BEARING CAPACITY OF SHALLOW FOUNDATIONS ON JOINTED ROCK

A DISSERTATION

Submitted to

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In partial fulfilment of the requirement for the award of degree of

Master of Technology

In

Geotechnical Engineering

Submitted by

NEERAJ GUPTA

(Roll No. 2K15/GTE/11)

Under the guidance of

Dr. Amit Kumar Shrivastava

(Associate Professor)

Department of Civil Engineering

DELHI TECHNOLOGICAL UNIVERSITY

Delhi - 110042

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CANDIDATES DECLARATION

I do hereby certify that the work presented in this report, entitled "**BEARING CAPACITY OF SHALLOW FOUNDATIONS ON JOINTED ROCK"** in the partial fulfilment of the requirements for the award of degree of "Master of Technology in Geotechnical Engineering", submitted in the Department of Civil Engineering, Delhi Technological University, is an authentic record of my own work carried under the guidance of Dr. A.K. Shrivastava, Department of civil Engineering.

I have not submitted the matter embodied in the report for the award of any other degree or diploma.

Date : 20/12/2017 Neeraj Gupta

. (2K15/GTE/11)

CERTIFICATE

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

Dr. A.K.Shrivastava

Associate Professor

Department of Civil Engineering

Delhi Technological University

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Neeraj Gupta

M. Tech (Geotechnical Engineering)

Roll No. 2K15/GTE/11

Department of Civil Engineering

Delhi Technological University, Delhi

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CHAPTER 1

INTRODUCTION

1.1 General

Rock is a very different engineering material and the process of designing a structure in a rock mass is a very complex problem. The distribution of stresses in the rock mass is equally important as the applied loads. Therefore measurement and judgment both are important while determining the material strength. The behavior of jointed rock mass depends upon many like joint frequency, location of joints, joint orientation, infill material, joint strength etc. The present study aims at studying the variation of bearing capacity of shallow foundation resting over a jointed rock mass through model study. As the process of determining in situ strength of a jointed rock mass is very expensive and difficult, many researchers have performed model studies to predict the strength behavior. The important factors on which strength of a rock mass depends are type of rock, bedding planes, condition of initial stress, presence of joints and cracks, nature of joint surface and the presence of infill material between the bedding planes.

Rocks are not found in intact state anywhere in the world, they essentially have faults and fissures in them which make it an anisotropic medium. These discontinuities like fissures, cracks, joint, bedding plane and faults make a rock weaker and more prone to deformations. In the case of important structures like a dam, leakage of water can take place due to this which can lead to loss of energy and erosion of the dam.

The strength and deformation behavior of a rock mass depends upon both, properties of the intact rock and the nature of discontinuities present in it. The strength of rock mass depends upon following factors (Hudson and Harrison, 1997):

1. The orientation of joints with respect to the direction of principle stress.
2. Joint spacing.
3. Joint opening
4. No. of joints per unit length
5. Shear strength of rock along the joints
6. Joint roughness
7. Joint frequency

In this study an effort is made to study the variation of Bearing Capacity of shallow foundation over a jointed rock mass model with the inclination of joints.

Before studying the nature and behavior of a jointed rock mass, first we have to study the two basic components which collectively constitute the system i.e. the intact rock materials and the discontinuities. The pieces of rock will slide, rotate or will be crushed in reaction to the loads applied on the rock mass, will depend on the number, nature and orientation of the joints. Large number of combinations of block shapes and sizes are possible, therefore it is required to find any trend which is common to all the possible combinations. For any study, the most important objective is to determine any such common trend. Some basic definitions are important to understand before starting the study of individual components and the whole system.

1.2 Intact rock mass

An intact rock is the aggregate of minerals in which there are no structural defects. Such rocks can be treated as homogenous, isotropic and continuous. They undergo brittle failure which means there is an abrupt decrease in the strength, whenever stress exceeds a limiting value.

1.2.1 Strength of intact rock mass

The strength of an intact rock mass depends primarily on the following factors (Goodman, 1989):

(1) Geological factors

(2) Lithological factors

(3) Physical factors

(4) Environmental factors

(5) Mechanical factors

If a rock mass is present on the earth's surface, there is no confining pressure is acting on it. But when the rock mass is situated beneath the earth's surface then there is large effect of confining pressures on the strength of rock mass. Many investigations have been conducted till date to study the effect of confining stress, the result of these studies show a non linear variation of strength with the confining pressure. An important result of the change in behavior from brittle to ductile is observed under uniaxial condition at high confining pressure.

1.2.2 Effect of rate of loading, confining pressure and temperature:

Apart from in situ conditions, there are some other factors which affect the strength of an intact rock specimen. These factors are summarized below:

1. Strength of the rock increases with increase in confining pressure. Also the degree of post yield axial strain hardening increases with confining pressure.
2. There is an increase in dilation at low confining pressure, which becomes max, at around 400 MPa and further reduces at higher confining pressure.
3. The effect of the pore water pressure is dependent on the viscosity of pore fluid, porosity of rocks, size of the specimen and the strain rate. Usually strength decreases with increase in pore pressure.
4. Generally there is a decrease in the strength of a rock with increase in temperature but the effect is unequal in different rocks.
5. Usually there is an increase in strength with the increase in rate of loading, but in some cases, an opposite trend has also been observed.

1.3 Jointed Rock Mass

The presence of joints, bedding planes, fissures, faults, folds, etc are found widespread in general engineering practices. There is a major role of discontinuities in deciding the engineering properties of a rock mass. Earthquakes play a major role in creation of discontinuities. According to Piteau (1970), the engineering behavior of rock mass depends on the following factors:

a) Nature of occurrence

b) Orientation or position in space

c) Continuity

d) Intensity

e) Surface geometry

The major indices adopted to describe discontinuity density are as following:

1) Rock quality designation (RQD) technique (Deere, 1964)

2) Determination of Measure of the discontinuities present per unit volume of a rock mass (Skempton, 1969)

3) The scan line survey technique (Piteau, 1970)

4) A relationship between the Rock Quality Designation (RQD) and the number of discontinuities per unit length (Bieniawski, 1973)

1.3.1 Joint Intensity

It is defined as the no. of joints per unit length, perpendicular to the joint planes. The strength and deformation behavior of a rock mass is influenced by joint intensity significantly, strength of the rock mass decreases with increase in no. of joints. It has been confirmed by the work performed Walker (1971), Goldstein (1966) and Lama(1971). Arora (1987) introduced a factor (J_f), to study the strength behavior of a jointed rock specimen. It is defined by the expression as :

$$J_f = J_n /(n.r) \qquad (1)$$

Where,

J_n = Number of joints per unit length

n = Joint inclination parameter (a function of joint orientation)

r = Roughness parameter (depends on joint condition).

Table 3.2: Value of the inclination parameter, n (Ramamurty, 1993)

Joint orientation (β^0)	Inclination parameter (n)
0°	0.810
10°	0.460
20°	0.105
30°	0.046
40°	0.071

50°	0.306
60°	0.465
70°	0.634
80°	0.814
90°	1.000

1.3.2 Scale Effect

Generally with increase in the volume of test specimen, strength of the rock material decreases. This property is known as scale effect which can also be observed in case of soft rocks. Bandis et al. (1981) performed experiments to study the scale effect on the shear behavior of rock joints. He performed direct shear tests on specimens with different sizes and various natural joint surfaces. Results of the study show significant scale effects on the deformation and shear strength characteristics. In case of rough and undulating joints, scale effects are more pronounced. Whereas for plane joints, they are virtually seen absent.

1.4 Objective of Project

Rocks are not found in intact state anywhere in the world, they essentially have faults and fissures in them which make it an anisotropic medium. The behavior of jointed rock mass depends upon many factors like joint frequency, joint location, orientation of joints, infill material, joint strength etc. The present study aims at studying the variation of bearing capacity of shallow foundation resting over a jointed rock mass through model study. The main objectives of the study are as follows:

- Physical modeling of the jointed rock mass with various joint inclinations using the model material.
- Testing of the model and analysis of results obtained.
- To study the effect of joint orientation on the bearing capacity of shallow foundation on jointed rock mass.
- To compare the results with previous studies and generation of empirical relation or correction in the existing relation if necessary.

1.5 Organisation of Thesis

This thesis is organised into 6 chapters, which sequentially elaborates the details of this work. The sequence of various chapters is as follows:

Chapter 1: This chapter is the introduction of the work. It contains the basic concepts of rocks and rock mass, which is a pre requisite to any beginner. This chapter also explains various factors on which strength of intact or jointed rock mass depends. Different modes of failure of rock masses are also described in the chapter.

Chapter 2: This chapter contains the details of literature review which is done before starting this work. Previous works performed by other researchers on this subject are quoted.

Chapter 3: In this chapter, the methodology adopted in this work is explained. The plan of experimental program, model preparation and testing procedure are described.

Chapter 4: This chapter contains all the test results obtained from testing, in the form of tables and charts. Compilation of results is done to study the variation of desired parameter. Results obtained are also compared with the previous studies.

Chapter 5: Based on the results obtained from the work, some conclusions drawn are presented in this chapter in point wise form.

Chapter 6: This chapter contains the list of all published articles, papers and books which are cited in the thesis at various places and whose reference is taken to carry out this work.

CHAPTER 2
LITERATURE REVIEW

Many structures these days are constructed over rocky subgrade. Rock mass is a stronger media than soil but determination of its strength accurately is a difficult task. Strength of intact rock specimens can be easily determined by laboratory tests but the strength of a jointed rock mass as a whole cannot be determined accurately because joints are neither regular nor equally strong. Some researchers have given empirical relations, which can be used to get a fair of strength of the rock mass. Some of the works are quoted here.

Arora (1987) and Ramamurthy and Arora (1994) provided solution for determining the unconfined compressive strength using the concept of joint factor. They give maximum importance to joint inclination, joint frequency and joint strength for predicting behavior of jointed rocks. By clubbing these three parameters, a factor called Joint Factor (J_f) was defined as.

$$J_f = \frac{J_n}{nr} \tag{7}$$

Where, J_n = Number of joints per meter in the direction of major principal stress,

n = Inclination factor which depends on orientation of joint with respect to loading direction,

r = Joint strength parameter = $\sigma_{cj} / \sigma_{ci} = \tan \phi_j$

ϕ_j = Discontinuity friction angle

The value of J_f thus obtained is an indicative of the extent of weakness which has been brought to intact rock by the presence of joints.

The average value of strength of the jointed rock mass is the unconfined compressive strength of the jointed rock mass that is given as

$$q_u = \sigma_{cj} = \sigma_{ci} \exp\left(-0.008 J_f\right) \tag{8}$$

Singh and Rao (2005) suggested a methodology to calculate the ultimate bearing capacity of shallow foundations in anisotropic rock masses. This approach considers the strength and deformation properties of the rock mass as a whole, which depends on both intact rock properties and joint properties. Bell's approach was used to compute the bearing capacity. In this approach, the major principal stress acting on the soil element below the corner of a smooth shallow foundation, at the time of failure under confining pressure, is taken as the ultimate bearing capacity. A simple parabolic equation was derived on the basis of critical state of rock mass, which is used to define the strength of rock mass. The uniaxial compressive strength of the jointed rock mass is an input parameter to the developed strength criterion, which is determined using the concept of joint factor. According to this approach, active and passive zones are developed in the rock mass beneath a smooth strip footing. These zones are assumed to be divided by a vertical line, passing through the edge of the strip footing. The ground surface is horizontal and the strip footing is assumed to be of infinite length. The rock mass below the footing and the surrounding mass is assumed to be in a triaxial state of stress. The vertical stress acting below the footing is considered the major principal stress for the active zone. For the passive zone, effective surcharge acts as the minor principal stress which acts in the vertical direction and the major principal stress acts in the horizontal direction. The equilibrium of two adjacent elements of the rock mass is considered at the time of failure, one just below the edge of the strip footing and the other just outside of it.

The uniaxial compressive strength of the jointed rock mass σ_{cj}, depends upon the mode of failure and the Joint Factor J_f (Singh et al., 2002). Its value is determined as:

$$\sigma_{cj} = \sigma_{ci}\exp(aJ_f) \tag{9}$$

Where, a is an empirical coefficient depending on the mode of failure.

Bindlish et. al. (2012) conducted an experimental study in which a rigid footing was placed over the top surface of the jointed rock mass and it was loaded upto failure. The effect of interlocking and the joint orientation on the ultimate bearing capacity of the rock mass was studied. A methodology had been suggested to determine the ultimate bearing capacity of shallow foundation placed on anisotropic rock mass in the field.

The model material used in the study was plaster of Paris mixed with medium sand. The size of the rock mass specimen was kept as 750 × 750 × 150 mm^3 while the size of elemental blocks used was 25 × 25 × 75 mm^3. The size of elemental plates used was 750 × 150 × 25 mm^3 or more.

The results of the study indicated that the degree of anisotropy in case of shallow foundations is much less than envisaged in the study by Singh et al. Also, the interlocking introduced by stepping of joints substantially enhances the strength if the continuous joints dip at angle less than about 45^o. Splitting and shearing were found to be the dominating failure mode of the rock mass beneath the footing. The results were analysed and a modified approach was suggested in this study. The suggested approach is applicable to both continuous as well as discontinuous joints and was found to be predicting the ultimate bearing capacity of the rock mass which closely match with laboratory results.

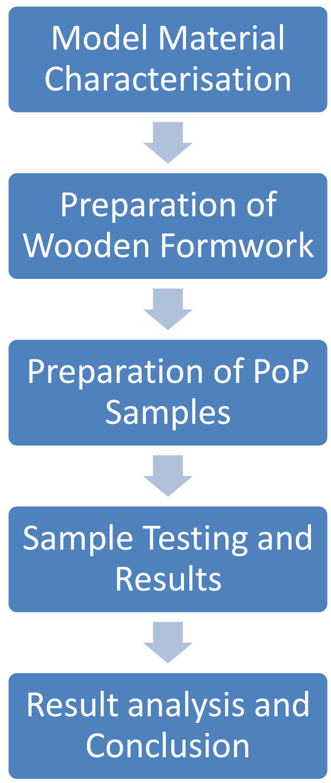
Shukla at. al. (2014) studied the strength and deformational behavior of slopping anisotropic rock mass had been assessed experimentally as well as analytically. The jointed rock mass was assembled using sand stone element of size 25 mm × 25 mm × 75 mm along different joint angles of 15°, 30°, 45°, 60°, 75° and 90° and slope angles of 30°,45°, 60°, 75°, and 90° with the horizontal in plane strain condition and 15 cm × 15 cm footing placed exactly at the edge of the slope as well as at 15 cm from edge. Joint angle, distance of footing from edge and modes of failure were important parameters, which govern the load intensity at slope apart from rock mass properties.

According to their results, bearing capacity of jointed rock mass is half of the total buckling load capacity when footing placed at edge of the slope. Average settlement of footing for joint angle (θ) = 0°, 15°, 30° (Buckling mode of failure) were less than the joint angle (θ) = 45°, 60° (Combination of Sliding and buckling mode of failure) due to mode of failure. Similarly the magnitude of bearing capacity was more for joint angle (θ) = 0°, 15°, 30° (Buckling mode of failure) as compare to the joint angle (θ) = 45°, 60° (Combination of Sliding and buckling failure). Settlements at failure were more when footing placed at 15 cm distance from edge compared to the footing placed exactly at edge.

CHAPTER 3

METHODOLOGY

3.1 Plan of the experimental program



Experiments were performed on models prepared using plaster of paris (PoP) so as to obtain identical, uniform and homogeneous specimen in order to obtain the desired results.

PoP was procured from the marked with brand name Trimurti Zip Plast. All the tests were conducted using a water content of 50% by mass, so as to produce the required workability to cast the required size of blocks. Before the preparation of rock mass test samples, following tests were conducted to determine the physical and engineering properties of model material:

Initial and Final Settling Time Test: This test was performed to obtain the initial and final setting time of pop after the addition of water. The test was performed using vicat's mould apparatus in the Concrete Engg. Lab.

Specific Gravity Test: The test was performed to determine relative density of pop. Pycnometer was used to determine the specific gravity of model material.

Direct Shear Test: This test was conducted to obtain the c and ϕ_j values of the material along the joints. The test was performed in the Geotechnical Engg. Lab of Civil Engg. Department (DTU).

Fig-3.1: Direct Shear Test apparatus

Following results were obtained from the test:

Table-3.1: Test results of the Direct Shear Test

Normal stress, σ_n (MPa)	Shear stress, τ(MPa)
0.049	**0.298**
0.098	**0.417**
0.147	**0.537**

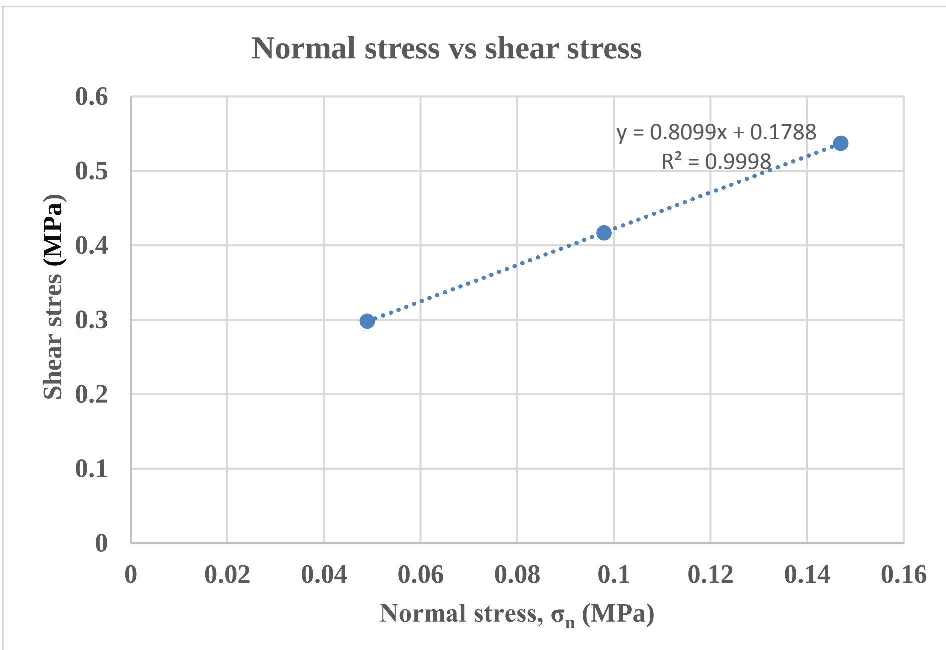


Fig-3.2: Test results of the Direct Shear Test

Unconfined Compressive Strength (UCS) Test: This test was performed to determine the UCS of the intact pop sample, which is the most important input parameter in any empirical or analytical calculation related to jointed rock mass. Cylindrical samples were prepared for the test with length and height 76 mm and 38 mm respectively. The test was conducted on the CBR machine in the Transportation Engg. Lab.

Table-3.2: Results of Unconfined compression test.

Strain (%)	Stress (MPa)
0.641	1.61
1.282	4.00
1.923	6.40
2.564	7.95
3.205	8.05
3.846	8.16
4.487	7.84

Fig-3.3: Unconfined compression test.

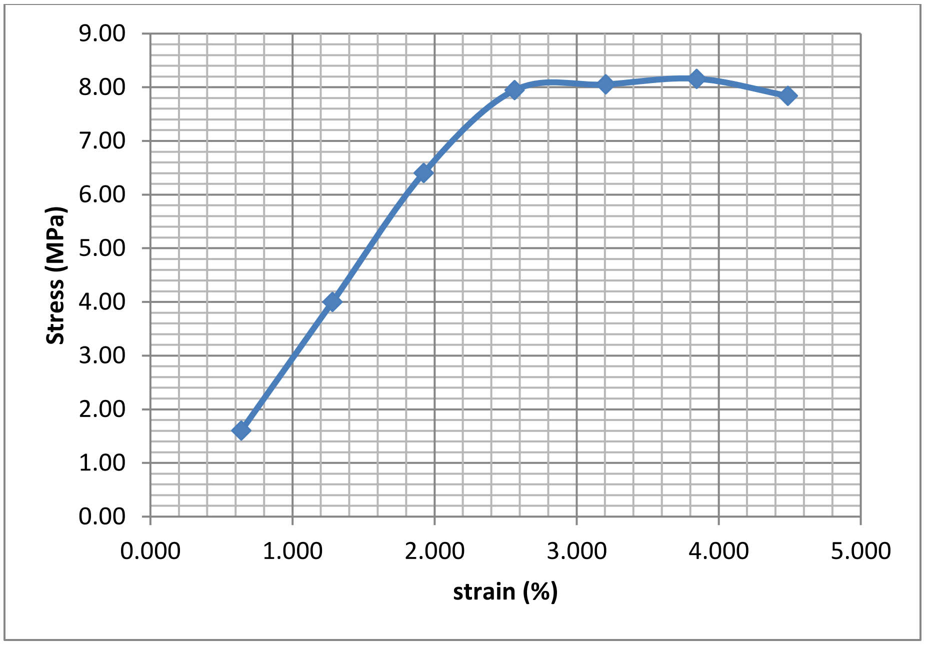


Fig- 3.4: Results of Unconfined compression test.

On the basis of above test results, following properties of the model material are obtained:

Table-3.3: Physical and engineering properties of model material

Property of model material	Value
Specific gravity (G)	**2.78**
Initial setting time	**6 min 30 sec**
Final setting time	**14 min**
Cohesion intercept (*c*)	**0.178 MPa**
Angle of friction(ϕ)	**39°**
Unconfined compressive strength (σ_{ci})	**8.16 MPa**

After the model material characterisation, calculations were performed to decide the size of each block of pop, so that they can be assembled into models of rock masses having dimensions 300mm×300mm×300mm, with a regular joint set on various inclinations.

3.2 Sample Preparation

Wooden Formworks: Formworks of various size and shapes were prepared using laminated ply boards, so that pop blocks can be easily removed from the mould after hardening.

L shaped steel clips and screws were used to hold the platens of formwork together, so that they can be opened and fitted again to prepare the pop blocks. Triangular and wedge shaped (corner) blocks were casted in the moulds of suitable shapes, while intermediate pieces were prepared by cutting the rectangular slabs.

Fig- 3.5: Wooden formwork for casting pop blocks

Cutting of pop blocks: After casting large number of triangular and rectangular blocks, they were cut in pre determined shapes so that they can be assembled to produce rock mass blocks of size 300mm×300mm×300mm with different orientation of joints.

Hacksaw and sandpaper were used to cut the pop blocks after marking them with pencil. After cutting, they were finished with sandpaper to fit with each other and give proper bearing against each other.

Fig- 3.6 (a): Prepared rock mass samples after cutting.

Fig- 3.6 (b): Prepared rock mass samples after cutting.

Fig- 3.6 (b): Prepared rock mass samples after cutting.

Assembling of rock mass samples in steel frame: After the preparation of test samples, they were finished and fitted in a mild steel frame to get a tight bearing against the frame, so that restraint can be provided aginst any lateral movement to simulate semi infinite conditions.

The steel frame consisted of 4 mild steel angles of length 300 mm each having dimensions 25mm×25mm×5mm. In shallow foundations, plain strain conditions are assumed to prevail. So to simulate such conditios, steel frame was used to restrict any lateral movement.

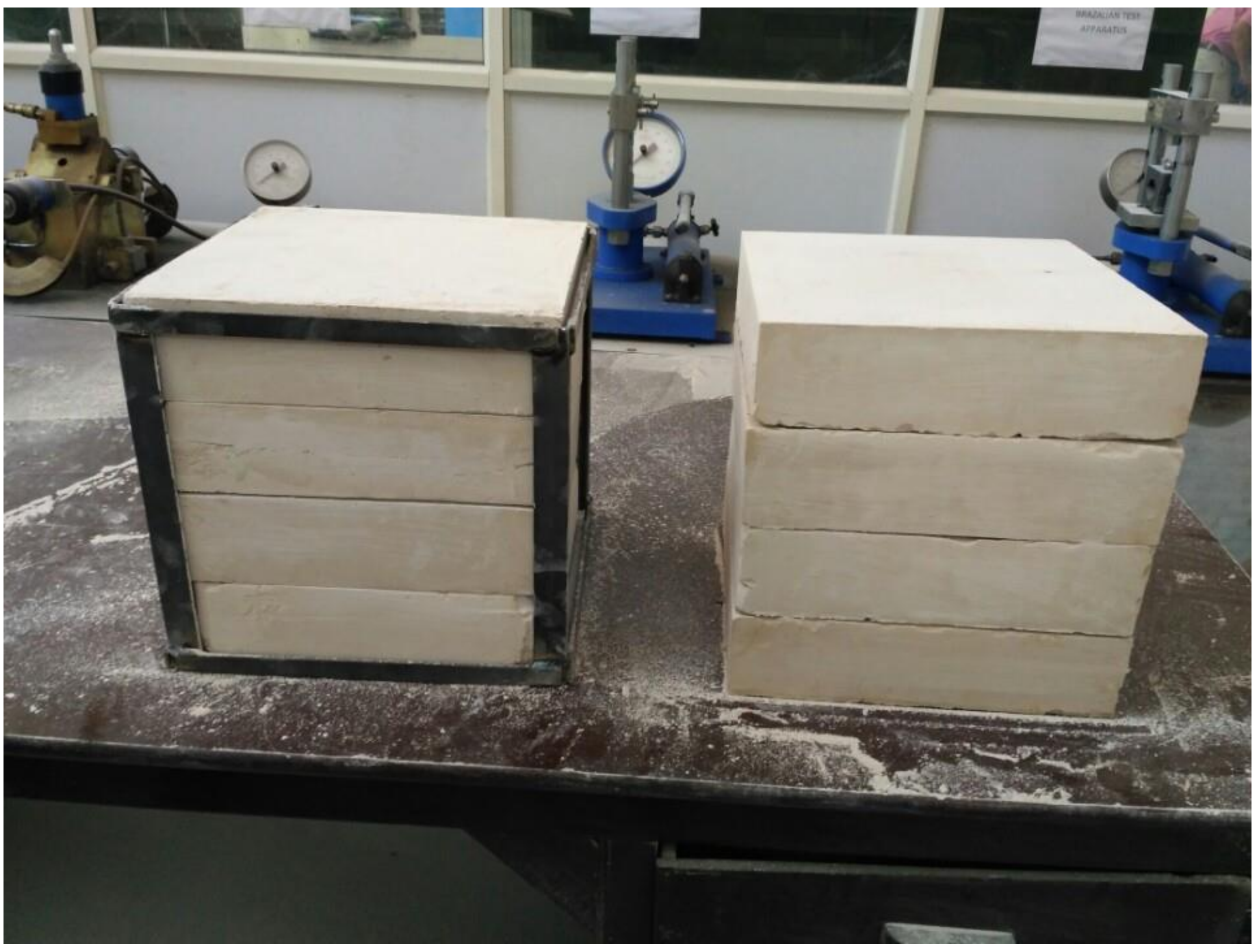


Fig- 3.7: Rock mass sample fitted in steel frame.

3.3 Testing of samples

After the preparation and assembling of samples, they were tested in the UTM of the Earthquake Engg. Lab. Due to some technical issues, load and deflection values could not be obtained from the machine. Therefore proving ring was used to obtain load values and dial gauge was used to obtain deflections.

A square footing plate of size 50mm×50mm was placed at the center of the sample.The samples were designed in a way that a joint essentially passed below the center of footing (Except for samples with joint inclination 0° and 15°). Samples were loaded till failure at a rate of 0.1mm/sec. Load and settlement values were plotted to get load-settlement curve, from which Bearing capacity of the samples were obtained. The failure of samples were brittle and sudden. The load at failure was taken to calculate the bearing capacity.

Fig- 3.8: Sample with joint orientation 0°

Fig- 3.9: Sample with joint orientation 15°.

Fig- 3.10: Sample with joint orientation 30°.

Fig- 3.11: Sample with joint orientation 45°

Fig- 3.12: Sample with joint orientation 60° (After failure)

Fig- 3.13: Sample with joint orientation 75°

Fig- 3.14: Sample with joint orientation 90°

CHAPTER 4

TEST RESULTS

4.1 Test results of rock samples with various inclinations

4.1.1 Sample with joint orientation 0°: Four PoP slabs of dimensions 300 mm × 300 mm × 75 mm were casted and finished. They were placed vertically over each other to form a rock mass sample. The sample was fitted in the steel frame and tested in UTM. Load and settlement values were recorded which are given below.

Table-4.1: Load-settlement values for joint inclination 0°

Load (KN)	Settlement (mm)
0	0
2.160	0.2
5.180	0.4
9.107	0.6
12.127	0.8
15.752	1
18.471	1.2
20.887	1.4
23.303	1.6
23.907	1.8
17.262	2
15.450	2.2
16.054	2.4
12.732	2.6

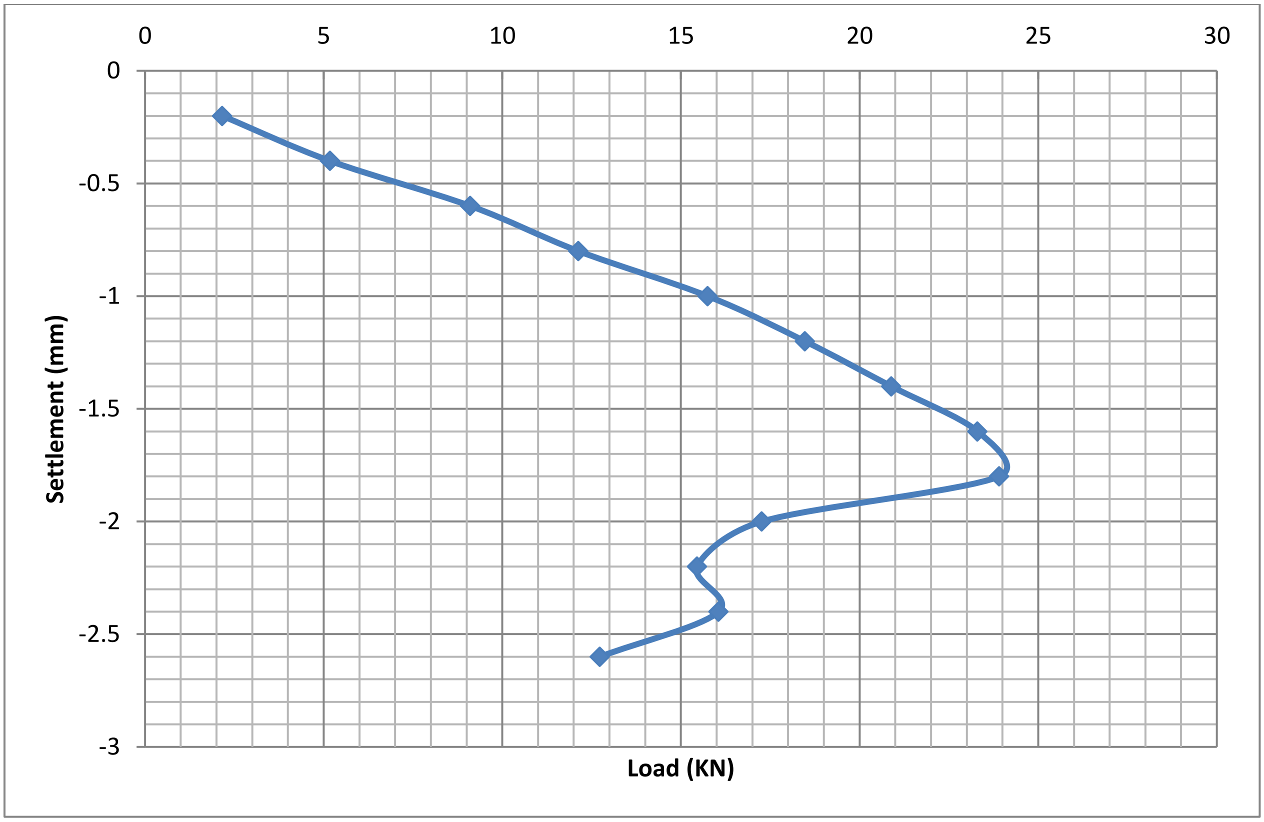


Fig-4.1: Load-settlement curve for sample with joint inclination 0°

From the observed values of load and settlement, a curve was drawn. Using the load – settlement curve, Bearing capacity of the foundation was observed.
From the curve, Bearing capacity of the foundation = **9.68 MPa**

4.1.2 Sample with joint orientation 15°: PoP blocks were casted and fitted after cutting in such a way, that a jointed rock mass sample was obtained with a joint set inclined at 15^{o}. The sample was again fitted in the steel frame and tested under UTM. Load settlement values were recorded and load – settlement curve was plotted.

Table-4.2: Load-settlement values for joint inclination 15°

Load (KN)	Settlement (mm)
0	0
4.33	0.2
6.02	0.4
6.02	0.6
7.17	0.8
11.7	1
17.38	1.2
21.49	1.4
17.26	1.6
11.64	1.8
11.64	2

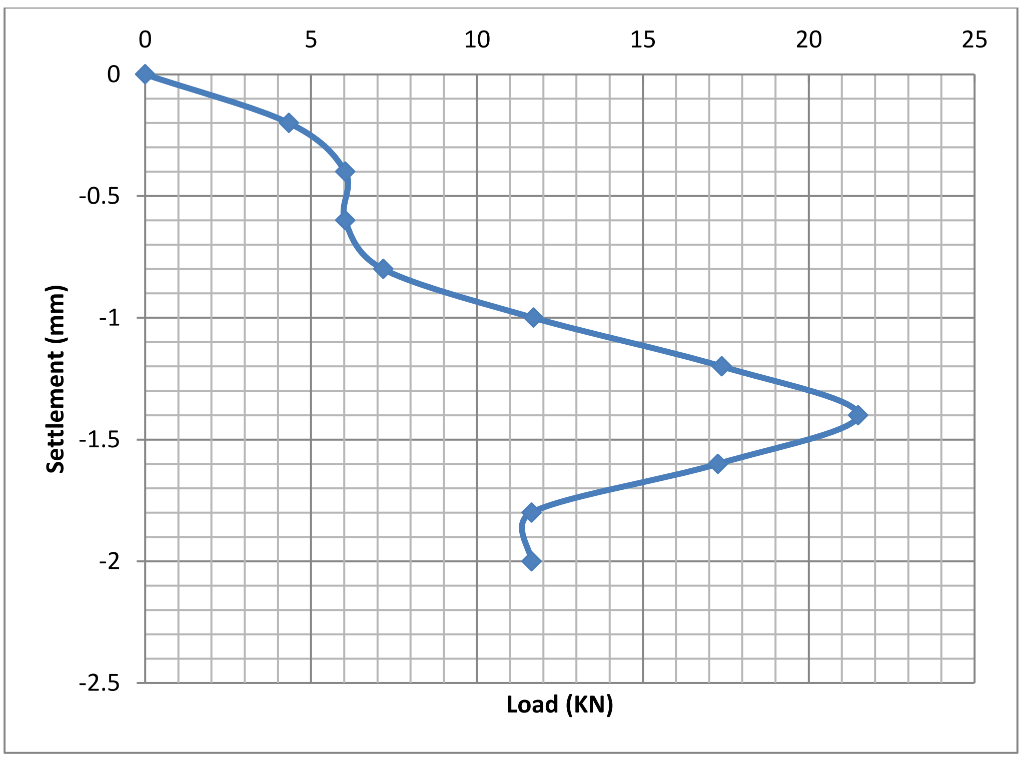


Fig-4.2: Load-settlement curve for sample with joint inclination 15°

From the curve, Bearing capacity of the foundation = **8.59 MPa**

4.1.3 Sample with joint orientation 30°: PoP blocks were casted and fitted after cutting in such a way, that a jointed rock mass sample was obtained with a joint set inclined at 30^0. The sample was fitted in the steel frame and tested under UTM. Load settlement values were recorded and load – settlement curve was plotted.

Table-4.3: Load-settlement values for joint inclination 30°

Load (KN)	Settlement (mm)
0	0
0.83	0.2
1.31	0.4
1.67	0.6
1.97	0.8
2.82	1
3.61	1.2
5.3	1.4
9.4	1.6
9.53	1.8
9.65	2
10.19	2.2
11.88	2.4
13.21	2.6
16.05	2.8
18.47	3
20.16	3.2
9.41	3.4
10.25	3.6

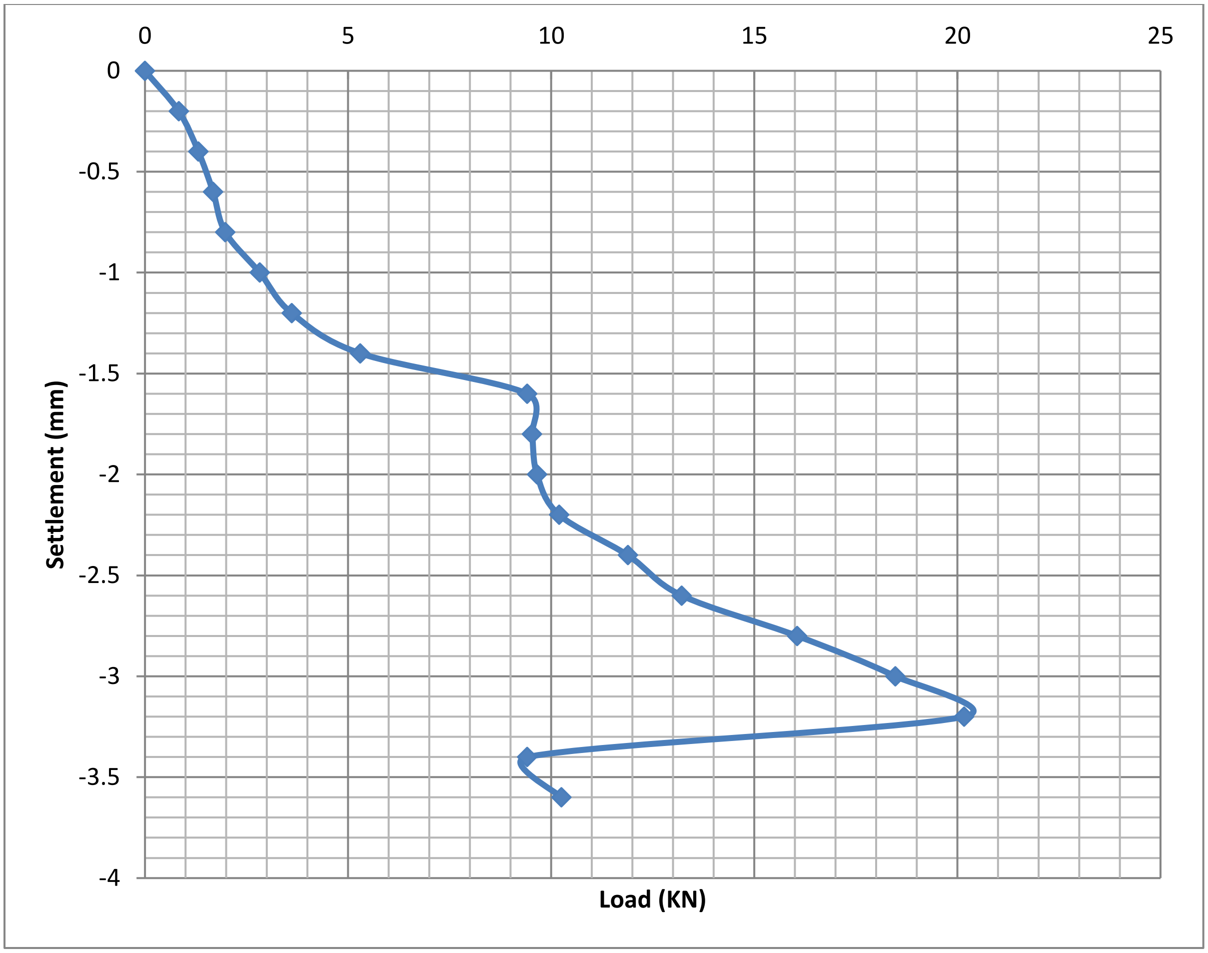


Fig-4.3: Load-settlement curve for sample with joint inclination 30°

From the curve, Bearing capacity of the foundation = **8.06 MPa**

4.1.4 Sample with joint orientation 45°: PoP blocks were casted and fitted after cutting in such a way, that a jointed rock mass sample was obtained with a joint set inclined at 45^0. The sample again fitted in the steel frame and tested under UTM. Load settlement values were recorded and load – settlement curve was plotted, from which Bearing capacity of the footing was calculated.

Table-4.4: Load-settlement values for joint inclination 45°

Load (KN)	Settlement (mm)
0	0
1.55	0.2
2.04	0.4
2.76	0.6
3.37	0.8
4.03	1
4.69	1.2
5.18	1.4
5.9	1.6
6.99	1.8
6.99	2
8.2	2.2
8.8	2.4
9.22	2.6
10.01	2.8
11.22	3
12.43	3.2
13.51	3.4
14.3	3.6
15.45	3.8
16.65	4
17.26	4.2
18.34	4.4
15.45	4.6

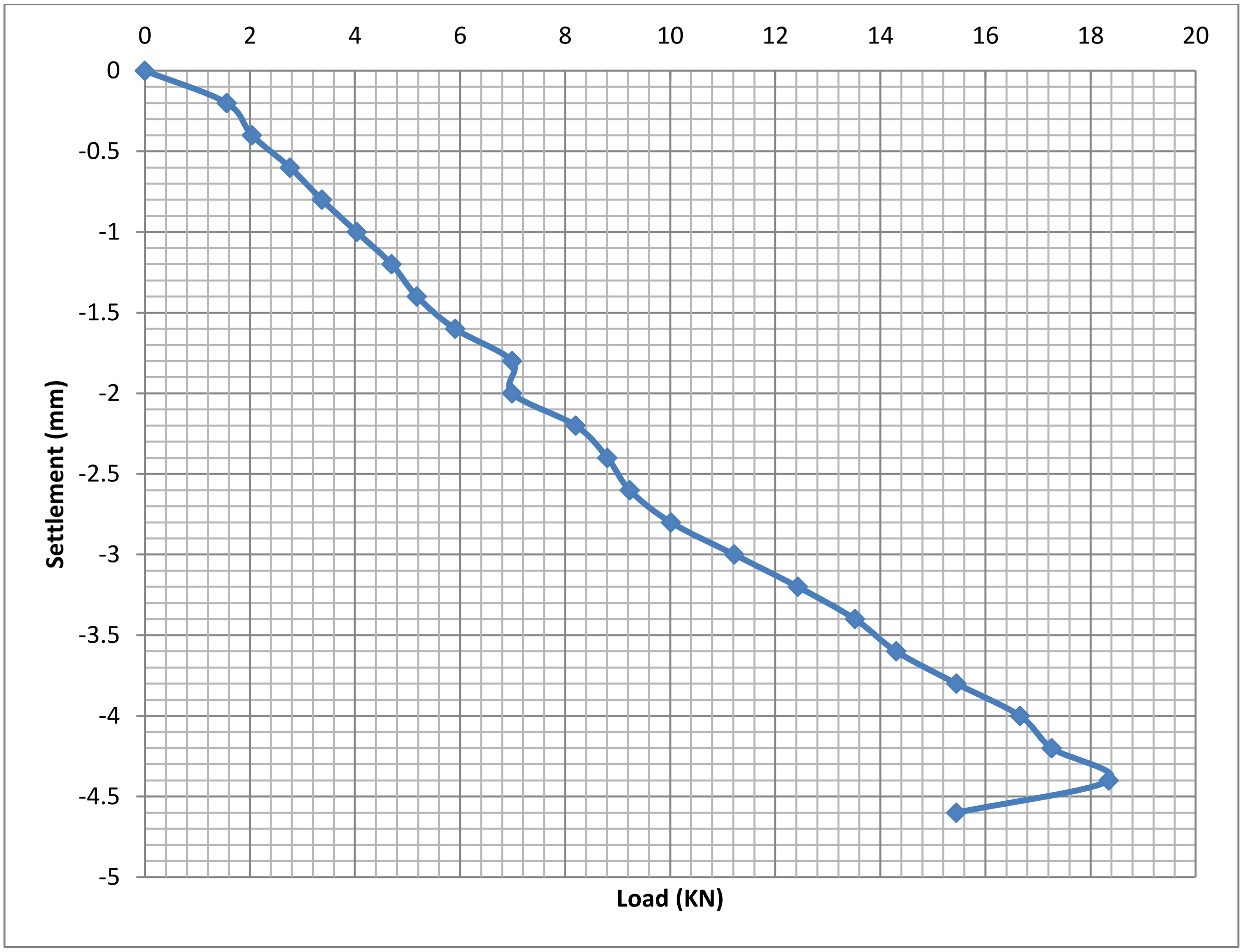


Fig-4.4: Load-settlement curve for sample with joint inclination 45°

From the curve, Bearing capacity of the foundation = **7.33 MPa**

4.1.5 Sample with joint orientation 60°: PoP blocks were casted and fitted after cutting in such a way, that a jointed rock mass sample was obtained with a joint set inclined at 60^0. The sample again fitted in the steel frame and tested under UTM. Load settlement values were recorded and load – settlement curve was plotted, from which Bearing capacity of the footing was calculated.

Table-4.5: Load-settlement values for joint inclination 60°

Load (KN)	Settlement (mm)
0	0
1.37	0.2
2.28	0.4
2.94	0.6
3.12	0.8
3.85	1
4.27	1.2
4.27	1.4
4.5	1.6
4.94	1.8
4.94	2
5.6	2.2
6.2	2.4
6.99	2.6
7.53	2.8
8.2	3
9.47	3.2
10.37	3.4
10.73	3.6
12.06	3.8
12.73	4
13.39	4.2
14.06	4.4
15.63	4.6
16.71	4.8
16.05	5
14.66	5.2

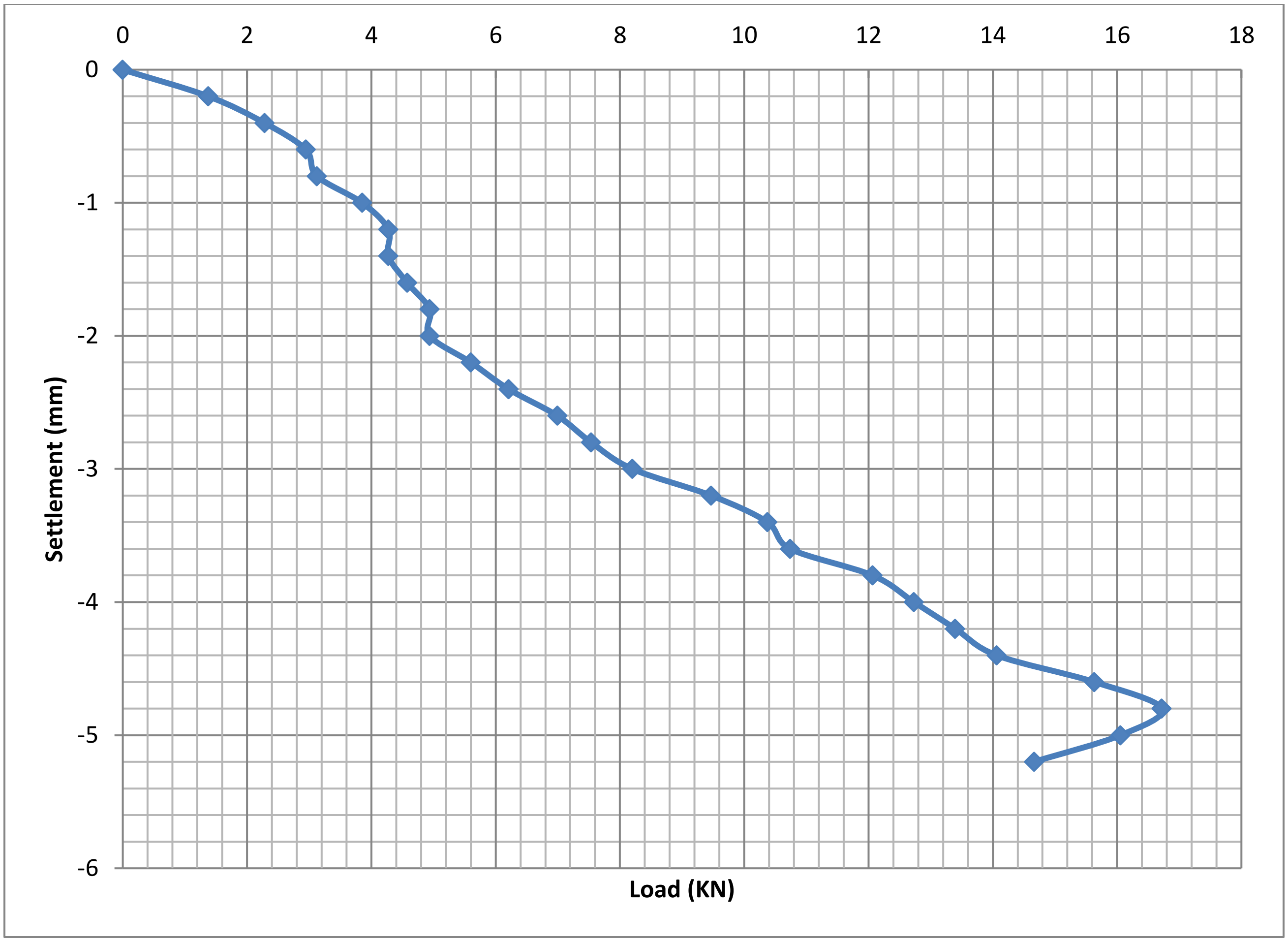


Fig-4.5: Load-settlement curve for sample with joint inclination 60°

From the curve, Bearing capacity of the foundation = **6.68 MPa**

4.1.6 Sample with joint orientation 75°: PoP blocks were casted and fitted after cutting in such a way, that a jointed rock mass sample was obtained with a joint set inclined at 75^0. The sample again fitted in the steel frame and tested under UTM. Load settlement values were recorded and load – settlement curve was plotted, from which Bearing capacity of the footing was calculated.

Table-4.6: Load-settlement values for joint inclination 75°

Load (KN)	Settlement (mm)
0	0
1.25	0.2
2.16	0.4
3.37	0.6
4.88	0.8
6.09	1
7.29	1.2
8.26	1.4
9.41	1.6
10.01	1.8
11.22	2
12.73	2.2
14.30	2.4
9.41	2.6
11.89	2.8

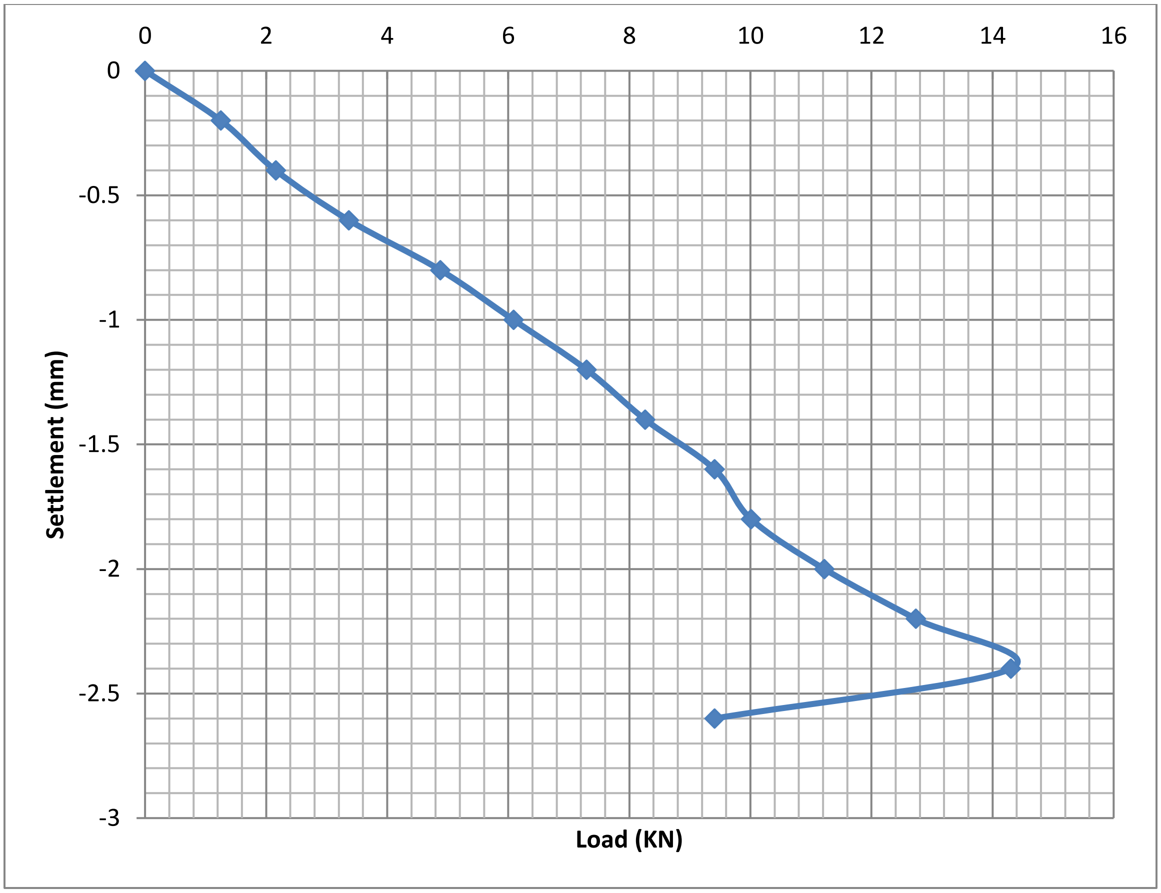


Fig-4.6: Load-settlement curve for sample with joint inclination 75°

From the curve, Bearing capacity of the foundation = **5.72 MPa**

4.1.7 Sample with joint orientation 90°: Four PoP slabs of dimensions 300 mm × 300 mm × 75 mm were casted and finished. They were placed horizontally besides each other to form a rock mass sample. The sample was fitted in the steel frame and tested in UTM. Load and settlement values were recorded which are given below.

Table-4.7: Load-settlement values for joint inclination 90°

Load (KN)	Settlement (mm)
0	0
0.77	0.2
1.19	0.4
2.34	0.6
3.85	0.8
5.6	1
7.05	1.2
7.84	1.4
8.92	1.6
9.59	1.8
10.25	2
10.49	2.2
10.91	2.4
11.34	2.6
11.88	2.8
12.48	3
12.55	3.2
12.67	3.4
13.15	3.6
12.24	3.8
12.42	4
12.91	4.2
12.97	4.4
13.09	4.6
12.67	4.8
13.33	5

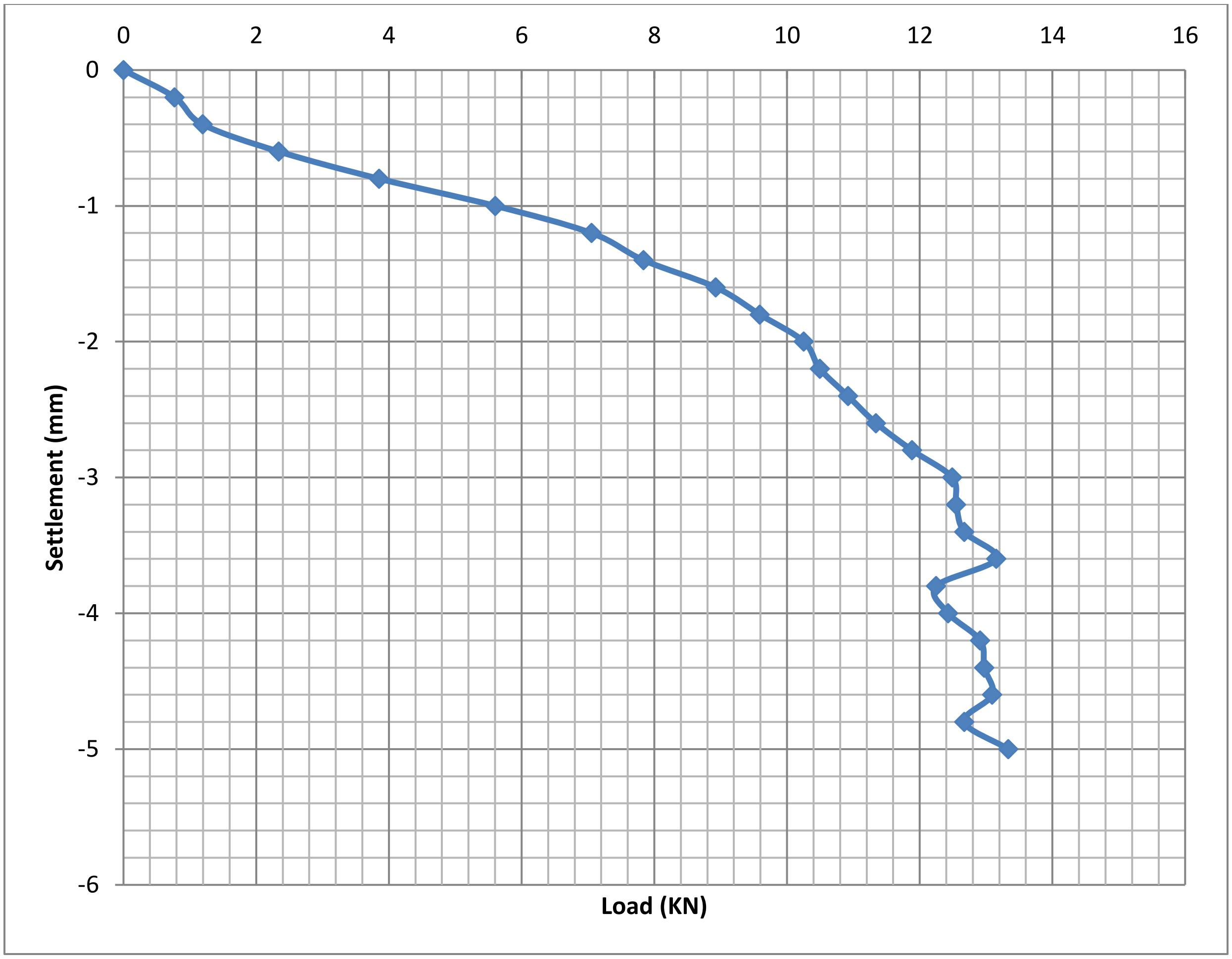


Fig-4.7: Load-settlement curve for sample with joint inclination 90°

From the observed values of load and settlement, a curve was drawn. Using the load – settlement curve, Bearing capacity of the foundation was observed.

From the curve, Bearing capacity of the foundation = **5.26 MPa**

4.2 Variation of Bearing capacity with joint orientation

The Bearing capacities calculated for all the samples with different joint inclinations are tabulated and plotted against the inclination of joints to see its variation. The variation of Bearing capacity with joint inclination is shown below.

Table-4.8: Variation of Bearing capacity with joint orientation

Orientation of joints w.r.t. horizontal	Bearing capacity
0	9.68
15	8.59
30	8.06
45	7.33
60	6.68
75	5.72
90	5.26

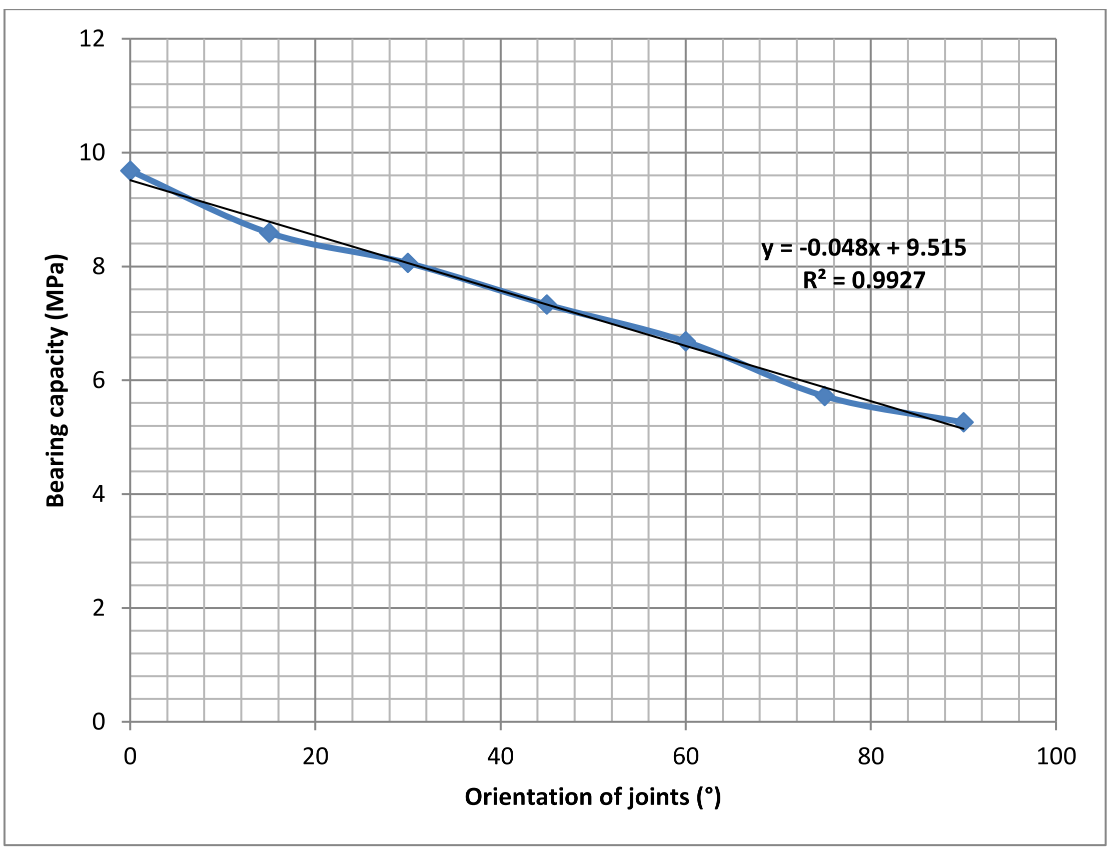


Fig-4.8: Variation of Bearing capacity with joint orientation

4.3 Comparison of results with previous studies

The results obtained in this study are compared with the results obtained by the empirical relation given by Bindlish et al.(2012).

$$\sigma_1 = \sigma_{cj} + (1- 2A_j \sigma_{ci}) \sigma_3 + A_j (\sigma_3)^3 \tag{10}$$

Where, $A_j = -1.23 (\sigma_{ci})^{-0.77}$

Their relation is valid for shallow foundation resting on non Hoek-Brown rock mass and can be applied to rock mass with even single joint set. Therefore their relation is suitable for the present study and hence adopted for comparison.

Their relation uses a modified joint inclination parameter for shallow foundation and ultimately the relation given by Singh and Rao (2005)

$$\sigma_{cj} = \sigma_{ci} \exp (a J_f) \tag{11}$$

where, *a* is an empirical coefficient depending on failure mode.

The process is based on Bell's approach to calculate the bearing capacity of foundation.

Table-4.9: Comparison of results with previous studies

Joint Orientation	**Bearing Capacity Values**	
	In this study	**According to Bindlish et al. (2012)**
0°	**9.68**	**8.22**
15°	**8.59**	**8.09**
30°	**8.06**	**7.92**
45°	**7.33**	**7.71**
60°	**6.68**	**7.4**
75°	**5.72**	**7.04**
90°	**5.26**	**6.89**

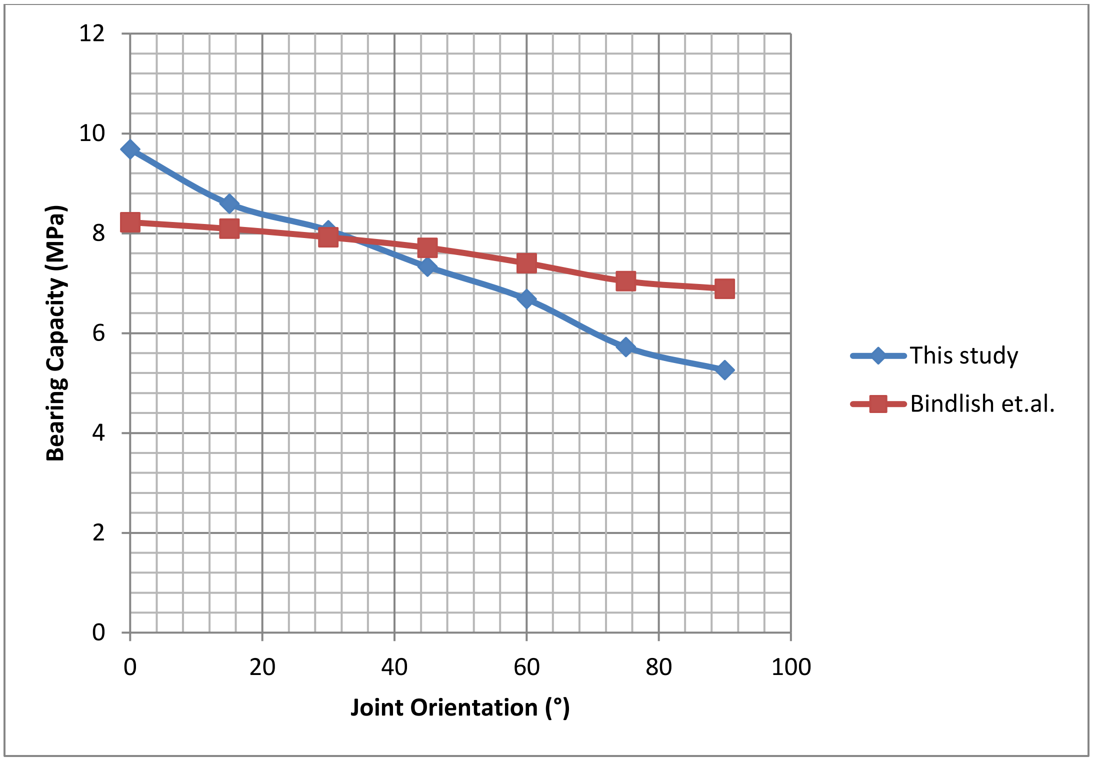


Fig-4.9: Comparison of results with previous studies

CHAPTER 5

CONCLUSION

On the basis of results obtained in this study, following conclusions can be drawn:

1. The failure of shallow foundations is more closely represented by laterally restrained conditions, as adopted in this study. On the other hand, unrestrained conditions simulate a slope failure condition better.

2. The anisotropy ratio obtained in the study is 1.84(Ratio of max. strength to min. strength), which is very low as compared to many UCS based experimental studies.

3. The most probable reason for the less variation in the bearing capacity of shallow footing may be due to modes of failure. In laterally restrained conditions, the modes of failure observed were only shearing and splitting as against all the four modes (shearing, splitting, rotation and sliding) in laterally unrestrained conditions.

4. The variation of bearing capacity of shallow foundation is found to decrease continuously with increase in dip of the continuous joints in a rock mass with single joint set. The decrease in Bearing Capacity is gradual.

5. The results obtained by empirical relation given by Bindlish, Singh and Samadhiya (2012), when applied to this model, also show the similar variation (continuous decrease). But the degree of variation is less than that obtained in this study.

6. For dip angle $< 30°$, the results obtained by previous study are little less than that of present results, while for dip angle $> 30°$, results by previous study are little more than that of present work.

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