A COMPARATIVE STUDY ON JOINTS

WITH AND WITHOUT GOUGE FILL

A THESIS SUBMITTED IN PARTIAL FULFILLMENT

OF THE REQUIREMENTS FOR THE DEGREE OF

Master of Technology

In

Civil Engineering

BY

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Roll .No- 210CE1020

DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY

ROURKELA-769008,

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NATIONAL INSTITUTE OF TECHNOLOGY

ROURKELA-769008,ORISSA, INDIA

CERTIFICATE

This is to certify that the thesis entitled,***"A Comparative Study On Joints With And Without Gouge Fill",*** submitted by **RajivLochanSahu** in partial fulfillment of the requirements for the award of Master Of Technology Degree in **Civil Engineering** with specialization in **GEOTECHNICAL ENGINEERING** at the National Institute of Technology ,Rourkela is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge,the matter embodied in the thesis has not been submitted to any other university /institute for the award of any degree or diploma.

Date

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Date Rajiv Lochan Sahu

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Abstract

Rock is a discontinuous medium and the discontinuity may be in the form of joints or faults. Hence, the strength of rock mass generally depends on the type of discontinuity present in the rock mass. The strength of jointed rock mass generally depends on the joint spacing, inclination parameter and the roughness parameter. This joint roughness is a vital parameter which generally governs the strength of the rock mass. The roughness is usually computed with the help of direct shear test ($r = \tan\varphi_j$). The deformation characteristics of rock mass are another parameter which is usually taken care in addition to the strength of rock mass and can be computed with the help of uniaxial compressive strength test. As collection of jointed rock mass and field testing of rock mass is tedious as well as difficult, hence jointed rock mass models are generally made in the laboratory itself. In the present study, plaster of Paris was used for modeling laboratory specimens as it is locally available and at the same time casting of jointed rock mass specimen can be done easily. The joints in rock mass specimen were made at various angles of orientation (β^0) which is varying from 0^0-90^0. These models were possessing joints with and without gouge fill. Clay was used as gouge material. Here, an attempt was made to compare the results of strength and deformation characteristics of jointed rock mass with and without gouge fill by using model material plaster of Paris. From the experiments it is found that for single jointed rock mass specimen without gouge fill at $\beta = 30^0$ strength was found to be 0.22 MPa which is minimum and at $\beta = 90^0$ strength was found to be 7.34 MPa which is maximum. For single jointed rock mass specimen with gouge , further trend of decrease in strength was found and this is due to decrease in roughness parameter.Here , also at $\beta = 30^0$ strength was found to be 0.11 MPa which is minimum and at $\beta = 90^0$ strength was found to be 6.79 MPa which is maximum. An empirical relationship $\sigma_{cr} = e^{-0.008 \times J_f}$ is applicable for joints with gouge.

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Nomenclature

J_n	joint frequency
n	inclination parameter
r	joint strength parameter
σ_{nj}	the normal stress on the joint
σ_{cm}	compressive strength of the composite specimen;
σ_{ce}	compressive strength of the element constituting the block;
L	length of the specimen;
l	length of rock element;
a	Constant
b	Constant
e	Constant
σ_c	compressive strength;
Ed	deformation modulus;
K	strength of the specimen containing more than 150 joints;
υ	Constant
L	length of the specimen;
1	length of the element.
E_r	initial tangent modulus (computed at 50% of failure stress);
σ_3	confining pressure,

P_a	atmospheric pressure
K	modulus number
M_{rj}	modulus ratio
σ_{cr}	uniaxial compressive strength ratio
σ_{cj}	uniaxial compressive strength of jointed rock
σ_{ci}	uniaxial compressive strength of intact rock.
J_f	joint factor
E_r	elastic modulus ratio
E_{tj}	tangent modulus of the jointed rock
E_i	tangent modulus of the intact rock
φ_r	the residual angle of friction
φ_b	basic friction angle
i	the angle of the saw-tooth face.
u	the water pressure.
σ_n'	the effective normal stress
c	Cohesion
φ	friction angle

CHAPTER 1

Introduction

Rock is a discontinuous medium with joints .These discontinuities may exist with or without gouge material. The strength of rock mass depends on the behavior of these discontinuities and at the same time field tests of rock joints are expensive, time consuming and difficult to carry out at sites so there is a need to develop a indirect method for computation of rock strength. In present study an attempt is made to compare the strength of joints with and without gouge .Variation of uniaxial compressive strength (UCS) with respect to a parameter named joint factor (J_f) was seen for both the cases. Joint factor can be computed with the help of photos of a given site and it can be used to compute the strength of rock joints with various angles of inclinations as present at the given site and thus we would save time, adopt economy and would be able to predict the stresses of jointed rock mass which would be helpful for design work of jointed rock mass. Often jointed rock specimens are difficult to obtain from the field to test them in the laboratory to assess their strength and modulus values .Testing of large specimens in situ particularly at greater depths has become quite time consuming and costly the only approach available is to introduce the influence of joints, their orientation with reference to the stress directions and the strength along the most unfavorable joint into the unconfined compressive strength of the intact rock to obtain that of the joint mass. In fact the first attempt was made by Hoek and Brown (1980) to characterize the jointed mass through the material parameters in m_j and s_j by adopting the field and laboratory data of Punguna andesite. To make a more realistic assessment of strength of jointed rocks, extensive laboratory tests were conducted (Yaji 1984, Arora 1987, Roy 1993, and Singh 1997) on various grades of plaster of Paris, sandstones, granite and block specimens of sand –lime bricks. The joints in the intact specimen were either cut, broken in the desired direction , step shaped or berm shaped, with or without gouge material. The analysis of data from uniaxial and triaxial tests confirmed that the important factors which influence the strength and modulus most are

- Joint frequency ,i.e number of joints per metre ,
- Critical joint orientation , i.e , angle β with respect to the major principal stress direction
- The strength along the critical joint. The combined effect of these three factors has been represented by the joint factor as(Ramamurthy 1993,Ramamurthy and Arora 1994).
- Joint factor(J_f)

  $J_{f=} J_n/ (n. r)$

- Where J_n is joint frequency ,n is an inclination parameter that depends on the orientation of the critical joint and r is the joint strength parameter , which depends on the condition of the joint , i.e. cemented tight , open , weathered or filled with gouge. The values of n were obtained by taking the ratio of log (strength reduction) at $\beta = 90^o$ to log (strength reduction) at the desired value of β. The values of n were found to be almost the same, irrespective of the joint frequency. The joint strength parameter, r, is obtained from a shear test along the joint and is given by $r=(\tau_j / \sigma_{nj})= \tan \varphi_j$ where τ_j is shear strength along the joint ,σ_{nj} is the normal stress on the joint , and φ_j is the equivalent value of the friction angle , i.e. it includes the influence of cohesion intercept in the case of cemented joints. In the absence of shear tests on the joints for rocks with tight joints (unfilled), the value of r depends on the uniaxial compressive strength of the rock. Gouge is defined as a clay like material occurring between the walls of a fault as a result of the movements along the fault surface as per (ISRM, 1978, dictionary of geological terms, 1962). Geological discontinuity in rock masses are formed due to geological process and tectonic movements. Continuous and intensive weathering of rock occurs mainly in tropical countries as a result of which surface and interior part of rock masses are affected. Joints provide a free passage for water and other weathering agents to move into the rock. Due to weathering the material of the joint surface gets disintegrated and decomposed to form a completely weathered material which is much weaker than the intact rock as a result of which a completely weathered material is sandwiched in between the joint blocks sometimes filling of joint aperture may also occur due to in situ deposition .Both weathering of joint surface and in situ deposition collectively leads to formation of a “filled joint”. Together with the weathered joint surface, the nature of contact between the interfacing joint surfaces and the nature of infill create a very complex deformational behavior of filled joints as compared to unfilled joint .Filled joint often exhibits high deformability and low shear strength when subjected to loading these characteristics are unfavorable for any civil engineering constructions particularly when it involves excavation of rock mass. They may induce instability into excavated surfaces such as rock slopes and tunnel walls. In present study joints with and without gouge fill is taken into consideration for comparison of their strengths.

CHAPTER 2

Literature cited

2.1 Introduction

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 depends 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. 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 shales, 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.

Rock mass is an inhomogeneous and anisotropic material. It is formed by intact rock substance and very often disrupted by different types of discontinuities such as joints, bedding planes, cleavage and fractures formed either by tectonic deformation, sedimentation or weathering process (Wan Mohd Kamil, 2002). The strength of rock mass does not normally depend on its material characteristics, but on the strength of the discontinuities in it. Unfortunately these discontinuities are generally weaker than the rock. Therefore, the strength and characteristics of discontinuities must be studied to interpret the stability of the rock mass involved. Among that filled joints are likely to be the weakest elements of any rock mass in which they occur and to exert a significant influence of its behavior (de Toledo and de Freitas ,1993).In the field of rock engineering ,certain important characteristics of filled joints can be interpreted through laboratory tests on simulated and artificial joints model. Many elements of the filled joint can be analyzed to estimate its behavior under different conditions. Throughout the years ,quite a number of researches have been carried out to study various characteristics of filled joints ,they include authors like de Toledo and de Freitas (1993 and 1995) ,Phien –Wej et al.(1990), Pereira(1990) , Papaliangas et al.(1993) and Ladanyi and Archambault (1977).

The outcomes of these studies have contributed significantly towards the understanding of the behavior and characteristics of filled joint, particularly the effect of this discrete discontinuity on the deformational behavior of rock mass. This understanding is vital in assessing and predicting potential slope failure or rock sliding ,which is most related to planar weaknesses, like filled joint(Waltham, 2002).

2.2. Joint

Price (1966) described joints as cracks and fractures in rock along which there has been extremely little or no movement .In geological terms, the word “joint” is frequently is treated as an omnibus term and has been used to describe structures that vary widely in character .They have occurred and are present within all types of rock (Bell, 1983) hence, joints are often encountered during excavation of any rock masses. Since early days, the formation and origin of joints have attracted many researchers interest. Many types of forces have been advocated to account for the formation of the joints, which include torsion, compression and shear , tension and also fatigue phenomenon .Price(1966) suggested that the majority of the joints are the post compression structures , formed as a result of the dissipation of residual stress after folding has occurred .Rock masses are continually affected and modified by weathering and erosion .Mechanical weathering or disintegration , breaks down rock mass into smaller blocks by physical interaction (friction between rock and water ,wind, rain drop,etc) and the action of temperature .

Beavis (1992) explained that joints develop through different processes in different rock masses .Igneous rock is formed when the hot lava (from the inner of earth) cools down and solidifies (when it flows to the outer surface of earth).However , hot lava continues to flow upwards to the surface . The up-pushing lava tends to crack the rock solid above and creates fracture or joints in it. Joints may also develop in igneous rock due to shrinkage of rock mass when magma cools down. In sedimentary rocks , joints develop when the rocks shrink , due to the drying process of rocks. In summary ,joint ,as other type of fractures ,are formed as a result of different processes , such as mineralization, metamorphism, crushing,brecciation, mylonitization , metasomatic replacement ,etc.(ChernyshevaandDearman ,1991).

Ladanyi and Archambault (1977) categorized joints into four classes to ease studies of joints:

1.Clean

2.Coated

3.Filled with clay like infilling

4.Filled with sand like infilling

2.2.1 Filled joints

Filled joints posses very unique characteristics .However some of them resemble the properties of an unfilled joint or fracture. It is believed that filled joints develop gradually from unfilled joints , so as to maintain their certain behaviours and characteristics. Generally there are two types of filed joints based on the origin of the infill. Infill within the apertures of the joints may result from continuous weathering of joint surface, or in situ deposition of ground surface material from the nearby area.

Mohd Amin et al. (2000) briefly described the formation of filled joint in granite through continuous weathering .Joints or fractures are discontinuities (weak plane) that are permeable .Water penetrates through joint surface , and cause weathering to happen. The least stable feldspars at joint surfaces is firstly broken down during weathering. Further weathering can be noticed by the penetration of discoloration inwards from the joint surfaces. Hydrolysis of feldspars and mica eventually increases the volume of rock material .Expansion of joint blocks tends to push and press the opposite joint surfaces together .When compressed, the joint surfaces will crack and break down into small pieces. Consequently ,joint surface opens up and fresh rock (initially deep inside the joint) is exposed, and subjected to continuous weathering .The torn pieces from the weathered joint surfaces ,forms the infilling between the joint apertures .As they are of small pieces ,they possess greater effective surface for weathering . Therefore, the infillings of a filled joint is often consists of highly weathered materials (grade V or Residual soil).

Beavis (1992) explained that weathering and the releasing of load above rock due to erosion would lead to the formation of an opened joint .These joint openings ,might be clean without infillings, or filled with secondary minerals .These minerals could have been caused by hydrothermal changes or transportation or weathering .Wide opened joints near to the surface of earth may contain infillings deposited from the earth surface Chernyshev and Dearman(1991) drew up a classification chart of joint filler , based on its mode of deformation

Table 2.1

Classification of joint filler by origin (after Chernyshev and Dearman(1991))

Deposition of fracture filler	Description of filler based on material	Composition and properties of fracture filler
Chemical or physicochemical	Magmatic Hydrothermal and pneumatolytic Hypergene Artificial	Rock healing fracture solidly Rock healing fracture Colloidal formations which cause fracture narrowing or healing chemical grout infilling fracture

Mechanical	Tectonic Hypergene Artificial	Mylonite,faultbreccias. Compact, impervious, low strength slightly compressed Clastic or clay ,loose rocks impervious, low strength ,compressed cement grout infilling fracture
Organic	Phytogenic Zoogenic	Plant roots, rotting residues. Permeable medium ,facilitates weathering organic residues and rotting products washed into fractures weakens rock mass and facilitates weathering

2.3 Filled joint elements

Filled joints pose very unique and complex behaviours due to their components are made up of materials of different properties. There are number of components having significant influence on their characteristics .These components are to be studied individually to enable the interpretation of their interactive effect on a filled joints behaviours to be made .Over the years number of studies on filled joints have been carried out .Generally ,certain joint elements have been recognized as having significant influence on joint behavior , such as the material of infillings ,the thickness of infillings and the contact condition between joint blocks and infillings .Changes in these elements directly leads to alteration of joint properties .

2.3.1Material of infilling

In filled joints ,the physical and mineralogical properties of the material separating the joint walls are the primary concern in determining its shear strength and deformation characteristics . Filling materials vary greatly in their mechanical characteristics ,from very soft to very hard and strong (Franklin and Dusseault ,1989).

Tulinov and Molokov(1971) defined five types of filling material according to their genesis:

1. Loose material of tectonic crushed zones.
2. Products of decompression and weathering of joint walls .
3. Soils of the shear zones of rock slides.
4. Filling materials of karst cavities ,which has been formed by leaching carbonaceous rocks and then shifted by the ground water flow.
5. Filling materials of joints and cavities brought from the surface ; or it may be of mixed origin.

Brekke and Howard (1972) ,on the other hand, distinguished seven major groups of joints/infilling materials according to their strength :

1. Healed joints
2. Clean discontinuities
3. Calcite fillings

4. Coatings /filling with chlorite, talc and graphite
5. Inactive clay
6. Swelling clay
7. Material that has been altered to a more cohesion less material (like sand)

The main difference between sand and clay is their permeability. Clay is considered soil of very low or non permeability while sand is a highly permeable soil .The low permeability increase the effect of pore water pressure on the strength of soils .In low permeable soil ,water is trapped inside the pores when the soil is compacted .Contrary ,in highly permeable soil like sand , pore water is drained out of soil immediately once the soil is loaded and does not influence to the strength of sand.

Cheng and Evett (1987) described that, since the shear strength of most cohesion less soil is resulted from the interlocking between grains, values of friction angle differ little whether the soil is wet or dry. This clearly explains that the moisture content display very small effect on the shear strength of cohesion less soil

Mohd Amin and Awang (2002) carried out uniaxial compression test on modeled filled joints and found that a significant reduction in joint stiffness and young's modulus may occur when weak material, like CW granite, is present in joint aperture .This is due to the high axial strain and low young's modulus exhibited by the infill material .The series of tests conducted strongly indicated the effect of infilling on the compressibility of joint.

2.3.2 Particle shape of infill material

Particle shape has a pronounced effect on properties of soil, such as ,void ratio, compressibility ,crushability, etc. Varying particle shapes can lead to different engineering properties even on granular soils at the same relative density (Holubec and D'Appolonia ,1973).

Generally, particle shape is defined by its angularity /roundness and sphericity.Sphericity is the ratio of the surface area of a sphere having the same volume as the particle to the surface area of the particle , while angularity is the measure of the curvature of the corners to the average curvature of the particle (Holubec and D'Appolonia ,1973).Judging from the aspect of angularity ,particle shape can be divided into five main categories ,which are angular, sub angular ,sub-rounded, rounded and well rounded (Franklin and Dusseault,1989).

(Holubec and D'Appolonia ,1973) studied the effect of particle shape on the engineering properties of granular soil .With the increase in particle angularity ,the maximum and minimum void ratio of a soil is found to be increasing .The shear strength or the friction angle is found to be greater for soils with more angular particles (also proven by Koerner ,1970) .It is because the angularity provides interlocking effect between grains ,thus increasing the resistance to shear .Whenever a grain is considered to be a polygon of finite number of sides (high angularity) ,the concept of rolling friction is no more valid and is to be replaced by overturning friction(Pereira ,1990).Besides the more angular particle results in greater failure strain for a given relative density .Tests carried out showed that crushed stone with angular particles has greater elastic and permanent deformations than crushed gravel composed of rounded particles (Haynes ,1966;Dunlap,1966;Holubec,1969).Particle angularity is also proven to contribute to the resistance to the dynamic penetration of soils .However ,angular particles are found to be more crushable than the spherical grains (Feda,2002).

2.3.3 Thickness of infilling

Thickness of infilling layer has significant influence on filled joints strength .The range of infill thickness with regard to the particle size limits the type of movement of the filler particles.

Pereira (1990) studied the movement of grains in a filler of thickness twice greater than the grains size .When sheared grains with contact to the flat and planar joint surface tend to roll .However, grains on the other side may block the rolling motion and force it into sliding motion .In the middle of infill layer (soil to soil contact), each grain moves over one or more grains to occupy the voids next to them.

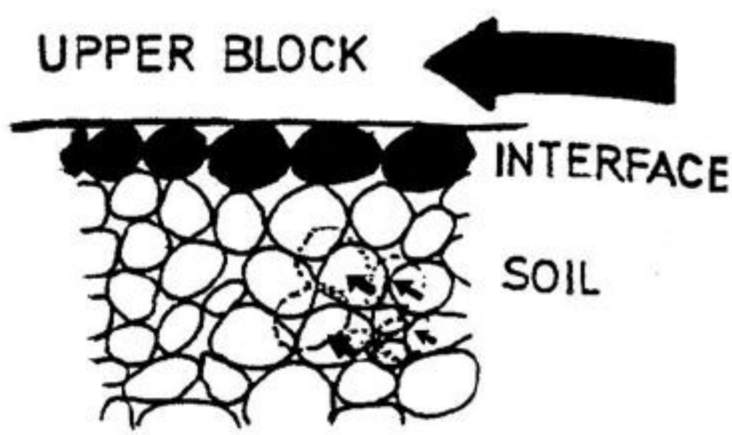


Fig 2.1.Layers and movement of grains of infill (after Pereira (1990))

At the same time ,it can be expected that when the filler is a grain thick contact of grains to flat surface on both sides allow a rolling motion to take place ,imposing only a low rolling friction rather than the high sliding friction .

Barton(1974) idealized four hypothetical thickness of clay filling in a rough ,undulating joint .The shear characteristics of these filled joints can be briefly described as below :

A. Almost immediate rock/rock asperity contact .Shear strength will be very little different from the unfilled strength because the rock/rock contact area at peak strength is always small. Dilation due to rock/rock contact will cause negative pore pressures to be developed in filling if shearing rate is fast.
B. Similar to A, but a larger displacement is required to reach peak shear strength reduced dilation reduces tendency for negative pore pressures.
C. No rock/rock contact occurs anywhere, but there will be a buildup of stress in the filling where the adjacent rock asperities come closer together .Greater shear strength obtained if shearing rate is low..Low shearing rate allows drainage to occur ,avoiding the increase in pore pressure that can reduce the effective stress on the filling.
D. The influence of the rock wall will disappear ,as the infillings are several times thicker than the asperity amplitude .If the filling is uniformly graded and mostly clay or silt ,the shear strength behavior can be estimated by basic soil mechanics principle.

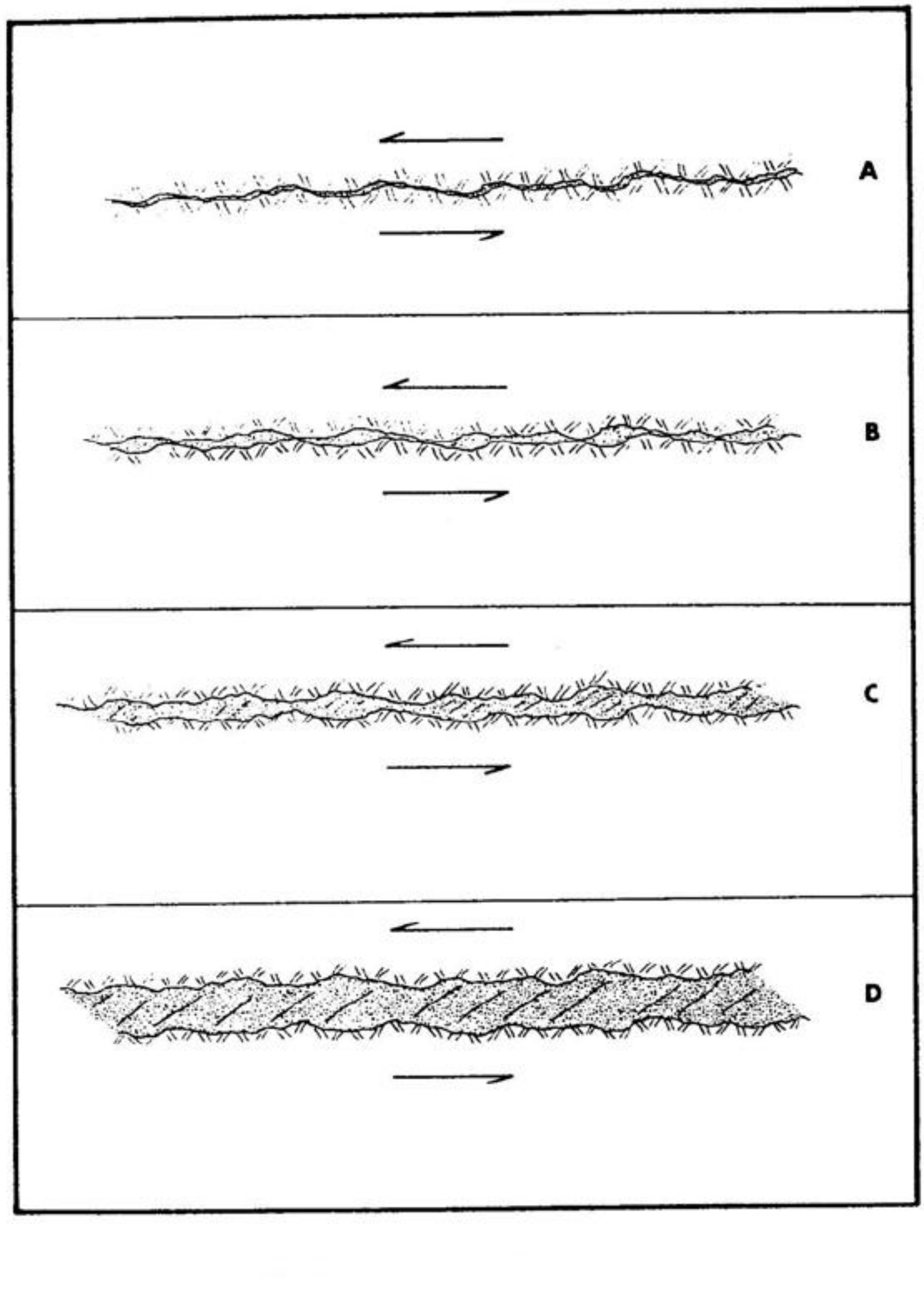


Fig 2.2Four categories of discontinuity filling thickness (After Barton, 1974)

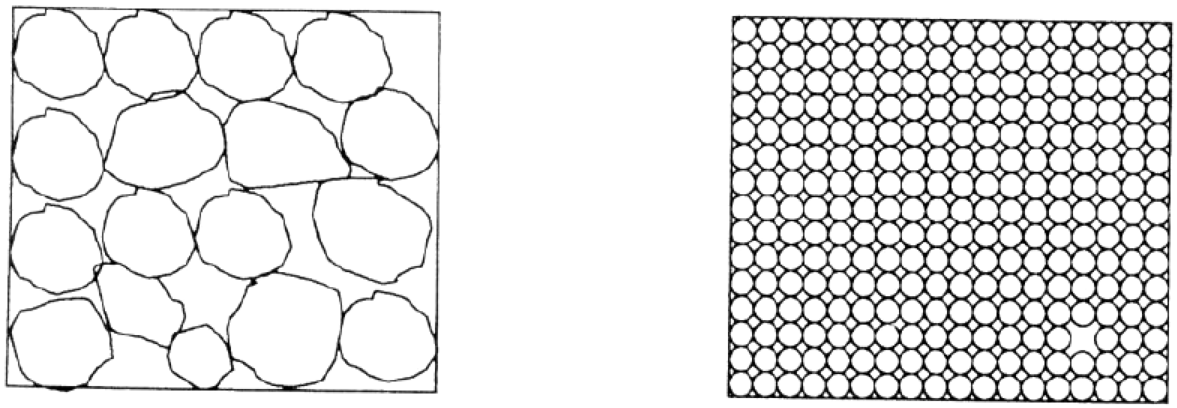
Over the years many researchers have done their studies on the effect of infill thickness to the strength of the joint systems. Majority of them have shown that when the infill layer is thicker, the joint system is weaker .Arora and Trivedi (1992) found out that for filled joint with thicker infilling, its uniaxial compressive strength is relatively smaller than the unfilled one. Through triaxial strength on filled joint, Sinha and Singh (2000) proved the weakening of joint system by the increasing infill thickness.

2.3.4 Particle size distribution

Particle size distribution is the content of grain of different sizes in a soil sample .It is an important parameter for classifying granular and relative coarse soil sample .It describes soil sample physically, from which, subsequently, the physical and mechanical behavior or sample can be interpreted.

Generally ,potential crushing of mineral grain increases with the grain size (Hardin,1985,Ong , 2000).Contact area between coarser grain is smaller compared to finer grain .Therefore when loaded or stressed the effective stress on each grain is much larger ,resulting in greater crushing of grains .

Feda (1971) proved that the poorly graded sample with high content of voids is more crushable than the well graded sample.



(a) (b)

Fig 2.3 Grain arrangement in (a) coarse grained sample(b) fine grained sample.(after ,Ong ,2000)

Farmer and Attewell (1973) proved that , apart from the crushability, compressibility of a soil sample also increases with its grain size .The presence of large amount of voids in coarse grained sample allows more particle rearrangement to take place .Compression comes mostly from the rearrangement of the grain particles to fill the voids within .

2.3.5 Surface roughness

Brady and Brown, (1985) Surface roughness is a measure of the inherent surface unevenness and waviness of the discontinuity relative to its mean plane .It is a measure factor determining the shear strength of a joint .The nature of the opposing joint surfaces influences the behavior of rock mass as the smoother they are, the easier movement can take place along them (Bell, 1983).

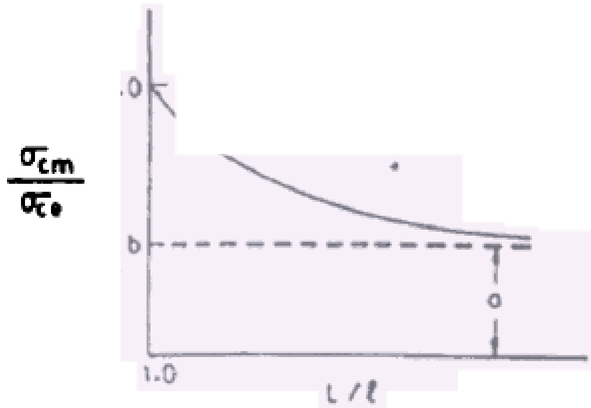


Fig. 2.4Relative strength of mass after Goldstein et al. (1966)

Summary of Experimental Studies on Jointed Rock

A brief review of the numerous experimental studies on the jointed rock samples with different joint fabrics is presented here. Uniaxial compression tests were conducted on composite specimens(Goldstein et al. 1966) made from cubes of plaster of Paris and the following relationship is suggested:

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

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

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.

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:

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

Where σ_c =compressive strength; Ed=deformation modulus; K = strength of the specimen containing more than 150 joints; υ = constant; L = length of the specimen; and 1 = length of the 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 for the intact rock and fitting the following relation:

$$E_r = KP_a(\sigma_3/P_a)^n$$

Where Er = initial tangent modulus (computed at 50% of failure stress); σ_3 = confining pressure, and P_a = atmospheric pressure. The results of these experiments were analyzed for strength and deformation purposes. It was found that the mode of failure is dependent on the confining stress and orientation of the joint. Joint specimens with rough joint surface failed by shearing across the joint, by tensile splitting, or by a combination of thereof. Arora (1987) conducted tests on intact and jointed specimens of plaster of Paris, Jamarani sandstone, and Agra sandstone.

2.4 Modes of failure in jointed rocks

When the joint sets are orthogonal and one of the joint set is continuously dipping often one of the four modes of failure in jointed mass may be expected. These modes are controlled primarily by the Orientation angle (β) of this continuous joint with the vertical and they are

1. Splitting of rock material

2. Shearing of rock material
3. Sliding on the continuous joint
4. Rotation of rock blocks

In the case of layered rock mass buckling failure of inclined layers may also be expected .in general the following ranges of angle (β) ,for different modes of failure are suggested based on the experimental results on jointed blocks

1. Splitting of rock material is likely to occur for (β) < 10^0 and (β) > 80^0
2. Shearing of rock material mostly ignoring the presence of joints for (β) < 20^0 and (β)=$70^0$to 80^0
3. Sliding along the continuous joint for (β) between $30^0$and 60^0i.e. (45-(φ'/2)) with the vertical .
4. Rotation of rock blocks is likely to take place for (β) = 10^0 to 20^0 with the increase in height to width ratio of blocks.
5. The other mode of failure is by buckling of thin layers ; this is a different from of rotation of layers of rock mass when (β)<30^0 and when the ratio of the width of the layer (B) to its thickness (t) (i.e slenderness ratio) is greater than 10 as in the case of plates and columns more precisely this slenderness ratio (B/t) depends on the modulus ratio ,M_{rj} of the material ,from Eulers theory for per metre length of layer as given in equation below

   $(B/t) = 0.5\,\pi\,(M_{rj} x t)^{0.5}$

   For a layer thickness of 1m , the slenderness ratio for buckling failure will be 11,16,22 and 35 for M_{rj} values of 50,100,200,500 respectively .For a lesser thickness of layer ,this ratio will be lower.

CHAPTER 3

Some basic concepts

3.1 Problem background

Filled joint is considered as the most critical discontinuity in rock masses, its deformability ,compressibility and its shear behavior is matter of concern when it comes to the stability of rock .Each component of filled joint ,such as joint surface ,filling material present in between the joints and joint blocks displays its own distinct characteristics. Hence forth in depth knowledge on the characteristics of each relevant component is essential to understand the overall behavior of the filled joint under loading .In present study an attempt is made in order to compare the strength of a joint with and without gouge fill. This comparison would help us to find the strength of the jointed rock mass with and without gouge fill as per our requirement (i.e. in any of one case or both the cases as said earlier) and would just save time and at the same time it would be economical. Thus this comparison would be helpful in evaluating geological hazards like landslides ,seismic activity etc. Selection and layout of the construction site can also be done with the help of this study. Similarly this study can be helpful in analysis of stability of rock masses, in design of blasting operations, in design of support system, in design of hydraulic fracturing programs, in design of instrumentation programs, in evaluation of excavation characteristics, in study of rock deformation at high temperatures and pressures (structural geology).

Table. 3.1

Generally rock is named as per description

Grade	description	Lithology	Excavation	Foundations
V I	Soil	Some organic content no original structure	May need to save and re-use	Unsuitable
V	Completely weathered	Decomposed soil, some remnant structure	Scrape	Assess by soil testing
I V	Highly weathered	Partly changed to soil,	Scrape	Variable and unreliable
I II	Moderately weathered	Partly changes to soil	Rip	Good for small structures
I I	Slightly weathered	Increased fractures and mineral staining	Blast	Good for anything except large dams
I	Fresh rock	Clean rock	Blast	Sound

3.2 *EDX-Energy dispersive X-ray analysis*

This technique is used in conjunction with SEM and is not a surface science technique, and electron beam strikes the surface of a conducting sample .The energy of the beam is typically in the range 10-20Kev . This causes X-rays to be emitted from the point of the material. The energy of the X-rays emitted depends on the material under examination .The X-rays are generated in a region about 2 microns in depth , and thus EDX is not a surface science technique .By moving the electron beam across the material an image of each element in the sample can be acquired in a manner similar to SAM . Due to low X-ray intensity images usually take a number of hours to acquire .Elements of low atomic number are difficult to detect by EDX.

3.3 SEM (Scanning electron microscope)

The first scanning electron microscope debuted in (1938) (Von Ardenne) with the first commercial instrument around (1965) .Its late development was due to the electronics involved in scanning the beam of electrons across the sample .

3.3.1 Following are the Characteristic information's (SEM)

(i) Topography

The surface features of an object, its texture, hence direct relation between these features and material properties can be made.

(ii) Morphology:

The shape and size of the particles making up the object, direct relation between these structures and material properties can be made.

(iii) Composition

The elements and compounds that the object is composed of and the relative amounts of them, direct relationship between composition and material properties can be made.

(iv) Crystallographic information

Here arrangement of the atoms in the object can be found out. Direct relation between these arrangements and material properties can be made.

3.4 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 J_f for the experimental specimens is estimated based on the joint orientation, joint strength. and joint spacing. 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.

3.5 Elastic Modulus

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

$$E_r = E_j/E_i$$

Where E_j=tangent modulus of the jointed rock and E_i= tangent modulus of the intact rock.

3.6Shear strength of discontinuities

3.6.1 Introduction

All rock masses contain discontinuities such as bedding planes, joints, shear zones and faults. At shallow depth, where stresses are low, failure of the intact rock material is minimal and the behaviour of the rock mass is controlled by sliding on the discontinuities. In order to analyze the stability of this system of individual rock blocks, it is necessary to understand the factors that control the shear strength of the discontinuities which separate the blocks. These questions are addressed in the discussion that follows.

3.6.2 Shear strength of planar surfaces

Suppose that a number of samples of a rock are obtained for shear testing. Each sample contains a through-going bedding plane that is cemented; in other words, a tensile force would have to be applied to the two halves of the specimen in order to separate them. The bedding plane is absolutely planar, having no surface irregularities or undulations.

In a shear test each specimen is subjected to a stress σ_n normal to the bedding plane, and the shear stress, required to cause a displacement, is measured. The shear stress will increase rapidly until the peak strength is reached. This corresponds to the sum of the strength of the cementing material bonding the two halves of the bedding plane together and the frictional resistance of the matching surfaces. As the displacement continues, the shear stress will fall to some residual value that will then remain constant, even for large shear displacements.

For planar discontinuity surfaces the experimental points will generally fall along straight lines. The peak strength line has a slope of φ and an intercept of c on the shear strength axis. The residual strength line has a slope of φr.

The relationship between the peak shear strength τ_p and the normal stress σ_n can be represented by the Mohr-Coulomb equation:

$$\tau_{p=}c + \sigma_n tan\varphi$$

where c is the cohesive strength of the cemented surface and φ is the angle of friction. In the case of the residual strength, the cohesion c has dropped to zero and the relationship between φ_r and σ_n can be represented by:

$$\tau_{r=}\sigma_n tan\varphi_r$$

Where φ_r is the residual angle of friction.

This example has been discussed in order to illustrate the physical meaning of the term cohesion, a soil mechanics term, which has been adopted by the rock mechanics community. In shear tests on soils, the stress levels are generally an order of magnitude lower than those involved in rock testing and the cohesive strength of a soil is a result of the adhesion of the soil particles. In rock mechanics, true cohesion occurs when cemented surfaces are sheared. However, in many practical applications, the term cohesion is used for convenience and it refers to a mathematical quantity related to surface roughness, Cohesion is simply the intercept on the τ axis at zero normal stress.

The basic friction angle φ_b is a quantity that is fundamental to the understanding of the shear strength of discontinuity surfaces. This is approximately equal to the residual friction angle φ_r but it is generally measured by testing sawn or ground rock surfaces. These tests, which can be carried out on surfaces as small as 50 mm x 50 mm, will produce a straight line plot defined by the equation:

$$\tau_{r=}\sigma_n tan\varphi_b$$

Most shear strength determinations today are carried out by determining the basic friction angle, and then making corrections for surface roughness In the past there was more emphasis on testing full scale discontinuity surfaces, either in the laboratory or in the field. There are a significant number of papers in the literature of the 1960s and 1970s describing large and elaborate in situ shear tests, many of which were carried out to determine the shear strength of weak layers in dam foundations. However, the high cost of these tests together with the difficulty of interpreting the results has resulted in a decline in the use of these large scale tests and they are seldom seen today. It makes both economical and practical sense to carry out a number of small scale laboratory shear tests. The roughness component which is then added to this basic friction angle to give the effective friction angle is a number which is site specific and scale dependent and is best obtained by visual estimates in the field.

3.6.3 Shear strength of rough surfaces

A natural discontinuity surface in hard rock is never as smooth as a sawn or ground surface of the type used for determining the basic friction angle. The undulations and asperities on a natural joint surface have a significant influence on its shear behaviour.

Generally, this surface roughness increases the shear strength of the surface, and this strength increase is extremely important in terms of the stability of excavations in rock. Patton (1966) demonstrated this influence by means of an experiment in which he carried out shear tests on 'saw-tooth' specimens. Shear displacement in these specimens occurs as a result of the surfaces moving up the inclined faces, causing dilation (an increase in volume) of the specimen. The shear strength of Patton's saw-tooth specimens can be represented by:

$$\tau_{p}=\sigma_n \tan(\varphi_b + i) \qquad \text{equation (i)}$$

where φ_b is the basic friction angle of the surface and *i* is the angle of the saw-tooth face.

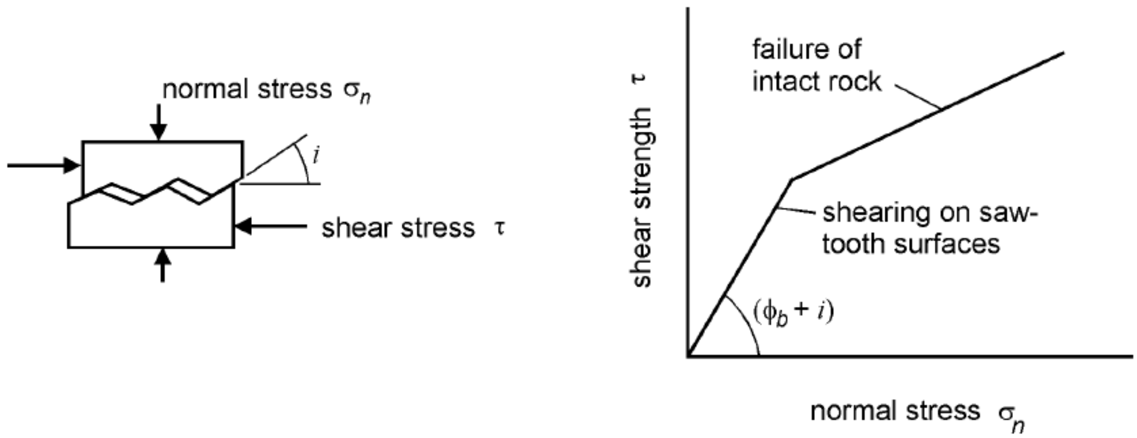


Fig. 3.1 Patton's experiment on the shear strength of saw-tooth specimens.

3.6.4 Barton's estimate of shear strength

Equation (i) is valid at low normal stresses where shear displacement is due to sliding along the inclined surfaces. At higher normal stresses, the strength of the intact material will be exceeded and the teeth will tend to break off, resulting in a shear strength behaviour which is more closely related to the intact material strength than to the frictional characteristics of the surfaces.

While Patton's approach has the merit of being very simple, it does not reflect the reality that changes in shear strength with increasing normal stress are gradual rather than abrupt. Barton (1973, 1976) studied the behaviour of natural rock joints and proposed that equation (i) could be re-written as:

$$\tau_{p=}\sigma_n \tan(\varphi_b + (JRC \log_{10} (JCS/ (\sigma_n)))$$

where *JRC* is the joint roughness coefficient and *JCS* is the joint wall compressive strength .

Barton developed his first non-linear strength criterion for rock joints using the basic friction Angle φ_b from analysis of joint strength data reported in the literature. Barton and Choubey (1977), on the basis of their direct shear test results for 130 samples of variably weathered rock joints, revised this equation to

$\tau_{p=}\sigma_n \tan(\varphi_r + (JRC \log_{10} (JCS/ (\sigma_n)))$

where $\varphi_r$is the residual friction angle Barton and Choubey suggest that φ_r can be estimated from

$\varphi_{r=} (\varphi_b\text{-}20)+ 20(r/R)$

where r is the Schmidt rebound number wet and weathered fracture surfaces and *R* is the Schmidt rebound number on dry un weathered sawn surfaces.

3.6.5 Field estimates of JRC

The joint roughness coefficient *JRC* is a number that can be estimated by comparing the appearance of a discontinuity surface with standard profiles published by Barton and others. One of the most useful of these profile sets was published by Barton and Choubey(1977).The appearance of the discontinuity surface is compared visually with the profiles shown and the *JRC* value corresponding to the profile which most closely matches that of the discontinuity surface is chosen. In the case of small scale laboratory specimens, the scale of the surface roughness will be approximately the same as that of the profiles illustrated. However, in the field the length of the surface of interest may be several metres and the *JRC* value must be estimated for the full scale surface.

3.6.6 Field estimates of JCS

Suggested methods for estimating the joint wall compressive strength were published by the ISRM (1978). The use of the Schmidt rebound hammer for estimating joint wall compressive strength was proposed by Deere and Miller (1966).

3.6.7 Influence of scale on JRC and JCS

On the basis of extensive testing of joints, joint replicas, and a review of literature, Barton and Bandis (1982) proposed the scale corrections for *JRC* defined by the following relationship:

$$JRC_n = JRC_0((L_n)/(L_0))^{-0.02JRC_0}$$

where JRC_0, and L_0 (length) refer to 100 mm laboratory scale samples and JRC_n, and L_n refer to in situ block sizes. Because of the greater possibility of weaknesses in a large surface, it is likely that the average joint wall compressive strength (*JCS*) decreases with increasing scale. Barton and Bandis (1982) proposed the scale corrections for *JCS* defined by the following relationship:

$$JCS_n = JCS_0((L_n)/(L_0))^{-0.03JRC_0}$$

where JCS_0 and L_0 (length) refer to 100 mm laboratory scale samples and JCS_n and L_n refer to in situ block sizes.

3.6.8 Shear strength of filled discontinuities

The discussion presented in the previous sections has dealt with the shear strength of discontinuities in which rock wall contact occurs over the entire length of the surface under consideration. This shear strength can be reduced drastically when part or all of the surface is not in intimate contact, but covered by soft filling material such as clay gouge. For planar surfaces, such as bedding planes in sedimentary rock, a thin clay coating will result in a significant shear strength reduction. For a rough or undulating joint, the filling thickness has to be greater than the amplitude of the undulations before the shear strength is reduced to that of the filling material.

A comprehensive review of the shear strength of filled discontinuities was prepared by Barton (1974) and a summary of the shear strengths of typical discontinuity fillings, based on Barton's review. Where a significant thickness of clay or gouge fillings occurs in rock masses and where the shear strength of the filled discontinuities is likely to play an important role in the stability of the rock mass, it is strongly recommended that samples of the filling be sent to a soil mechanics laboratory for testing.

3.6.9 Influence of water pressure

When water pressure is present in a rock mass, the surfaces of the discontinuities are forced apart and the normal stress σ_n is reduced. Under steady state conditions, where there is sufficient time for the water pressures in the rock mass to reach equilibrium, the reduced normal stress is defined by $\sigma_n' = (\sigma_n - u)$, where u is the water pressure. The reduced normal stress σ_n' is usually called the effective normal stress, and it can be used in place of the normal stress term σ_n in all of the equations presented above.

3.6.10 Instantaneous cohesion and friction

Due to the historical development of the subject of rock mechanics, many of the analyses, used to calculate factors of safety against sliding, are expressed in terms of the Mohr-Coulomb cohesion (c) and friction angle (φ). Since in 1970s it has been recognized that the relationship between shear strength and normal stress is more accurately represented by a non-linear relationship such as that proposed by Barton and Bandis (1990). However, because this relationship (e.g. is not expressed in terms of c and φ, it is necessary to devise some means for estimating the equivalent cohesive strengths and angles of friction from relationships such as those proposed by Barton and Bandis.

3.7 Factors influencing the shear strength

Shearing mechanism of rock joints depends on Rock type, weathering, surface geometry, water, overclosure, scale effect, test method and filler material.

(i) Rock type

Rock type influences the shear strength of rock joints through its texture, mineralogy and fabric, which contributes to the strength of surface irregularities and friction between planar joint surfaces.

(ii)Weathering

Weathering affects shear strength of rock joint to a great extent .The compressive strength of joint wall is an important component of the shear strength, weathering causes reduction in this compressive strength and hence reduces the shear strength. The depth of penetration of weathering into joint walls depends largely on the rock type , and in particular on its permeability .Barton (1973) indicated the mechanical effects of weathering is that slight alteration of the fresh rock may cause a much more severe drop in mechanical strength than subsequent steps in the alteration of the weathered rock . However ,Dearman et.al (1978) showed that compressive strength decreases linearly with increasing degree of weathering .

(iii)Surface geometry

The geometry of the joint surface (roughness) has an important effect on shear behavior, and is particularly significant in determining dilation and the effect of asperities in general. Schneider (1976) tested model samples of granite, sandstone and limestone with the same material strength and different joint surface roughness.

(iv)Water

The presence of water in rock joints leads to several mechanical and some chemical effects, the most important of which will probably be the reduction ineffective stress. the water will also tend to reduce surface energy and crystal strength, with the result that the mechanical strength of the rock is lowered .This has the subsequent effect of lowering the shear strength .however some types of rock joints appear to be little affected by water (besides the effective stress effect) and may have a slightly higher shear strength when wet. Most of the smooth polished surfaces are unaffected or increase in strength slightly when wet .Brownell (1966) and Dickey (1966) explained that the effect of water on smooth surfaces is due to surface cleanliness .The presence of water has no effect on smooth, clean surfaces. However, if smooth surfaces are unclean, then water causes an increase in strength from the dry condition. Barton (1973) explained that the reduction in strength from the dry condition. Barton (1973) explained that the reduction in strength due to water on rough surface is related to the reduction in tensile and compressive strength of brittle material due to adverse effect of water.

(v)Over – closure

Barton (1973) indicated that non –planar joints intersecting the rock may be pre-loaded or over-closed .The methods of obtaining or exposing rock joints for shear testing involve such a degree of disturbance that any potential over –closure effect will be destroyed, and the strength measured therefore rather conservative .The only way of recovering the effect is to pre-consolidate the joints, before shear testing .

(vi)Scale effect

Scale effect of specimens on the compression strength and frictional strength of rocks has been demonstrated in rock mechanics .Pratt et al.(1972) indicated that compressive strength decreases with increase in specimen length , but the scale effect appears to die out when specimen sizes exceeds about 1 m Pratt et al.(1974) showed that shear strength of joint decreases with increase in joint length. But the scale effect appears to die out for joint length in excess of 2m to 3m.However, literature on scale effect is scarce, and the scale effect needs therefore to be further investigated.

(vii)Test methods

Shear strength of rock joints can be investigated in the laboratory with triaxial tests or direct shear tests or others .the results of these two methods on the same rock type basically agree well .Because of changes of geometry during sliding ,the triaxial method is not well suited to the study of continued sliding . In addition, it is not good for testing seams, or joints of shear origin. because of the large size of the shear box, the direct shear test has the advantage that comparatively large surfaces may be used and thus a better simulation of natural conditions obtained .However, it is not a simple matter to produce a reasonably uniform stress distribution inside a direct shear box, and there is the problem of direct shear tests, concerning the lateral boundary conditions. Fortunately, different elaborate direct shear machines like the direct shear machine by Hoek and Pentz (1968), the shear machine by Krsmanovic (1967), etc. were developed to solve the above mentioned problems.

(viii)Filler materials

For sliding of a filled joint, there will be an initial failure corresponding to the shear failure of the filling material, and subsequently there will be sliding on this surface of failure **.**If the filler material thickness is greater than the height of surface irregularities the characteristic of the filler can completely dominate the behavior of the discontinuity. If the filler thickness is less than that height then the filler material and the irregularities will interact in some way. Goodman (1974) indicated that the behavior of filled joint is different from that of unfilled joints, Ladanyi and Archambault (1975) indicated that the strength of the filled joint is located between that of the filling alone and that of the same type of joint when unfilled. The strength decreases steadily with the filling thickness to approach that of the filler when the thickness exceeds the height of the irregularities by about 50 percent.

3.8 Factors that influences deformability

The important factors that influence deformability are joint type, test methods and filler material .these factors affect the shear stiffness more than the normal stiffness.

(i) Joint type

Goodman (1970) generalized the characteristics of the shear stress displacement curves for different types of weak surfaces. The irregularities in the load displacement curve result from overriding of successive asperities. Water content reduces stiffness to a great extent.

(ii)Test methods

The stiffness properties for a joint can be determined experimentally .Rosso(1976) generated comparable stiffness values for direct shear test and triaxial test for a similar shale. He also ran in situ test on quartz diorite. The shear stiffness not only varies due to the parameters such as rock type, joint type, roughness etc. But also due to differences between testing techniques.

(iii)Filler material

Goodman (1920) carried out studies on the effect of joint filling thickness on the joint deformability .Ladanyi and Archambault (1975) ran test on model (concrete bricks) material with clay filling. They showed that for thickness of clay filling up to 60 percent, which is the ratio of thickness of filling material to the height of asperities, the joints of the model materials show a locking character in shear i.e. Their rigidity increases with the displacement .For medium filling thickness (30 to 60 percent) .Clay filled joint show first a distinct yield point in shear at a small displacement and the peak strength at a much larger displacement.

3.9 Geological basis for rock joint system models

3.9.1 Scope

Rock joint system models are inevitably simplifications of the complex geometries found in nature .Simplifications are necessary to make mathematical and numerical conceptualization possible. Nevertheless it is essential that the relationship of models to actual geologic conditions be clearly understood**.**

3.9.2 Sources of information on joint system geometry

Geometric characteristics which define joint systems are joint size, shape, location and orientation. Joint systems are three dimensional structures contained within rock masses and direct observation is therefore extremely difficult. Sources of information can be divided into three classes: one dimensional sources such as boreholes, two dimensional sources such as essentially planar surface exposures, and three dimensional sources such as tunnels and irregular surfaces. The most common source of information on joint system geometry is data from boreholes cored into the rock mass .Bore holes provide only a one dimensional view of the joint system .Information available from boreholes consists of the distribution of spacing's between intersections between the borehole and jointing, the autocorrelation process of the location of these intersections, and the distribution of the joint orientations. Information on the location and spacing of intersections between boreholes and joints provides a measure of the quantity of joints within the rock mass, the intensity of jointing, and also provides a partial perspective on the location of joints within the rock mass .information on joint shape and size (extent) cannot be obtained directly from boreholes. Information on joint orientation obtained from boreholes may be either apparent dips, if boreholes are not marked for absolute orientation or both dip and dip direction if cores are oriented .Two dimensional sources of information include visual inspection and mapping of essential planar surface.

Joint intersections with planar exposures are referred to as joint traces, and can provide information about joint intensity, location, size and orientation, but not joint shape. Jointing intensity and location can be obtained from joint traces in a number of ways .The most rigorous approach is the enumeration of the location and length of every joint trace. Within a given area of the exposure, this is a very expensive procedure and is therefore rarely undertaken .More frequently trace intersections with a line drawn on the exposure are used to obtain a measure of spacing between joint intersections with a one dimensional feature comparable to that obtained from boreholes. Joint size can be obtained in terms of joint trace length on exposures. This is a incomplete measure ,since it does not describe the true size of the two dimensional joint ,but can be used for inference of joint size .joint orientation information can be obtained in terms of both dip and strike from exposures, using photographic techniques or a Brunton compass. Joint shape information cannot be obtained from two dimensional exposures. The direct three dimensional sources of joint system geometry information are irregular surfaces, tunnels and the simultaneous excavation and observation of rock joints using hand tools. Only the last of these provides detailed and complete three dimensional information. This type of observation is however rarely done ,due to the time and expense involved ,but is part of current radioactive waste repository characterization programs (NTS,STRIPA,AECL,etc).finally three dimensional joint system geometry information may eventually be obtainable by geophysical means .Techniques are currently under development (Doe ,1984)for measurement of joint size , shape and orientation by geophysical means .

Table 3.2

Strength of jointed and intact rock mass

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

(After Ramamurthy and Arora ,1994)

Table 3.3

Modulus ratio classification of intact and jointed rocks

Class	Description	Modulus ratio
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

(After Ramamurthy and Arora ,1994)

CHAPTER 4

Laboratory investigation

In this chapter experimental investigation was carried to find out the shear strength and deformation properties of the rock joints. This chapter describes materials used; preparation of specimens, curing, making joints in specimen, experimental set up and test procedure, parameters studied.

4.1 Materials used

In past plaster of Paris was used as a model material in order to simulate weak rock jointed mass in the field. Plaster of Paris was used by researchers because of its ease in casting as well as it is flexible and it hardens instantly .Its low cost and easy availability made it a appropriate model material for many researchers. Plaster of Paris can be used to simulate any kind of joint as required by the researchers. The reduced strength and deformed abilities in relation to actual rocks has made plaster of Paris one of the perfect materials for modeling in geotechnical engineering and hence it was used to prepare model for the present study. Here clay was used as a gouge material. Clay was taken from a site near to jharsuguda, Orissa ,India.

4.2 Preparation of specimens

Two bags of plaster of Paris was procured from the local market. Both the bags of plaster of Paris was mixed uniformly in a big container. Container was made air tight by providing a plastic cap at the top of the container and then it was covered by two layers of polythene bag .Number of trial tests were carried out with different percentage of distilled water mixed with plaster of Paris .Different specimens were tested for uniaxial compressive strength. The water content of specimen corresponding to the maximum uniaxial compressive strength was taken as the required moisture content and all samples were made at this moisture content .The moisture content was found out to be 32 percent. 135 gm of Plaster of Paris was mixed with 32 percent of distilled water (i.e. 43.2cc of distilled water).A uniform paste was made in a bowl. Uniform mix was transferred into a mould in three layers .Care was taken that while transferring the mix into the mould, it was kept vibrating on a vibrating table for about two minutes. Vibration of mould was done in order to achieve proper compaction and thus making the specimen free from air voids. After that it was allowed to set and finally specimen was taken out of the mould manually with the help of an extruder. Similarly many specimens were made at the same moisture content of 32 percent and same procedure was repeated for all specimens. All the specimens were kept at room temperature for 48 hours.

4.3 Curing

A solution of concentrated sulphuric acid (47.7cc) mixed with distilled water (52.3 cc) was made. This was done mainly to maintain the relative humidity in range of 40% to 60% .This solution was poured in desiccators. All specimens were kept in the desiccators for curing until a constant weight was obtained (about 15 days).All specimens were polished with the help of a sand paper to have length of 76mm.Before testing care was taken that each specimen of plaster of Paris was made with (L/D) ratio as 2: 1 (i.e. L = 76 mm and D= 38 mm).

4.4 Making joints in specimens

For making rough joints, the following accessories were used:

1. Scale
2. Pencil

3. Protractor
4. Light weight hammer
5. Chisel
6. “v” block

On the surface of specimen two longitudinal lines were drawn opposite to each other. Protractor was used to mark the desired orientation angle with respect to the central longitudinal line. Marked specimen was kept on the “V’ block .Chisel was kept on the drawn line of the specimen and with the help of hammer required joint was made on the specimen .Same procedure was adopted for different specimen for different orientation angle. It was seen that the joint formed was coming under the category of rough joint.

4.5 Experimental setup and test procedure

In the present study, specimens were tested to obtain their uniaxial compressive strength, deformation behavior and shear parameters. Uniaxial compression test, direct shear test was carried out in order to acquire these parameters as mentioned above. These tests were carried as per ISRM and IS codes. On the prepared specimen of jointed rock mass uniaxial compression test was carried out (as per ASTM D2938)in order to obtain the ultimate compressive strength of jointed rock mass with respect to various orientation angles starting from 0^0 to 90^0 at an interval of 10^0 respectively. This test was repeated for both the cases (i.e. joints with and without gouge fill).Here clay gouge of 3mm thick was used in the present study.

4.6 Direct shear test

In order to find the roughness of joint (i.e $r = tan\varphi$) direct shear test was done. This r value was used to predict the joint factor as per (Arora 1987).These tests were carried out on conventional direct shear test apparatus (IS :1129 ,1985) with certain modifications required for placement of specimens inside the box .Two identical wooden blocks of size 59mm x 59mm x 12mm each was taken and a central hole of diameter 39mm was made in wooden blocks. These wooden blocks were put into two halves of the shear box. Specimen was placed inside the modified shear box of size 60mm x 60mm.Cylindrical specimen was broken into two equal parts and then it was placed inside the circular hole of the wooden blocks such that broken parts match together and were laid on the plane of shear (i.e. contact surface of two halves of the shear box).This procedure was repeated for different specimens.

4.7 Uniaxial compressive strength test

In this test the cylindrical specimen is subjected to major principal stress until the specimen fails .The test specimen was prepared as per the specifications (ISRM 1981) , i.e.

1. Circular cylindrical specimens were made straight with slenderness ratio preferably between 2 and 3 .
2. Ends of the specimen were made flat to 0.02 mm.
3. The sides of specimen were made straight to within 0.3 mm over the full length.
4. The ends of the specimen were made parallel with the axis of the specimen perpendicular within 0.05 mm in 50 mm.
5. The diameter was measured in the two perpendicular directions at three locations, near the top , middle and near the bottom .

The specimen were tested within 30 days .The finished specimen was put in between the plates of the testing machine and then specimen was loaded until it failed. Failure load was noted down. Deformation of specimen was measured with the help of a separate dial gauge which was fixed near to the specimen. After occurrence of the brittle failure a rapid decrease in load taken by specimen was noticed with further increase in strain. The failure load was divided by the cross sectional area of the specimen in order to get the uniaxial compressive strength of the specimen perpendicular to the direction of load. Same procedure was repeated for different specimens as per the given requirement.

4.8 Parameters studied

The main objective of the experimental investigation was to study the following aspects

1. The shear strength behavior of plaster of Paris specimen.
2. The deformation behavior of jointed specimen.
3. The effect of joint factor in the strength characteristic of the specimen.
4. To make a comparative study of joint with and without gouge fill.

   Uniaxial compressive strength of specimens were conducted in order to determine the strength as well as the deformation characteristic of intact and jointed specimens with single and double joint. The same procedure was repeated for single jointed specimen with gouge fill. The specimens were tested for different orientation starting from 0^0 to 90^0 at an interval of 10^0respectively. Some of these specimens are shown in the Fig 4.1. To avoid slippage of joints just after application of the load the jointed specimen were placed inside a rubber membrane before testing.

Direct shear test were conducted on joints with and without gouge fill to know the c_j and φ_j value.

Table 4.1

Types of single joint studied

Types of joint	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
Single joint with gouge fill	3	3	3	3	3	3	3	3	3	3

Table 4.2

Types of double joint studied

Types of joint	2j-10	2j-20	2j-30	2j-40	2j-50	2j-60	2j-70	2j-80	2j-90	1j-60 and 1j-90
Double joint	3	3	3	3	3	3	3	3	3	3
Double joint with gouge fill	0	0	0	0	0	0	0	0	3	3

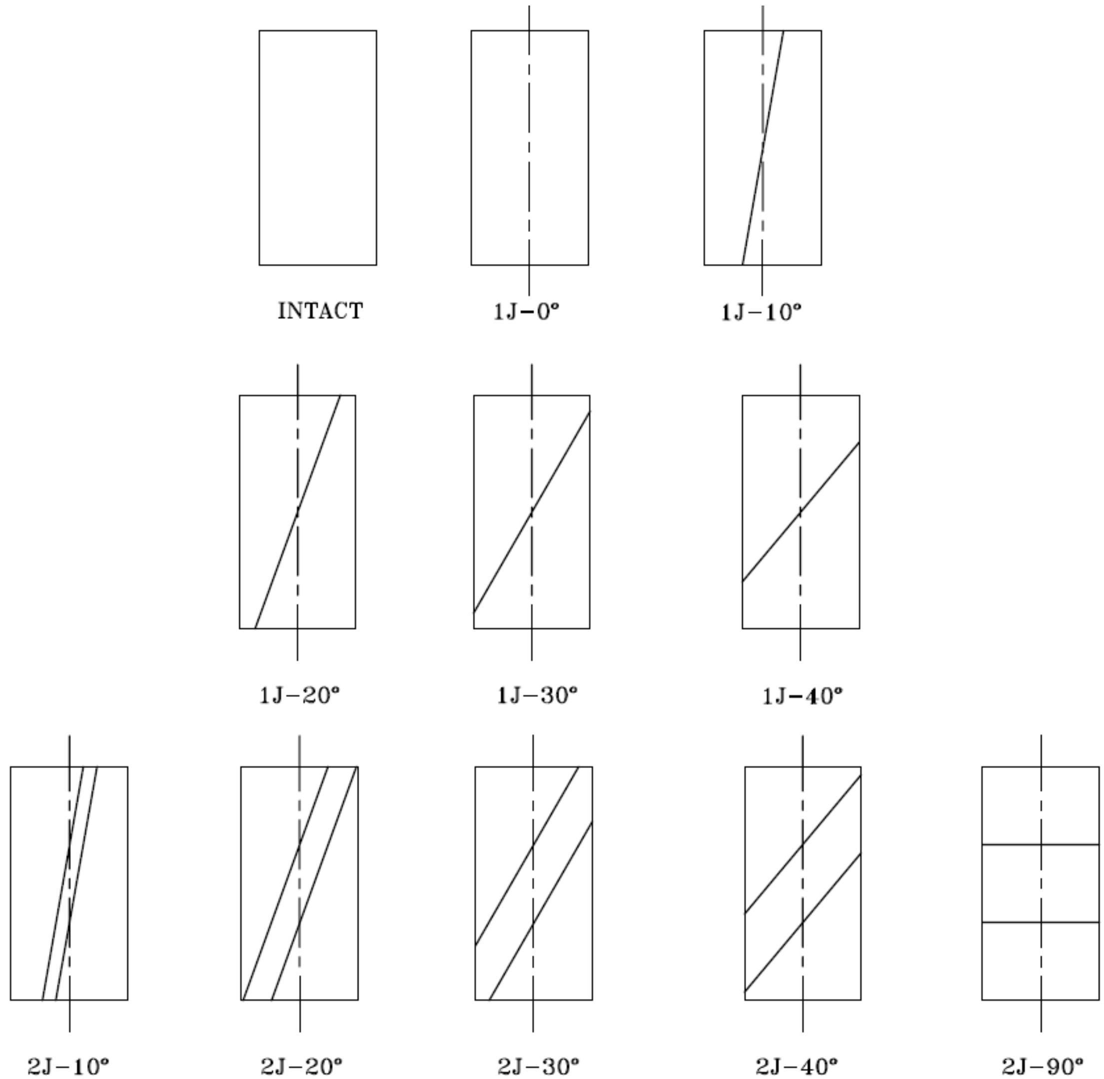


Fig. 4.1 Types of joints studied in plaster of Paris specimens. (single and double jointed specimen can be seen here).

CHAPTER 5

Results and discussions

5.1 Results from SEM/EDX

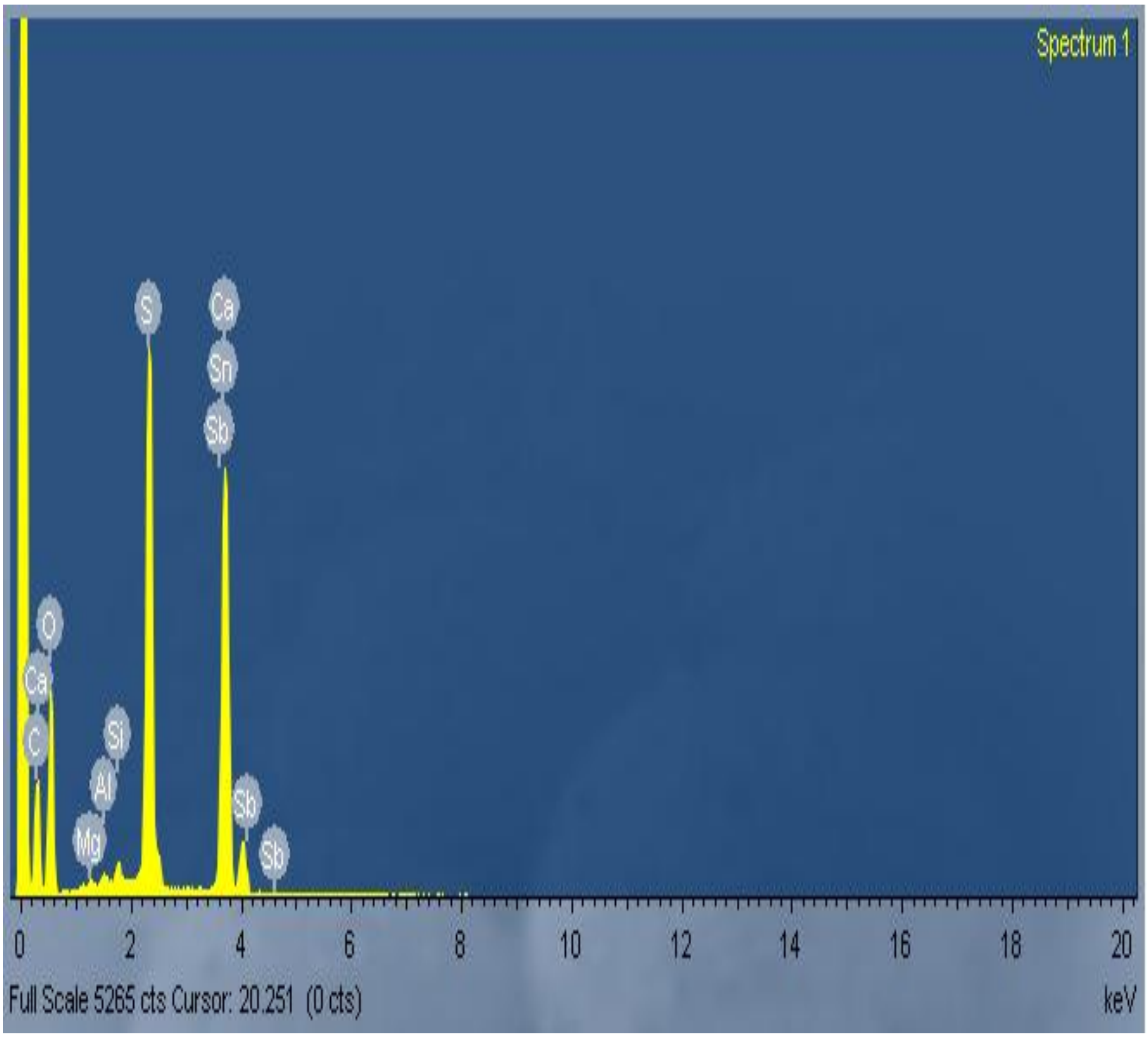


Fig. 5.1 Microscopic pattern of plaster of Paris taken from bottom part of the container showing the image of each element in the sample taken from the bottom part of the container.

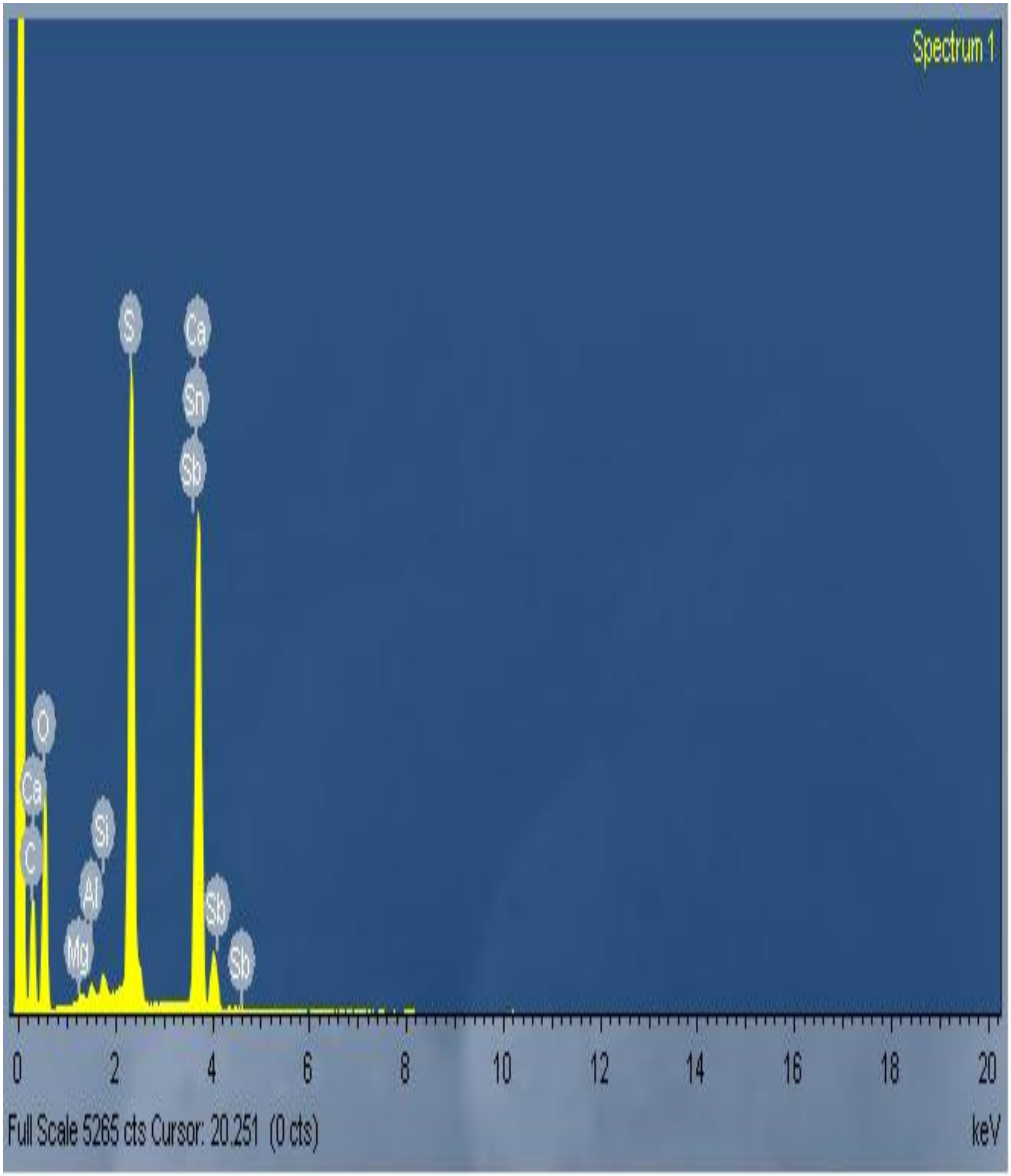


Fig. 5.2 Microscopic pattern of plaster of Paris taken from top part of the container showing the image of each element in the sample taken from the top part of the container.

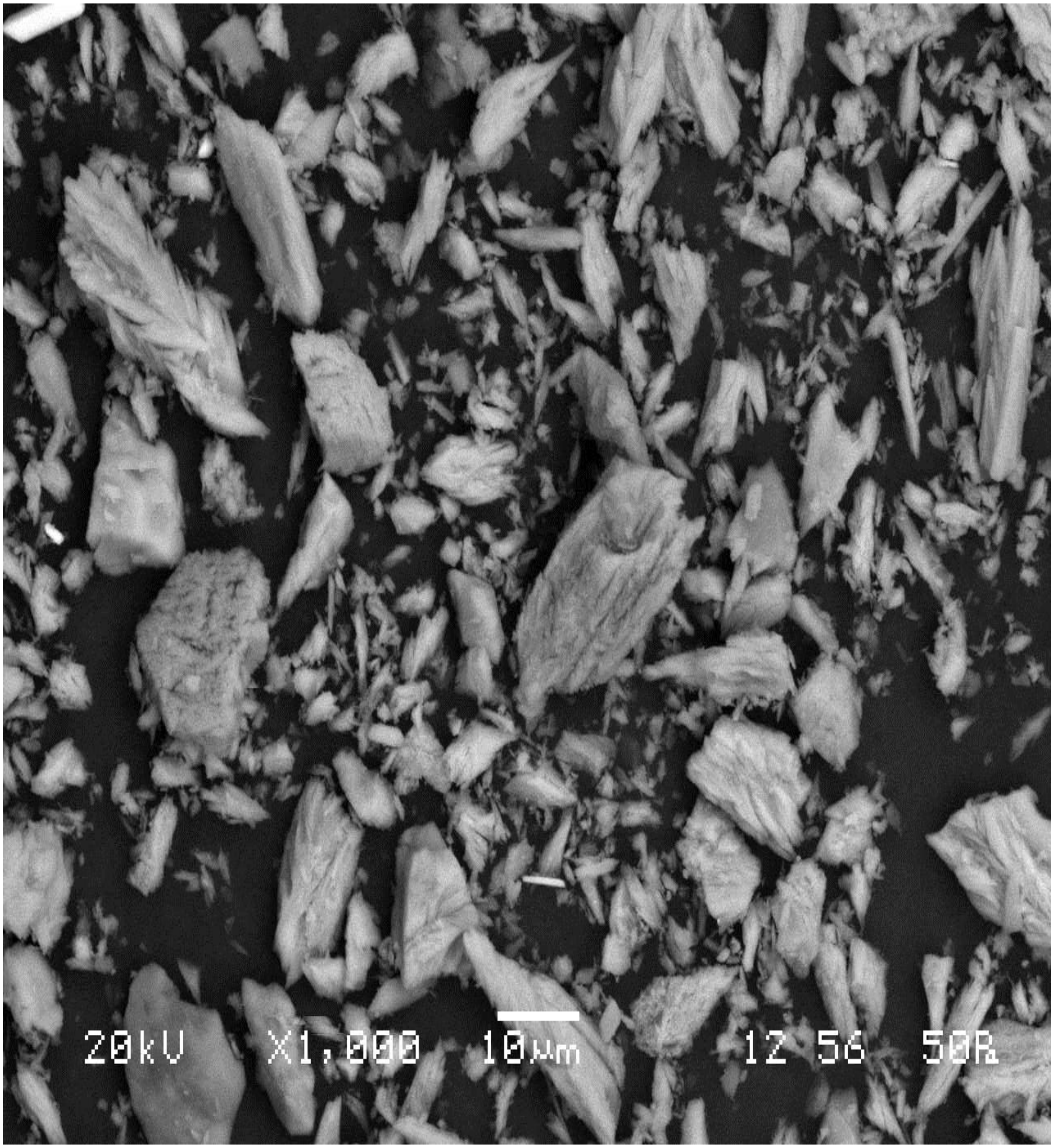


Fig. 5.3 Microstucture of plaster of Paris sample taken from the Bottom part of the container.(X1000) shows that particles at bottom part of container are possessing angular structure with irregular surfaces.

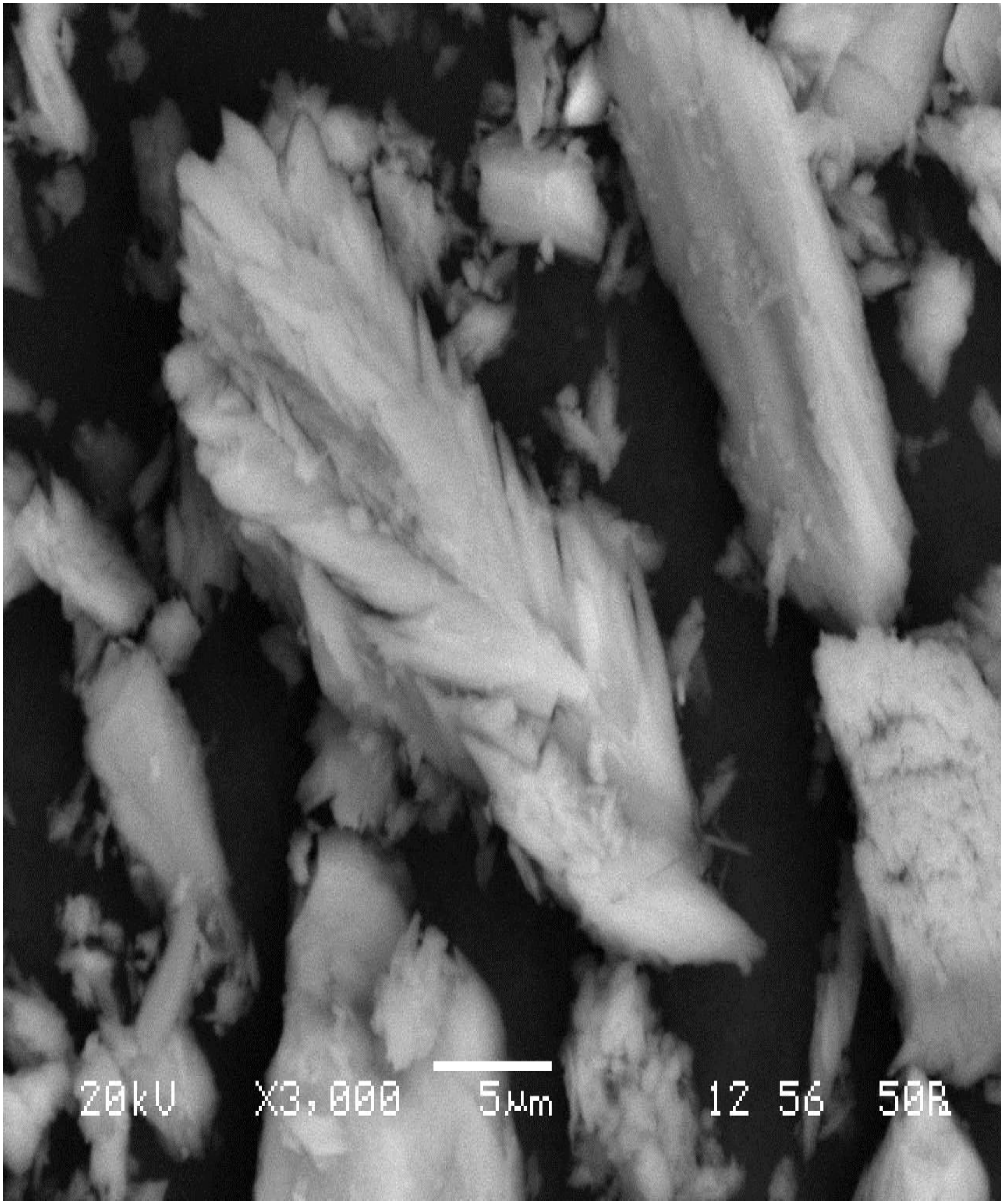


Fig. 5.4 Microstucture of plaster of Paris sample taken from the Bottom part of the container.(X3000)shows that particles at bottom part of container are possessing angular structure with irregular surfaces.

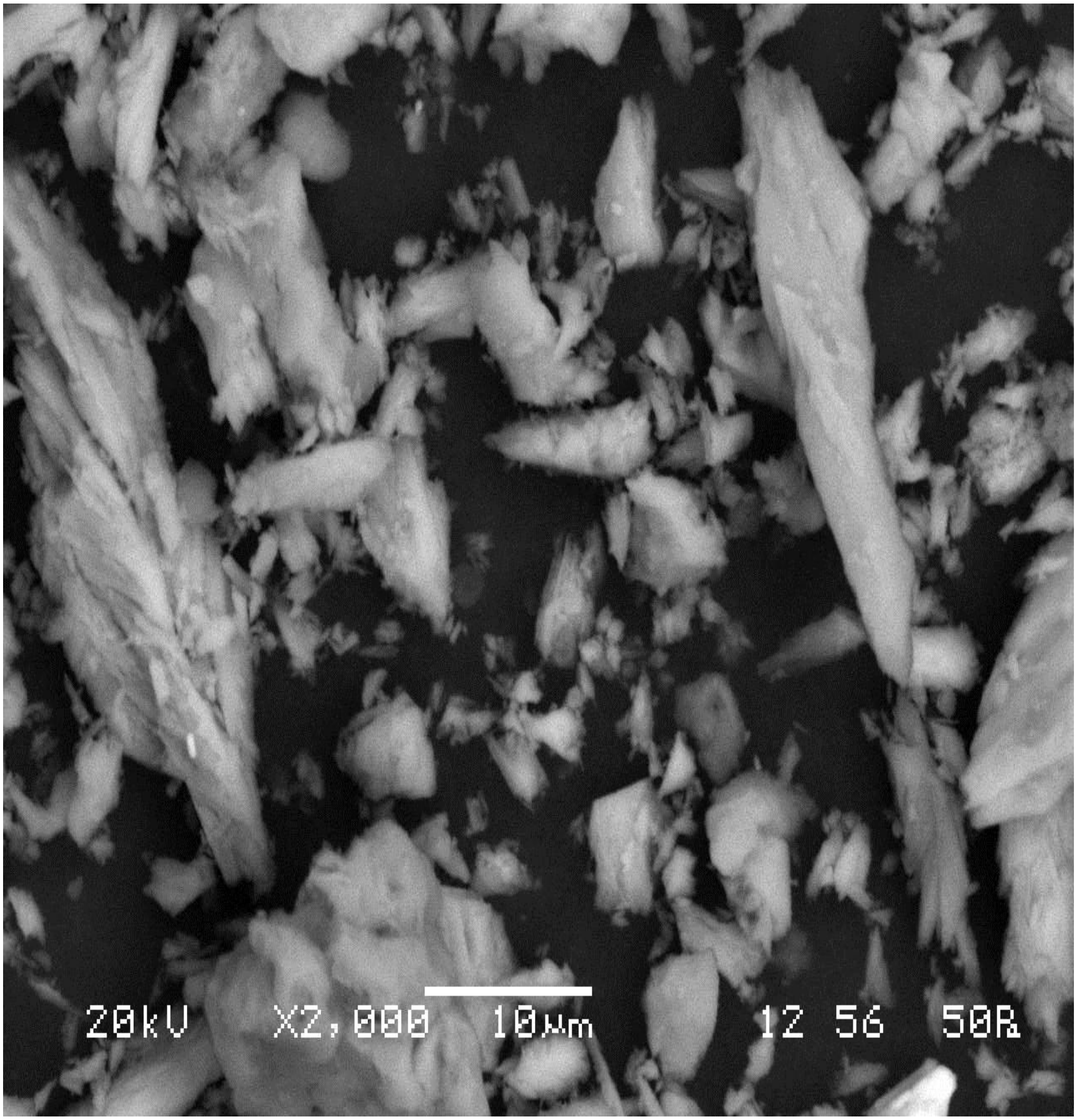


Fig. 5.5 Microstucture of plaster of Paris sample taken from the Bottom part of the container.(X2000)shows that particles at bottom part of container are possessing angular structure with irregular surfaces.

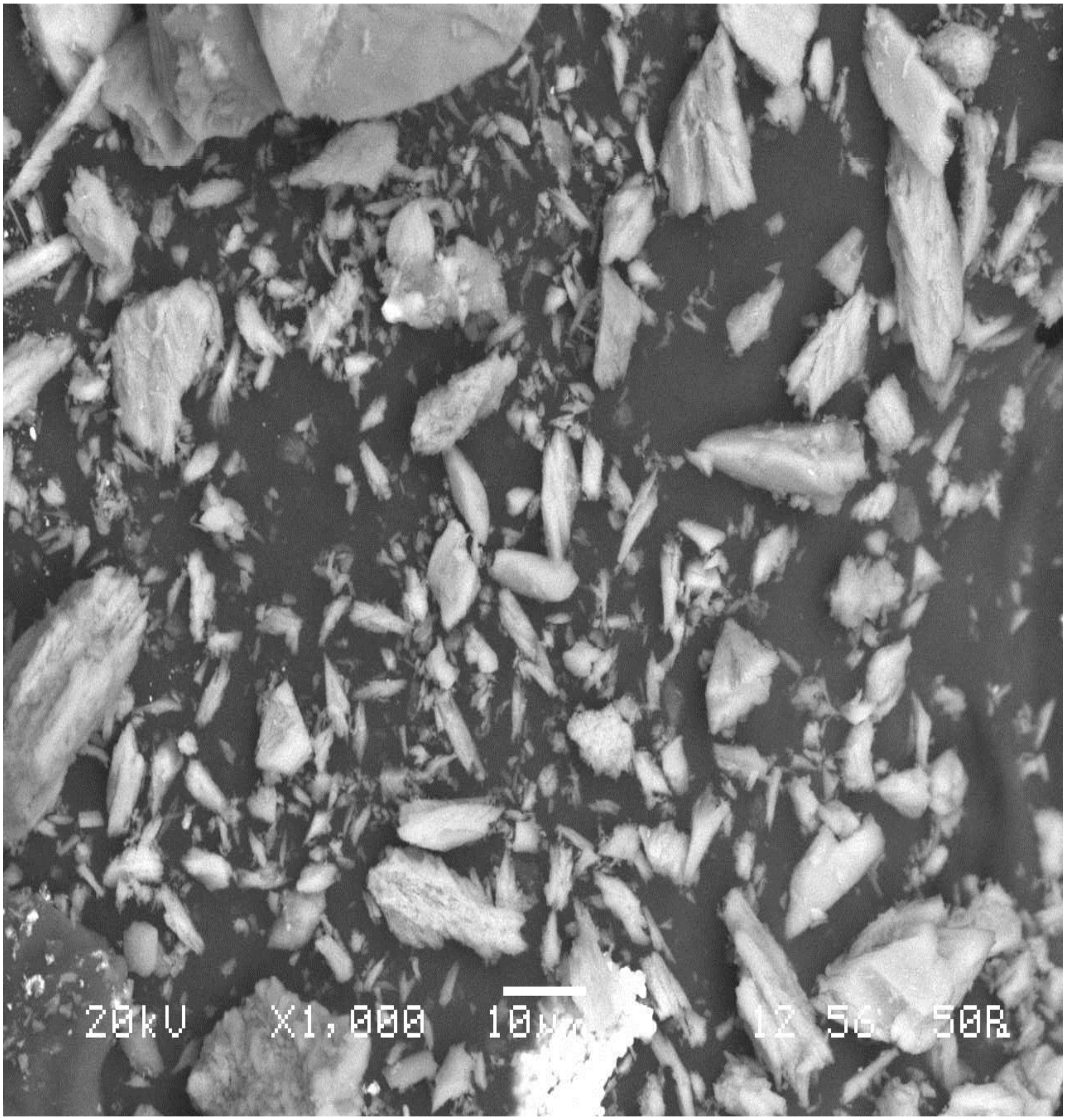


Fig. 5.6 Microstucture of plaster of Paris sample taken from the Top part of the container.(X1000) shows that particles at top part of container are possessing angular structure with irregular surfaces.

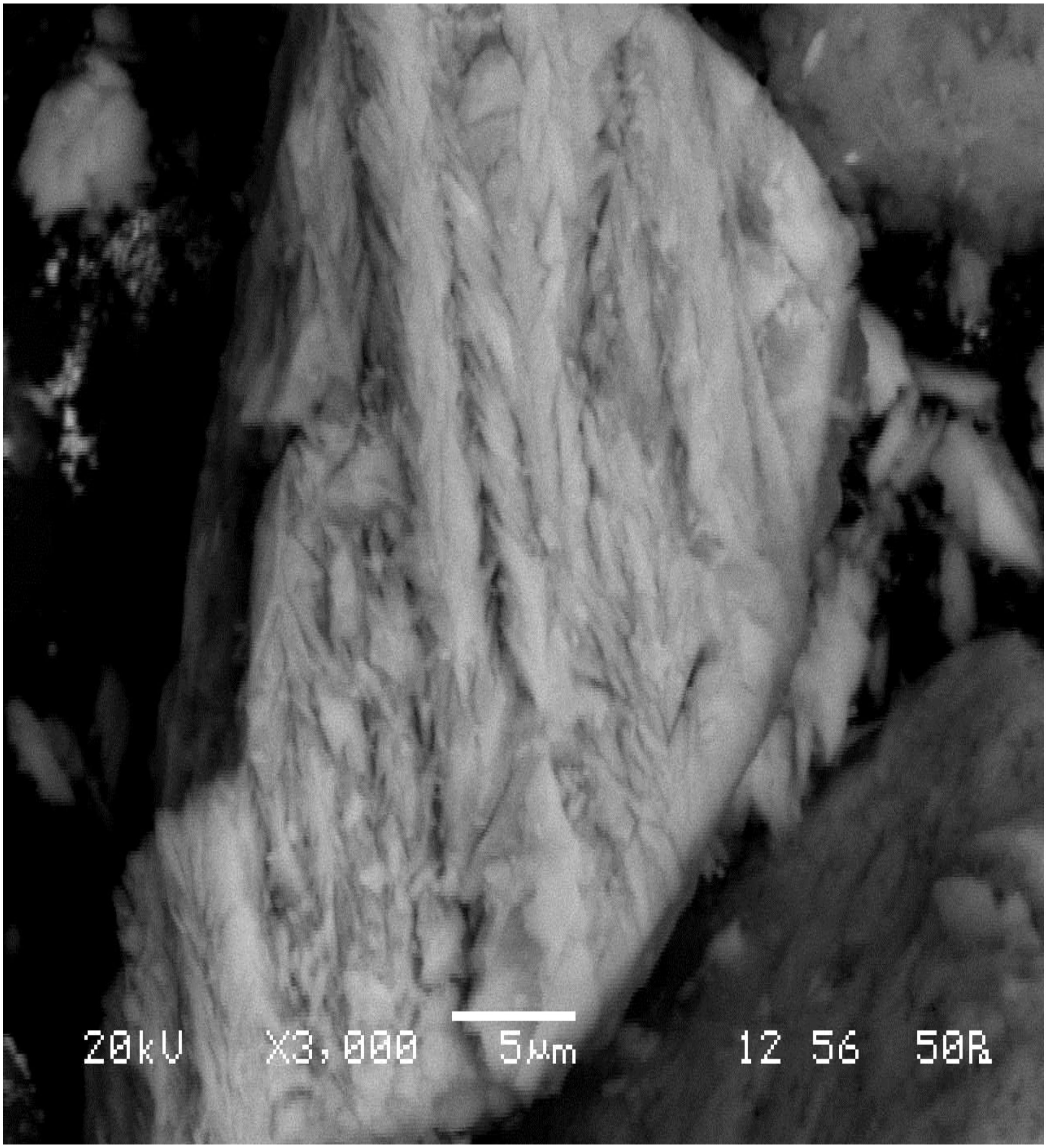


Fig. 5.7 Microstucture of plaster of Paris sample taken from the top part of the container.(X3000)shows that particles at top part of container is possessing angular structure with irregular surfaces.

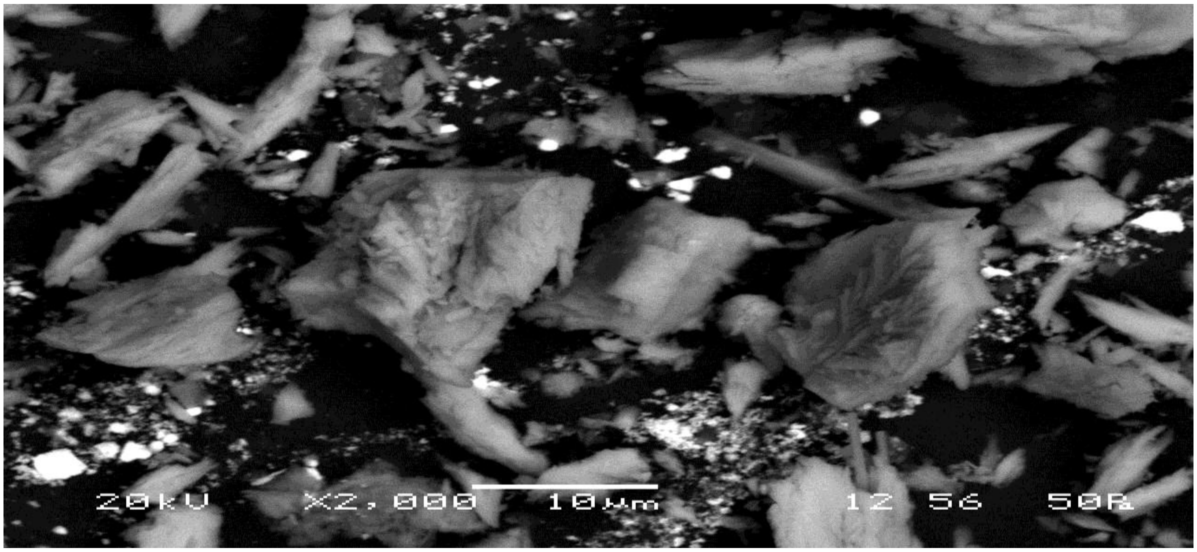


Fig. 5.8 Microstructure of plaster of Paris sample taken from the top part of the container.(X2000)shows that particles at top part of container are possessing angular structure with irregular surfaces.

5.2 Direct shear test results

The roughness parameter (r) which is the tangent value of the friction angle (φ_j) was obtained from the direct shear test conducted at different normal stresses. The variation of shear test with normal stress for specimens tested in direct shear tests are given in the fig-5.9 and their corresponding values are given in the Table 5.1.The value of cohesion (c_j) for jointed specimens of plaster of Paris has been found as 0.229 MPa and value of friction angle (φ_j) was found as 40^0. Hence the roughness parameter (r) is taken as 0.842(i.e r = tan(φ_j)) for plaster of Paris specimens.

Table 5.1

Values of shear stress for different values of normal stress on jointed specimens of plaster of Paris in direct shear test

Normal stress ,MPa	Shear stress, MPa
0.16	0.37
0.32	0.48
0.48	0.64

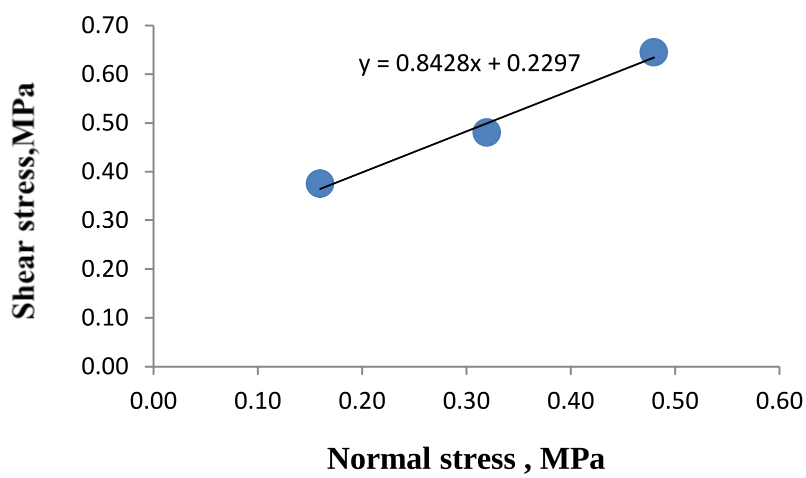


Fig. 5.9 Shear stress versus Normal stress (for joints without gouge fill)

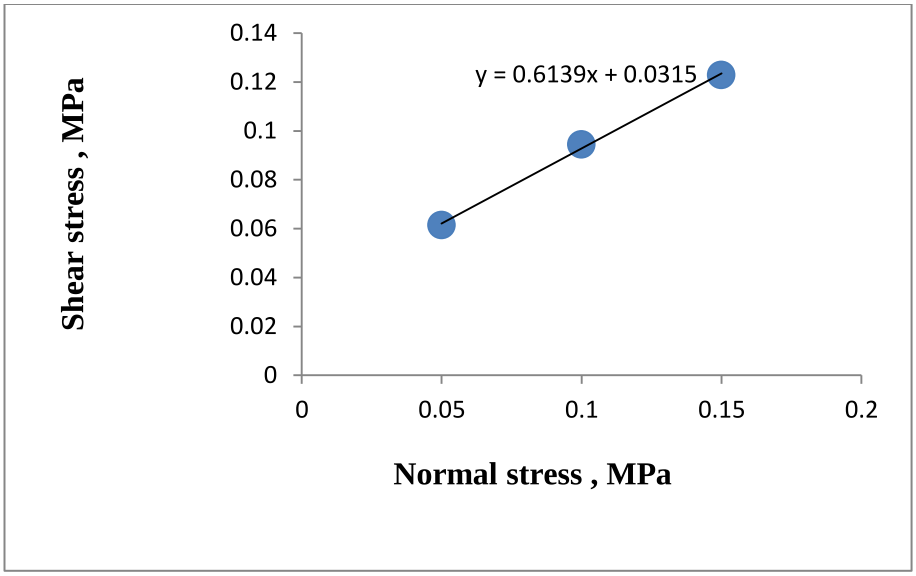


Fig. 5.10 Shear stress versus Normal stress (For joints with Gouge fill)

5.3 Uniaxial compression test results

(i) For intact specimens:

The variation of stress strain curve as obtained from uniaxial compression test for intact specimen of plaster of Paris is shown in fig-5.11 and corresponding stress versus strain values are shown in Table 5.2 .The value of uniaxial compressive strength (σ_{ci}) obtained from above test was found to be as 8.32 MPa.The modulus of elasticity of intact specimen is computed at 50 percent of σ_{ci} value to account the tangent modulus. The modulus of elasticity was found to be 462.5 MPa for intact specimen of plaster of Paris.

Table 5.2

Values of stress and strain for intact specimen

Length of the specimen=76mm

Diameter of the specimen=38mm

Strain rate =0.5 mm/minute.

Axial strain	Stress,MPa
0	0
0.006578947	0.98537685
0.013157895	3.83202109
0.019736842	6.13123374
0.026315789	8.32096008
0.032894737	8.32096008
0.039473684	8.32096008
0.046052632	8.21147376

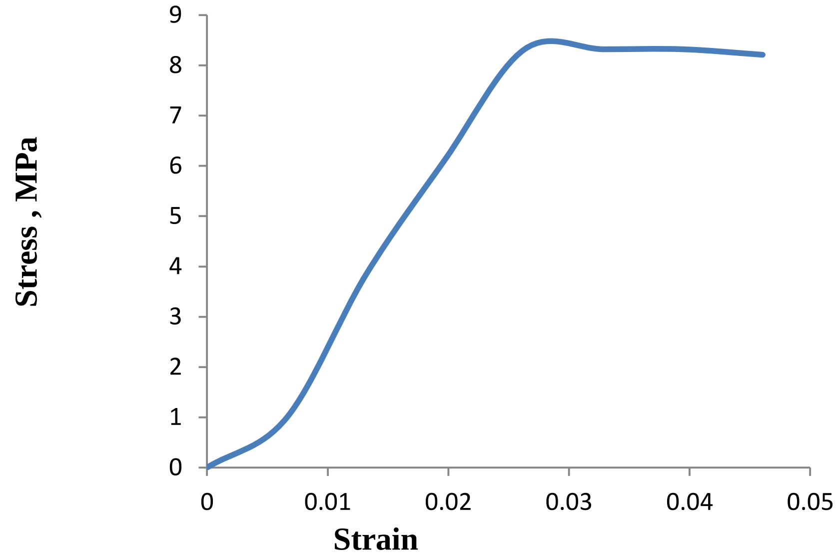


Fig. 5.11Stress versus strain curve for intact specimen

Table 5.3

Engineering properties of plaster of Paris

Serial Number	Property/Parameter	Values
1	Uniaxial compressive strength,MPa	8.32
2	Tangent modulus,MPa	462.5
3	Cohesion,MPa	0.229
4	Angle of friction in degrees	40

(ii) For jointed specimen:-

Same procedure was repeated to determine the uniaxial compressive strength of jointed specimens as it was done for intact specimen .Similarly modulus of elasticity for jointed specimens was also evaluated and was noted down. Jointed specimens were kept inside the rubber membrane in order to prevent the slippage along the critical joints. After noting down the values of (σ_{cj}) and (E_{tj}) for different orientation angle (β) it was seen that minimum strength of joint was found at ($\beta=30^0$) .Following relation was used in order to compute (σ_{cr}) corresponding to different values of joint orientation (β).

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

the values of joint factor J_f was evaluated using the following relation as,

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

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

$$\sigma_{cr} = e^{-0.008 \times J_f}$$

Table 5.4

The value of inclination parameter “n”

Orientation of joint β^0	Inclination parameter n
0	0.81
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

(After Roy .N , Ramamurthy , 1993)

Table 5.5

Values of Jn, J_f and σ_{cr} for jointed specimens (single joint)

β (angle)	**Jn**	**r**	**n**	**J_f**	**J_f * 0.008**	**σ_{cr} Predicted Arora ,1987**	**σ_{cr} Experimental values**
0	13	0.842	0.81	19.061	0.15249	0.859	0.842
10	13	0.842	0.46	33.564	0.26851	0.765	0.697
20	13	0.842	0.105	147.042	1.17634	0.308	0.25
30	13	0.842	0.046	335.64	2.68512	0.068	0.026
40	13	0.842	0.071	217.457	1.73965	0.176	0.1579
50	13	0.842	0.306	50.4557	0.40365	0.668	0.526
60	13	0.842	0.465	33.2031	0.26562	0.767	0.684
70	13	0.842	0.634	24.3524	0.19482	0.823	0.803
80	13	0.842	0.814	18.9674	0.15174	0.859	0.855
90	13	0.842	1	15.4394	0.12352	0.884	0.882

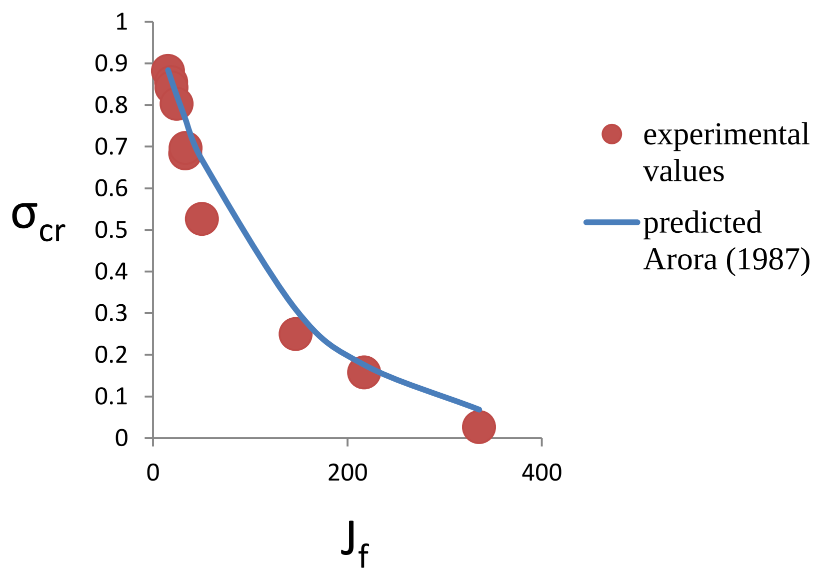


Fig. 5.12Joint factor versus compressive strength ratio(single joint)

Table 5.6

Values of Jn, J_f and σ_{cr} for jointed specimens (Double joint)

β(angle)	**Jn**	**r**	**n**	**J_f**	**J_f * 0.008**	**σ_{cr} Predicted Arora ,1987**	**σ_{cr},Experimental values**
10	26	0.842	0.46	67.128	0.53702	0.584	0.59
20	26	0.842	0.105	294.084	2.35268	0.095	0.18
30	26	0.842	0.046	671.28	5.37024	0.005	0.01
40	26	0.842	0.071	434.914	3.47931	0.031	0.12
50	26	0.842	0.306	100.911	0.80729	0.446	0.38
60	26	0.842	0.465	66.4062	0.53125	0.588	0.53
70	26	0.842	0.634	48.7048	0.38964	0.677	0.55
80	26	0.842	0.814	37.9347	0.30348	0.738	0.68
90	26	0.842	1	30.8789	0.24703	0.781	0.82

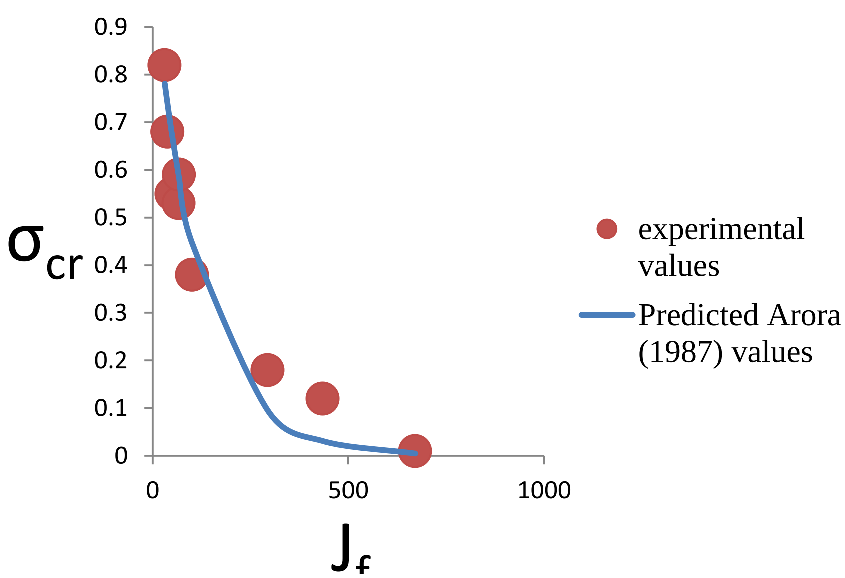


Fig. 5.13Joint factor versus compressive strength ratio (Double joint)

Table 5.7

Values of Jn, J_f and σ_{cr} for jointed specimens (Single joint with gouge fill)

β(angle)	**Jn**	**r**	**n**	**J_f**	**J_f * 0.008**	**σ_{cr} Predicted Arora ,1987**	**σ_{cr} Experimental values**
0	13	0.613	0.81	26.1817	0.20945	0.811	0.79
10	13	0.613	0.46	46.1026	0.36882	0.692	0.67
20	13	0.613	0.105	201.973	1.61578	0.199	0.18
30	13	0.613	0.046	461.026	3.6882	0.025	0.013
40	13	0.613	0.071	298.693	2.38954	0.092	0.04
50	13	0.613	0.306	69.3045	0.55444	0.574	0.26
60	13	0.613	0.465	45.6068	0.36485	0.694	0.59
70	13	0.613	0.634	33.4498	0.2676	0.765	0.62
80	13	0.613	0.814	26.053	0.20842	0.812	0.75
90	13	0.613	1	21.2072	0.16966	0.844	0.82

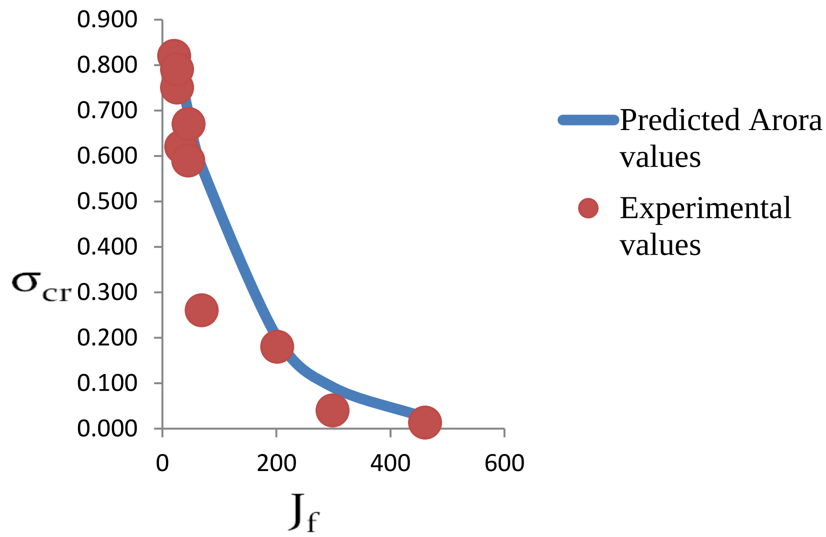


Fig. 5.14Joint factor versus compressive strength ratio (Single joint with gouge fill)

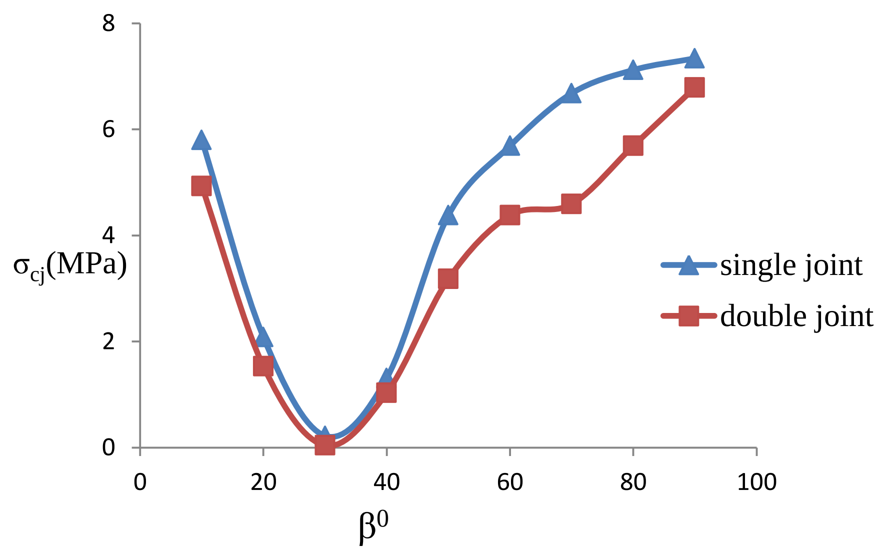


Fig. 5.15 σ_{cj} versus orientation angle (β^0)(joints without gouge fill)

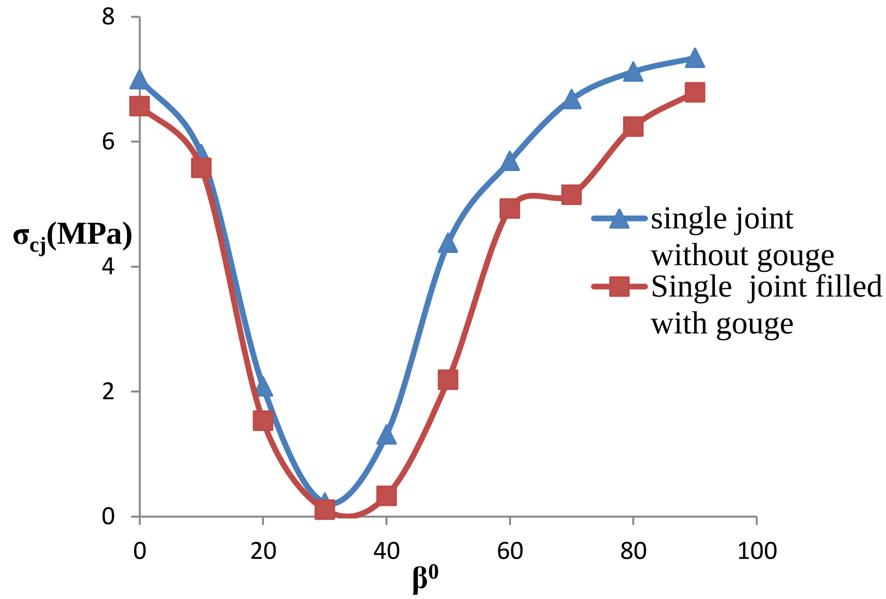


Fig. 5.16 σ_{cj} versus orientation angle (β^0)(joints with and without gouge fill)

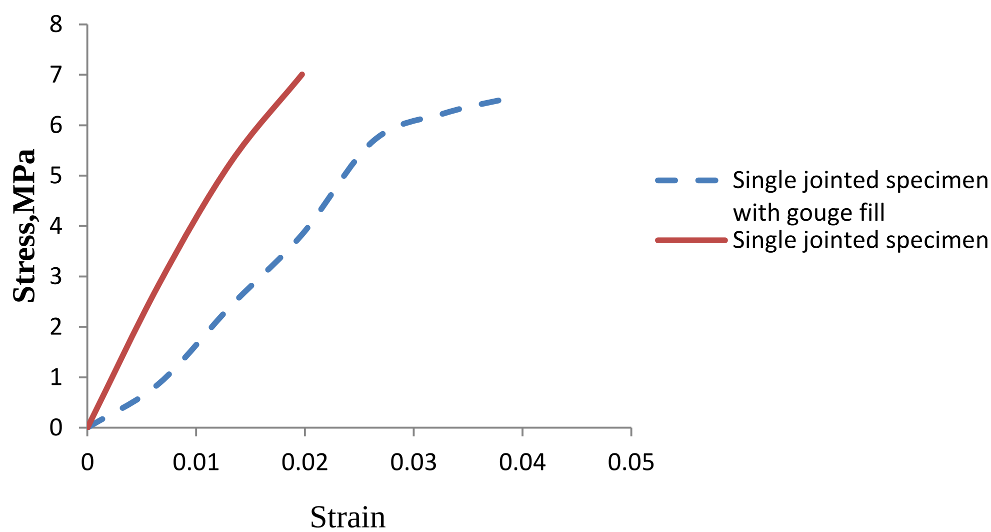


Fig. 5.17Stress strain curve for gouge filled joint at $\beta=0^0$.

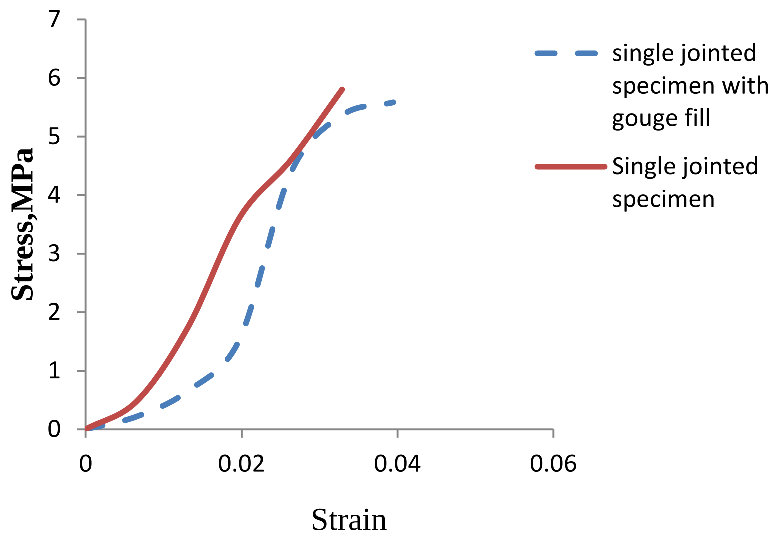


Fig. 5.18Stress strain curve for gouge filled joint at $\beta=10^0$.

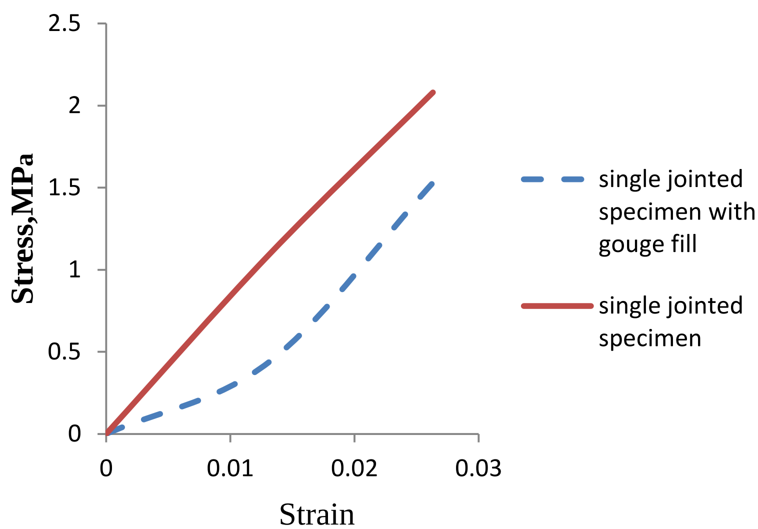


Fig. 5.19Stress strain curve for gouge filled joint at $\beta=20^0$.

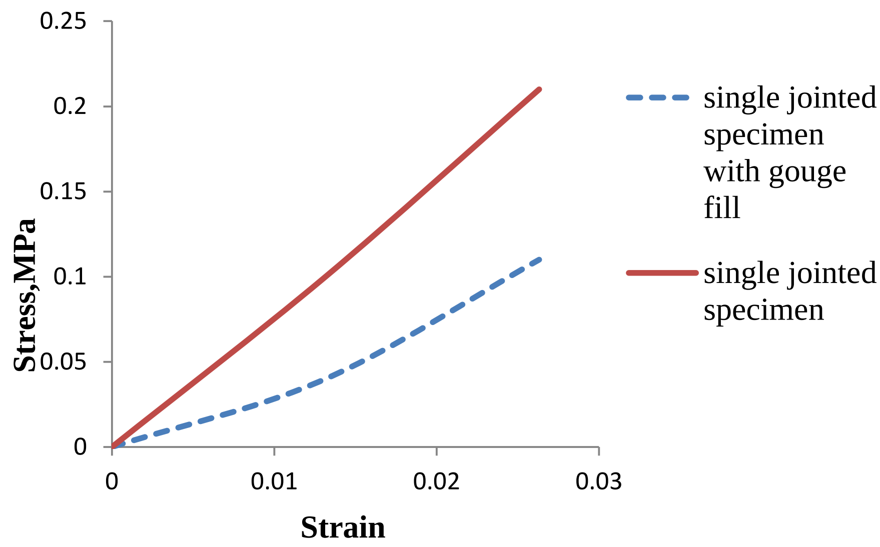


Fig. 5.20Stress strain curve for gouge filled joint at β=30^{0}.

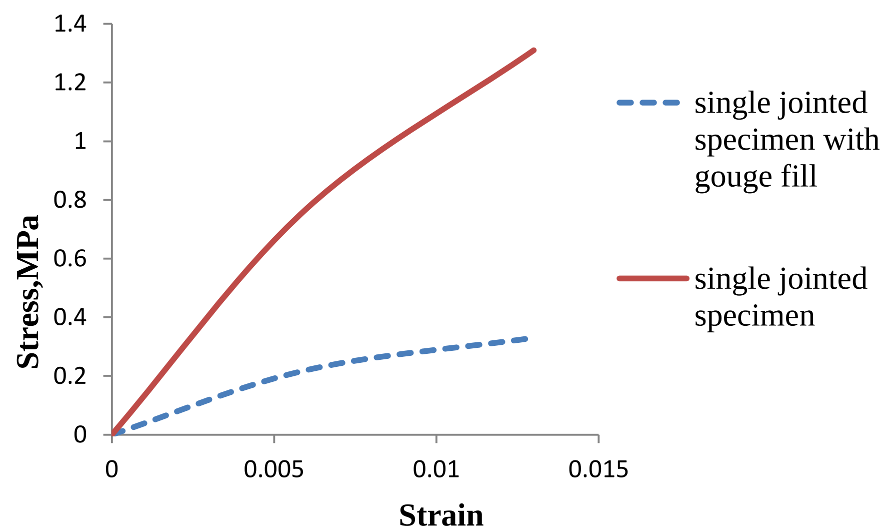


Fig. 5.21Stress strain curve for gouge filled joint at β=40^{0}.

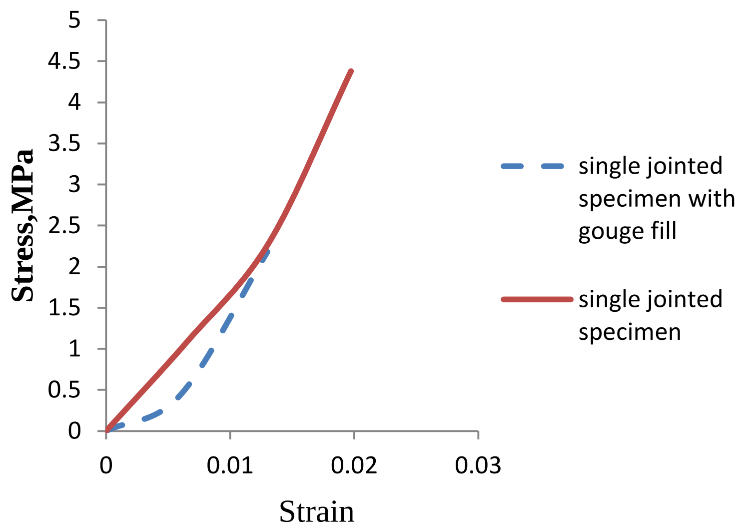


Fig. 5.22Stress strain curve for gouge filled joint at β=50^0.

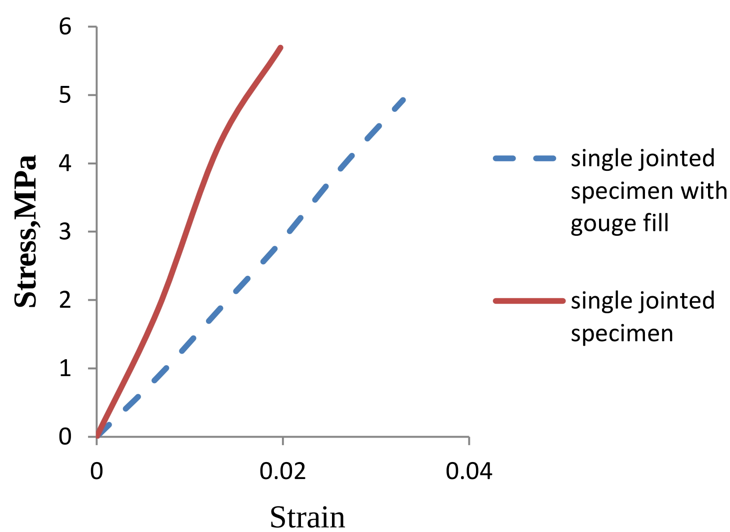


Fig. 5.23Stress strain curve for gouge filled joint at β=60^0.

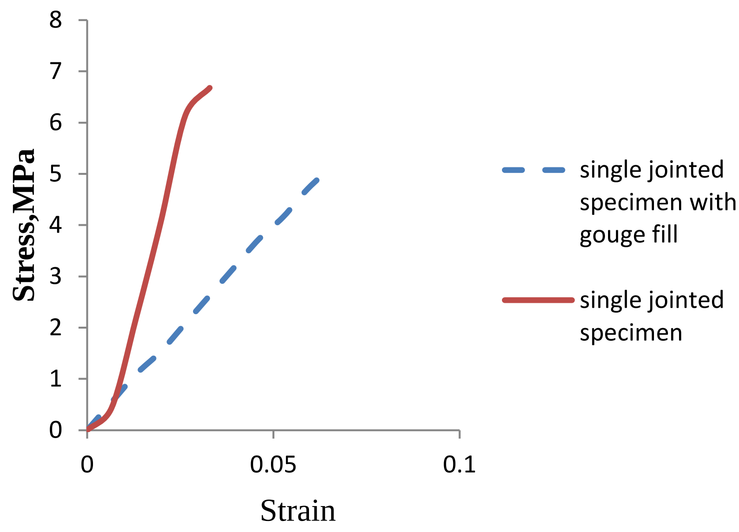


Fig. 5.24Stress strain curve for gouge filled joint at β=70^{0}.

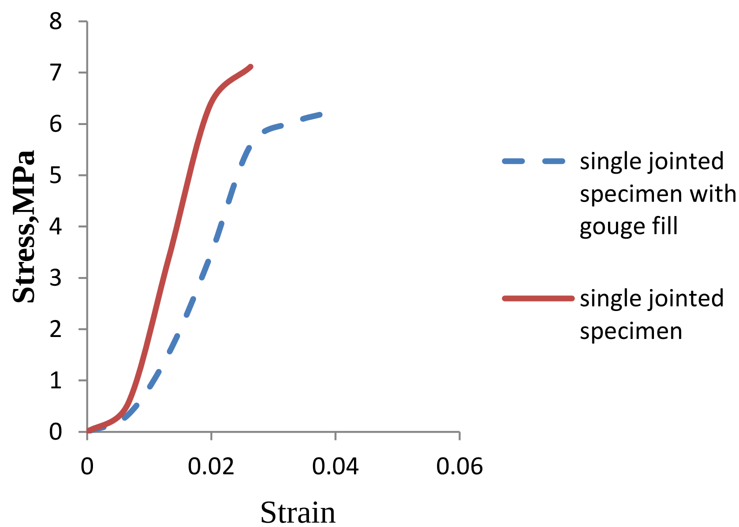


Fig. 5.25Stress strain curve for gouge filled joint at β=80^{0}.

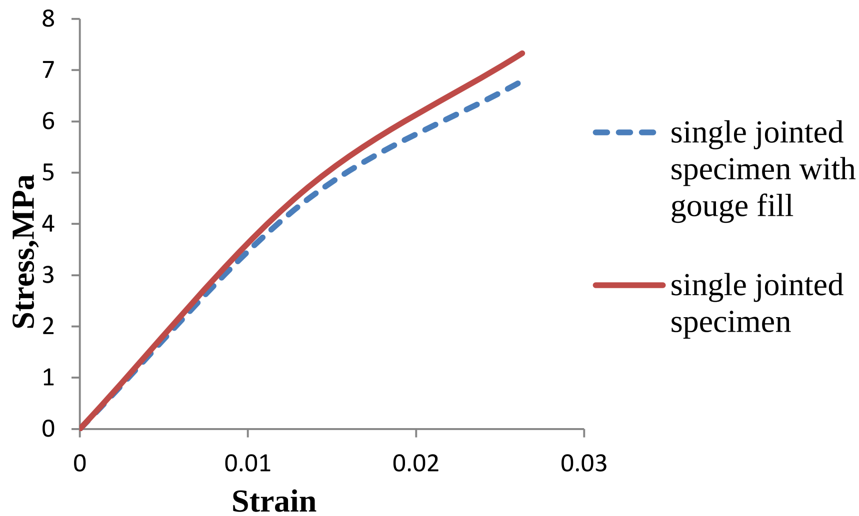


Fig. 5.26Stress strain curve for gouge filled joint at β=90^0

Table 5.8

Values of σ_{cj} for different joint Orientation angle,β^0

β^0	SINGLE JOINT($\sigma_{cj,}$,MPa)	DOUBLE JOINT($\sigma_{cj,}$,MPa)	DECREASE IN ($\sigma_{cj,}$,MPa)
10	5.8	4.93	0.87
20	2.08	1.53	0.55
30	0.22	0.04	0.18
40	1.31	1.03	0.28
50	4.38	3.18	1.20
60	5.69	4.38	1.31
70	6.68	4.59	2.09
80	7.12	5.69	1.43
90	7.34	6.79	0.55

Table 5.9

Values of σ_{cr} for different joint Orientation angle,β^0

β^0	SINGLE JOINT $\sigma_{cr} = (\sigma_{cj} / \sigma_{ci}$,) MPa	DOUBLE JOINT $\sigma_{cr} = (\sigma_{cj} / \sigma_{ci}$), MPa	DECREASE IN σ_{cr} , MPa
10	0.70	0.59	0.1
20	0.25	0.18	0.07
30	0.03	0.01	0.02
40	0.16	0.12	0.03
50	0.53	0.38	0.14
60	0.68	0.53	0.16
70	0.80	0.55	0.25
80	0.86	0.68	0.17
90	0.88	0.82	0.07

Table 5.10

Values of E_{tj} for different joint Orientation angle, β^0

β^0	Single joint E_{tj}, MPa	Double joint E_{tj}, MPa	Reduction in E_{tj}, MPa
10	300	250	50.0
20	160	125	35.0
30	11	6.25	4.8
40	133	80	53.0
50	300	150	150.0
60	350	250	100.0
70	364	275	89.0
80	375	300	75.0
90	388	333.333	54.7

Table 5.11

Values of σ_{cj} for different joint Orientation angle,β^0 (joints with and without gouge fill)

	Single joint	Gouge filled single joint	Decrease in
β^0	σ_{cj}, MPa	σ_{cj}, MPa	σ_{cj} ,MPa
0	7	6.57	0.43
10	5.8	5.58	0.22
20	2.08	1.53	0.55
30	0.22	0.11	0.11
40	1.31	0.33	0.98
50	4.38	2.18	2.20
60	5.69	4.93	0.76
70	6.68	5.15	1.53
80	7.12	6.24	0.88
90	7.34	6.79	0.55

Table 5.12

Values of E_{tj} for different joint Orientation angle ,β^0 (joints with and without gouge fill)

β^0	Single joint E_{tj}, MPa	Single joint with gouge fill E_{tj}, MPa	Reduction in E_{tj}, MPa
0	369.23	300	69.23
10	300	257.14	42.86
20	160	40	120
30	11	8	3
40	133	15.63	117.37
50	300	208	92
60	350	229	121
70	364	266.67	97.33
80	375	300	75
90	388	337.5	50.5

Table 5.13

Values of σ_{cj} and E_{tj} for different types of joint (joints with and without gouge fill)

Types of joint	Double joint(σ_{cj},,MPa)	Double joint with gouge fill(σ_{cj},,MPa)	Double joint $E_{tj\ ,\ MPa}$	Double joint with gouge fill $E_{tj\ ,\ MPa}$	Reduction in σ_{cj},,MPa	Reduction E_{tj},MPa
2j-90	6.78	5.25	333.33	200	1.53	133.33
1j-60 and 1j-90	3.94	2.63	266.67	100	1.31	166.67

CHAPTER 6

Conclusions

Based on the laboratory tests on model filled with and without gouge following conclusions are drawn.

1. The Uniaxial compressive strength and elastic modulus E_{ti} of intact specimen was found to be 8.32 MPa and 462.5MParespectively. Hence as per ISRM (1979) classification of intact rocks, the plaster of Paris tested in this study is classified as low strength rock.

2. The following relationship given by Arora (1987) has been used for predicting the strength of jointed rocks which is almost matching with that of the present experimental strength values.

   $\sigma_{cr} = e^{-0.008 \times J_f}$

   Where $\sigma_{cr} = \sigma_{cj} / \sigma_{ci}$, $J_f = (J_n/(n \times r))$,

   σ_{cj}= uniaxial compressive strength of jointed rock

   σ_{ci}= uniaxial strength of intact rock .

   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.

3. The strength of jointed specimen depends on the joint orientation β^0 with respect to the direction of major principal stress .The strength at $\beta = 30^0$(for single jointed specimen)

was found to be 0.22MPawhich is minimum and the strength at $\beta = 90^0$(for single jointed specimen) was found to be 7.34 MPa which is maximum.

4. σ_{cr} versus β^0 depicts a "U" shape curve as shown in Fig 5.16.

5. As the number of joints increases, the uniaxial compressive strength decreases.

6. The values of (E_r) which is ratio of (E_j/E_i) depends on the joint orientation β^0 . The modulus ratio is least at $\beta = 30^0$ and maximum at $\beta = 90^0$.

7. The empirical relationship suggested by Arora (1987) as $\sigma_{cr} = \sigma_{cj}/\sigma_{ci} = e^{-0.008 \times J_f}$ is also applicable for gouge filled joints.

CHAPTER 7

Scope of future work

1. Studies can be done for multiple joints at various angles of orientation.
2. The effect of rate of loading, temperature and confining pressure on the strength characteristics can be studied.
3. Strength and deformation behaviour of jointed specimens can be done under triaxial loading conditions for samples with single or multiple joints.
4. Strength and deformation behaviour of jointed specimens under triaxial loading conditions can be studied with gouge filled joints.
5. Prediction of strength and deformation behavior of specimens with any arbitrary orientation can be done by using artificial neural network with the help of these data's mentioned in the study.
6. Different theories can be used for developing numerical models and the results can be compared with the experimental results to reach at the best possible numerical model.
7. Different software's can be used to analyse the experimental results.

CHAPTER 8

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