CORRELATION BETWEEN SHEAR STRENGTH OF SOIL AND WATER CONTENT RATIO AS A SUBSTITUTE FOR LIQUIDITY INDEX

A DISSERTATION

SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE

OF

MASTER OF TECHNOLOGY

In

GEOTECHNICAL ENGINEERING

Submitted By

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(2K16/GTE/09)

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CANDIDATE'S DECLARATION

I do hereby certify that the work presented is the report entitled "Correlation Between Shear Strength Of Soil And Water Content Ratio As A Substitute For Liquidity Index" in the partial fulfillment of the requirement for the award of the degree of "Master of Technology" in Geotechnical Engineering submitted in the Department of Civil Engineering, Delhi Technological University, Delhi is an authentic record of my own work carried out under the supervision of Prof. A.K Gupta, Department of Civil Engineering. The work is original and not copied from any other source without proper citation. The work has not previously formed the basis for the award of any Degree, Diploma Associate ship, Fellowship or other similar title or recognition.

Place: Delhi (HARSHDEEP SINGH)

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CERTIFICATE

I hereby certify that the Project Dissertation titled "Correlation Between Shear Strength Of Soil And Water Content Ratio As A Substitute For Liquidity Index" which is submitted by Harshdeep Singh, 2K16/GTE/09, Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfilment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by the student under my supervision. To the best of my knowledge this work has not been submitted in part or full for any Degree or Diploma to this University or elsewhere.

Place: Delhi

Date: Signature

(Prof. A.K. Gupta)

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(Harshdeep Singh)
2K16/GTE/09

ABSTRACT

Undrained shear strength of saturated clays is a vital property in Geotechnical engineering practice. If any relationship between shear strength of soil and index properties of soil is developed, it would be exceptionally alluring. A few endeavors have been made in the past to associate shear strength with Liquidity index. The Liquidity index requires the estimation of plastic limit calculated by Casagrande thread rolling method which does not provide the correct assurance of plastic limit of the soil particularly in less plastic soils. Shear strength variation with water content does not follow a regular trend which makes the analysis difficult. It has been observed in the past researches that Shear strength of soil correlates very well with the consistency limits of soils. The present paper develops the correlation between shear strength and Water Content Ratio (w_X) and between shear strength and Liquidity Index, to find out the better parameter to evaluate shear strength between Water Content Ratio (w_X) and Liquidity Index. The experimental results on 3 different Highly compressible soils having water content ranging from 5% to 25% showed that the Regression coefficient value of relation between undrained shear strength with Water Content Ratio came out to be closer to 1 compared with Regression coefficient value of relation between Undrained shear strength with Liquidity Index, for the soils of same geological origin. Liquidity Index variation with Water Content Ratio suggests that there is a definite relation between Liquidity Index and Water Content Ratio and Liquidity Index can be substituted by Water Content Ratio. However, the results obtained from both are more or less same for the soils irrespective of their origin.

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LIST OF ABBREVIATIONS

SYMBOL	TITLE
kN	Kilo Newton
ϕ	Angle of internal friction
c	Cohesion
γ_d	Dry unit weight
OMC	Optimum moisture content
G	Specific gravity
I_L	Liquidity Index
w_N	Natural water content
w_X	Water content ratio
Cu	Undrained Shear strength
CH	Highly compressible Clays
CU	Consolidated Undrained
UU	Unconsolidated Undrained
w_L	Liquid limit
w_P	Plastic limit
w_S	Shrinkage limit

CHAPTER 1

INTRODUCTION

Shear strength of the soil is an imperative property in geotechnical engineering so as to survey the strength of the soil. In soil mechanics, the soil is by and large thought to be completely saturated or in totally dry conditions. However the vast majority of the regular soils or compacted soils are unsaturated.

A soil may derive its shear strength from following parameters:

- Interlocking between molecules
- Friction between molecules
- Intermolecular attraction
- ✓ A coarse grained soil derives its shear strength from interlocking and friction.
- ✓ A fine-grained soil derives its shear strength from cohesion and friction. (Pure clay from cohesion)

The two parameters mentioned primarily, defines the soil maximum ability to resist shear Stress under defined load.

Cohesion Increases at the starting of application of stress and reaches its maximum around the plastic limit, i.e. when the structure starts collapsing. (Mencl.V, 1997).

Cohesion decreases at water content heading towards the liquid limit (w_L) and increases towards the shrinkage limit (w_s).

In general, Cohesion does not increase as the stress increases but for clays molecular binding increases with the increase in stress. (Mencl.V, 1997).

In order to find the undrained shear strength of Remolded clays in an economical way ,Liquidity Index is not a stable parameter as it requires the plastic limit which is not an accurate method for fine grained saturated soils and also, most of the representative samples collected are disturbed. So, a parameter which would strongly relate with the Shear strength would be much appreciated. One such parameter is w_X (Water Content Ratio) which is defined as

$$\mathbf{w_X = w_N/w_L} \qquad (1.1)$$

In earlier researches, Wroth and Wood (1978), Yilmaz (2000) established relationship between shear strength and liquidity index. Leroueil et al. (1983) examined undrained cohesion relation with liquidity index in an inverse power form .Schofield and Wroth (1968), from the vane shear data collected from Skempton and Northey (1952) observed that Liquid limit corresponds well approximately to fixed strengths that are in the ratio of 1:100.

The shear strength of soil reduces with increase in water content for a particular soil. But at same water content, due to plastic behavior, different soil behaves differently and has different shear strengths. So, water content alone would not describe the relation with shear strength in an absolute manner.

Also, water content increases with increase in liquid limit values. These two parameters combined, correlates strongly with shear strength and shows definite trends. Therefore, a parameter w_X (Water Content Ratio) which is defined as $w_X = w_N / w_L$ is taken into analysis and discussed. Shear strength is inversely proportional to w_X.

1.1 Scope of the Project:

The project work aims to develop a relationship between shear strength and a parameter called Water Content Ratio, w_X for better prediction of shear strength empirically.

The Purpose of this research is-

1. To investigate the undrained shear Strength of clay samples collected from different regions and of different consistency limits by UU (Unconsolidated Undrained) Triaxial test on Digital Triaxial equipment to examine cohesion (C) and angle of shearing resistance (ϕ) of different soils at different water contents and at OMC.
2. To plot the variation of shear strength with liquidity index and find out the correlation coefficient.
3. To plot the variation of shear strength with water content ratio (w_X) and to develop a suitable relation between them.
4. To compare the correlation between Liquidity Index and Water content ratio for all soils and to determine whether the I_L can be replaced by $w_{X,}$ which eliminates the determination of plastic limit?

1.2 Thesis Outline

The basic focus of this thesis is to find the comparison between w_X and Liquidity Index so that a suitable relation may be developed between shear strength and w_X which will not depend on plastic limit of soils.

Chapter 2 includes an extensive literature review of research on the plastic analysis of soils and relations developed in the earlier researches.

Chapter 3 provides the background of material that was used. The properties of the material used will be discussed in this chapter. Different type of experiments performed will be discussed in this chapter.

Chapter 4 includes detailed discussion on the UU test performed in the laboratory on triaxial test apparatus and its results. The relation between shear strength, W_X and Liquidity Index will be discussed in this Chapter for soils of different origin.

Chapter 5 includes the summary and conclusion of this research presented along with limitations of the project.

CHAPTER 2

LITERATURE REVIEW

In this chapter, earlier research works performed related to the plasticity characteristics of the clays and difficulty in its determination are discussed. Also, the relations established for shear strength determination are discussed.

Schofield and Wroth (1968) evaluated the Plastic Limit determined experimentally from the rolling method and stated that the crumbling of soil in plastic limit is due to tension failure, same as that observed in split-cylindrical tests on concrete. This explains the failure of the thread of soil which cannot be a test of soil strength, tensile or otherwise.

Schofield and Wroth (1968) related the shear strength with Liquidity Index, which is an index property of soil. For this, they examined the CL natural soil and performed laboratory experiment on onshore soil and also examination of vane shear test data from Skempton and Northey (1952). They concluded that the liquid limit and plastic limit correspond approximately to fixed strengths which are in the proposed ratio of 1:100.

$$Cu = 170\ e^{4.6I_L}\ \text{kPa} \qquad (2.1)$$

Leroueil et al. (1983) performed the experiments on CL and CH natural soils in laboratory which had different Liquidity index values ranging from 0.5-2.5 and tried to develop a relation between shear strength, Cu and I_L values.

$$Cu = 1/\ (I_L - 0.21)^2 \qquad (2.2)$$

They Predicted that infinite strength occurs at $I_L = 0.21$ and it cannot be extended beyond this value.

Abraham Bennys Mathew (1993) in his thesis work evaluated the consolidation and shear strength of Cochin marine clays which were highly compressible. CU tests were performed by Triaxial test and showed that the samples grow stiffer with increasing values of confining pressure and the stress-strain curves did not show any clear peak point which makes the evaluation of shear strength difficult for such soils.

Toll (2000) examined that, it is expected for the shear strength to grow with the decrease in water content. In his study, he concluded that clayey materials compacted drier than optimum moisture content behave in a coarser fashion, due to aggregation, that would be justified by the grading. Therefore reduction of water content in clayey soils results in higher friction angle, due to the fact, that clay particles group into aggregates which have larger eff ective particle size.

Lee (2004) in his study, tried to develop a suitable relation between shear strength and other index properties for the better prediction of shear strength. He developed a parameter called water content ratio (w_N/w_L) and shear strength of CL and CH soils which were highly compressible soft soils. He developed an empirical relation which is given by

$$Cu = 8.779\, e^{2.3714\,(w_N/w_L)} \qquad (2.3) \quad (R^2=0.86)$$

Lavasann and Ghazavi (2012) in there paper ,described an experimental investigation conducted to investigate the ultimate bearing ability, the settlement and the two types of closely spaced footings, one having square shapes and the other having circular shapes, on reinforced and without reinforced soil. They performed total number of 20 large-scale model tests on sets of closely spaced circle and square shaped footings placed on reinforced sand and without reinforced sand with more than one layers of geo-grid. From the experimental results they conclude that there is a huge enhancement in the ultimate bearing capability, whereas the settlement and tilting of the interfering footings at the ultimate load increased.

Haigh S.K. (2013), reviewed the original definitions of plastic limit as proposed by Atterberg, and proposes that the brittle failure observed in the plastic limit test is caused by either air entry or cavitations in the clay. He used Critical state soil mechanics to show that the observed range of undrained shear strengths of soils at plastic limit is consistent with this hypothesis. The fallacy that strength at plastic limit is a constant is highlighted and concluded that for the soft saturated clays showed varied strength The plastic limit as measured by a fall cone or static cone apparatus relies on an assigned ratio of strengths between the plastic and liquid limits, and hence is not the plastic limit described by Atterberg (1911).A quantity termed the plastic strength limit, PL_{100}, is suggested for correlations with strength properties, but not for analysis of the water content at which the soil becomes brittle.

Giovanni Spagnoli and Feinendege Martin (2017) investigated 40 natural clayey samples of various mineralogies and origins and other publicly available data, where Atterberg limits and undrained shear strength values obtained with the vane shear tests were given. The liquidity index and water-content ratio correlate very well for defined undrained shear strength values of the clays. Solving the liquidity index equation for the plastic limit, estimated plastic limit values obtained by the liquidity index/water-content ratio relationship were compared with laboratory plastic-limit values. Preliminary results based on 62 values show an exponential trend with a multiple regression coefficient of 0.79. The data need to be confirmed on a larger database, however.

Shridharan et.al (2017) in their experiments, tried to develop a correlation of undrained shear strength values with a parameter water content ratio (w_N/w_L) which eliminates the evaluation of plastic limit which was used in earlier researches to correlate shear strength with liquidity index, which is dubious property due to observation error in thread rolling method and weight error in fall cone test. The strength of various marine clays collected from different regions of cochin were evaluated using vane shear test and correlation was developed with a determination coefficient of 0.989.There test was limited to water content from 98% to 119%

$$\text{Log } C_u = 0.644 - 2.55\, w_N/w_L \qquad (2.4)$$

$$(R^2=0.989)$$

CHAPTER 3

MATERIALS AND METHODS

3.1 Material and Equipments used

Different material and methods used to fulfil the objective of this research work. The material and equipment used are as follows.

3.1.1 Classification of Soil

3.1.1.1 Soil 1

The CH clay classified according to Indian Soil classification System (IS 1498(1970)) with I_L=55% was taken from Gwalior which passed through 4.75 mm IS sieve. A Soil of this soil was properly wrapped in plastic sheets which was wrapped with tape to not to lose the water content which was then tested for the natural water content. The clay should be free from grass roots and other organic material so it was dried in oven for 24 hours.

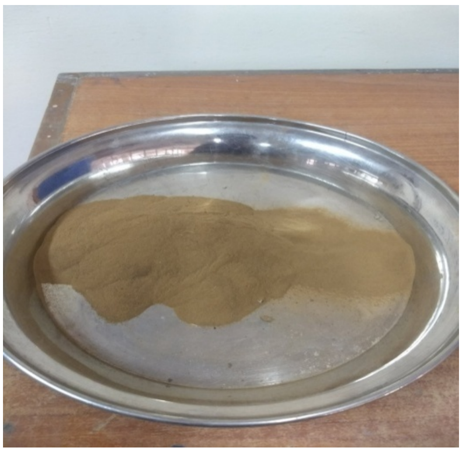

Fig.3.1: Soil 1

3.1.1.2 Soil 2

Black Cotton Soil was collected from Bhopal and was classified according to Indian Soil classification System (IS 1498(1970)) and a Soil of soil which was passed through 4.75 mm sieve. A Soil of this soil was properly wrapped in plastic sheets which was wrapped with tape to not to lose the water content which was then tested for the natural water content. The clay should be free from grass roots and other organic material so it was dried in oven for 24 hours. It was CH clay.

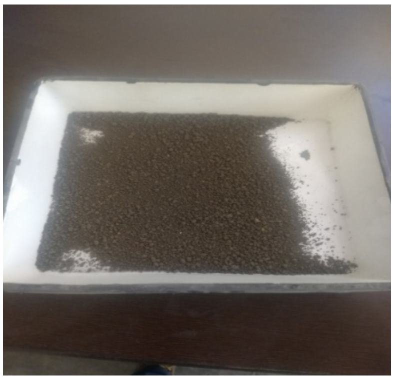

Fig. 3.2: Soil 2

3.1.1.3 Soil 3

CH Clay with w_L=58% was collected from local soil from Rithala, New Delhi. A Soil of this soil was properly wrapped in plastic sheets which was wrapped with tape to not to lose the water content which was then tested for the natural water content. The clay should be free from grass roots and other organic material so it was dried in oven for 24 hours.

Fig. 3.3: Soil No. 3

3.2 Experiments Performed

3.2.1 Consistency Limits

3.2.1.1 Plastic Limit test

Thread rolling Method was used in accordance with IS: 2720 (Part 5) – 1985 (Reaffirmed-2006) to determine the Plastic limit of the soil in which the water content at which soil just crumbles when rolled into a dia. Of 3mm were noted. The rate of rolling was between 60 to 90 strokes per min.

Fig. 3.4: Plastic limit test performed at DTU laboratory

3.2.1.2 Liquid Limit test

Casagrande method is used to determine the Liquid limit of different samples in accordance with IS: 2720 (Part 5) – 1985 (Reaffirmed-2006).

250 gm of each soil passing through 425µ sieve is taken which were placed in cup and was cut by a groove of standard dimensions (11 mm wide at top, 2 mm at bottom, and 8 mm deep). The cup is lifted and dropped by turning crank at a rate of approx. 2 revolutions per sec. no. Of blows required for the soil to flow together by a distance of 12mm is noted. 4

samples of each soil were tested and plot of water content vs. $Log_{10}N$ is plotted and water content corresponding to 25 no. Of blows were noted.

Fig. 3.5: Liquid limit test performed at DTU laboratory

3.2.1.3 Shrinkage Limit test

Shrinkage Limit Test was performed in accordance with IS-2720-PART-6-1972 (Reaffirmed-2001).50 gm of soil passing through 425µ sieve was taken and water was added which was equal to the liquid limit of soil. The soil was placed in evaporating dish and was air dried for about 6 hours until the colour changed from dark to light. Then the soil was placed in oven for about 14-15 hours at 105°C. Mercury was used to find the volume of the soil by mercury displacement method.

Shrinkage Limit was calculated by

$$(w_S) = [w - \{(V-V_d) / w_o\}] * 100 \quad (3.1)$$

w_S = Shrinkage limit of soil

w = water content of soil (%)

V = volume of wet soil pat (cm^3)

V_d = volume of dry soil pat (cm^3)

w_o = weight of oven dried soil (gm)

Fig. 3.6: Shrinkage limit test apparatus at DTU laboratory

3.2.2 Specific Gravity Test

Specific gravity was determined by using Pycnometer method as per IS 2720 part 3.The oven dried Soil of 3 different soils passing through 425 µ sieve were taken to test for their specific gravity.

Fig. 3.7: Pycnometer used for the determination of specific gravity

3.2.3 UU Triaxial Test

Digital Triaxial Apparatus was used for UU test which has Load Frame, Motorised with 50 kN Loading capacity. It can accommodate triaxial cells up to 100mm diameter specimen. Operates on 220V, 50Hz supply. Main component requirement for this test are:

i) Load Frame
ii) Triaxial Cell
iii) Lateral Pressure Assembly
iv) Measuring Unit

UU tests are performed in two stages:

1. Cell Pressure Stage/Confining Pressure stage
2. Deviatric stress stage/Shear stress stage

UU tests were performed at a strain rate of 1.25mm/min. Plot of Deviatric stress vs. axial strain was noted up to 20% axial strain. On one Soil, 3 tests were performed at three different confining pressures of 0.2 kg/cm^2, 0.3 kg/cm^2, 0.4 kg/cm^2. Mohr circle was plotted for the different confining pressures and failure envelope was drawn which gave Cohesion value(c) and angle of internal friction (ϕ) which was used to find out the shear strength of soil (Cu) by the following relation:

$$C_u = c + \sigma_n \tan\phi \qquad (3.2)$$

$$(\sigma_n)_f = \{(\sigma_1+\sigma_3)/2\}_{max.} \qquad (3.3)$$

$(\sigma_n)_f$ = Normal stress on failure plane

σ_1= Major principle stress on failure plane

σ_3= Minor principle stress on failure plane

Area at the time of failure is calculated by:

$$A_f = A_0/(1-\varepsilon_l) \qquad (3.4)$$

A_f = Area at the time of failure

A_0 = Original Area of the specimen

ε_l = Axial strain at the time of failure

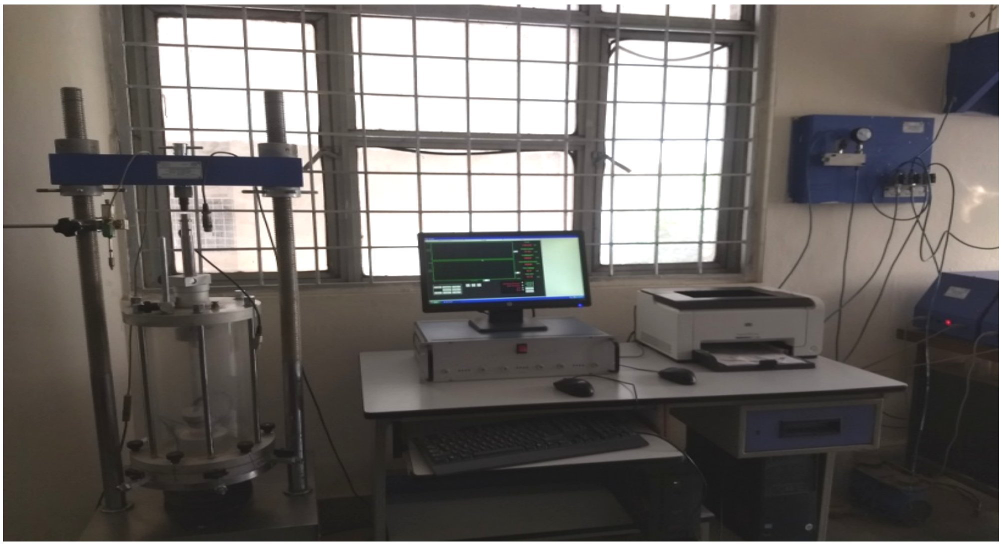

Fig. 3.8: Triaxial Test Apparatus at DTU laboratory

3.2.4 Compaction Characteristics

Standard proctor test was performed as per IS 2720- part 7- 1980 to find out the OMC and γ_d (max.) of soils. Air dried Soil of various soils of known weight passed through 4.75 mm IS sieve mixed at different water content was noted which was determined by oven drying method (IS 2720- part 2- 1980).

The soil was filled in Proctor mould in 3 layers. and then filled in proctor mould in 3 layers.25 number of blows were given to the soil in each layer by rammer of weight 2.6kg which was dropped from 31cm height and was weighed. The dry density was calculated by the mass of compacted soil and its water content. Then the compaction curve was plotted between the dry density and water content. From the compaction curve, OMC and γ_d (max.) can be obtained.

Fig. 3.9 Standard Proctor Test Apparatus at DTU Laboratory

Stated above are the materials and equipment used. Various tests have been performed and discussed in the next chapter.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Results

4.1.1 Soil Classification

All the soils tested were classified according to Indian Soil Classification System (IS 1498(1970)). All samples were Highly Compressible Clays ($W_L > 50$).

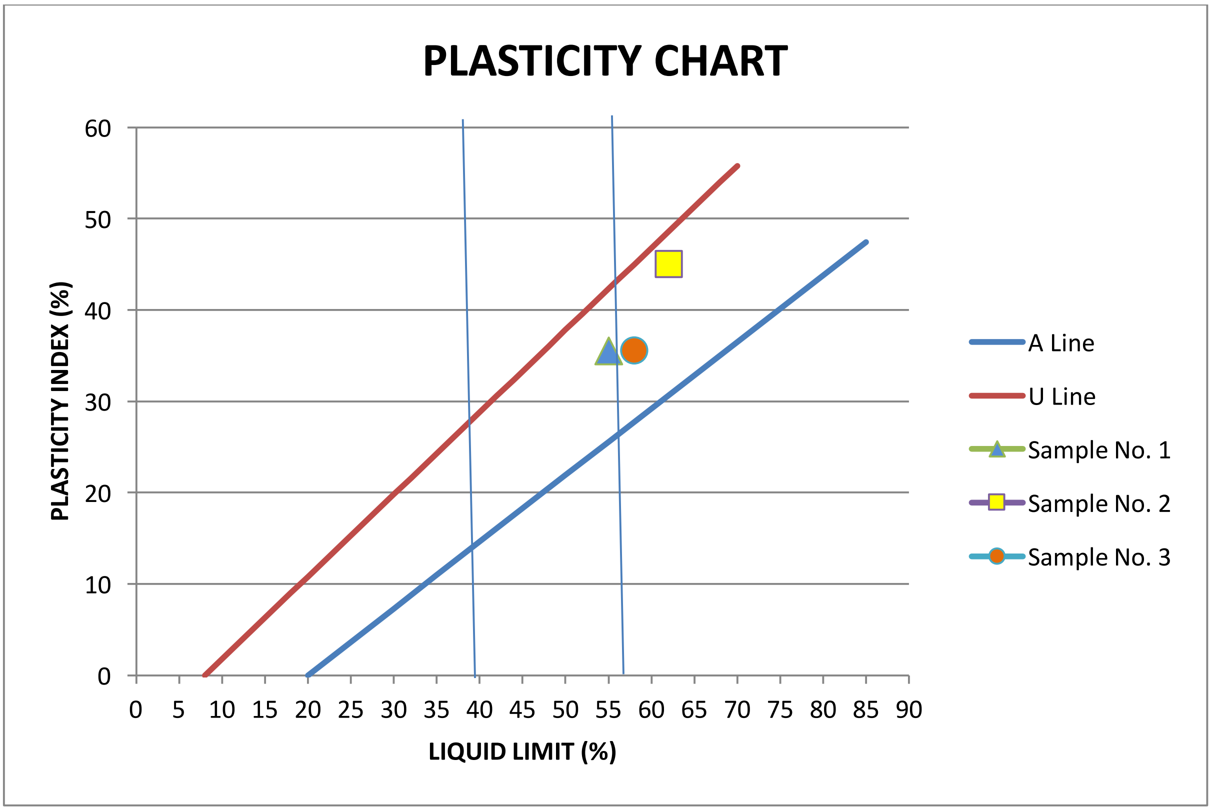

Fig. 4.1 Plasticity Chart

4.1.2 Liquid Limit

All Soils tested were Highly Compressible Clays. (Liquid Limit > 50)

Table- 4.1: Liquid Limit

Soil Classification	Liquid Limit
Soil 1	55
Soil 2	62
Soil 3	58

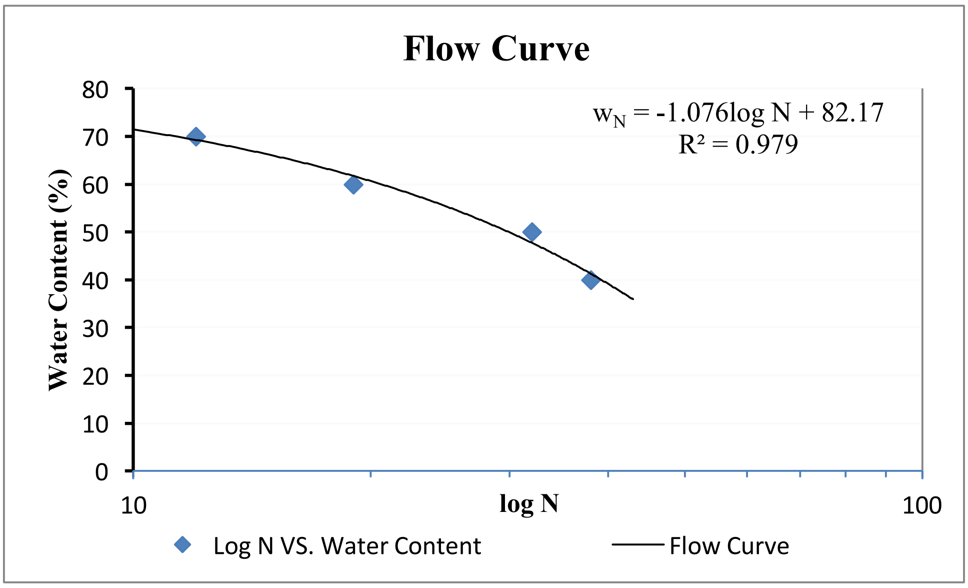

Fig. 4.2 Flow curve Soil 1

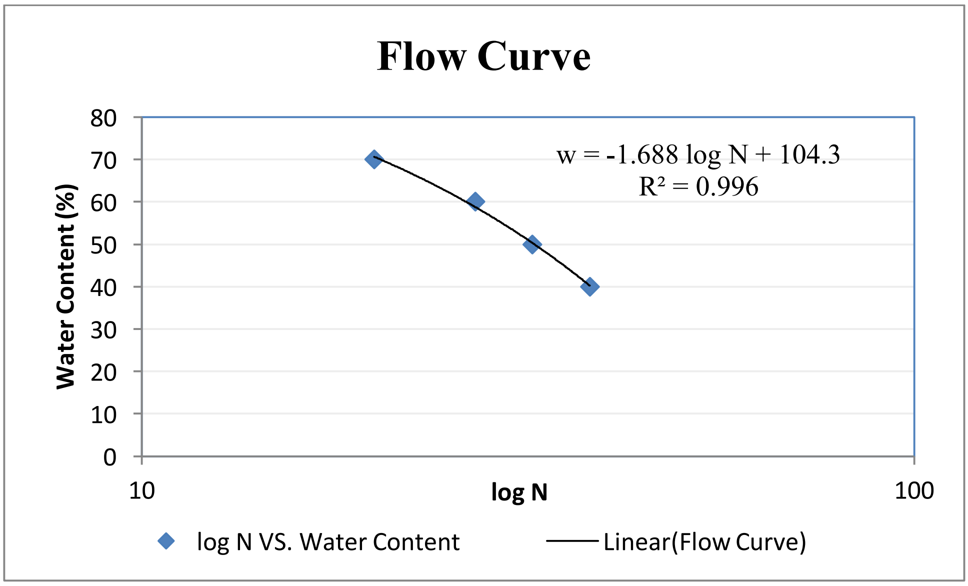

Fig. 4.3 Flow curve Soil 2

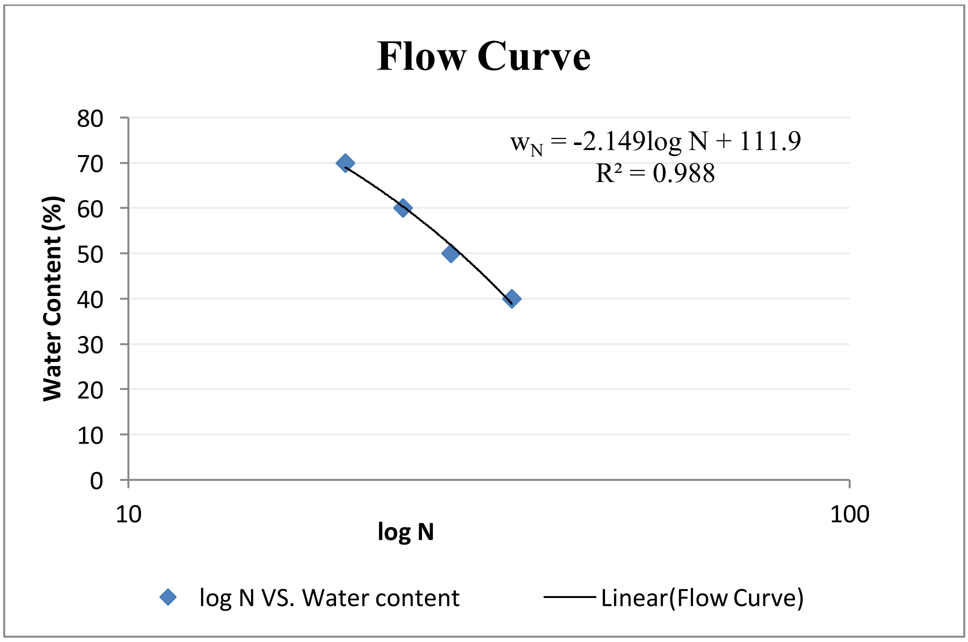

Fig. 4.4 Flow curve Soil 3

4.1.3 Compaction Characteristics

Standard proctor test was performed as per IS 2720- part 7- 1980 to find out the OMC and Y_d (max.) of soils. Air dried Soil of various soils of known weight passed through 4.75 mm IS sieve mixed at different water content was noted which was determined by oven drying method (IS 2720- part 2- 1980).Test results are as follows:

4.1.3.1 Compaction characteristics for Soil 1

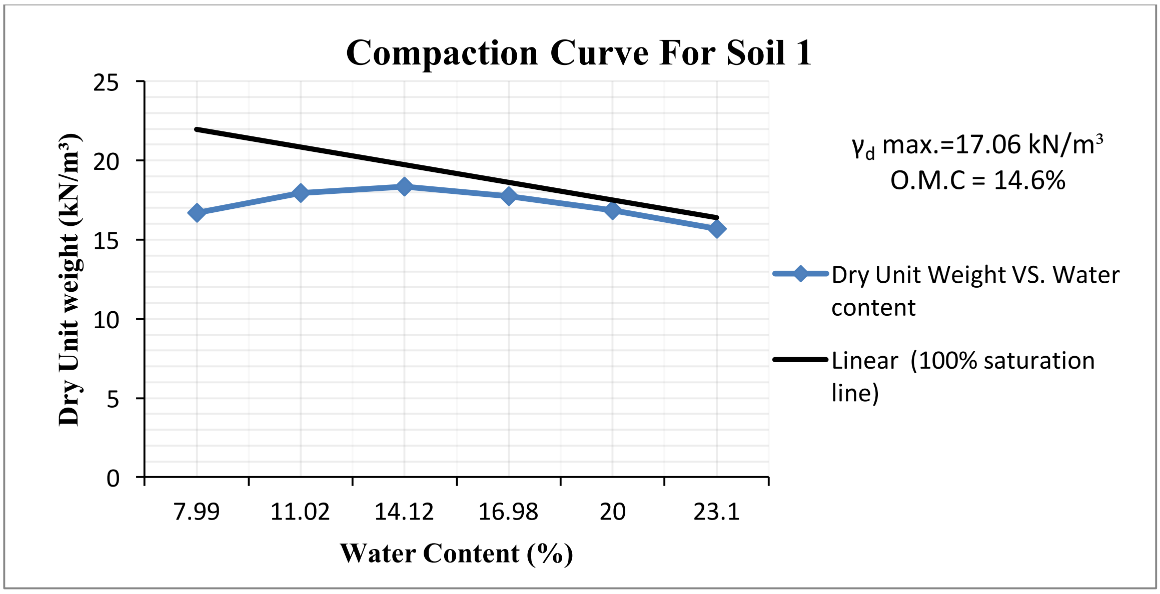

Fig. 4.5 Compaction Curve Soil 1

4.1.3.2 Compaction characteristics for Soil 2

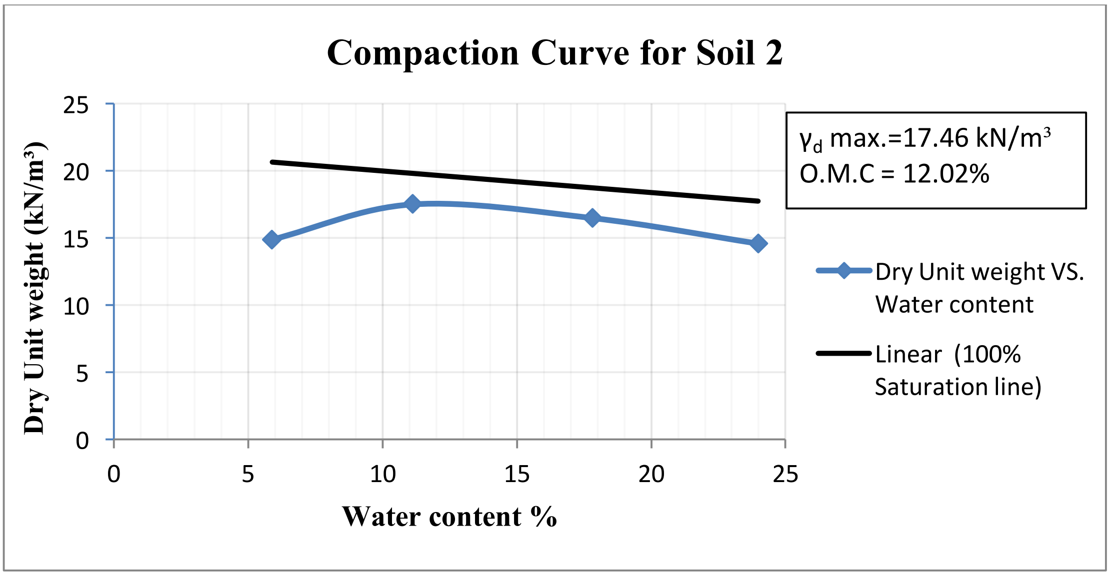

Fig. 4.6 Compaction curve Soil 2

4.1.3.3 Compaction characteristics for Soil 3

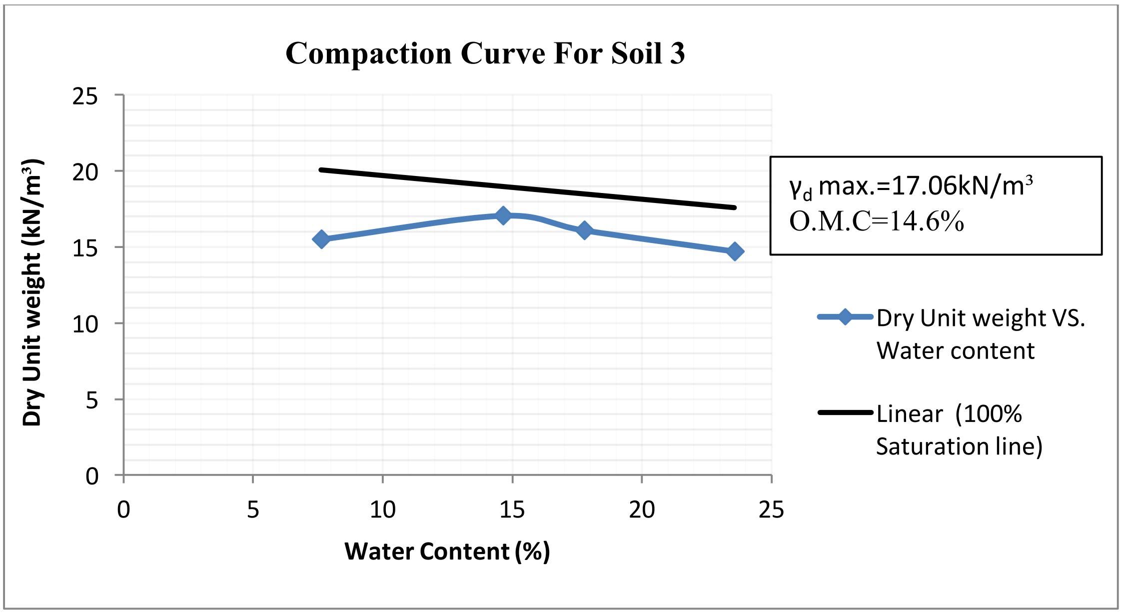

Fig. 4.7 Compaction curve Soil 3

Table 4.2 Basic properties of soils

Properties	Soil 1	Soil 2	Soil 3
Specific Gravity	2.434	2.265	2.231
Liquid Limit (w_L) (%)	55	62	58
Plastic Limit (w_P)%	21	28	22.5
Shrinkage Limit	18	24	20
Plasticity Index (I_P)	34	34	35.5
OMC (%)	14.6	12.02	14.12
Y_d (kN/m³)	17.06	17.46	18.02

4.1.4 Unconsolidated Undrained Triaxial Test

UU Test results of all samples are as follows:

4.1.4.1 UU Test Results for Soil 1

Table 4.3 UU Test Results for Soil 1 at w = 14.6%

<table>
<tr><th colspan="5">SOIL NO. 1
SOIL DESCRIPTION : CH</th></tr>
<tr><td rowspan="7">SOIL DETAILS</td><td>Confining pressure,σ_3(kN/m^2)</td><td>19.61</td><td>29.42</td><td>39.22</td></tr>
<tr><td>Initial Diameter (cm)</td><td>50</td><td>50</td><td>50</td></tr>
<tr><td>Initial Length (cm)</td><td>100</td><td>100</td><td>100</td></tr>
<tr><td>Bulk Unit weight (kN/m^3)</td><td>14.88</td><td>14.88</td><td>14.88</td></tr>
<tr><td>Dry Unit Weight (kN/ m^3)</td><td>17.06</td><td>17.06</td><td>17.06</td></tr>
<tr><td>Moisture content (OMC) (%)</td><td>14.6</td><td>14.6</td><td>14.6</td></tr>
<tr><td>Peak Deviatric Stress,$(\sigma_1-\sigma_3)_f$ (kN/m^2)</td><td>186.32</td><td>188.286</td><td>190.25</td></tr>
<tr><td rowspan="4">TEST RESULTS</td><td>Failure Strain, ε_f (%)</td><td>19.7</td><td>19.7</td><td>21.1</td></tr>
<tr><td>**Cohesion Intercept ,C (kN/m^2)**</td><td colspan="3">82.375</td></tr>
<tr><td>**Angle of internal friction ,ϕ**</td><td colspan="3">3.8</td></tr>
<tr><td>**Shear Strength ,Cu(kN/m^2)**</td><td colspan="3">95.22</td></tr>
</table>

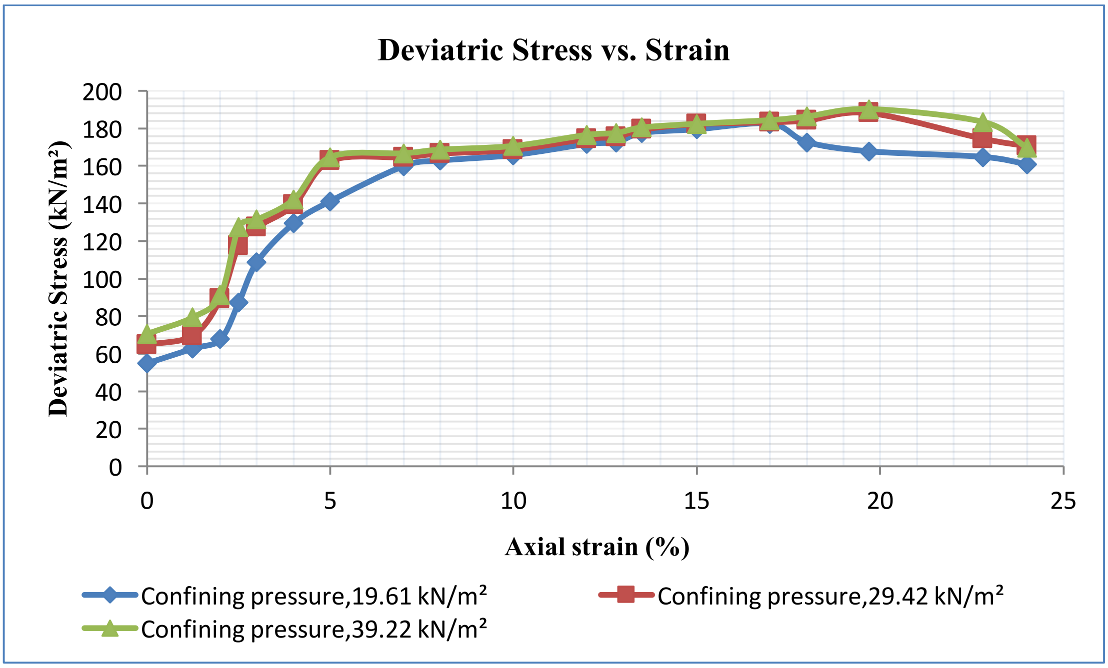

Fig. 4.8 Deviatric Stress VS. Axial strain Soil 1

(1) **(2)**

Fig. 4.9 Soil 1 (1) Before Test

(2) After Test

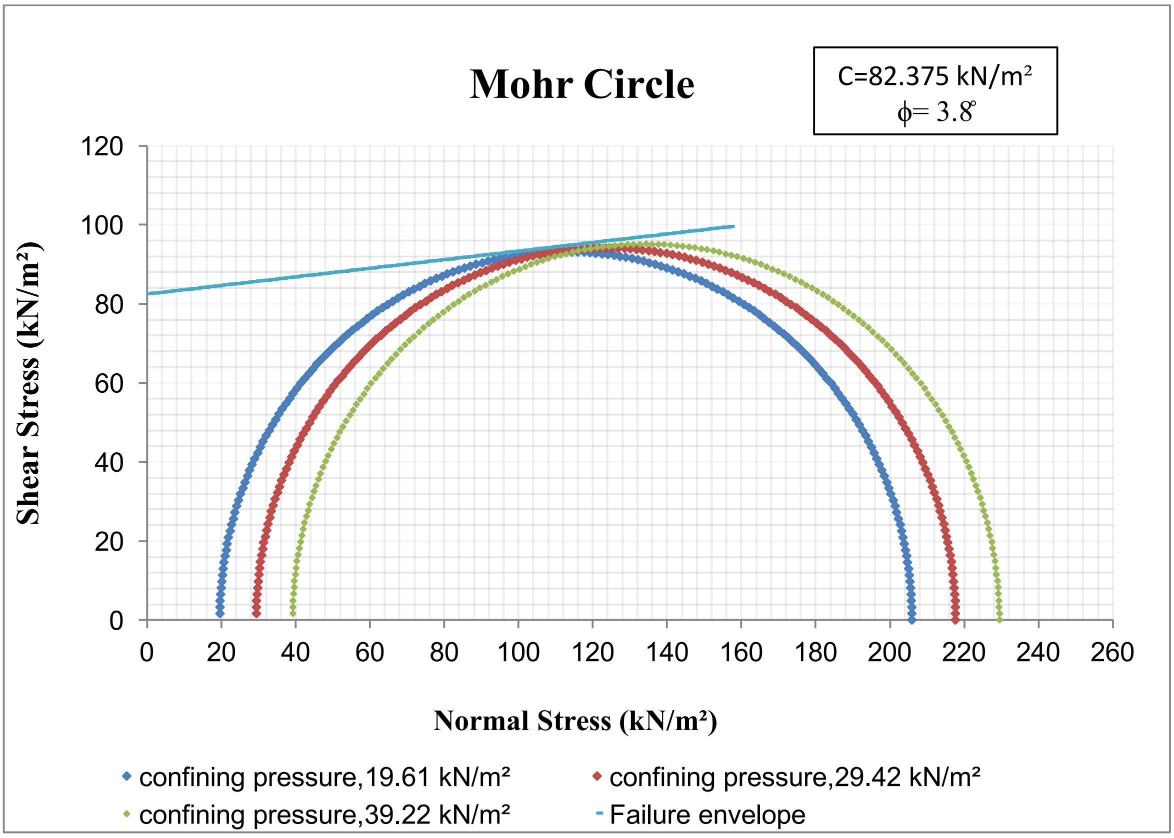

Fig. 4.10 Mohr Circle Soil 1

4.1.4.2 UU Test Results for Soil 2

Table 4.4 UU Test Results for Soil 2 at w = 12.12%

SOIL 2 SOIL DESCRIPTION : CH				
SOIL DETAILS	Confining pressure,σ_3(kN/m^2)	19.61	29.42	39.22
	Initial Diameter (cm)	50	50	50
	Initial Length (cm)	100	100	100
	Bulk Unit weight (kN/m³)	15.23	15.23	15.23
	Dry Unit Weight (kN/ m³)	17.46	17.46	17.46
	Moisture content (OMC) (%)	12.12	12.12	12.12

	Peak Deviatric Stress,$(\sigma_1-\sigma_3)_f$ (kN/m^2)	81.394	84.337	86.298
TEST RESULTS	Failure Strain, ε_{f}(%)	21.2	21.4	22.3
	Cohesion Intercept ,C (kN/m²)	35.3		
	Angle of internal friction ,ϕ	5.9°		
	Shear Strength ,Cu(kN/m²)	43.75		

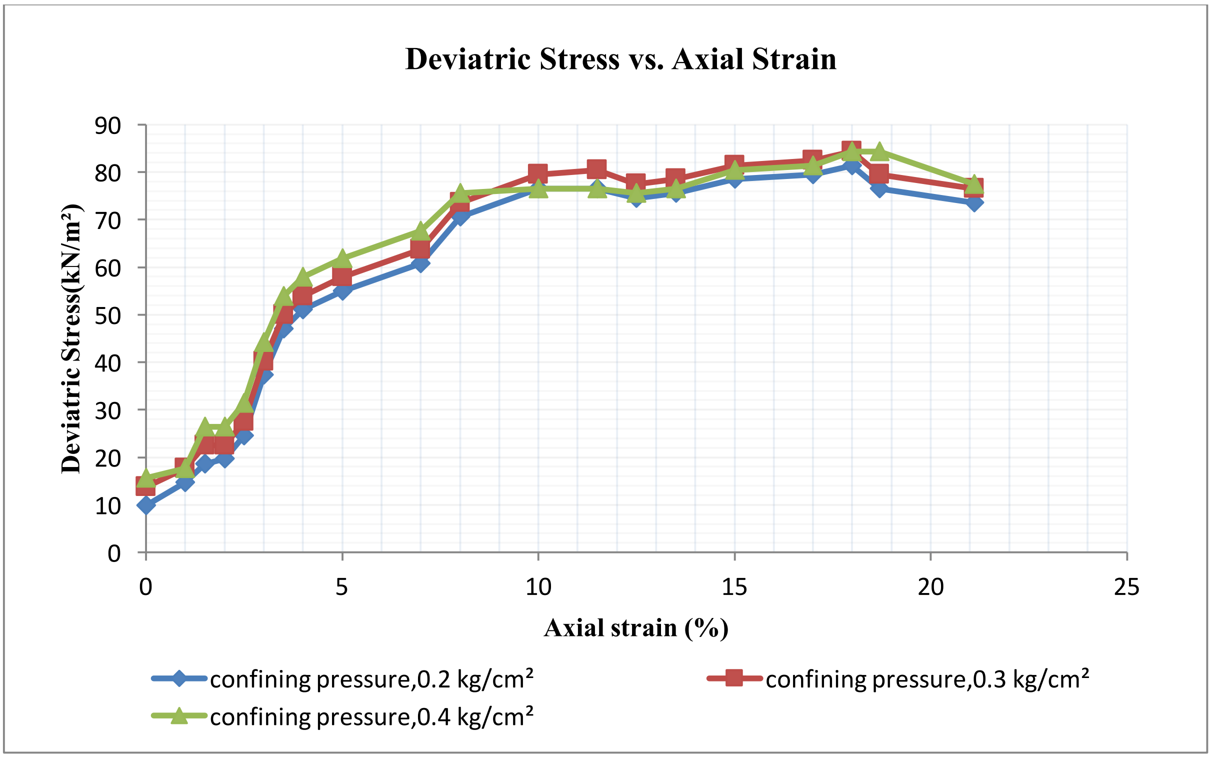

Fig. 4.11 Deviatric Stress VS. Axial strain Soil 2

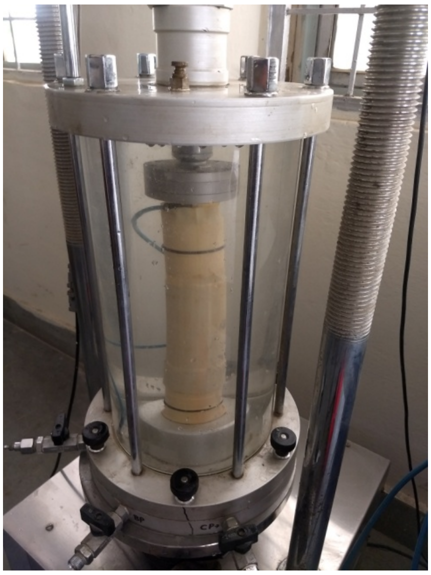

Fig. 4.12 UU Test performed at DTU Laboratory

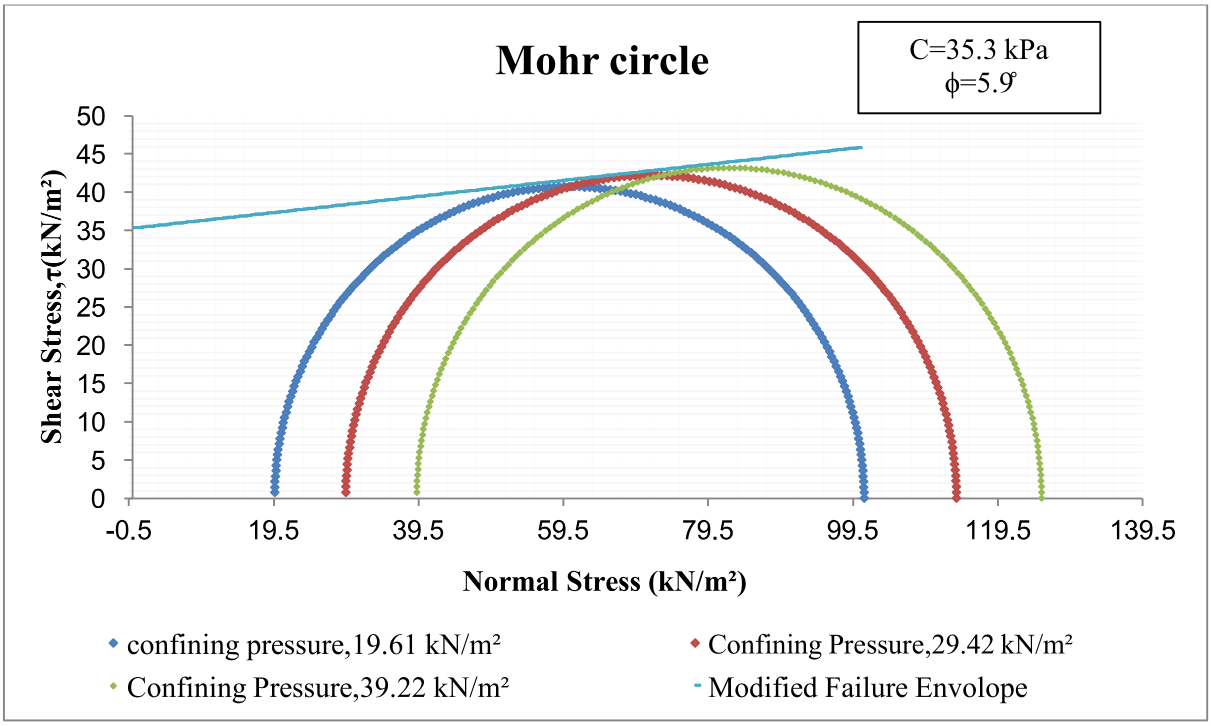

Fig. 4.13 Mohr circle Soil 2

4.1.4.3 UU Test Results for Soil 3

Table 4.5 UU Test Results for Soil 3 at w = 14.12%

SOIL NO. 3 SOIL DESCRIPTION : CH				
SOIL DETAILS	Confining pressure,σ_3(kN/m^2)	19.61	29.42	39.22
	Initial Diameter (cm)	50	50	50
	Initial Length (cm)	100	100	100
	Bulk Unit weight (kN/m^3)	15.23	15.23	15.23
	Dry Unit Weight (kN/ m^3)	18.02	18.02	18.02
	Moisture content (OMC) (%)	14.12	14.12	14.12
	Peak Deviatric Stress,$(\sigma_1-\sigma_3)_f$ (kN/m^2)	156.9	158.867	159.84
TEST RESULTS	Failure Strain, ε_f (%)	19.2	19.6	20
	Cohesion Intercept ,C (kN/m^2)	73.55		
	Angle of internal friction ,ϕ	3.48°		
	Shear Strength ,Cu(kN/m^2)	80.795		

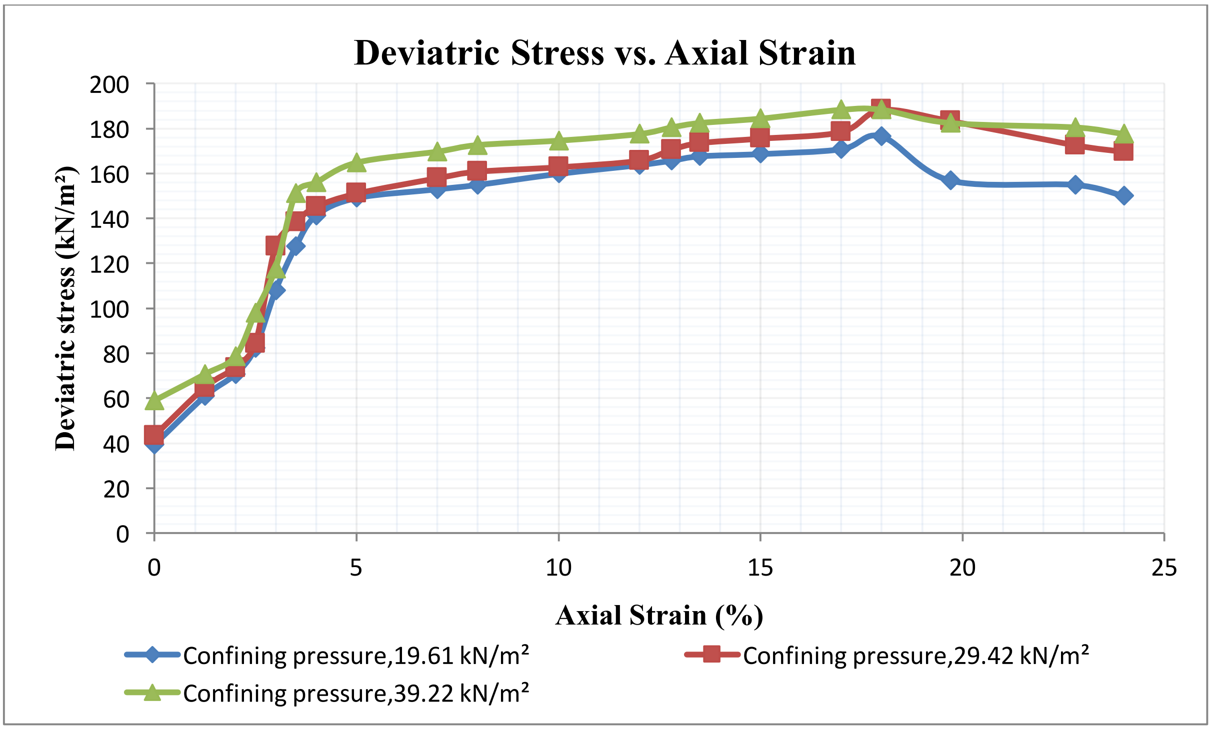

Fig. 4.14 Deviatric stress VS. Axial strain Soil 3

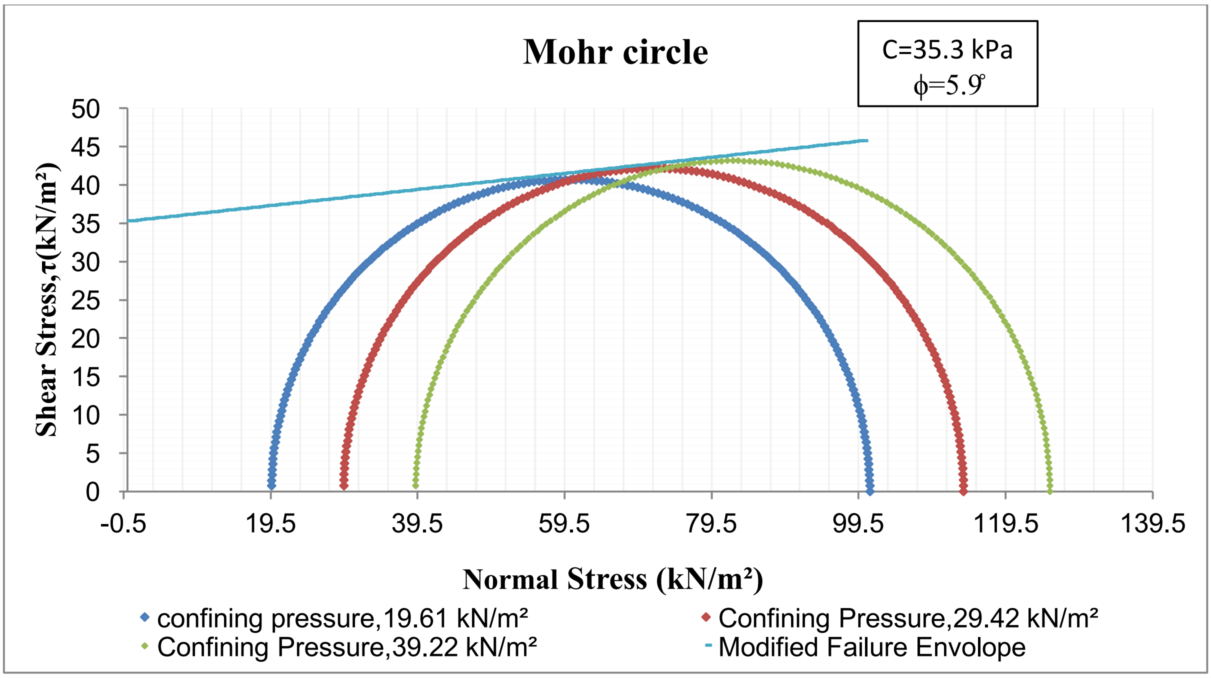

Fig. 4.15 Mohr Circle Soil 3

Fig. 4.16 Soil 3 after failure

4.1.5 Plot of Shear Strength VS. Water content

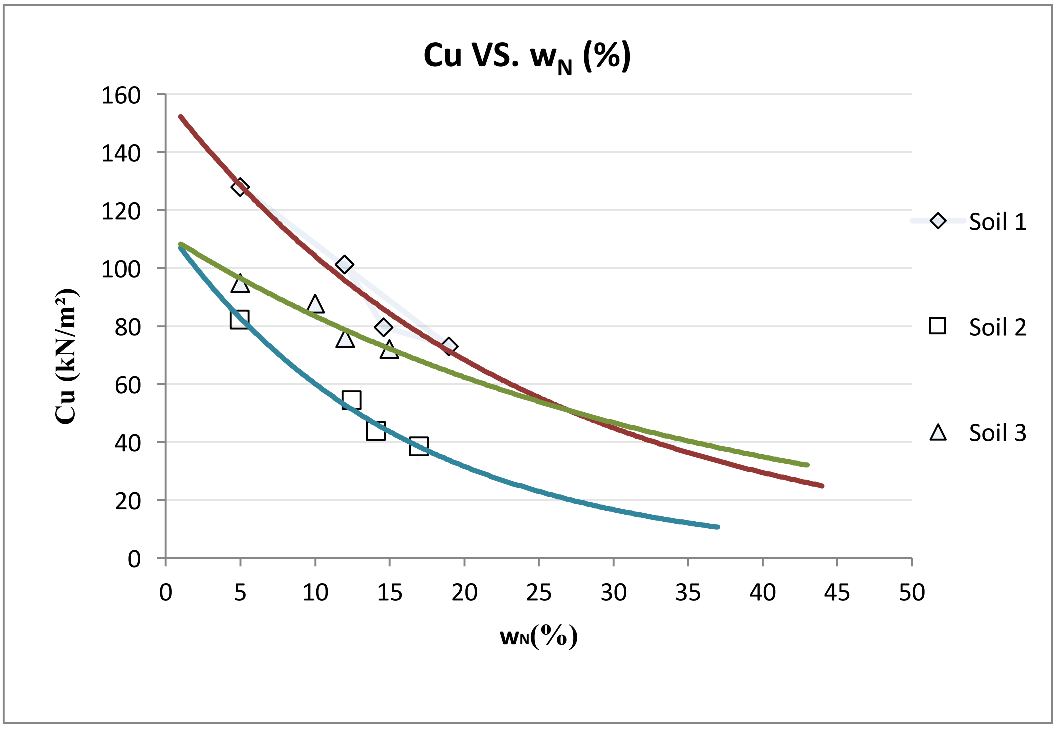

Fig. 4.17 Undrained Shear strength VS. Water content

4.1.6 Plot of Shear Strength VS. Liquidity Index

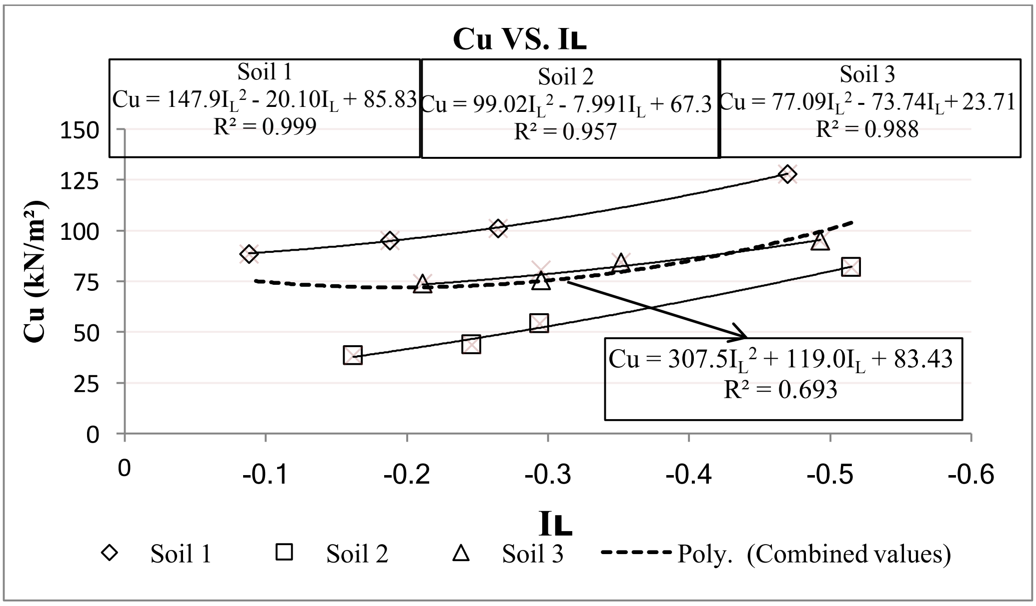

Fig. 4.18 Shear strength VS. Liquidity Index

4.1.7 Plot of Shear Strength VS. w_X

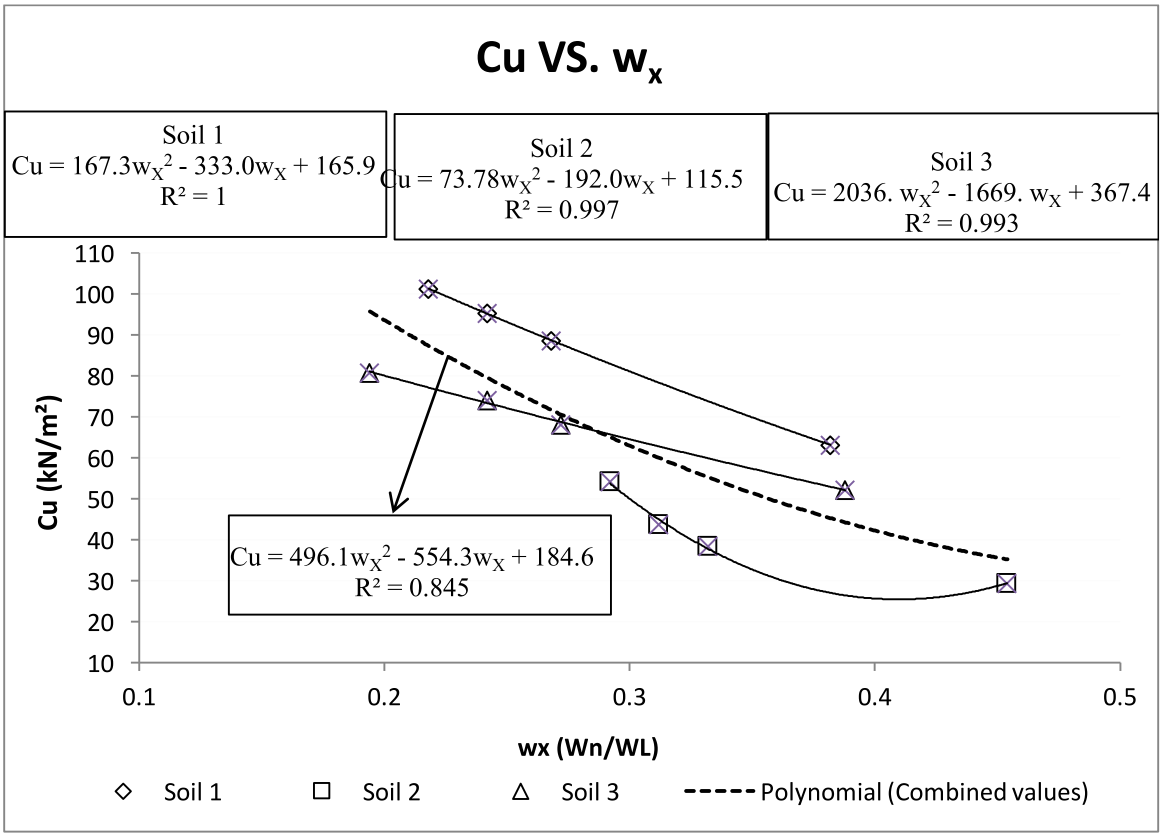

Fig. 4.19 Cu VS. w_X

4.1.8 Plot of Liquidity Index VS. w_X

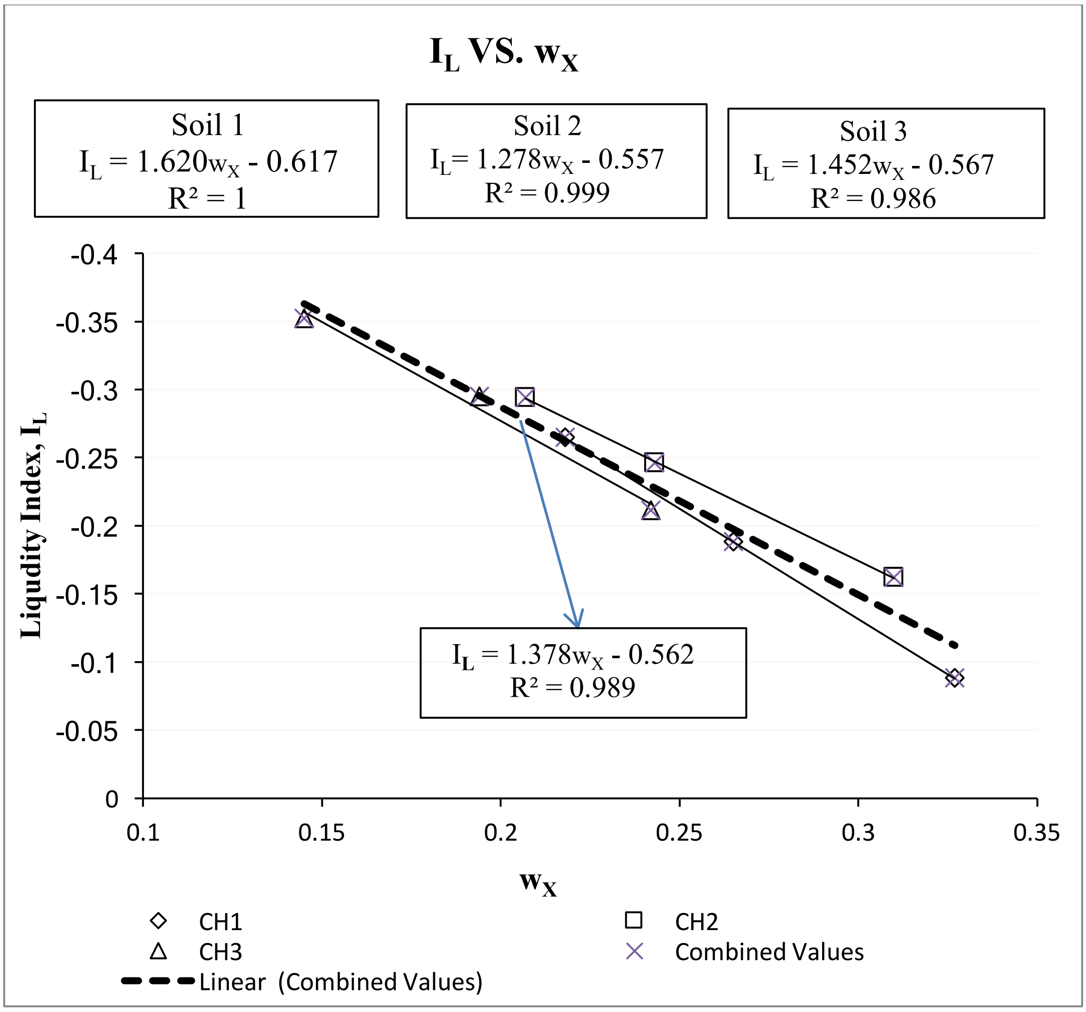

Fig. 4.20 Liquidity Index VS. w_X

4.2 Discussions

1. Fig. 4.17 suggests that Shear strength is inversely proportional to water content. However no clear relation can be estimated if only water content is taken as basis.
2. Fig. 4.18 shows the plot of three samples tested at different Liquidity Index values and for the combined values of all soils. Plot suggests that shear strength can be related with Liquidity index by equation:

$$Cu = a' I_L^2 - b' I_L + c' \quad (4.1)$$

where, a', b' and c' are constants which depends on type of soil.

The relation exists well for a soil of similar geological origin but not for different geological origin. However the above data contains only negative values of Liquidity indices which need to be verified with positive values of liquidity indices which will occur at higher water content and would require Vane shear test for shear strength calculation.

3. Fig. 4.19 shows the plot of three samples tested at different w_X values and for the combined values of the all soils. It suggests that shear strength can be related with Water content ratio (w_X) by equation:

$$Cu = a\, w_X^2 + b\, w_X + c \quad (4.2)$$

Where a, b and c are constants which depends on type of soil. The relation exists well for a soil of similar geological origin but not for different geological origin.

4. Fig. 4.20 suggests that liquidity index relates well with w_X irrespective of the geological origin of soil. Therefore, w_X can be replaced by Liquidity Index to find out shear strength of soils, as it eliminates the determination of Plastic limit of soils.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE

5.1 Conclusions

1. Shear Strength correlates well with the I_L for the soils of same origin and follows a relation given by :

$$Cu = a' I_L^2 – b' I_L + c' \qquad (5.1)$$

   Where a', b' and c' are constants which depends on type of soil.
2. Shear strength correlates well with the w_X of soils of same origin and follows a relation which is given by :

$$Cu = a\, w_X^{\,2} + b\, w_X + c \qquad (5.2)$$

   Where a, b and c are constants which depends on type of soils.
3. Definite relationship exists between I_L and w_X irrespective of the geological origin of soil. Therefore, I_L can be substituted by w_X which eliminates the determination of plastic limit of soils.

5.2 Recommendations for Future

1. Schofield and Wroth (1968) made the analysis that plastic limit calculation by crumbling implies tensile failure, like split cylinder tests in concrete but the split cylinder applies the restricted stress to the Soil by load cell or other devices. And that is not the same with stress applied though hands, which cannot be controlled .that makes its calculation unreliable .so any method, which eliminates the use of plastic limit to determine shear strength would be of greater advantage.
2. In this test, membrane effect is not considered in the UU tests which may slightly vary the results otherwise. Hence, membrane effect shall be taken into consideration for better results.
3. In the earlier research work performed by A.Shridharan et. al on Cochin marine clays gave the following equation:

$$\text{Log } Cu=0.644-2.55w_X \quad (R^2=0.989) \qquad (5.3)$$

However they tested the soils having water content between 98% to 119% and it gave good results than the above analysis. Also, they used Vane shear test data but UU tests were performed for this project work. So , for the combined analysis of soils of same origin, for less saturated soils, UU tests shall be performed and for highly saturated soils ,Vane shear test data shall be taken to determine the shear strength of soils.

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