The Major Project-II
On

STUDY OF SHEAR BEHAVIOUR OF SAND BLENDED WITH SILT.

Submitted in Partial Fulfillment for the Award of the Degree of

MASTERS OF TECHNOLOGY

IN

CIVIL ENGINERING

With Specialization In

GEOTECHNICAL ENGINEERING

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2015

DELHI TECHNOLOGICAL UNIVERSITY, DELHI

CERTIFICATE

This is to certify that the major project-II report entitled "**STUDY OF SHEAR BEHAVIOUR OF SAND BLENDED WITH SILT**" is a bona fide record of work carried out by **Arvind Kumar (Roll No. 2K13/GTE/04)** under my guidance and supervision, during the session 2015 in the partial fulfillment of the requirement for the degree of Master of Technology (Geotechnical Engineering) from Delhi Technological University, Delhi.

The work embodied in this major project has not been submitted for the award of any other degree to the best of our knowledge.

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ACKNOWLEDGEMENT

As I write this acknowledgement, I must clarify that this is not just a formal acknowledgement but also a sincere note of thanks and regard from my side. I feel a deep sense of gratitude and affection for those who were associated with the project and without whose co-operation and guidance this project could not have been conducted properly.

I express my deep gratitude to **Prof. Pradeep Kumar**, Hon'ble Vice Chancellor and **Prof. Nirendra Dev**, Head, Department of Civil Engineering, Delhi Technological University, Delhi for the facilities provided.

Words fail me to express my regards towards my project guide, **Prof. A. K. Sahu**, Civil Engineering Department, Delhi Technological University, Delhi for giving me an opportunity to work under his guidance, which really instilled in me the requisite confidence.

Last but not the least, I would like to thank my family and friends who stimulated me to bring this work to a successful close.

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DECLARATION

I hereby declare that the work being presented in this Project Report entitled "**STUDY OF SHEAR BEHAVIOUR OF SAND BLENDED WITH SILT**" is a bona fide record of work carried out by me as a part of major project in partial fulfillment of the requirement for the degree of Master of Technology in Geotechnical Engineering.

I have not submitted the matter presented in this report for the award of any degree.

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ABSTRACT

The structure derived from compacting the soil at different water contents and energy levels can have a substantial effect on its shear strength. While the shear strength can be estimated based on the saturated shear strength parameters and the unsaturated angle of shearing resistance, limited studies have explored the variation of shear strength properties with different compaction states. In this project report, the shear strength of a sandy soil was investigated using a conventional direct shear box and triaxial test (CU) with three different normal pressures. In this project, it was aimed to observe shear strength behavior of sand blended with various percentage of silt. Three series of experiments were performed. In all series, behavior of shear strength under different testing conditions was investigated against increasing fine materials in the mixtures. Silt is used as fine material. Shear strength parameters, failure strains, stress-strain behaviors were studied. The changes in basic characteristics such as particle size distributions, consistency limits and index properties were also studied.

In this investigation laboratory study on Sand (SP) blended with Silt (ML) has been carried out. Various test conducted on sand and silt were performed for the determination of following parameters: Field moisture content, Atterberg's Limits, Grain Size Analysis, Standard Proctor's Compaction test, Direct Shear and Triaxial Consolidated Undrained test on varying percentages of silt (by weight of 5%,10%,15%).

These primary conclusions were obtained from this investigation. With the addition of silt there was considerable decrease in the value of angle of internal resistance and small increase in cohesion in the soil. During the design of structure, most of the cases the plain strain problem is carried out. Therefore an attempt has been carried out to establish a relation between the angle of shearing resistance obtained from Direct Shear test and Triaxial test.

LIST OF TABLES

LIST OF FIGURES

LIST OF PHOTOGRAPHS

NOTATIONS

SYMBOL	DESCRIPTION
Cu	Coefficient of uniformity
Cc	Coefficient of curvature
D_{10}	Effective size
D_{60}	Grain diameter corresponding to 60% finer.
D_{30}	Grain diameter corresponding to 30% finer.
G	Specific gravity
DST	Direct shear test
CU	Consolidation undrained
CD	Consolidation drained
NP	Non Plastic
OCR	Over consolidation ratio
LL,PL,PI	Liquid limit, Plastic limit, Plasticity index
Φ	Angle of shearing resistance
$\Delta\sigma_3$	Confining stress
$\Delta\sigma_1$	Major principal stress
$\Delta\sigma_d$	Deviator stress
Δu	Pore pressure
P	$(\Delta\sigma1+ \Delta\sigma3)/2$
Q	$(\Delta\sigma1- \Delta\sigma3)/2$
SM,SP	Silty sand, Poorly graded sand

CONTENTS

CHAPTER 1- INTRODUCTION

1.1 INTRODUCTION

Mostly man-made earth structures involve the use of compacted soils. The compaction process enhances a soil with a degree of saturation usually in the range of 70 -90%. Embankments, earth fill dams and highways are the examples of earth structures made of compacted, unsaturated soils. The theory and measurement of shear strength of silty sand have gained increasing attention during the past three decades.

A brief review on the development of our understanding of the shear strength behavior of silty sand is presented in this thesis

This thesis presents the results of a series of direct shear tests and triaxial consolidated undrained test on silt blended with sand. Direct shear testing of silty sand is desirable since less time is required to fail the soil specimen than when using the triaxial test. The time to failure in the direct shear test is greatly reduced because the specimen is relatively thin. A multistage test procedure was used and description of the Direct Shear box and Triaxial testing procedure is presented.

The present project is composed of five Chapters. The theoretical background of the present study and a review of the literature about subject are given in Chapter 2. In Chapter 3, the experimental methods are explained by presenting the procedures of samples preparation and describing the testing program followed. In the 1st series of experiments, soil mixtures having were consolidated and tested under cell pressure of 100 kPa keeping back pressure to be 90 kPa in the consolidated undrained triaxial compression test for obtaining shear strength in terms of total stresses. In the 2nd series, soil mixtures were consolidated under 200 kPa cell pressure keeping pore pressure to be 90 kPa. In the 3rd series, soil mixtures were tested under 300 kPa cell pressure and they were repeated for different soil mix in consolidated undrained triaxial test in order to measure effective shear strength parameters. The test results obtained in each experimental series are analyzed and given in Chapter 4. In Chapter 5, the conclusions obtained from the present study are given.

The mechanical behavior of clean sands was investigated first analyzed in the 18th century. Studies of the mechanical behavior of pure clays were reported only approximately 150 years later. Studies of these soils continued over the years as clean sands and pure clays define distinct boundaries of a wide spectrum of natural. Most of the studies concerning the stress–strain and shear strength behavior of granular soils mainly inspected the response of clean sands. However, field observations show that granular soils may contain a considerable amount of clay or silt. Therefore, these fines are expected to be influence the engineering behavior of sandy soils.

In the guidelines of retaining structures, soils with fines are disqualified as backfill materials. For example, according to the AASHTO (American Association of State Highway and Transportation Officials) specifications, the content of fines used in a reinforced soil retaining wall must be less than 15%. However, geotechnical engineers usually face practical concerns, like the availability of good quality backfill materials and the construction costs in meeting these criteria.

1.2 OBJECTIVES OF STUDY

- To study the effect of varying percent of silt blended with sand on Gradation analysis and Atterberg's limit of the soil.
- To study the effect of varying percent of silt blended with sand on Compaction properties of the soil
- To study the effect of varying percent of silt blended with sand on Shear Strength Parameters of the soil.
- To study the effect of different varying percent of silt blended with sand on Pore pressure of the soil.
- To compare the results of Direct Shear Result with Consolidated Undrained Triaxial Results.

To fulfill these objectives, these Experiments have been carried out which is reflected in the following chapters.

CHAPTER 2- LITERATURE REVIEW

On these basis, aims and objectives the literature has been surveyed with respect to compaction and shear behavior of the soil which is reflected in the following paragraph.

The consolidated-drained direct shear tests on the artificial mixtures with increasing percentage of clay, including two types of sand (fine and coarse) were performed and found that maximum and limiting shear stresses showed a tendency to decrease as the clay content increases and described the existence of three zones of behavior of the mixtures as a function of clay content 1) Incoherent behavior where the cohesion is null and the angle of friction is high (above 30^0) and the effects of fluctuations in soil grain size variations are not significant. 2) Where the soil is sensitive to grain size fluctuations. 3) Coherent behavior where the cohesion is high and the angle of friction is low. [1]

When the behavior of clayey sands under monotonic and cyclic loading was tested, the finer content has a remarkable influence on the stress-strain response of the soil mass. As the finer content increases the dilatants behavior of the soils is decreased, and the response gradually becomes controlled by the fine matrix at 40 percent fines content. [2]

When stress- strain behavior of anisotropically consolidated clayey sands using triaxial test were performed ,the specimens were prepared by sedimenting Ham River sand into a kaolin suspension, they observed the effects of variations in clay content and initial granular void ratio and concluded that this method creates a material which is markedly less stable, which has a higher granular void ratio and exhibits a higher undrained brittleness behaviour, which is the engineering characteristic like ductile behavior and it is determined by stress history, rate of shearing and fabric of clays, if compared with the same sand that is sedimented through clean water (i.e. contains no clay).Moreover they showed that a sand that has 30 % clay fraction the normally consolidated material is no longer dilatant and exhibits the response that would be expected in a sedimented clay. They also stated that for clay fractions up to 20 %, the clay does not significantly reduce the angle of shearing resistance of the granular component. [3]

The influence of fines and gradation on the behavior of loosely prepared sand samples formed by moist tamping and consolidated to the same effective stress level were prepared. Samples were isotropically consolidated and subjected to monotonic undrained triaxial compression. They stated that undrained brittleness decreased as the fines content, for both plastic and nonplastic type, increased and concluded that undrained brittleness may not be controlled by the plasticity of the fines but more by the amount of fines, at least for percentages greater than 10 percent. [4]

The effects of the fine particles (diameter < 0.074 mm.) on the shear strength and compressibility properties of the soil mixtures were investigated. Soil mixtures having wide range of grain size from sand to silt-clay mixtures were studied. Drained shear box and consolidated- undrained triaxial tests were performed on normally consolidated clay-sand mixtures to obtain strength and compressibility parameters, on mixtures containing 5 %, 15 %, 35 %, 50 %, 75 %, and 100 % fines, the internal friction angles varied between 30-38 degrees until 50 % fines and a slight decrease existed in the friction angle with increasing fine content. At fine contents higher than 50%, the reduction in the friction angle was significant and decreased to about 10^0. According to the results of consolidated-undrained triaxial tests, on 35%, 50%, 75%, and 100% fines, there was no a current relation between undrained friction angle and percentage of fines and the measured angle of shearing resistances were in the same order of magnitude irrespective of percent fines. [5]

It has been noted that there is a distinct relationship between the types of behaviour and the relative density of the specimen.[6]

The effects of nonplastic fines on the shear strength of sand were studied. A series of laboratory tests was performed on samples of Ottawa sand with fines content in the range of 5-20 % by weight. They used triaxial tests that were conducted to axial strains in excess of 30 %. They used the concept of the skeleton void ratio, which is the void ratio of the silty sand calculated as if the fines were voids. They suggested that silty sand with non floating fabric in the 5-20 % silt content range is more dilatant than clean sands; dilatancy appears to peak around 5 % silt content, but even at 20 % silt content it remains above that of clean sand. [7]

When sand-clay mixtures were studied, it is observed that when clay content is just enough to fill the voids of the granular portion at its maximum porosity, the structure of the mixture changes and the linear relationship between the Atterberg's limits (plastic and liquid limits) and the clay content is no more valid and soil changed its behavior from sand to clay. For mixture including kaolin clay at its liquid limit, they showed out that this threshold value exist about 25 % kaolin content. [8]

The behavior of a model soil formed from Ham River sand and kaolin was observed.the model soil was selected in order to display relatively closer response of the soil at the field. These reconstituted specimens have been subjected undrained shear in the triaxial compression test under displacement control and concluded that undrained brittleness in compression increases as the clay content increases from 4.5 % to 11.5 %, but reduces as the OCR increases. They also showed that the clayey sand reaches its peak resistance at small axial strains: ε_a in compression increases from 0.1 % to 0.3 % as OCR increases from 1 to 2. [9]

When large strain undrained shear strength in triaxial compression for a particular host sand mixed with different amounts of nonplastic fines were studied, the results indicate that the inter granular void ratio, which is the void of the sand-grain-matrix plays an important role on undrained shear strength of silty sands. [10]

Undrained state is considered as critical state in soil mechanics because it increases the excess pore water pressure which decreases the effective stress in the soil and might reduces the shear strength of saturated cohesion less soils.[11]

Reconstitution technique of water pluviation leads to an inherently cross anisotropic structure using triaxial tests. In cohesion less soils, the spatial arrangements of solid particles progressively change during deformation. This change in soil may gradually increase the degree of anisotropy; such phenomenon has been called induced anisotropy [12]

When the anisotropy and the effects of principal stress rotation in medium-loose sand were studied under undrained conditions using a hollow cylinder apparatus, it is observed that principal stresses have been rotated at a constant shear stress during both monotonic and cyclic loading. Pore pressures are shown to be generated by rotation of principal stress directions at constant shear stress and their accumulation during cyclic principal stress rotation can lead to failure. [13]

CHAPTER 3 - MATERIALS & METHODS

3.1 AIM OF THE INVESTIGATION

The current investigation aims to study the "Study of Shear Behavior of Sand blended with Silt".

3.2 MATERIALS USED FOR THE INVESTIGATION

SAND: It is a naturally occurring material composed of finely divided rock and mineral particles. It is defined by size i.e. soil which passes through 4.75 mm sieve and retained on 75 micron sieve is called sand.

In this project work pure sand is i.e. firstly soil is washed and wetted soil is passed through 4.75 mm sieve and retained on 75 micron sieve, then soil is oven dried, is procured from Badarpur, Delhi.

SILT: It is a granular material of a size varies between sand and clay i.e. the soil which passed through 75 micron sieve and coarser then 2 micron size called silt, is taken in this project work as finer material.

MIXES: Silt with varying percentage blended with pure sand is taken and designated as follows:-

SM0: Pure sand/Clean sand

SM5: Pure sand with 5 percent silt (i.e. between 0 to 5%)

SM10: Pure sand with 10 percent silt (i.e. between 5 to 12%)

SM15: Pure sand with 15 percent silt (i.e. greater than 12%)

3.3 METHODOLOGY FOR THE INVESTIGATION

In the current investigation, after finding out the index properties of the soil, following tests have been performed on the soil.

- Direct shear Test
- Triaxial Consolidated Undrained Test

3.4 TESTING PROGRAM

3.4.1SPECIFIC GRAVITY (*G*)

Specific gravity of soil solids is the ratio of weight, in air of a given volume of dry soil solids to the weight of equal volume of water at 4ºC defined as per **IS-2720-PART-3-1980**. Specific gravity of soil grains is used in the calculation of void ratio, porosity and degree of saturation, by knowing the moisture content and density. Its value helps in identifying and classifying the soil type.

3.4.3 LIQUID LIMIT

The liquid limit was determined with the help of standard liquid limit apparatus defined as per **IS: 2720-PART-5–1985**. About 120g of the soil passes through 425µ sieve was taken and groove was made by groove tool which is designates by. One brass cup was raised and allowed to fall on the rubber base and then water content correspond to 25 blows was taken as the liquid limit.

3.4.4 PLASTIC LIMIT

This test is performed to determine the plastic limit of soil defined as per **IS: 2720-PART-5–1985**. The plastic limit of fine-grained soil is the water content below which soil ceases to be plastic. Its crumble when rolled into the threads of 3mm dia.

3.4.6 PROCTOR's COMPACTION TEST

This laboratory test is performed to determine the relationship between the moisture content and the dry density of a soil at a specified compactive effort defined as per **IS-2720-PART-7-1980**.

3.4.7 DIRECT SHEAR TEST

General

- This test is carried out on soil to determine the shear parameters of soil.
- A standard size (60mm*60mm) Direct Shear box was used for the investigation.
- The tests were conducted on three different normal stresses i.e. 50, 100 & 150 kPa and the angle of internal friction & cohesion values were obtained by plotting a straight line through the plot of shear stress versus the normal stress.

- Direct Shear tests were performed strictly according to **IS 2720: part 13 (1986).**

Analysis:

- Calculate the density of the soil sample from the mass of soil and volume of the shear box.
- Convert the dial readings to the appropriate length and load units and enter the values on the data sheet in the correct locations. Compute the sample area A, and the vertical (Normal) stress
- Plot the Normal stress (kPa) versus horizontal (lateral) displacement (mm).

3.4.8 TRIAXIAL TEST

General

- In this investigation Consolidated-Undrained Triaxial tests were conducted in order to determine the compressive and shear strength of the soil.
- While conducting the tests the valves were closed during shearing the stages, in order to prevent the dissipation of pore water.
- The specimen of aspect ratio 2 was used i.e. diameter of 38mm and a length of 76mm.
- The tests were conducted on three different cell pressures (σ_3) i.e. 200, 300 & 400 kPa, thus three different deviatric stress (σ_d) were obtained.
- By using the Minor Principal Stress as σ_3 and Major Principal Stress as ($\sigma_1 = \sigma_3 + \sigma_d$), Mohr circle is drawn.
- A tangent is drawn on the above Mohr Circle to obtain the Mohr failure envelope. The angle of internal friction and cohesion intercept values can be recorded from the failure envelope itself.
- Consolidation undrained tests were performed strictly according to **IS 2720: part 12 (1981).**
- Effect of varying percentage of silt blended with sand has been observed on the change in values of angle of internal friction and cohesion

CONSOLIDATED UNDRAINED TRIAXIAL TEST

The consolidated undrained (CU) test is the most common triaxial test to know strength parameter based on the effective stresses (i.e. ϕ' and c') while permitting a faster rate of shearing compared with the CD test. This is achieved by recording the excess pore pressure change within the specimen as shearing takes place.

Specimen & System Preparation

The test specimen firstly prepared from a sample of soil before placing into the triaxial cell. For cohesive soils it may involve trimming undisturbed specimens pushed out from Shelby tubes or cut from block samples.

Following steps are done.

I. Saturation

II. Consolidation

III. Shearing

Saturation

In saturation process, all the voids within the test specimen were filled with water and the pore pressure transducer and drainage lines were properly de-aired. This may be achieved by firstly applying a partial vacuum to the specimen to remove air and draw water into the transducer and drainage lines, followed by a linear increase of the cell and back pressures. To check the specimen in reaching full saturation stage, the following steps may be taken:

- Use of de-aired water to fill specimen voids
- Increase of back pressure to force air into solution

To check the degree of specimen, saturation is sufficiently high before moving to the consolidation stage, a short test was performed to determine Skempton's B-value. This is called a B-check, requires specimen drainage to be closed while the cell pressure is raised by approximately 50kPa, with $B \geq 0.95$ typically used to confirm full specimen saturation

Consolidation

The consolidation stage was used to bring the sample to the effective stress state for shearing. It was conducted by increasing the cell pressure by maintaining a constant back pressure (often equal to the pore pressure reached during the final saturation B-check).This process was continued until the volume change ΔV of the specimen was no longer significant, and at least 95% of the excess pore pressure was dissipated. The consolidation response can also be used to estimate a suitable rate of strain when shearing of cohesive specimens.

Shearing

The soil was sheared by applying an axial strain to the test sample at a constant rate through upward or downward movement of the load frame platen. This rate, along with the specimen drainage condition, was dependent on the type of triaxial test being performed and specimen response during the shear stage was typically monitored by plotting the deviator stress or effective principal stress ratio $\sigma'1/\sigma'3$ against the axial strain. The stage was continued until a specified failure criterion has been reached, which may include identification of the peak deviator stress , observation of constant stress and excess pore pressure / volume change values, or simply a specific value of axial strain being reached (for example *axial strain* = 20%).

Analysis:

- Plot the Deviatric Stress (kPa) versus axial strain (%).
- Plot the Pore Pressure (kPa) versus axial strain (%).
- Plot the Normal Stress (kPa) versus axial strain (%).
- Plot the p-q curve.
- Plot the Mohr's circle and get the value of cohesion and angle of friction.

(a) (b)

Photograph 1: (a) Direct shear box, (b) Failed sample

(a) (b) (c)

Photograph 2: (a) Sample extractor, (b) Sample sizer & (c) Rubber membrane on soil sample.

(a)

(b)

Photograph 3: (a) Sample during saturation stage, (b) Sample during shearing stage .

Photograph 4: Bulged sample after shearing

CHAPTER -4 RESULTS & DISCUSSION

4.0. INTRODUCTION

The effect of silt intrusion on the shear strength of the sand under consolidated undrained condition is observed. The results in this regard are discussed in the following section.

4.1. Physical properties and Classification of Sand Blended with Silt.

4.1.1 Specific Gravity

The results obtained from the Pycnometer test of soil mix are given below. It has been observed that with increase in silt percentage, specific gravity of soil mix reduces. These changes occur due to increase of fines percentage in sand, due to shape & size of silt particle.

Table 4.1: Specific gravity of soil mix

Sample	Specific gravity
Clean sand	2.692
Clean sand + 5% silt	2.675
Clean sand + 10% silt	2.667
Clean sand +15%silt	2.658
Silt	2.612

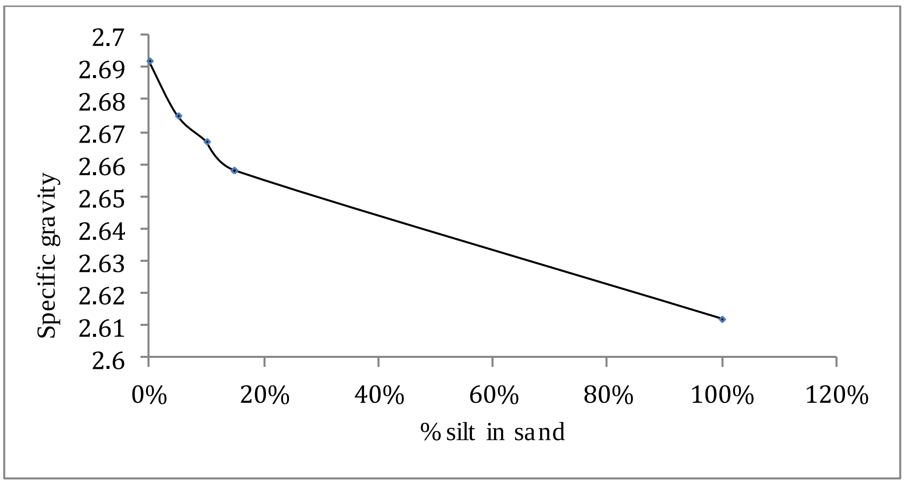

Figure 4.1:- Variation of Specific gravity of soil mix.

4.1.2. Gradation Analysis

Sieve analysis of soil was performed and observed that the soil is Poorly Graded Sand. The effective size (D_{10}),the mean grain size(D_{50}), coefficient of uniformity (Cu), and coefficient of curvature(Cc), for sand were 0.19 mm, 0.50 mm, 2.9, 1.007, respectively. Table 4.2 shows the sieve analysis of clean sand. Clean sand particles are round to angular.

Total weight of soil sample taken was 1000 grams.

Table 4.2: Grain Size Analysis

Sieve size (mm)	**% RETAINED**				**% CUMMULATIVE RETAINED**				**% PASSING**			
	SM0	SM5	SM10	SM15	SMO	SM5	SM10	SM15	SM O	SM5	SM10	SM15
4.75	1.5	4.55	3.85	2.91	1.5	4.55	3.85	2.91	94.7	95.4	96.1	97.0
2.36	2.0	2.31	1.62	0.68	3.5	6.86	5.47	3.59	91.8	93.1	94.5	96.4
1.18	3.1	7.95	7.26	6.32	6.6	14.81	12.73	9.91	83.1	85.1	87.2	90.0
.60	2.9	25.10	24.41	23.47	9.5	39.91	37.14	33.38	57.4	60.0	62.8	66.6
.30	4.2	22.10	21.41	20.47	13.7	62.12	58.55	53.85	34.6	37.8	41.4	46.1
.15	3.7	20.00	19.31	18.37	17.4	82.12	77.86	72.22	13.9	17.8	22.1	27.7
.075	20	11.47	10.78	9.84	32.4	93.19	88.64	82.06	1.8	6.8	11.3	17.9

PERCENTAGE OF SOIL PASSING 4.75MM SIEVE: 94.78% (> 50%)

Classification as per Indian standard code (IS 2720: 1985 part 4):
Sand with very less amount of fine (S.P.) POORLY GRADED SAND

Classification

Table 4.3: Grain Size Analysis classification

Sample Description	**% finer**	**Classification**
Clean sand	1.8	SP
Clean Sand + 5% Silt	6.81	SP-SM
Clean Sand +10%Silt	11.36	SP-SM
Clean Sand +15%Silt	17.94	SM

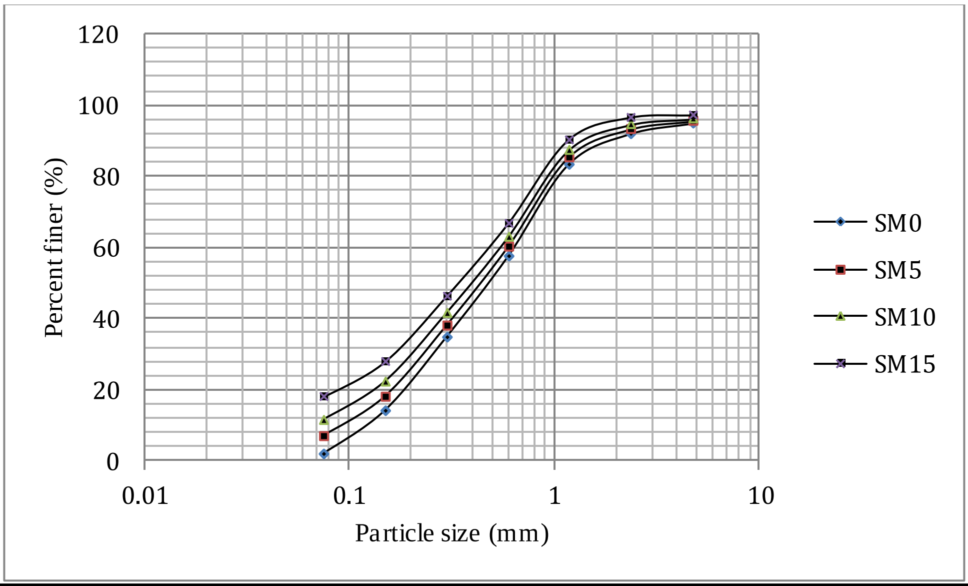

Figure 4.2:- Grain size curve of soil

4.1.3. Consistency Limits

For finding out the consistency limits of soil, tests were performed and observed that with increase in silt percentage, liquid limit of soil increase. This was happening because when the silt added to sand, amount of finer particle tends to increase in the soil which overall amplify the overall electro chemical nature of soil, water is easily bounded with the surface of the soil particle and thus there is modification in the consistency limit

Table 4.4: Consistency Limits of soil mix.

S. No.	Sample Description	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)
1	Clean sand	NP	NP	NP
2	Clean Sand + 5% Silt	NP	NP	NP
3	Clean Sand +10%Silt	11	NP	NP
4	Clean Sand +15%Silt	16	NP	NP
5	Silt	22.2	10.0	12.2

4.1.4. Classification

After doing all the tests, we get various parameters like Moisture content, Specific Gravity, MDD & OMC. Thus soil is classified as Poorly Graded Sand (SP).

Table 4.5.: Results of test on soil

Property of the Soil	**Values**
Moisture Content (%)	5.6
Specific Gravity	2.67
Liquid Limit (%)	NP
Plastic Limit (%)	NP
Plasticity Index (%)	NP
I.S. Classification (%)	Poorly Graded Sand (SP)
Maximum Dry Density(KN/m^3)	20.2
Optimum Moisture Content (%)	9.85

4.2. Compaction Characteristics of sand Blended with Silt.

Standard Proctor's test was done on sand with varying silt percentage and observation is given below.

Table 4.6.: Maximum Dry Density & Optimum Moisture Content

MOISTURE ADDED (%)	WEIGHT OF SOIL (kg)	NET MOISTURE (%)	BULK DENSITY (kN/m^3)	DRY DENSITY (kN/m^3)
3	2.02	3.2	19.21	18.62
2	2.08	5.1	20.18	19.22
2	2.14	7.0	20.87	19.50
2	2.21	9.0	21.56	19.79
2	2.17	10.7	21.26	19.20

Maximum Dry Density = **19.81 kN/m^3** Optimum Moisture Content = **9.85 %**

Table 4.6.: Compaction characteristics of soil mix

S. No.	Sample Description	MDD (kN/m^3)	OMC (%)
1	Clean sand	19.81	9.85
2	Clean Sand + 5% Silt	19.41	10.00
3	Clean Sand +10%Silt	19.23	10.25
4	Clean Sand +15%Silt	19.11	10.60

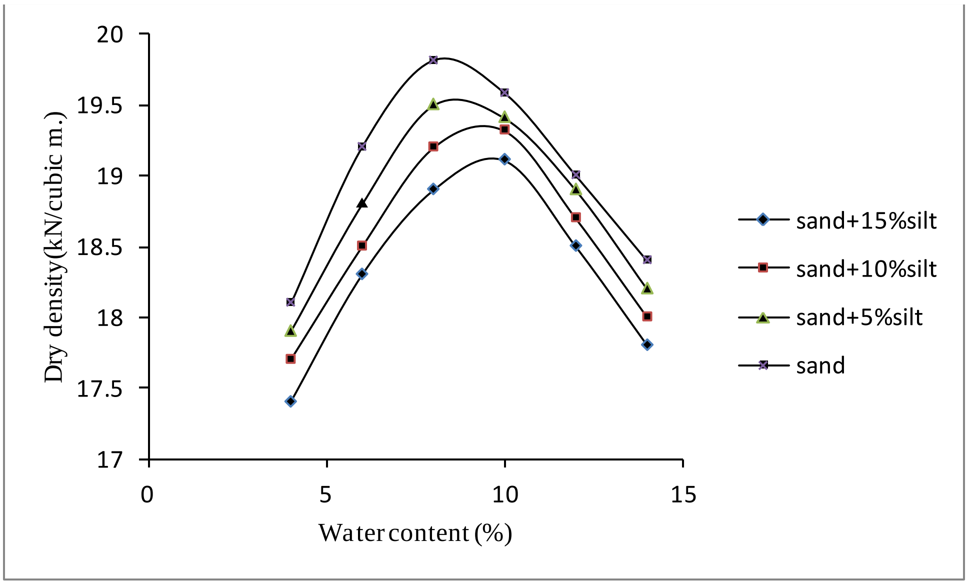

Figure 4.3:- Compaction curve of soil mix.

4.3. Shear Characteristics of Sand blended with Silt.

4.3.1. Direct Shear Test Results

The results obtained from direct shear test are given in following tables and figures.

Table No. 4.8.**:** Shear stress with Displacement readings of SM0

H.displacement(mm)	**50kN/m^2**	**100kN/m^2**	**150kN/m^2**
0	0	0	0
0.2	20	51	67
0.4	47	111	110
0.6	65	138	160
0.8	89	169	191
1.0	101	187	212
1.2	115	201	234
1.4	125	211	250
1.6	135	226	267
1.8	143	236	282
2.0	152	246	297
2.2	157	256	311
2.4	164	262	324
3.0	180	285	358
3.2	182	292	363
3.6	186	301	376
4.0	189	308	385

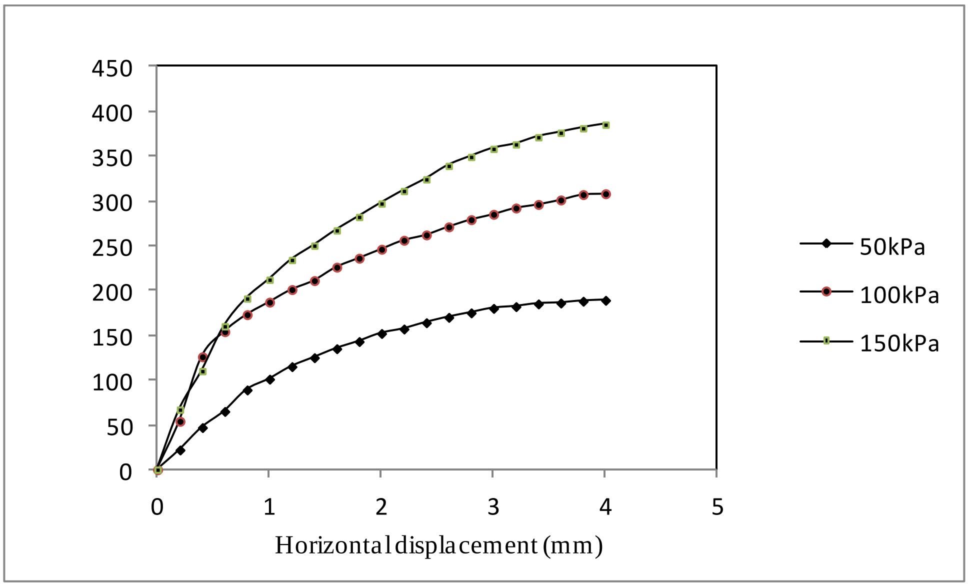

Figure 4.4:- Shear stress vs. displacement curve of clean sand

Table No. 4.9.: Shear stress with displacement readings of SM5

H.displacement(mm)	50kN/m^2	100kN/m^2	150kN/m^2
0	0	0	0
0.2	26	54	60
0.4	35	85	120
0.6	43	90	140
0.8	50	101	160
1.0	60	109	178
1.2	70	114	198
1.4	80	129	208
1.8	90	134	230
2.0	100	150	249
2.2	110	154	256
2.4	120	159	262
2.8	130	165	279
3.0	140	178	291
3.2	145	180	295
3.4	148	202	304
3.6	150	206	308
4.0	153	212	312

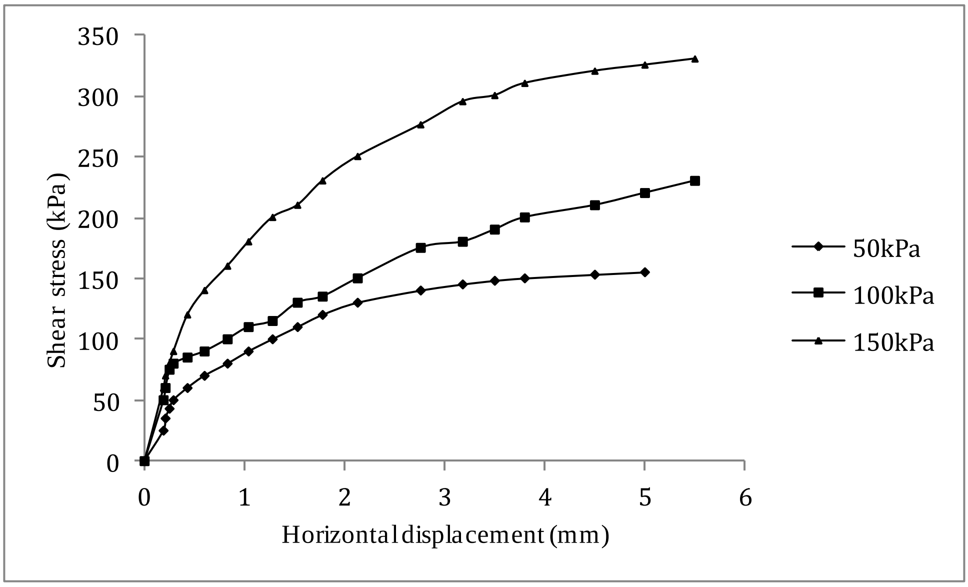

Figure 4.5:- Shear stress vs. displacement curve of clean sand+5%silt.

Table No. 4.10.: Shear stress with displacement readings of SM10

H.displacement(mm)	50kN/m^2	100kN/m^2	150kN/m^2
0	0	0	0
0.2	18	49	54
0.4	39	68	98
0.6	58	79	128
0.8	73	92	148
1.0	88	106	169
1.2	101	121	187
1.4	113	137	203
1.6	123	154	215
2.0	136	182	238
2.2	140	193	250
2.4	143	206	258
2.6	144	214	268
2.8	144	224	278
3.0	145	231	290
3.2		238	299
3.4		244	305
3.6		248	313
4.0		253	319

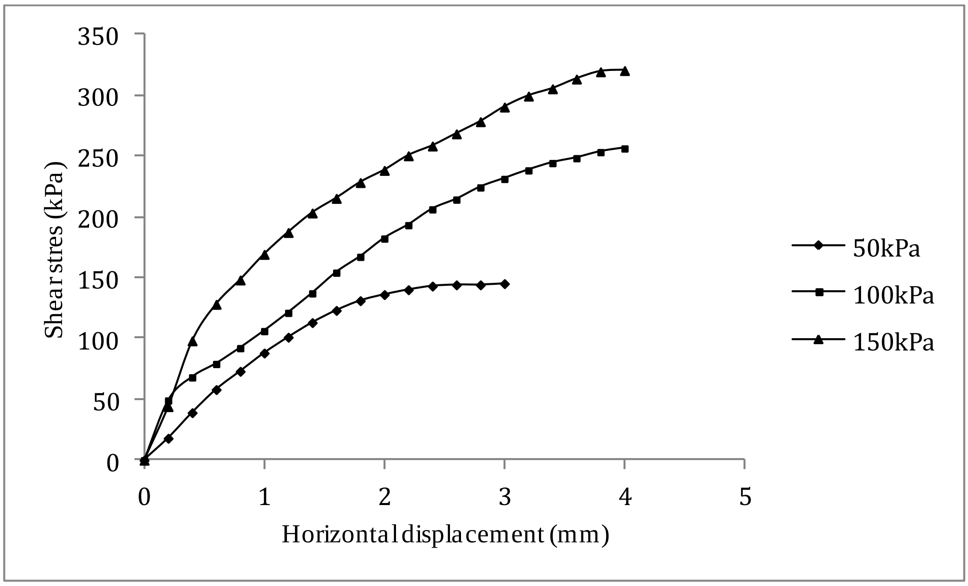

Figure 4.6:- Shear stress vs. displacement curve of clean sand+10%silt

Table No. 4.11.: Shear Stress with displacement readings of SM15

H.displacement(mm)	50kN/m^2	100kN/m^2	150kN/m^2
0	0	0	0
0.2	16	43	48
0.4	38	65	94
0.6	56	83	125
0.8	71	97	145
1.0	86	111	172
1.2	97	123	184
1.4	113	138	201
1.6	120	155	214
1.8	129	164	223
2.0	132	179	234
2.2	136	191	248
2.6	141	211	267
2.8	142	221	277
3.0	143	228	286
3.2	143	235	294
3.4		241	301
3.6		245	309
4.0		247	310

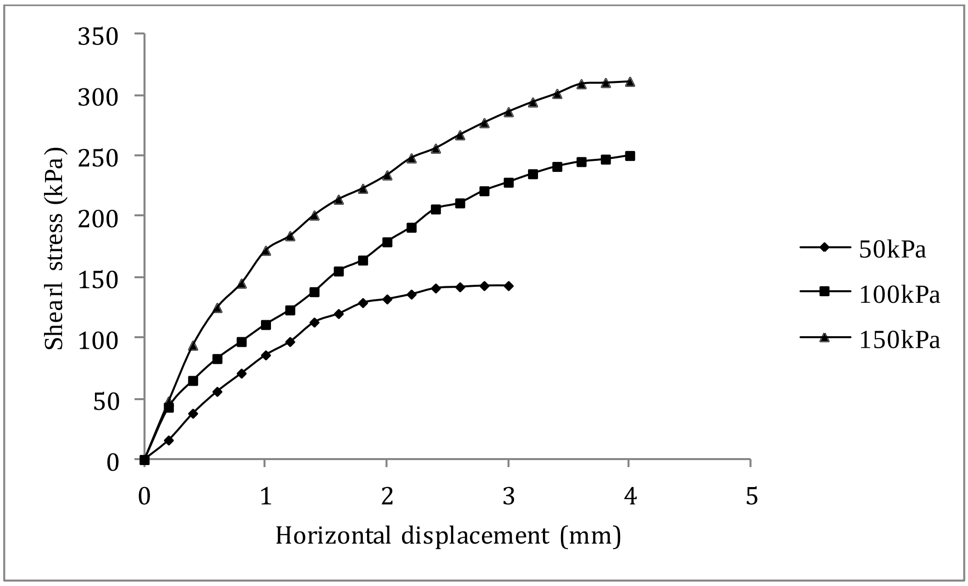

Figure 4.7:- Shear stress vs. displacement curve of clean sand+15% silt.

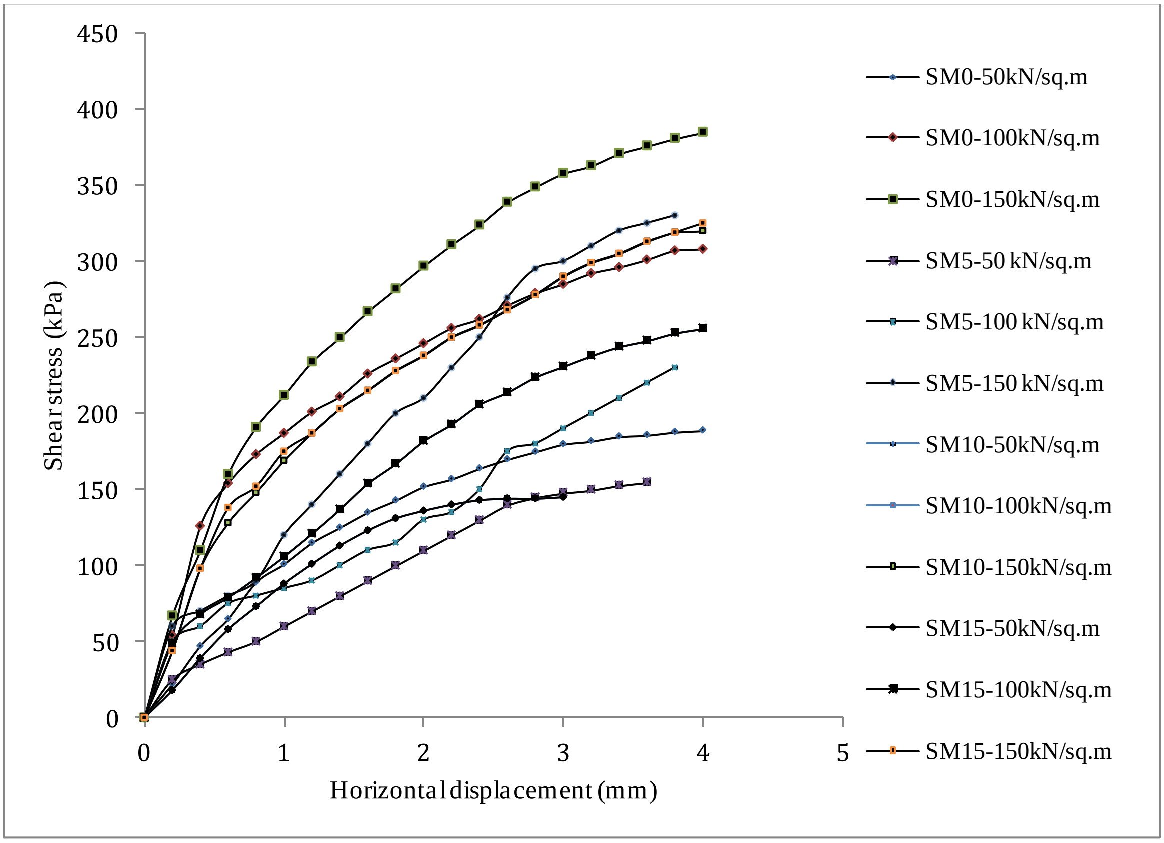

Figure 4.8:- Shear stress versus H.displacement plot of soil mix.

Table 4.12: Shear parameters of soil mix from direct shear test

S. No.	Sample Description	Angle of internal resistance	Cohesion (kN/m^2)
1	Clean sand	39^0	--
2	Clean Sand + 5% Silt	37.5^0	32.34
3	Clean Sand +10% Silt	36.8^0	56.84
4	Clean Sand +15% Silt	35.6^0	73.5

4.3.2. Triaxial Consolidated undrained test.

The results obtained from Triaxial (CU) test are given in following tables and figures.

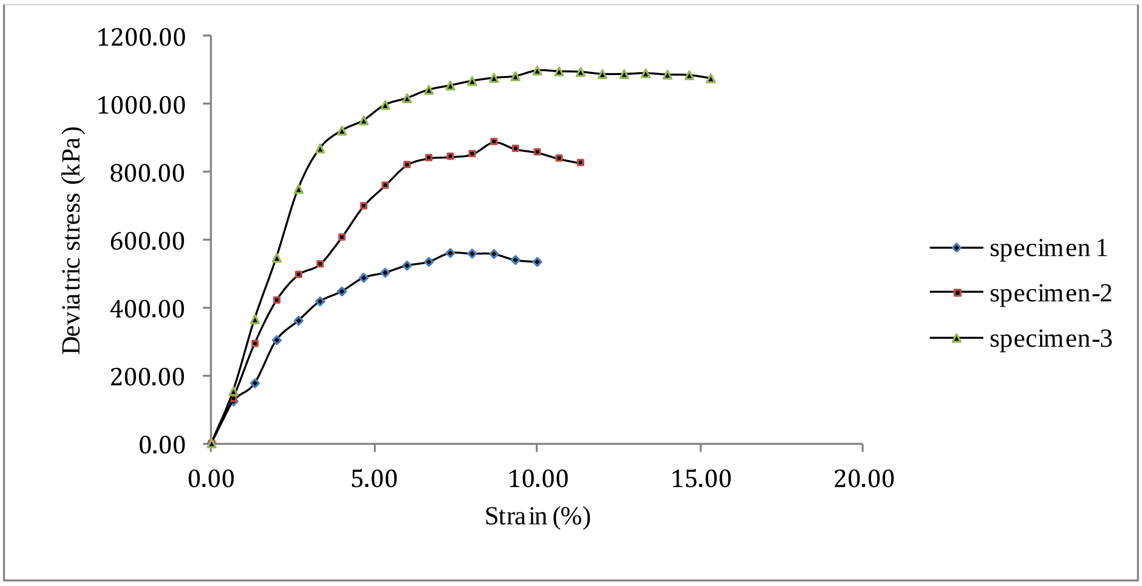

Figure 4.9:- Variation of deviatric stress vs. axial strain of clean sand

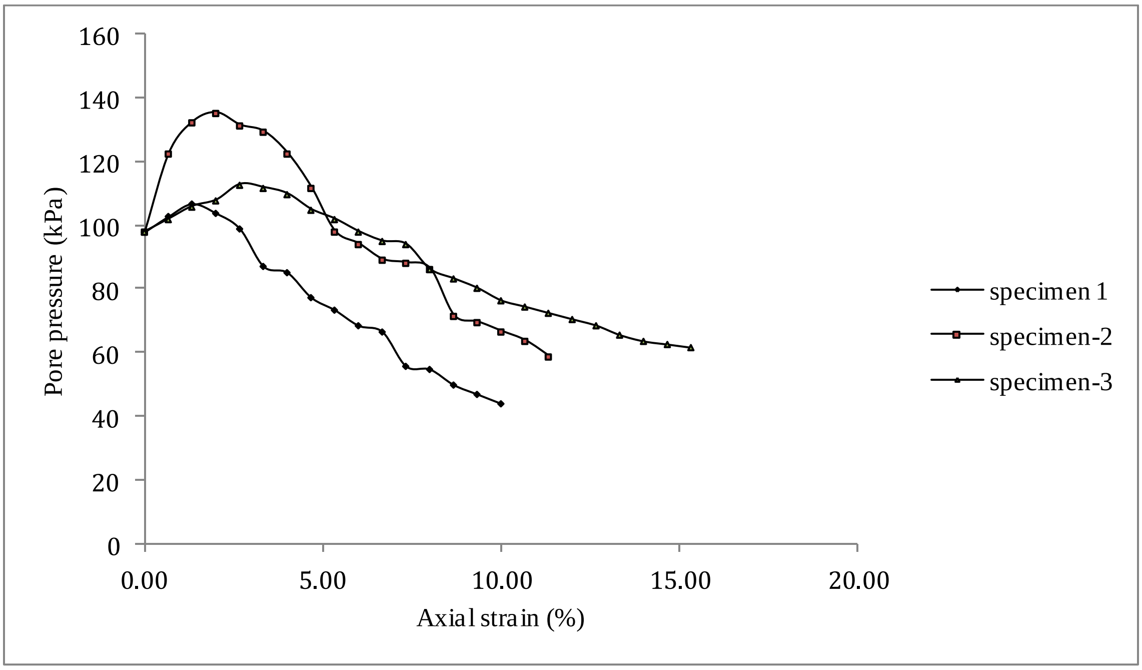

Figure 4.10:- Variation of Pore pressure vs. axial strain of clean sand

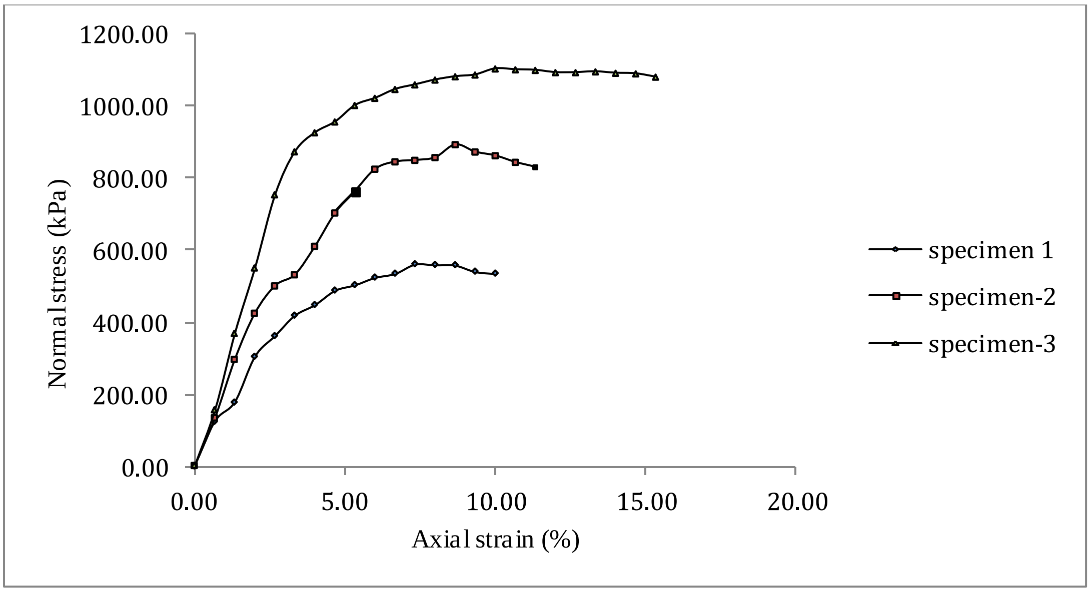

Figure 4.11:- Variation of normal stress vs. axial strain of clean sand

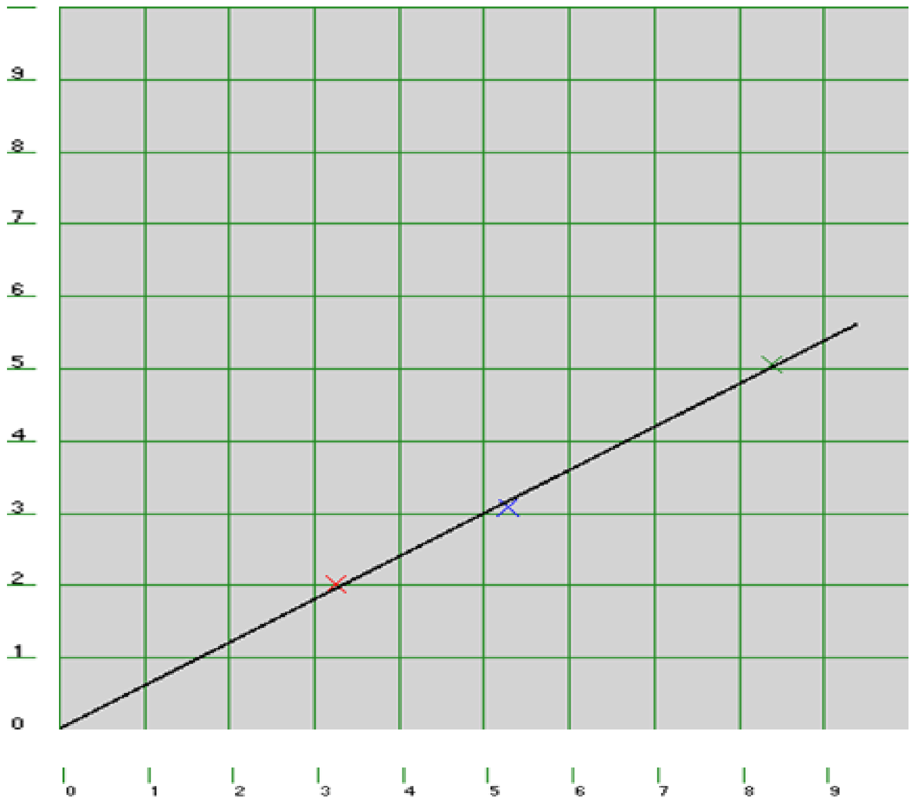

Figure 4.12:- p-q curve of clean sand (obtained from digi triaxial machine)

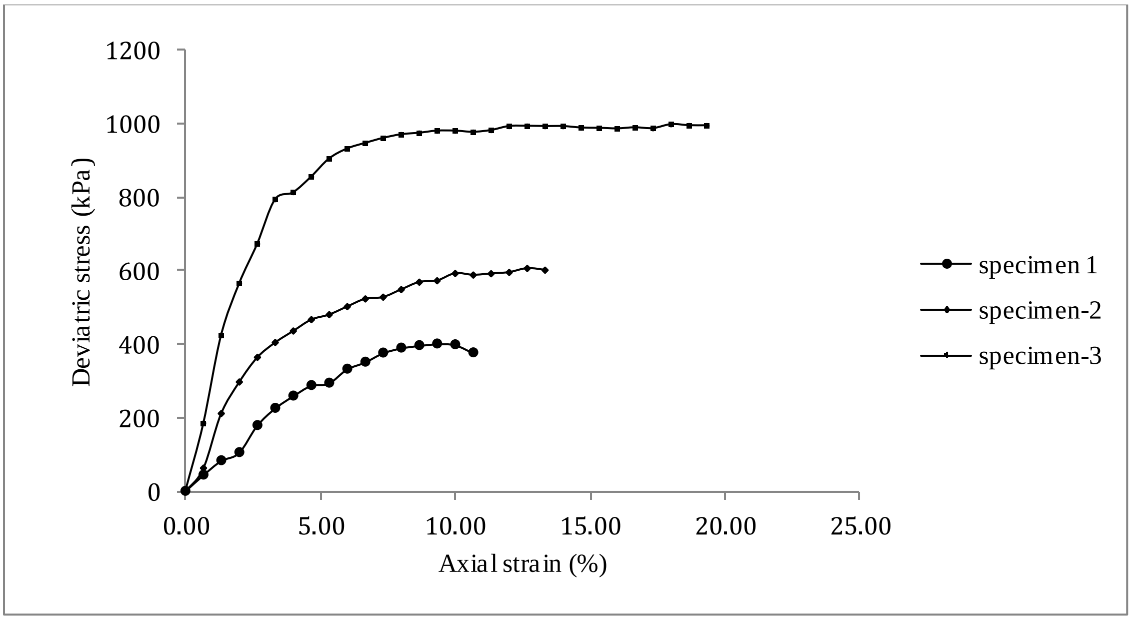

Figure 4.13:- Variation of deviatric stress vs. axial strain of clean sand+5%+silt

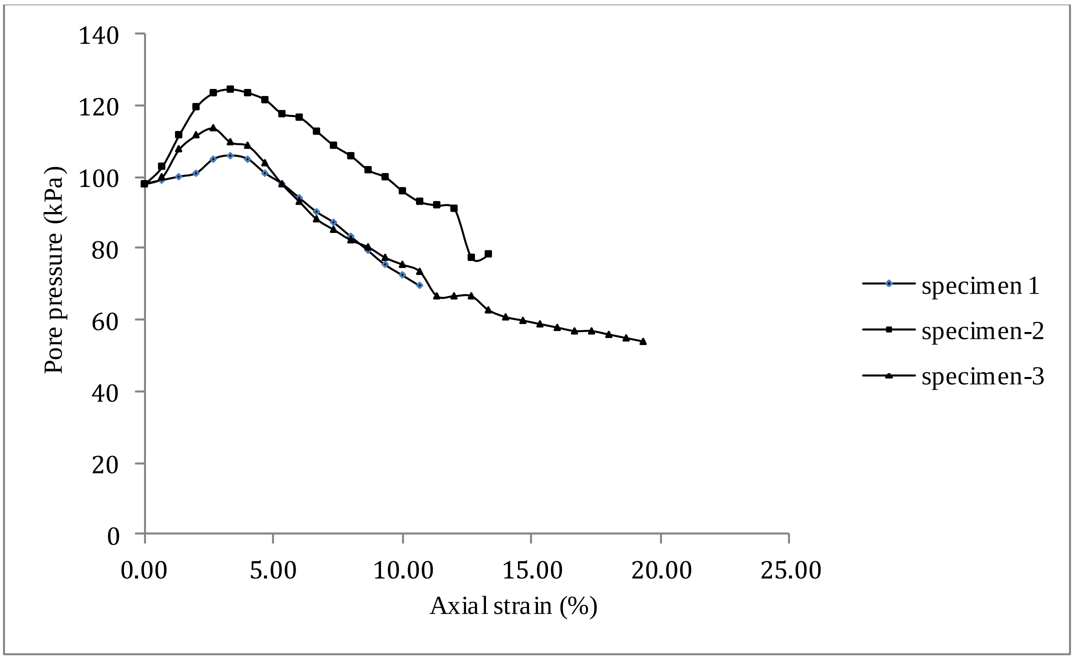

Figure 4.14:- Variation of pore pressure vs. axial strain of clean sand+5%silt

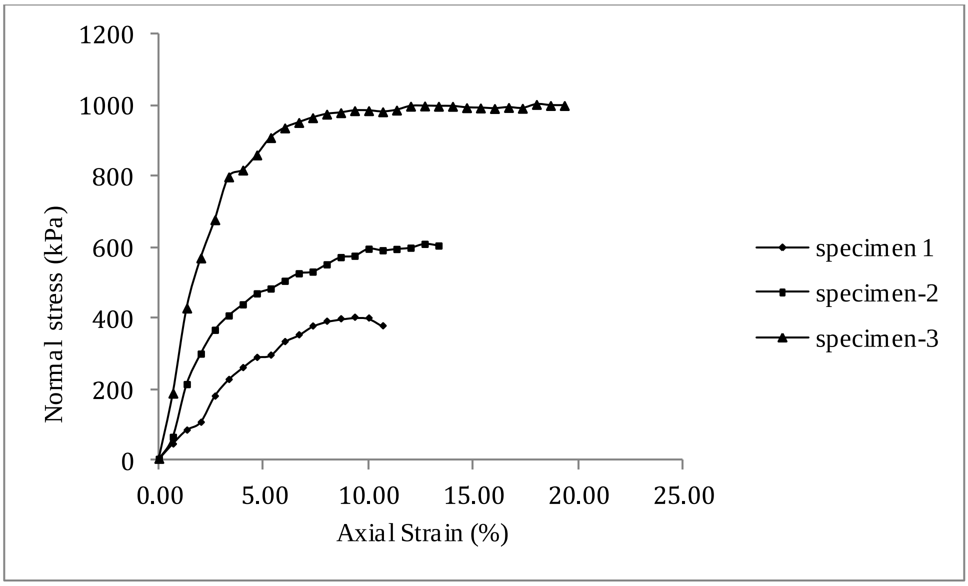

Figure 4.15:- Variation of normal stress vs. axial strain of clean sand+5%silt

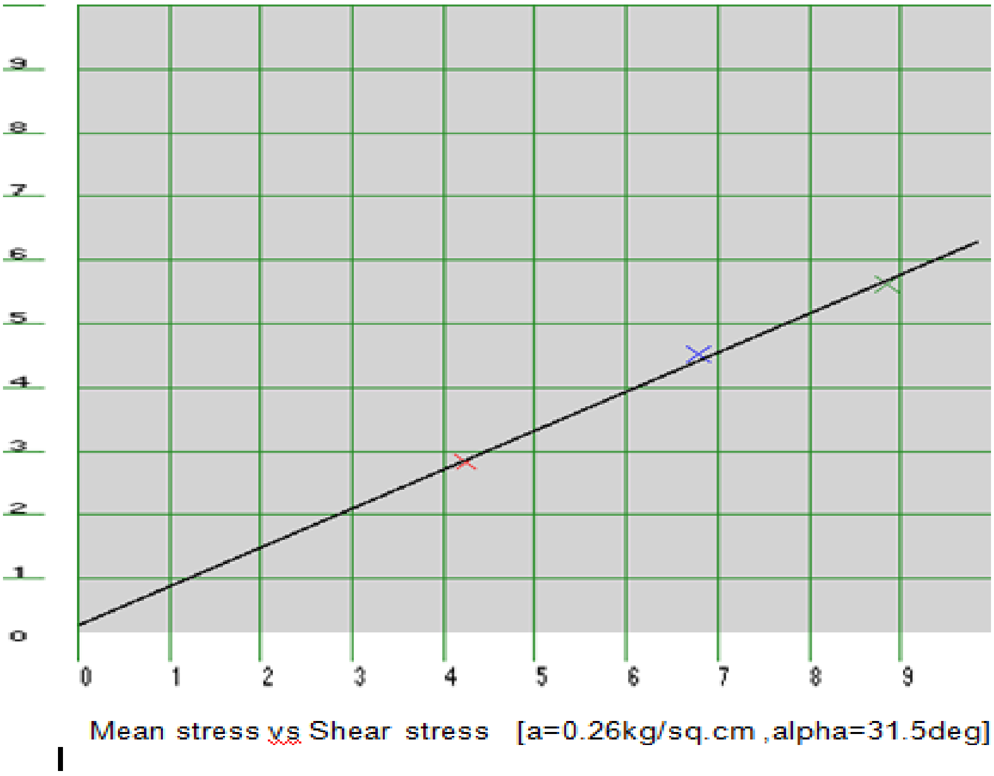

Figure 4.16:- p-q curve of clean sand+5%silt (obtained from digi triaxial machine)

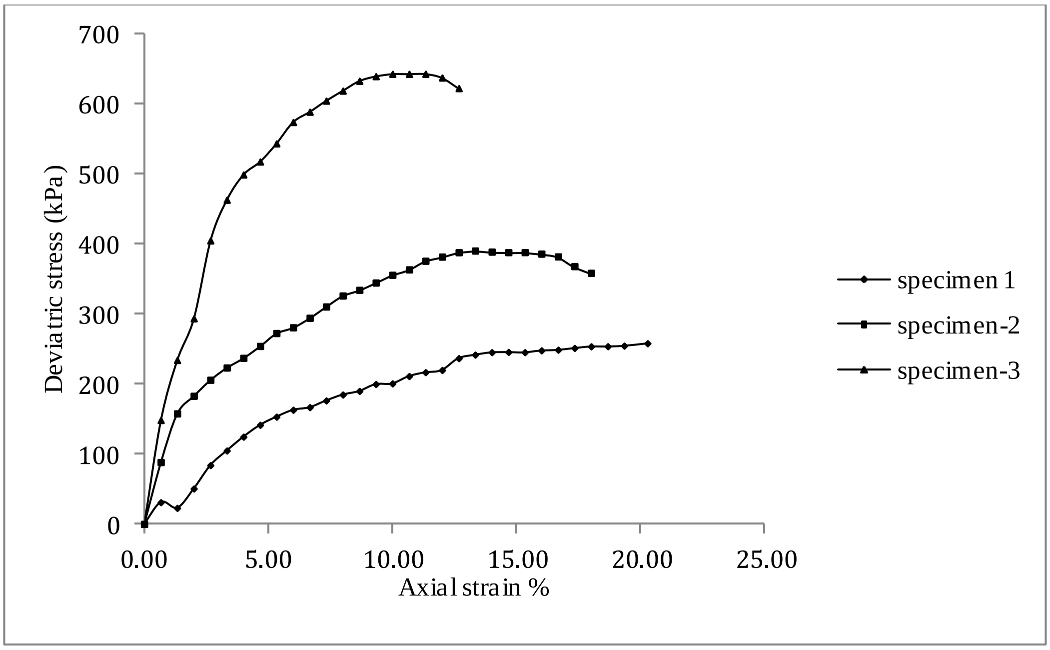

Figure 4.17:- Variation of deviatric stress vs. axial strain of clean sand+10%silt

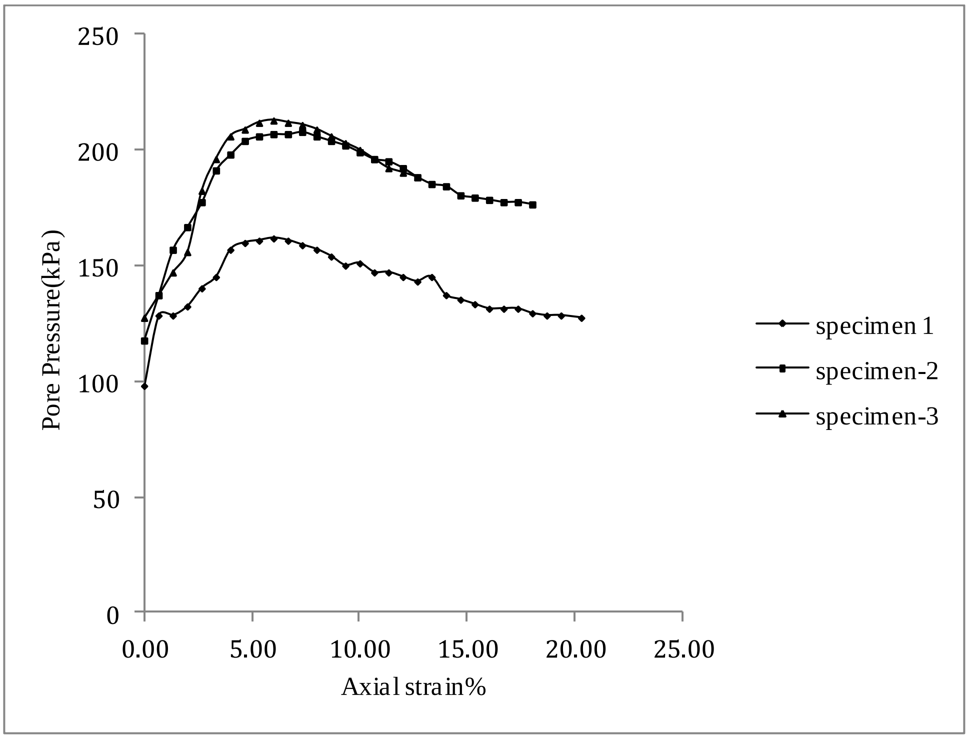

Figure 4.18:- Variation of pore pressure vs. axial strain of clean sand+10%silt

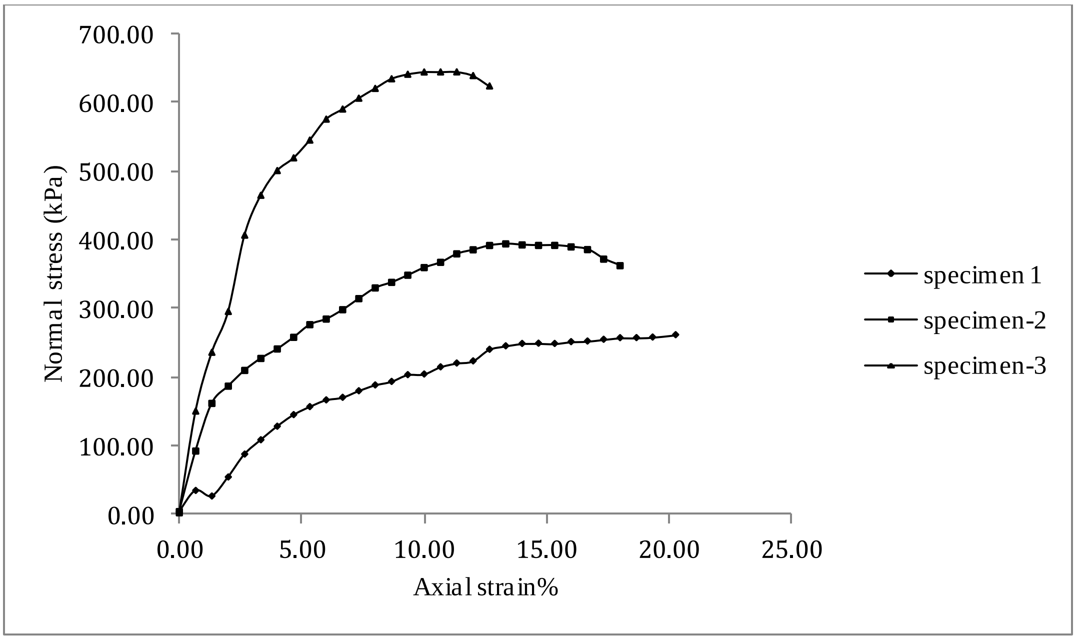

Figure 4.19:- Variation of normal stress vs. axial strain of clean sand+10%silt

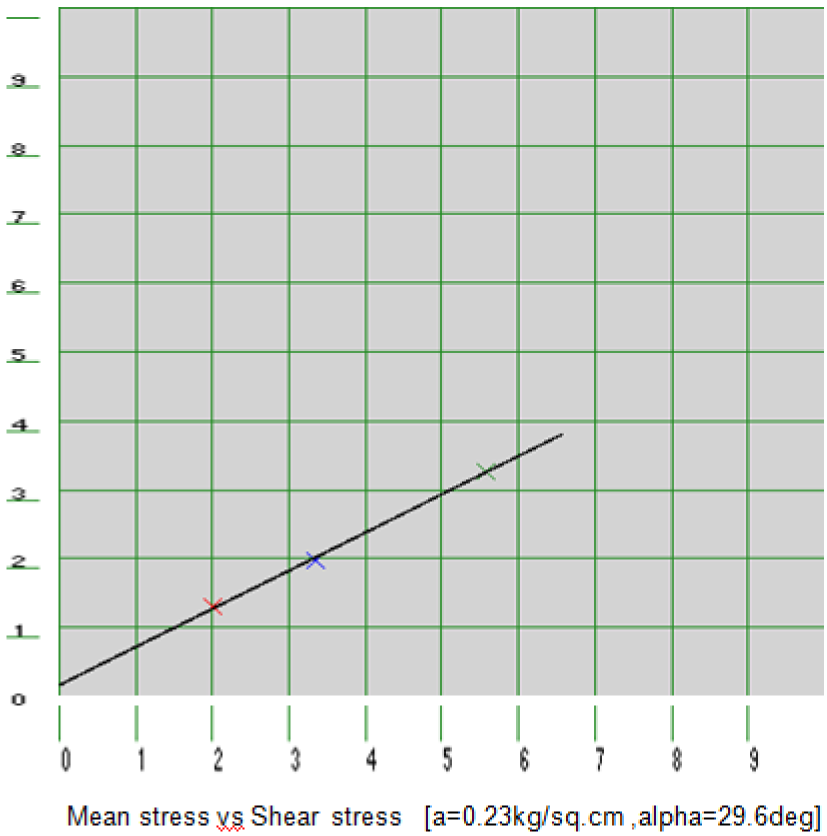

Figure 4.20:- p-q curve of clean sand+10%silt (obtained from digi triaxial machine)

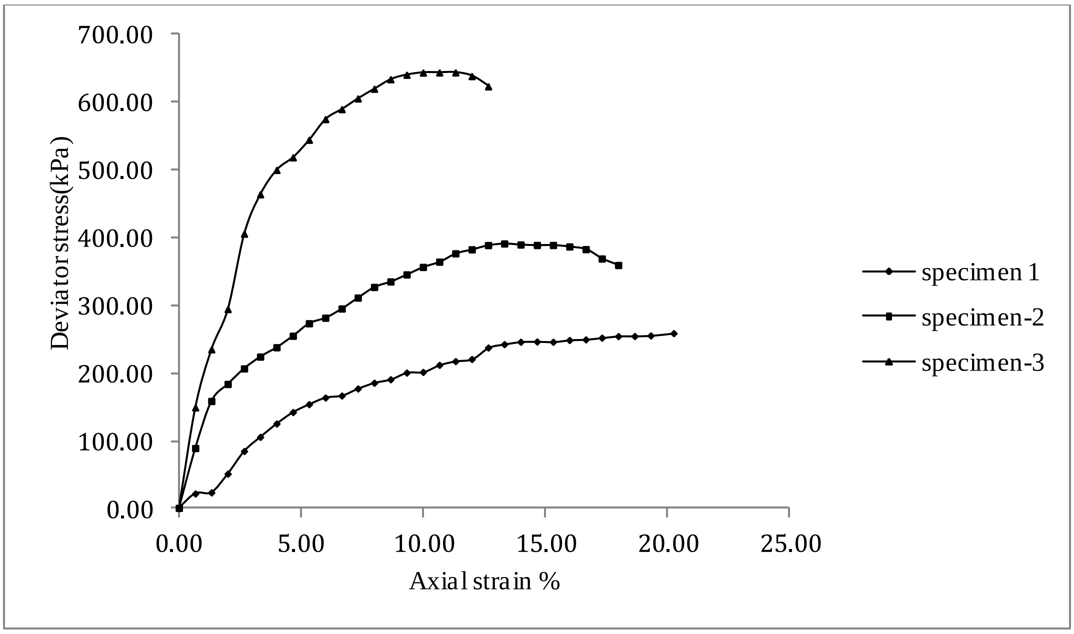

Figure 4.21:- Variation of deviatric stress vs. axial strain of clean sand+15%silt

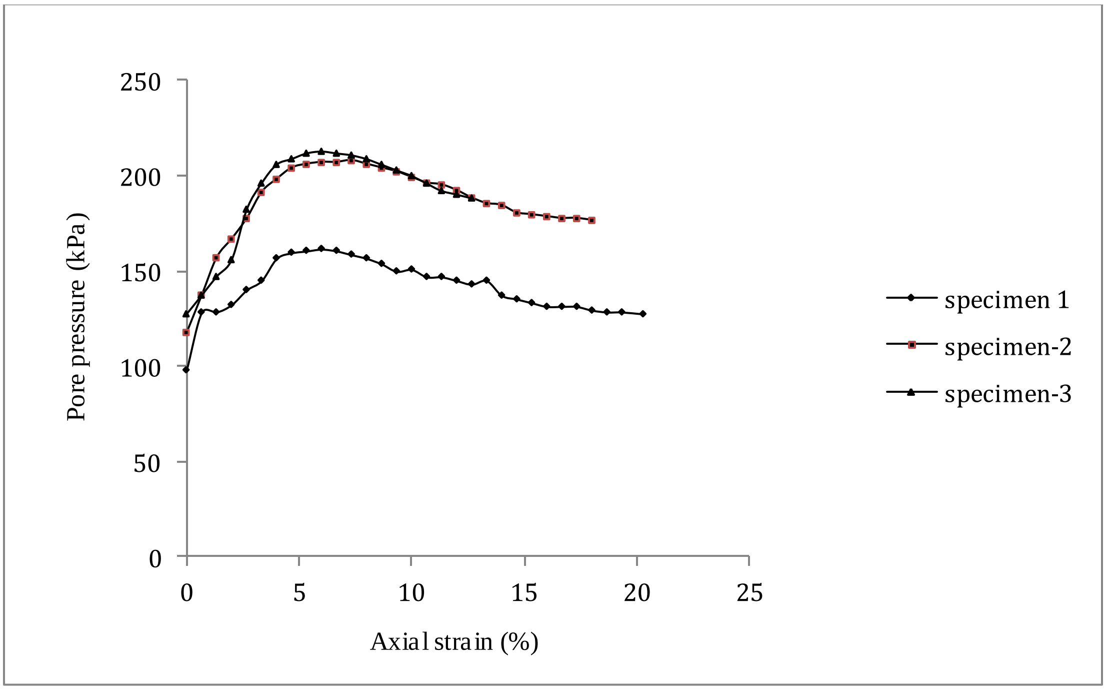

Figure 4.22:- Variation of pore pressure vs. axial strain of clean sand+15%silt

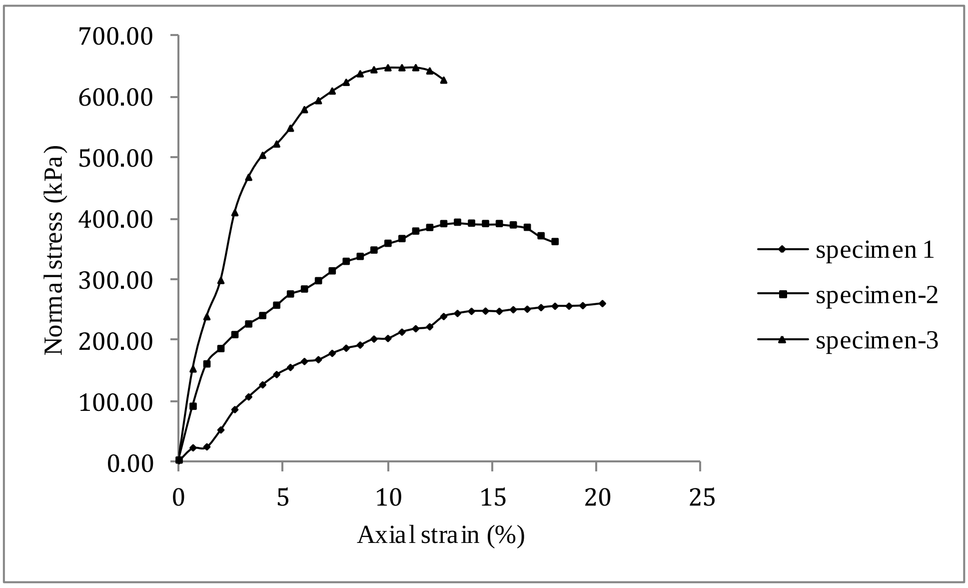

Figure 4.23:- Variation of normal stress vs. axial strain of clean sand+15%silt

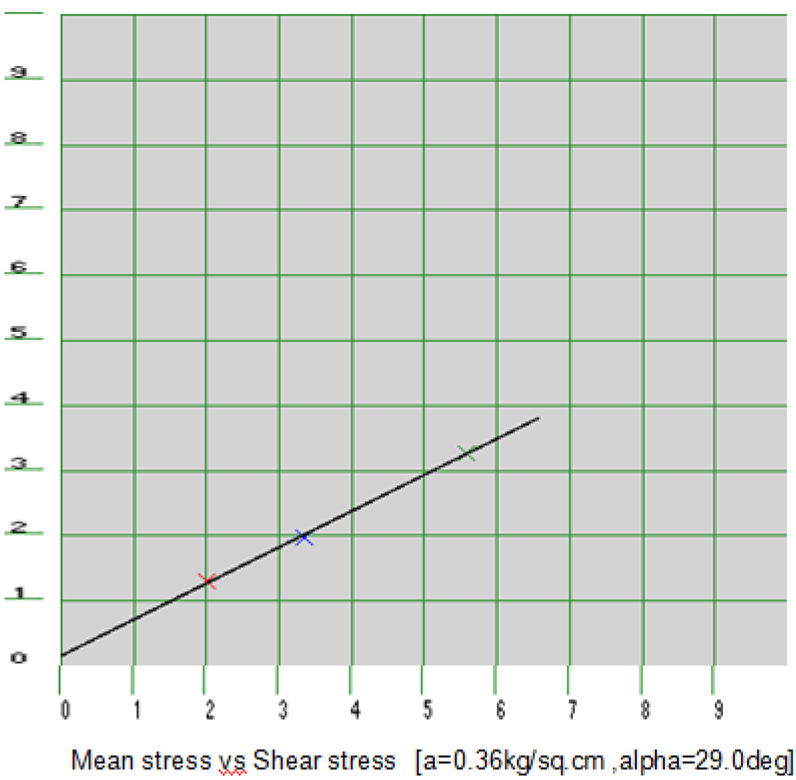

Figure 4.24:- p-q curve of clean sand+15%silt (obtained from digi triaxial machine)

Skempton's "*B*" parameter

Below table shows the variation of the generated excess pore water pressure as a function of the hydrostatic stress increment $\Delta\sigma$ for the tests. Skempton's *B* value under an initial hydrostatic load increment is a clear indication that the specimens were well-saturated initially but with increase in silt percentage, fines content increases in the soil, due to which sample saturation is affected may be due to the shape and size of silt particle.

Table 4.13.: Skempton's B parameter during saturation stage

Specimen name	**Pore pressure** Δu_1 (kPa)	**Cell pressure** $\Delta\sigma_3$ (kPa)	**Parameter-B** ($\Delta u_1/\Delta\sigma_3$)
SM0-specimen1	49.30	50	0.986
SM0-specimen2	98.91	100	0.989
SM0-specimen3	148.98	150	0.993
SM5-specimen1	49.08	50	0.982
SM5-specimen2	98.52	100	0.985
SM5-specimen3	148.52	150	0.990
SM10specimen1	49.01	50	0.980
SM10specimen2	99.05	100	0.991
SM10specimen3	148.53	150	0.990
SM15specimen1	48.85	50	0.977
SM15specimen2	98.10	100	0.981
SM15specimen3	147.75	150	0.985

Skempton's "A" parameter

For partially saturated soil, parameter **Ā** is calculated (where Ā=A.B) and then Skempton's "**A**" parameter is calculated by dividing parameter **B** from **Ā**. An assessment of the Skempton's formulation for shear induced pore pressure indicates that the **A** value is a function of the initial stress state The computed **A** value varied between about 0.3 to 0.6 for the given load increment.

Table 4.14.: Skempton's A parameter during shear stage

Specimen name	**Pore pressure** Δu_2(kPa)	**Principal stress** $\Delta\sigma_1$ (kPa)	**Parameter Ā** **(**$\Delta u_2/\Delta\sigma_1$)	**Parameter A** (=Ā/B)
SM0-specimen1	101.82	321.19	0.31	0.32
SM0-specimen2	135.24	527.29	0.21	0.21
SM0-specimen3	105.84	775.43	0.13	0.13
SM5-specimen1	103.46	256.62	0.40	0.41
SM5-specimen2	124.46	310.00	0.38	0.39
SM5-specimen3	113.68	592.89	0.19	0.19
SM10specimen1	151.10	252.89	0.59	0.61
SM10specimen2	182.30	362.22	0.50	0.50
SM10specimen3	196.00	417.02	0.47	0.47
SM15specimen1	159.74	250.23	0.60	0.62
SM15specimen2	197.96	364.90	0.45	0.46
SM15specimen3	208.74	409.29	0.41	0.42

Table 4.15.: Volume change during consolidation stage (in terms of column height)

Time (min)	**SM0** (m)	**SM5** (m)	**SM10** (m)	**SM15** (m)
.25	0.145	0.140	0.135	0.130
1.0	0.145	0.138	0.130	0.125
2.25	0.144	0.137	0.128	0.113
4.0	0.144	0.136	0.128	0.112
6.25	0.143	0.136	0.127	0.111
9.0	0.143	0.135	0.127	0.111

Deviator stress

- The results of Maximum Deviator, Normal Stress and Pore Pressure with strain are shown in following figures and tables given below. It is observed that with increase in cell pressure, Deviator stress increases with strain .With increase in silt percentage in sand deviator stress decreases.

Table 4.16.: Ultimate Deviator stress with strain of soil mix

Specimen name	Deviator stress(kPa)	Strain (%)
SM0-specimen1	560.35	*7.33*
SM0-specimen2	889.42	8.67
SM0-specimen3	1098.21	10.00
SM5-specimen1	400.69	9.33
SM5-specimen2	604.75	12.67
SM5-specimen3	990.94	12.00
SM10-specimen1	257.75	20.27
SM10-specimen2	390.00	13.33
SM10-specimen3	642.23	11.33
SM15-specimen1	257.75	20.27
SM15-specimen2	391.79	15.33
SM15-specimen3	642.23	10.67

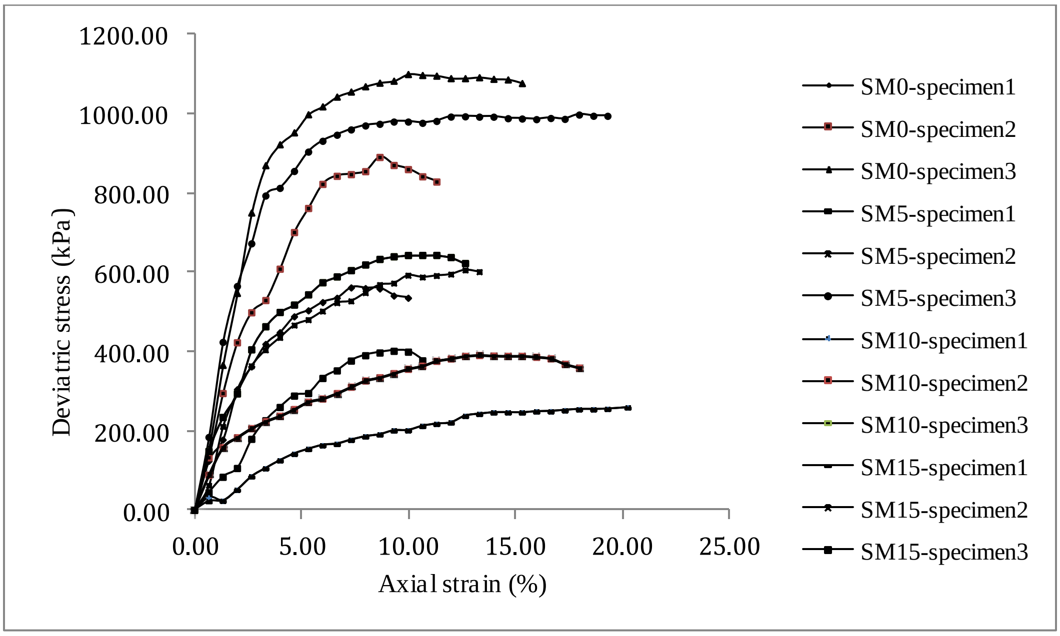

Figure 4.25:- Variation of deviatric stress vs. axial strain of soil mix.

Pore pressure

➢ It is observed from the table and figure given below that with increase in Cell pressure, Pore pressure increases but strain decreases respectively but due to increase in silt percentage in sand, Pore pressure increases unlike in the case of deviator stress. It is observed as percentage of finer is increasing due to which volume decreases (in table 4.15) during application of deviator stress and hence positive pore pressure is developed.

Table 4.17.: Ultimate Pore pressure with strain of soil mix .

Specimen name	Pore pressure (kPa)	Strain(%)
SM0-specimen1	77.42	4.67
SM0-specimen2	135.24	2.00
SM0-specimen3	112.70	2.67
SM5-specimen1	105.84	4.00
SM5-specimen2	124.46	3.33
SM5-specimen3	113.68	2.65
SM10-specimen1	161.70	6.00
SM10-specimen2	207.76	7.33
SM10-specimen3	212.66	6.00
SM15-specimen1	162.80	6.33
SM15-specimen2	207.76	7.33
SM15-specimen3	212.61	6.00

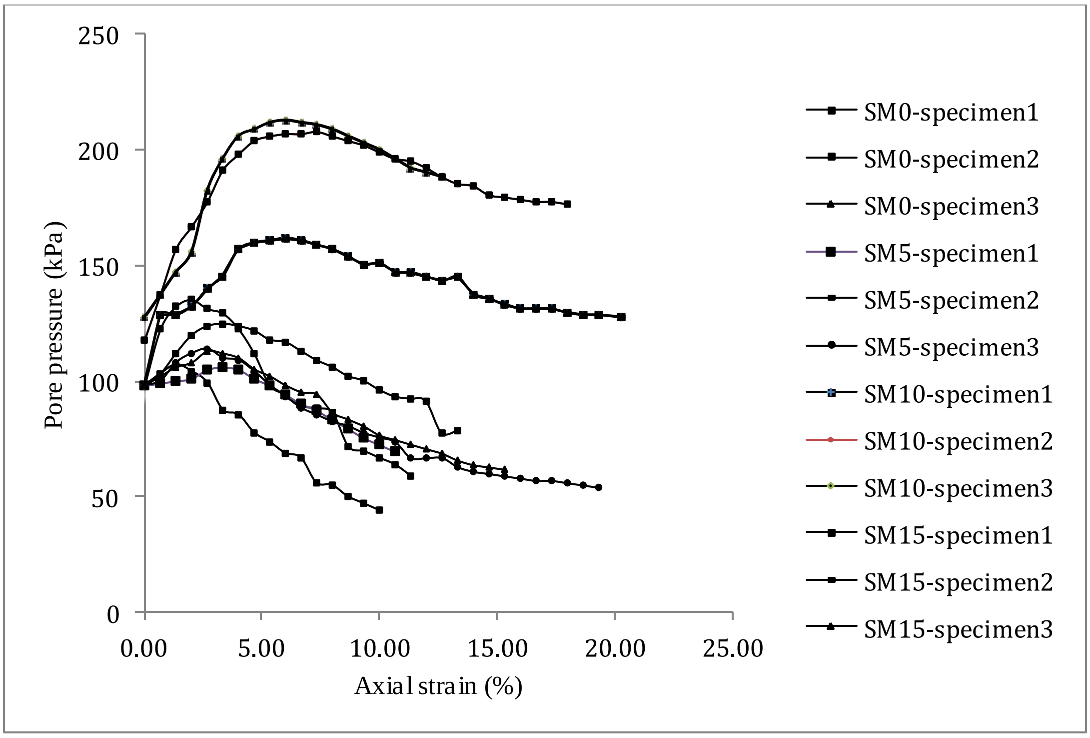

Figure 4.26:- Variation of pore pressure vs. axial strain in silty sand

Normal stress

- It is observed from the table and figure, given below that with increase in Cell pressure, Normal stress increases with strain. But with increase in silt percentage, Normal stress decreases.

Table 4.18.: Ultimate Pore pressure with strain of soil mix .

Specimen name	Normal Stress(kPa)	Strain%
SM0-specimen1	562.35	7.33
SM0-specimen2	892.21	8.67
SM0-specimen3	1102.21	10.00
SM5-specimen1	402.69	9.33
SM5-specimen2	607.75	12.67
SM5-specimen3	1000.02	18.00
SM10-specimen1	261.75	20.27
SM10-specimen2	391.76	14.67
SM10-specimen3	644.61	11.00
SM15-specimen1	259.75	20.27
SM15-specimen2	391.69	14.00
SM15-specimen3	646.61	11.33

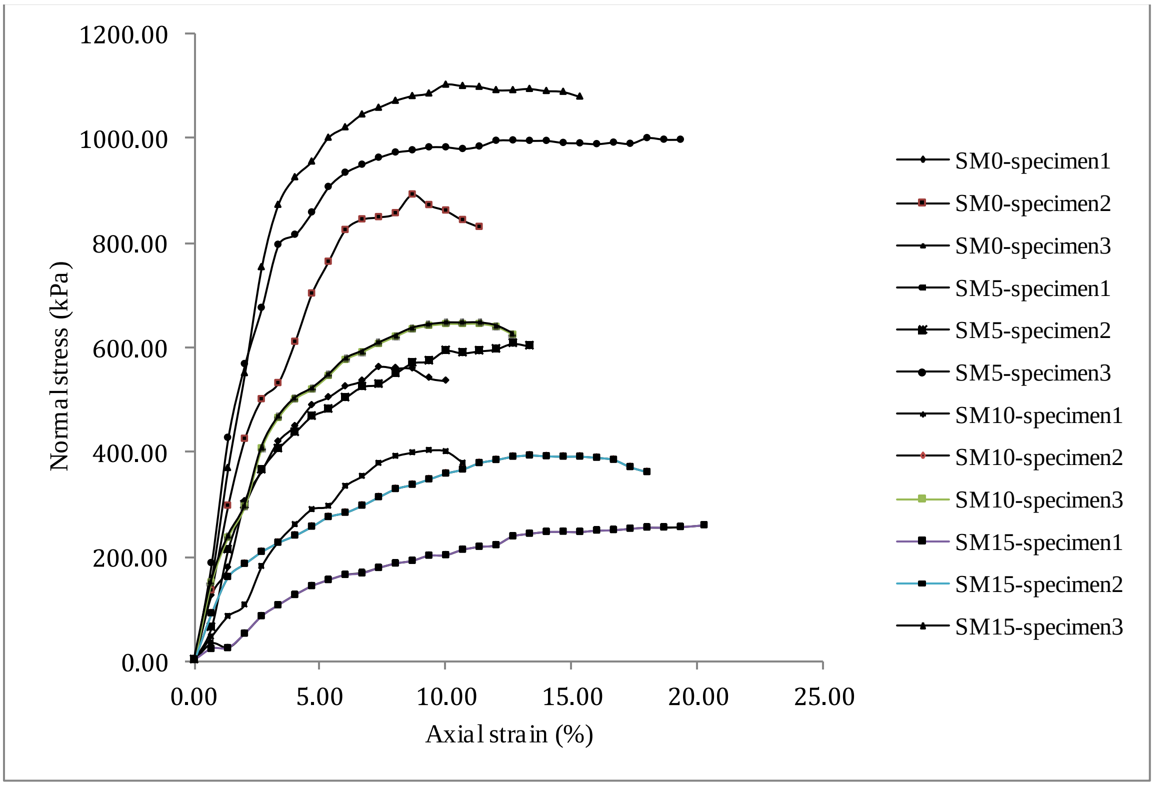

Figure 4.27:- Variation of normal stress vs. axial strain of soil mix .

MOHR –COULOMB PLOT OF TRIAXIAL TEST RESULTS

With increase in the percentage of silt, cohesion and angle of internal resistance increases and decreases respectively. The grain shape, grain size, uniformity of gradation is the factors influencing the value of *Φ*.

Table 4.19.: Shear parameters of soil mix from CU Triaxial test

S. No.	Sample Description	*Φ* (angle of resistance)	Cohesion(kN/m^2)
1	Clean sand	37.8^0	2.94
2	Clean Sand + 5% Silt	36.3^0	24.50
3	Clean Sand + 10% Silt	35.4^0	27.44
4	Clean Sand + 15% Silt	33.7^0	35.28

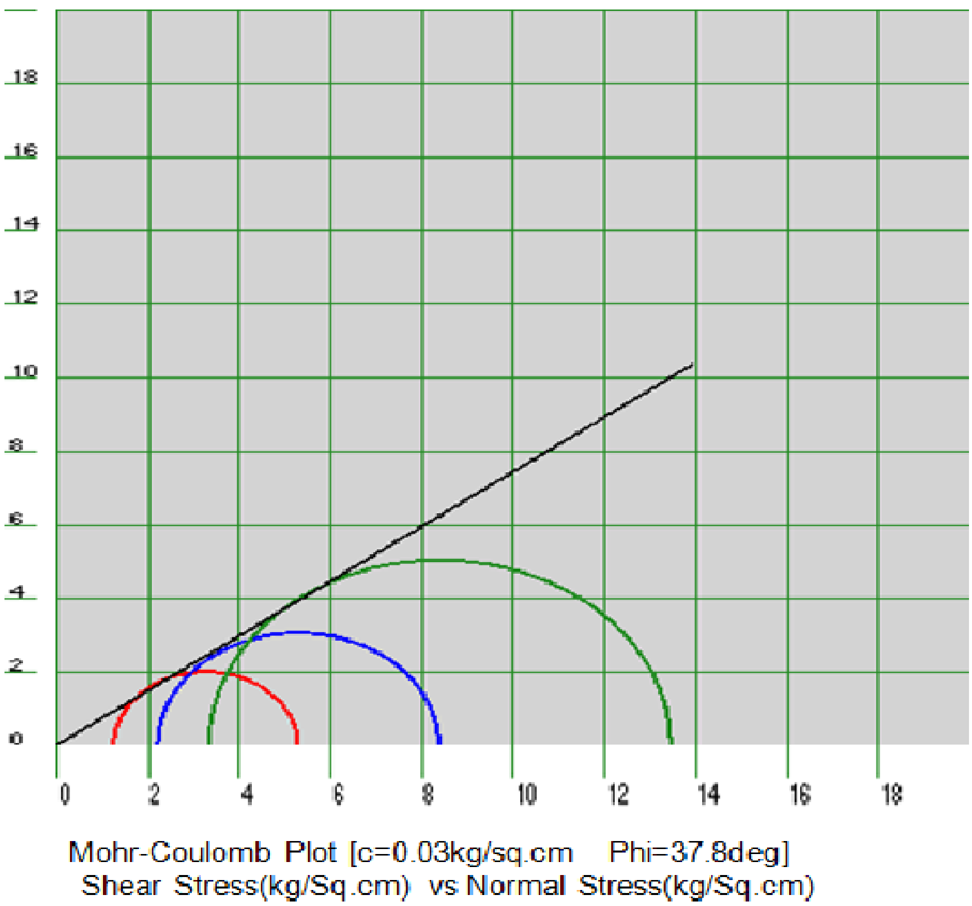

Figure 4.28:- Mohr-Coulomb Plot of clean sand (obtained from digi triaxial machine)

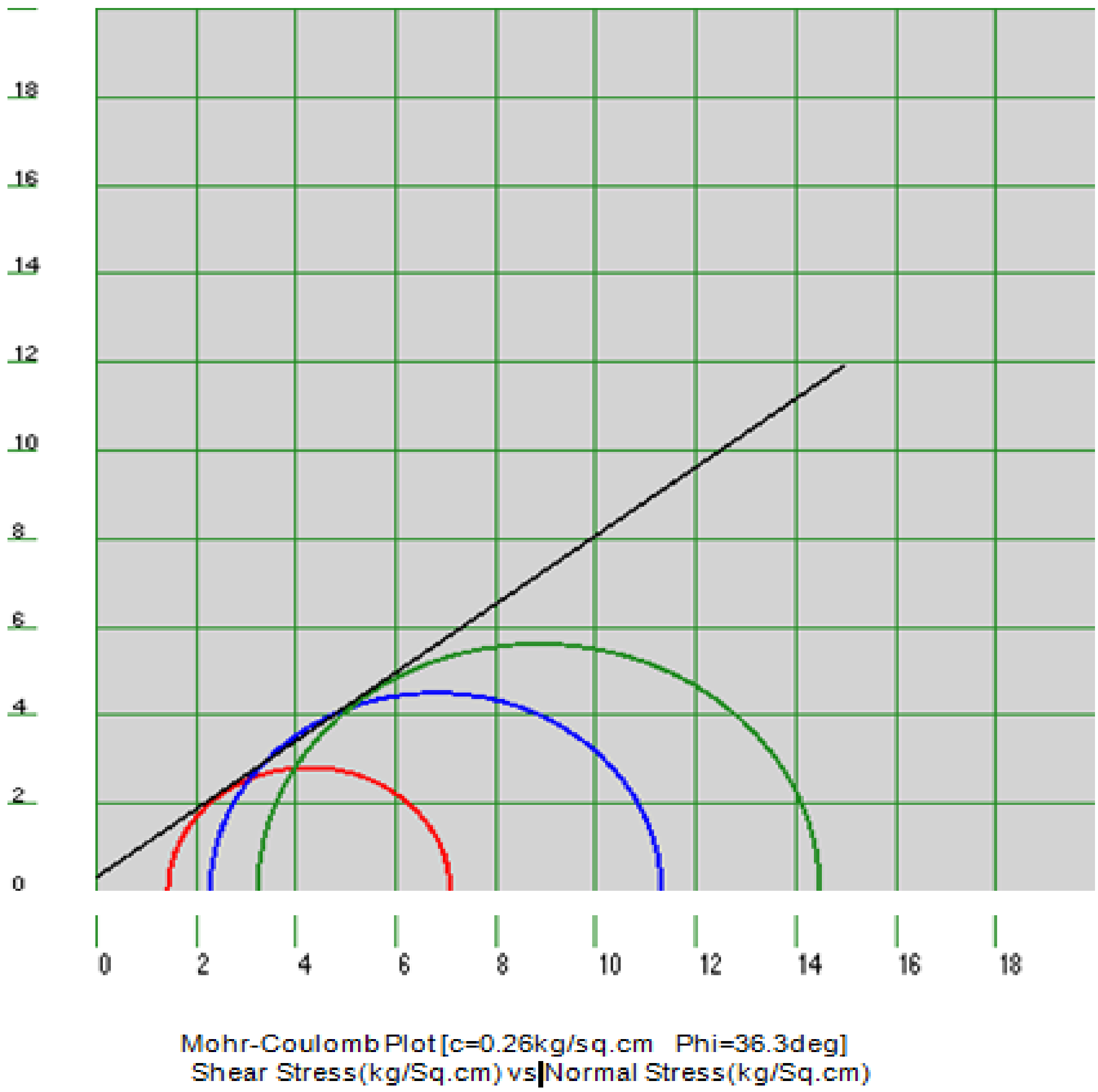

Figure 4.29:- Mohr-Coulomb Plot of clean sand+5% (obtained from digi triaxial machine)

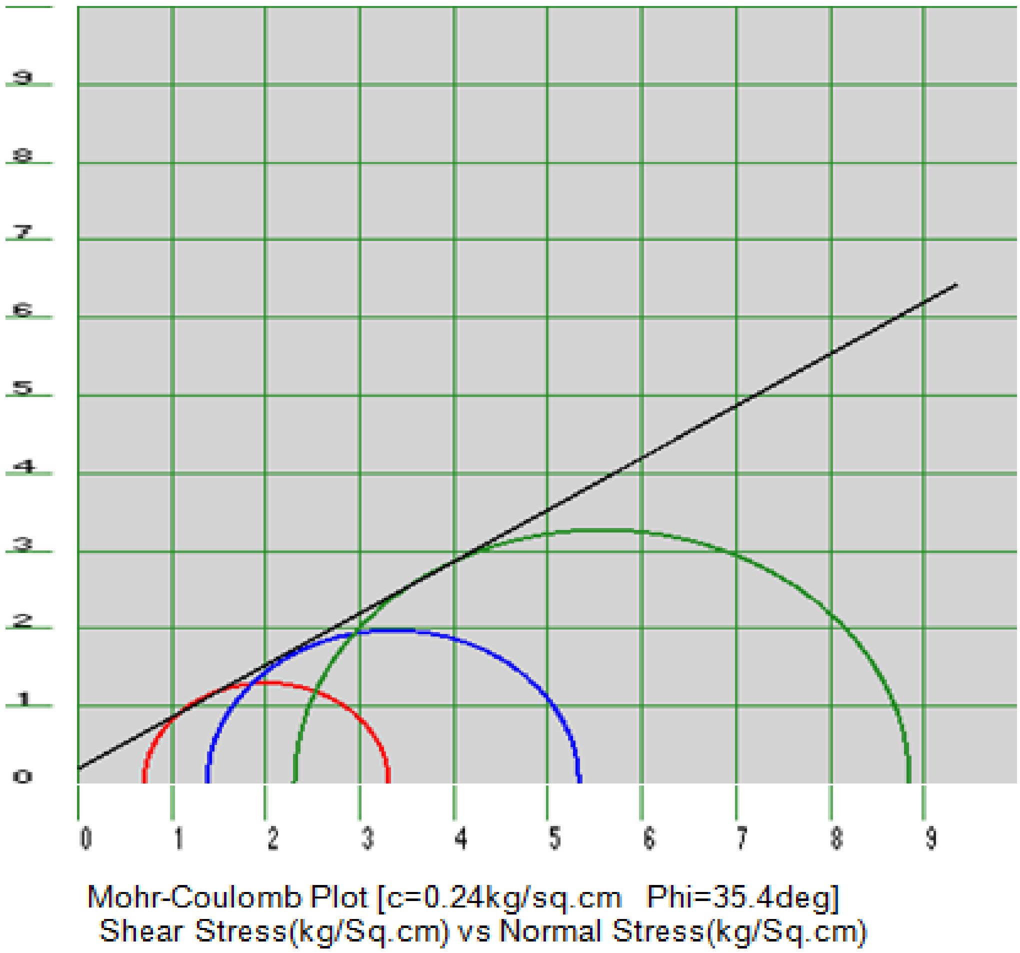

Figure 4.30:- Mohr-Coulomb Plot of clean sand+10%silt (obtained from digi triaxial machine)

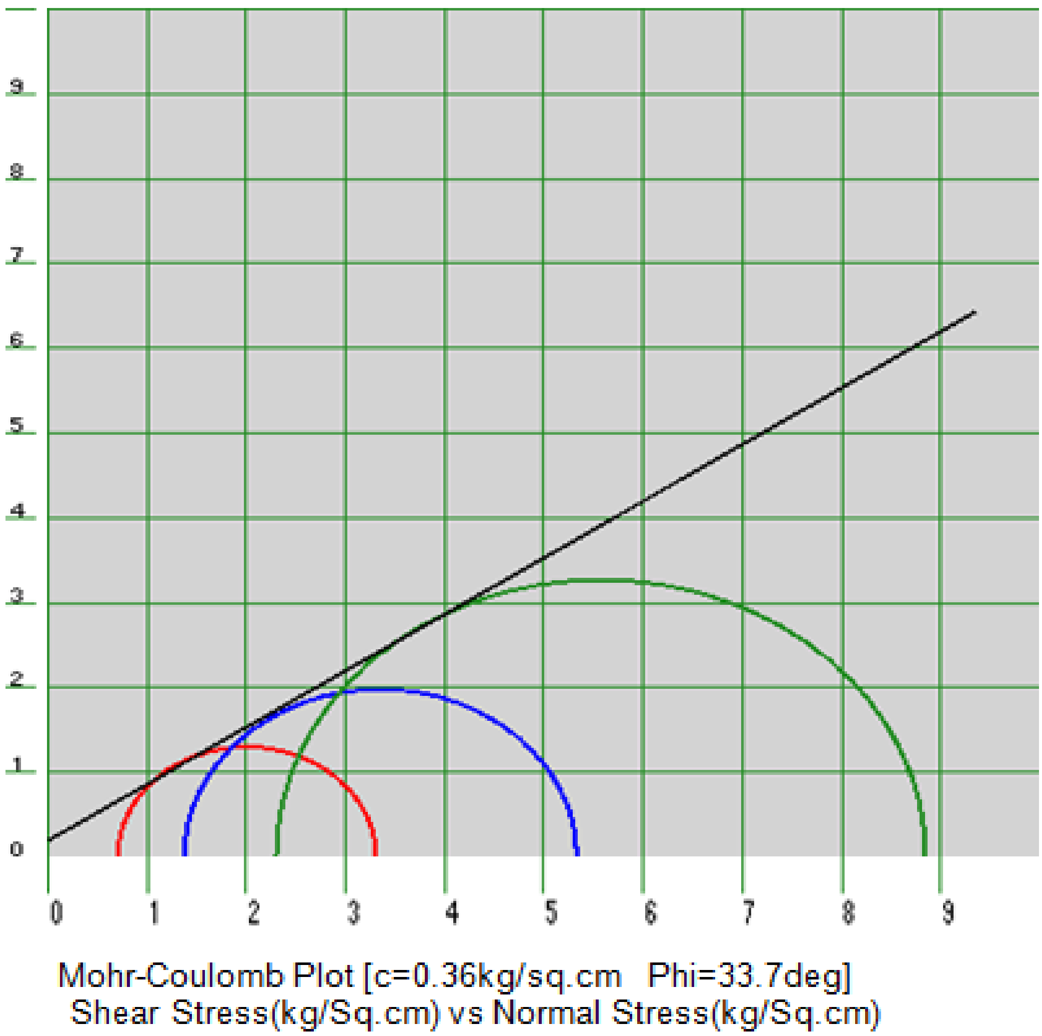

Figure 4.31:- Mohr-Coulomb Plot of clean sand+15%silt (obtained from digi triaxial machine)

4.3.3. Relationship between angle of friction obtained from DST and Triaxial.

A relationship is observed in the frictional angle of soil obtained from DST and Triaxial i.e. Angle of internal resistance (Triaxial) = k (Angle of internal resistance (DST))

- $\boldsymbol{\Phi_{triaxial} = k \cdot \Phi_{plain}}$

Table 4.20.: Relationship between frictional angle of soil obtained from DST & Triaxial.

Sample Name	Angle of internal resistance (DST) (Φ_{plain})	Angle of internal resistance (Triaxial) $(\Phi_{triaxial})$	k (constant)
SM0	38.98^0	37.80^0	1.031
SM5	37.50^0	36.30^0	1.033
SM10	36.60^0	35.40^0	1.034
SM15	34.88^0	33.70^0	1.035

CHAPTER 5- CONCLUSIONS

With the increase of silt content in sand following conclusions are obtained:

1. Specific gravity decreases with increasing silt content.
2. Optimum moisture content increases with increasing silt content.
3. Maximum dry density decreases with increasing silt content.
4. Liquid limit increases with increasing silt content.

In Direct Shear test:

1. Direct shear tests on saturated specimens show that the ultimate shear strength is relatively independent of the initial compaction state.
2. With the increase in normal load, deformation increases but at 5 percent silt blended with sand, Normal Stress value decreases as compared to pure sand sample and similarly with increasing silt percentage displacement increased but normal stress value decreases.

Triaxial (CU) test:

This test is essential to understand soil behavior. We can measure the strength and stiffness, monitor the internal response of the particulate medium, monitor pore pressures as they build, and watch volume changes taking place during the test.

Proper understanding of material behavior followed by the proper assessment of its characteristics allows the Engineer to improve designs and to reduce the risk of failures.

According to consolidated –undrained triaxial test results performed on mixtures of silt with sand, following conclusions are obtained:

1. Shear strength and stress – strain characteristics of mixtures show significant changes at fine content of about 5%.
2. The undrained angle of friction and therefore undrained shear strength is decreased with increase in fine content.
3. Strains at failure in undrained tests markedly increase in the specimens with 5 % silt content with increase in deviatric stress.
4. Due the presence of water in a sand sample, it becomes slightly cohesive and consequently its pore pressure increases and normal stress decreases but this happens up to a certain limit of silt content.

5. It is concluded that with increase in cell pressure, Deviator stress increases with strain .With increase in silt percentage in sand deviator stress decreases.
6. It is concluded that with increase in Cell pressure, Pore pressure increases but strain decreases respectively but due to increase in silt percentage in sand, Pore pressure increases unlike in the case of deviator stress.
7. It is concluded that with increase in Cell pressure, Normal stress increases with strain. But with increase in silt percentage, Normal stress decreases.

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APPENDIX

Table 7.1.: Deviatric stress and pore pressure of SM0 sample with strain.

	Deviatric stress (kN/m^2)			Pore Pressure (kN/m^2)		
Strain(%)	specimen-1	specimen-2	specimen-3	specimen-1	specimen-2	specimen-3
0.00	0.00	0.00	0.00	98.00	98.00	98.00
0.67	123.49	132.31	153.48	102.9	122.5	101.92
1.33	176.91	294.26	365.20	106.82	132.3	105.84
2.00	303.75	422.11	546.57	103.88	135.24	107.81
2.67	361.23	497.78	749.26	98.98	131.32	112.72
3.33	417.92	528.62	868.44	87.22	129.36	111.72
4.00	447.39	607.60	921.20	85.26	122.5	109.76
4.67	487.46	699.88	951.23	77.42	111.72	104.86
5.33	502.61	760.63	996.81	73.50	98.0	101.92
6.00	523.39	821.40	1016.73	68.60	94.08	98.00
6.67	534.32	841.66	1041.30	66.64	89.18	95.06
7.33	560.35	845.88	1054.06	55.86	88.2.0	94.08
8.00	558.60	853.42	1067.38	54.88	86.24	86.24
8.67	557.68	889.21	1076.46	49.98	71.54	83.32
9.33	539.89	868.97	1081.38	47.04	69.58	80.36
10.00	534.33	858.60	1098.21	44.10	66.64	76.44
10.67	-	840.45	1095.76	-	63.70	74.48
11.33	-	827.29	1094.14	-	58.80	72.52
12.00	-	-	1087.85	-	-	70.56
12.67	-	-	1087.77	-	-	68.61
13.33	-	-	1090.09	-	-	65.66
14.00	-	-	1086.15	-	-	63.72
14.67	-	-	1084.56	-	-	62.72
15.33	-	-	1075.43	-	-	61.74

Table 7.2.: Deviatric stress and pore pressure of SM5 sample with strain.

	Deviatric stress (kN/m^2)			Pore pressure (kN/m^2)		
Strain (%)	specimen -1	specimen-2	specimen-3	specimen- 1	specimen-2	specimen-3
0.00	0.00	0.00	0.00	98.00	98.00	98.00
0.67	44.10	61.75	183.47	98.98	102.90	99.96
1.33	83.20	210.19	423.00	99.96	111.72	107.80
2.00	105.31	295.91	563.98	100.94	119.56	111.72
2.67	178.89	362.96	671.48	104.86	123.48	113.68
3.33	225.69	403.33	792.07	105.84	124.46	109.76
4.00	259.06	434.61	811.27	104.86	123.48	108.78
4.67	287.74	465.46	853.90	100.94	121.52	103.88
5.33	294.17	479.07	902.68	98.00	117.60	98.00
6.00	332.23	500.85	929.91	94.08	116.62	93.10
6.67	351.24	521.89	945.21	90.16	112.70	88.20
7.33	376.04	526.62	958.61	87.22	108.78	85.26
8.00	389.55	547.17	968.57	83.30	105.84	82.32
8.67	396.38	567.41	972.70	79.38	101.92	80.36
9.33	400.69	571.26	978.39	75.46	99.96	77.42
10.00	398.55	591.04	978.40	72.52	96.04	75.46
10.67	376.62	586.73	975.24	69.58	93.10	73.50
11.33	-	590.36	980.00	-	92.12	66.64
12.00	-	593.94	990.94	-	91.14	66.64
12.67	-	604.75	991.63	-	77.42	66.64
13.33	-	600.00	990.77	-	78.40	62.72
14.00	-	-	990.69	-	-	60.76
14.67	-	-	986.82	-	-	59.78
15.33	-	-	986.01	-	-	58.80
16.00	-	-	984.47	-	-	57.82
16.67	-	-	987.40	-	-	56.84
17.33	-	-	985.14	-	-	56.84
18.00	-	-	996.02	-	-	55.86
18.67	-	-	993.00	-	-	54.88
19.33	-	-	992.89	-	-	53.90

Table 7.3.: Deviatric stress and pore pressure of SM10 sample with strain.

	Deviatric stress(kN/m^2)			Pore pressure (kN/m^2)		
Strain (%)	specimen- 1	specimen-2	specimen-3	specimen- 1	specimen-2	specimen-3
0.00	0.00	0.00	0.00	98.00	117.60	127.40
0.67	34.87	92.21	150.49	128.38	137.20	137.20
1.33	26.77	161.64	236.13	128.38	156.80	147.00
2.00	54.48	186.77	295.60	132.30	166.60	155.82
2.67	87.83	209.68	406.74	140.14	177.38	182.28
3.33	108.69	227.12	464.84	145.04	191.10	196.00
4.00	128.42	240.90	500.82	156.80	197.96	205.80
4.67	145.33	257.89	519.38	159.74	203.84	208.74
5.33	156.97	276.32	545.25	160.72	205.80	211.68
6.00	166.78	284.48	575.78	161.7	206.78	212.66
6.67	170.51	298.08	590.47	160.72	206.78	211.68
7.33	180.09	314.21	606.26	158.76	207.76	210.70
8.00	188.57	329.85	620.52	156.80	205.80	208.74
9.33	203.54	348.37	641.15	149.94	201.88	202.86
10.00	204.47	359.42	644.45	150.92	198.94	199.92
10.67	214.91	367.14	644.53	147.00	196.00	196.00
12.00	220.47	379.47	644.61	145.04	192.08	190.12
12.67	223.60	385.37	639.22	143.08	188.16	188.16
13.33	240.47	391.66	-	145.04	185.22	-
14.00	245.54	394.00	-	137.20	184.24	-
14.67	249.00	392.49	-	135.24	180.32	-
15.33	249.38	391.76	-	133.28	179.34	-
16.00	249.00	391.79	-	131.32	178.36	-
16.67	251.61	389.59	-	131.32	177.38	-
17.33	252.51	385.65	-	131.32	177.38	-
18.00	255.06	371.78	-	129.36	176.40	-
18.67	257.37	-	-	128.38	-	-
19.33	257.49	-	-	128.38	-	-
20.27	261.75	-	-	127.40	-	-

Table 7.4.: Deviatric stress and pore pressure of SM15 sample with strain.

	Deviatric stress (kN/m^2)			Pore Pressure (kN/m^2)		
Strain (%)	specimen-1	specimen-2	specimen-3	specimen -1	specimen-2	specimen-3
0.00	0.00	0.00	0.00	99.10	118.92	128.83
0.67	21.13	88.03	147.89	129.82	138.74	137.65
1.33	22.72	157.32	233.36	129.82	158.56	148.65
2.00	50.38	182.40	292.71	133.79	168.47	157.569
2.67	83.66	205.26	403.62	141.71	179.37	184.326
3.33	104.48	222.66	461.60	146.67	193.25	198.22
4.00	124.16	236.42	497.50	158.56	200.18	208.11
4.67	141.04	253.37	516.03	161.53	206.13	211.083
5.33	152.66	271.76	541.84	162.52	208.11	214.056
6.00	162.44	279.90	572.30	163.52	209.10	215.047
7.33	165.34	293.48	586.97	160.54	210.09	213.065
8.00	175.73	309.58	602.73	158.56	208.11	211.083
8.67	184.19	325.19	616.96	155.59	206.13	208.11
9.33	189.29	333.28	630.97	151.62	204.15	205.137
10.00	199.13	343.67	637.55	152.61	201.17	202.164
11.33	200.06	354.69	640.84	148.65	197.21	194.236
12.00	216.02	374.70	641.00	146.67	194.24	192.254
12.67	219.15	380.59	635.62	144.69	190.27	190.272
13.33	235.99	386.87	-	146.67	187.30	-
14.00	241.05	389.20	-	138.74	186.31	-
14.67	244.50	387.70	-	136.76	182.34	-
15.33	244.88	386.97	-	134.78	181.35	-
16.00	244.50	387.00	-	132.79	180.36	-
16.67	247.11	384.80	-	132.79	179.37	-
17.33	248.01	380.87	-	132.79	179.37	--
18.00	250.54	367.02	-	130.81	178.38	-
18.67	252.86	-	-	129.82	-	-
19.33	252.97	-	-	129.82	-	-
20.27	253.79	-	-	128.83	-	-