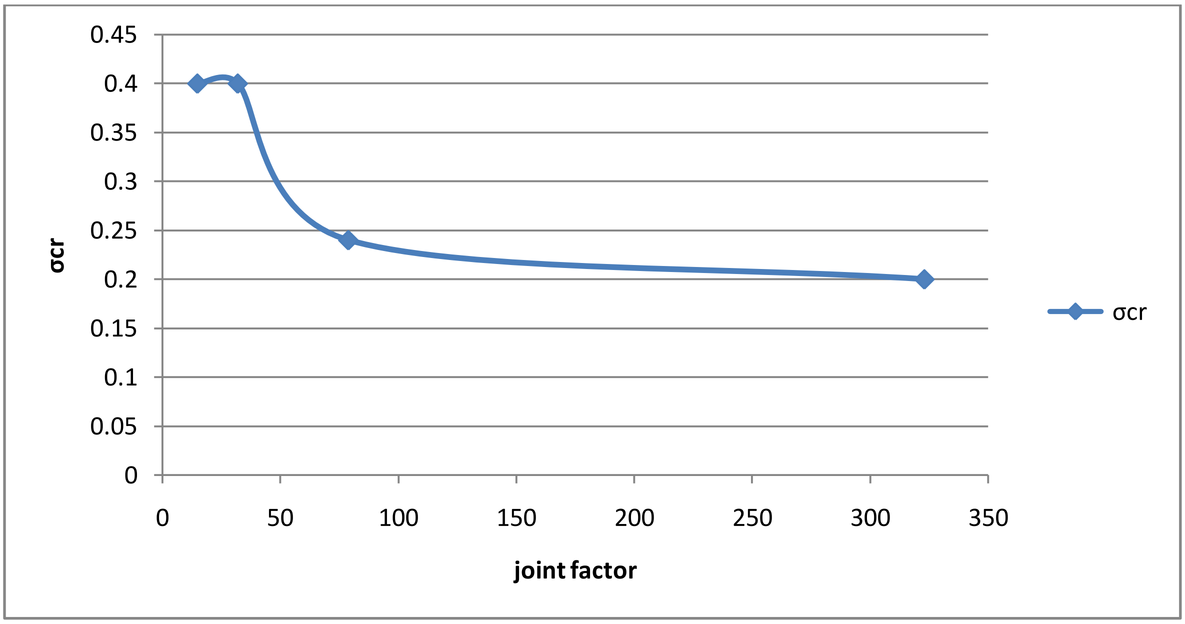

FIG 23 - VARIATION OF STRENGTH RATIO OF JOINTED SPECIMENS TO INTACT SPECIMENS WITH JOINT FACTOR IN CASE OF CEMENT (DIA. 63.5 MM).

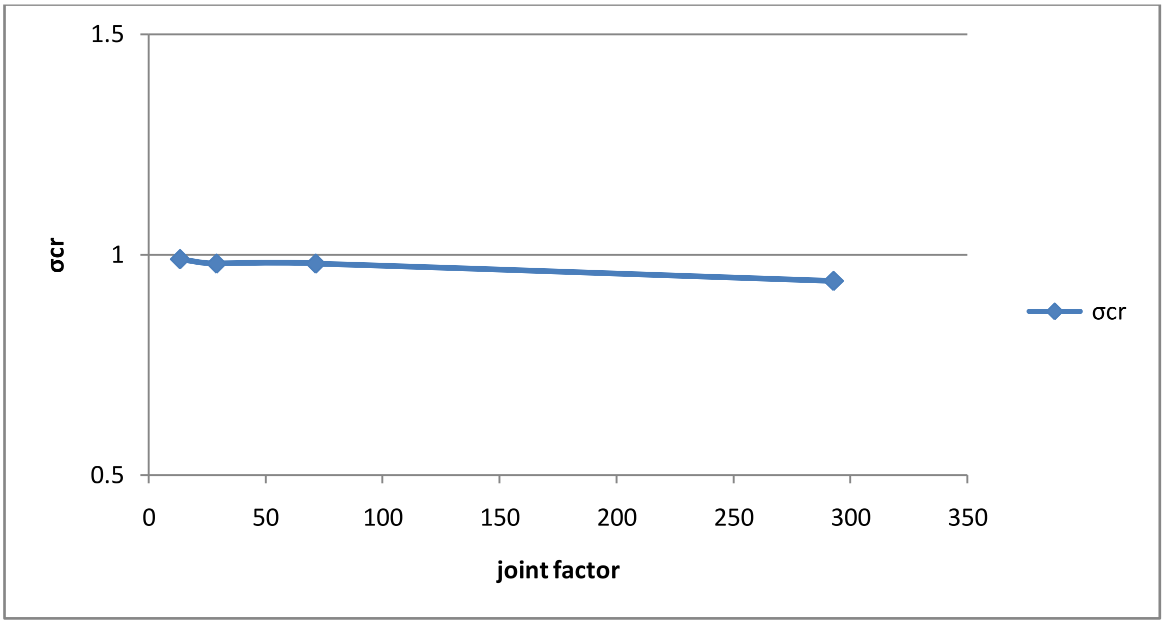

FIG 24 - VARIATION OF STRENGTH RATIO OF JOINTED SPECIMENS TO INTACT SPECIMENS WITH JOINT FACTOR IN CASE OF ARALDITE (DIA. 90 MM).

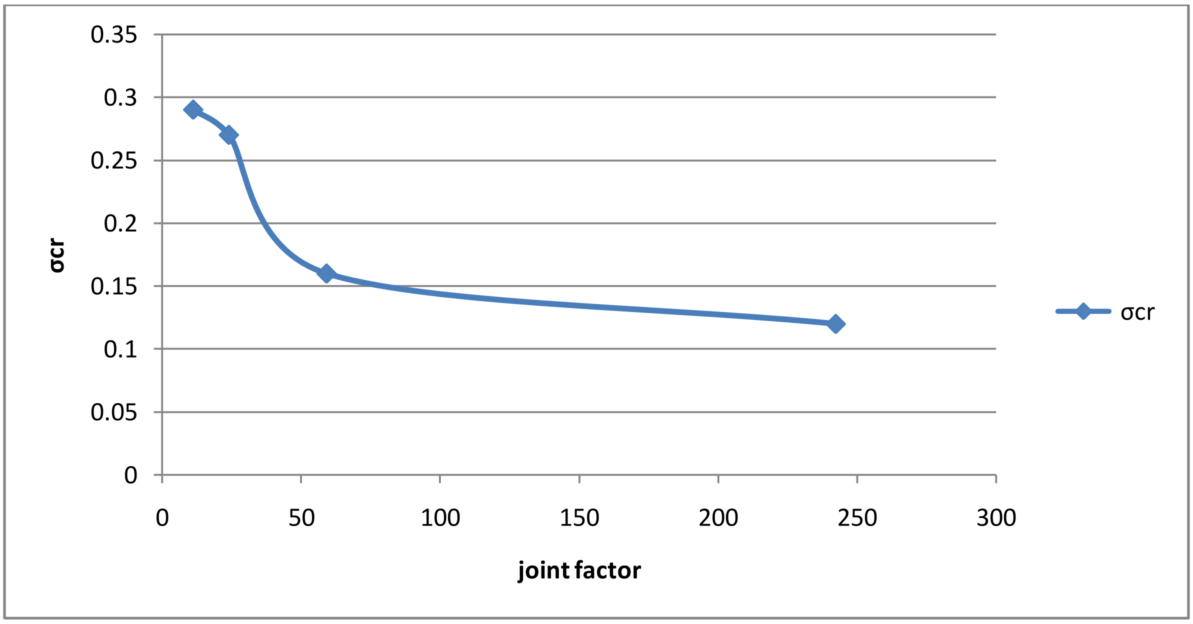

FIG 25 - VARIATION OF STRENGTH RATIO OF JOINTED SPECIMENS TO INTACT SPECIMENS WITH JOINT FACTOR IN CASE OF CEMENT (DIA. 90 MM).

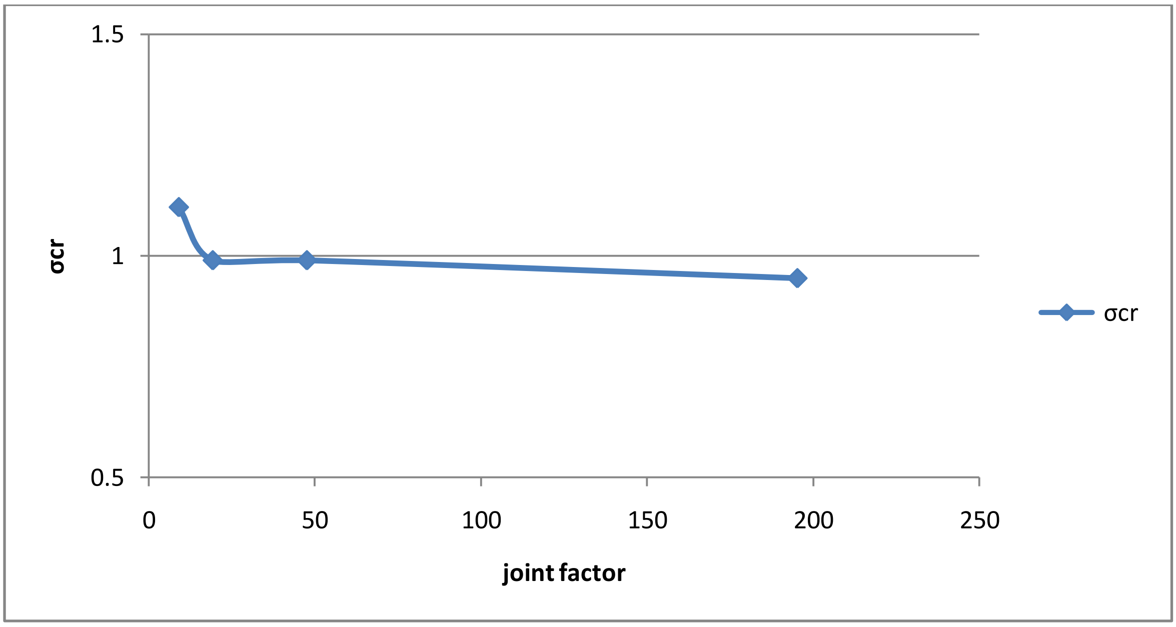

FIG 26 - VARIATION OF STRENGTH RATIO OF JOINTED SPECIMENS TO INTACT SPECIMENS WITH JOINT FACTOR IN CASE OF ARALDITE (DIA. 114 MM).

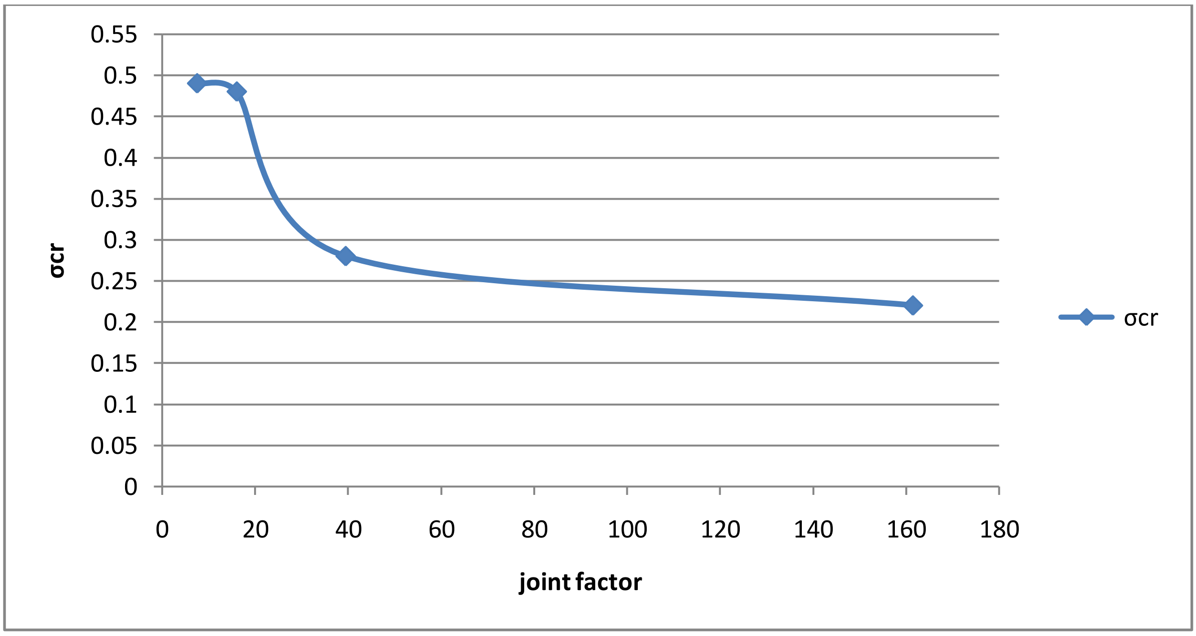

FIG 27 - VARIATION OF STRENGTH RATIO OF JOINTED SPECIMENS TO INTACT SPECIMENS WITH JOINT FACTOR IN CASE OF CEMENT (DIA. 114 MM).

CHAPTER – 5

CONCLUSION

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

1. Uniaxial compressive strength of intact specimen of plaster of paris is depending upon the diameter of specimen.
2. As we increase the diameter of specimen from 63.5 mm to 90 mm strength increases and when increased from 90 mm to 114 mm strength decreases.
3. The strength of jointed specimen depends upon the diameter of specimen, inclination of joint and material used to join the specimens.
4. When Araldite is used as a binding material there is approximate no difference between the strength of intact specimens and jointed specimens.
5. When cement is used as a binding material there is considerable difference between the strength of intact specimens and jointed specimens.
6. As the value of joint factor increases the strength of jointed rock decreases.

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. Different binding material can be used and their behavior on jointed rocks may be studied.

REFERENCES

1. Arora, V. K. (1987). "Strength and deformation behavior of jointed rocks." PhD thesis, IIT Delhi, India.

2. Arora, V.K. and Trivedi, A. (1992). “Effect of kaolin gouge on strength of jointed rock”. Regional symposium on rock slopes, India.

3. Barton, N. (1988). Rock Mass Classification and Tunnel Reinforcement Selection using the Q-System. Rock Classification Systems for Engineering Purposes, ASTM STP 984, pp. 59-84.

4.Barton, N. and Bandis, S. (1990). Review of predictive capabilities of JRC-JCS model in engineering practice, Proc. Int. Conf. on Rock Joints, Rotterdam, Balkema, pp. 603-610.

5. Bieniawski, Z.T. (1973). Engineering classification of jointed rock masses, J.S. Afr.Inst. Min. Metall., Vol. 15, pp. 335-344

6. Brown, E. T. (1970). "Strength of models of rock with intermittent joints." J. Soil Mech. Found. Div., 96(6), 1935-1949.

7. Brown. E. T., and Trollope, D. H. (1970). "Strength of model of jointed rock." J. Soil Mech. Found. Div., 96(2), 685-704.

8. Deere, D.U. and Deere, D.W. (1988). The Rock Quality Designation (RQD) Index in Practice, Rock Classification Systems for Engineering Purposes, ASTM Special Publication 984, pp. 91-101.

9. Einstein, H. H., Nelson, R. A., Bruhn. R. W., and Hirschfeld, R. C., (1970). "Model studies of jointed rock behaviour." Proc., Symp. Of Rock Mechanics, Berkeley, California, 83-103.

10. Einstein, H. H., and Hirschfeld, R. C. (1973). "Model studies in mechanics of jointed rocks." J. SIJiI Mech. Found. Div.. 99(3), 229-248.

11. Goldstein, M., Goosev, B., Pyrogovsky, Tulinov, R., and Turovskaya A. (1966). "Investigations

of mechanical properties of cracked rock." Proc.,1st Congress of Society of Rock Mechanics, Lisbon, I, 521-524.

12. Goodman, Richard E, Introduction to Rock Mechanics. New york, John Wiley, 1980

13. Gerrard, C. M. (1982). Elastic models of rock masses having one, two and three sets of joints, Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.,Vol.19, pp 5-23.

14. Hayashi, M., (1966). "Strength and dilatancy of brittle jointed mass. The extreme stochastic and anisotropic failure mechanism." Proc.,1 st Int. Congress on Rock Mechanics, Libson, I, 295-302.

15. Hoek, E. and Brown, E. T. (1997). Practical estimates of rock mass strength, Int *J.* Rock. Mech. Min. Sci., V 01. 34, No.8, pp. 1165-1186.

16. I.S. 11229 – 1985, Specifications for shear box for testing of soils.

17. I.S. 2720 (part XIII) – 1972, Determination of shear strength parameters from direct shear test.

18. Jaeger C., Fundamentals of Rock Mechanics. London, Chapman and Hall, 1969.

19. Laboratory testing of rocks manual

20. Lama, R. D. (1974). "Uniaxial compressive strength of jointed rock." Institute of soil mechanics and rock mechanics, P-Muller Festschrift.ed., Karlsruhe,Germany, 67- 77.

21. Ramamurthy,T. (1994). "Strength and modulus responses of anisotropic rocks." Rock Eng.. 1(13), 313-329

22. Roy, N. (1993). "Engineering behavior of rock masses through study of jointed models." PhD thesis, Indian Institute of Technology, Delhi, India.

23. Shahin, M.A., Jaksa, M.B. and Maier, H.R. (2000). “Predicting the settlement of shallow foundations on cohesion less soils using back-propagation neural networks” Research report No. R 167, Dept. of Civil and Environmental Engineering, University of Adelaide, Australia.

24. Simon Hilykin (2001) Neural Networks (A Comprehensive Foundation) Addison Wesley Longrnan (Singapore) Pte. Ltd, Indian Branch Delhi.

25. Sitharam, T.G., Sridevi, J. and Shimizu, N. (2001). Practical equivalent continuum characterization of jointed rock masses, International Journal of Rock Mechanics and Mining Sciences, Vol. 38, pp. 437-448.

26. Terminology, ISRM, 1975

27. T G. SITHARAM, K ET. Al (2001).Intelligent Prediction of Elastic Properties of Jointed Rocks, J. of Rock Mech. & Tunneling Tech. Vol. 9.

28. Varadarajan, A., Sharma, K.G., Desai, C.S. and Hashemi, M. (2001) Analysis of a powerhouse cavern in the Himalayas, Int. J. Geomech., Vol. I, No.1, pp.109-127.

29. Wiliam F. Bawden, Mahdi Moosavi (2002). Shear strength of Portland cement grout.

30. Yaji, R. K. (1984). "Shear strength and deformation of jointed rocks."PhD thesis, Indian Institute of Technology, Delhi, India.