



# **Investigation on Engineering Properties of Soils Found in Dangila**

**A thesis submitted to  
Addis Ababa institute of technology**

**In partial fulfillment of the requirements  
for the Degree of Masters of Science in Civil Engineering  
(GEOTECHNIQUES)**

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**December, 2014**

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## **Declaration**

I, the undersigned, declare that this thesis is my work and that all sources of material used for the thesis have been duly acknowledged.

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## **Acknowledgement**

I thank the Almighty God above all because he hath dealt bountifully with me. I wish to express my deepest gratitude to my advisor, Dr. Samuel Tadesse, for his guidance and advice to the realization of this thesis. I would like to thank the Municipality of Dangila town for cooperation in surveying the location of test pits.

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Symbols	Designation	Units
LL	Liquid limit	%
PL	Plastic limit	%
PI	Plasticity Index	%
LS	Linear shrinkage	%
$G_s$	Specific gravity	
W	Moisture content	%
A	Cross sectional area	$m^2$
$a_v$	Coefficient of compressibility	$m^2/N$
C	Cohesion	$N/m^2$
$E_s$	Modulus of compressibility	$N/m^2$
$C_c$	Compression index	
e	void ratio	
K	Coefficient of permeability	m/sec
$m_v$	Coefficient of volume compressibility	$m^2/N$
S	Saturation	---
P	Pressure	$N/m^2$
OMC	Optimum moisture content for compaction test	%
$\phi$	Angle of internal friction	deg
$\gamma$	Unit weight	$N/m^3$
ASTM	American Society for testing and Materials	---
BS	British Standard	---
UCS	Unconfined compression strength	$kN/m^2$

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### Abstract

The town of Dangila, having adequate land area for expansion will have a high potential for future development. Due to rapid development of the town and appearance of construction works, the characteristics of the soil are important to be investigated. The aim of this present research is to investigate the engineering properties i.e. index properties, compressibility characteristics and shear strength properties of soils of natural soils. A laboratory research was conducted to determine such properties.

The USCS classification reveals that Dangila soils are classified as MH or CH. The PI values lie between 24.8 and 44.9. The Specific gravity results range between 2.61 and 2.9. For the Consolidation tests, Coefficient of Consolidation varies between  $0.94 \times 10^{-3}$  and  $3.6 \times 10^{-3}$  cm<sup>2</sup>/sec, While the Coefficient of Permeability ranges between  $1.8 \times 10^{-9}$  and  $9.89 \times 10^{-8}$  cm/sec. The compression indexes of undisturbed samples are calculated and the values of  $C_c$  and  $C_s$  are in the range of 0.13 - 0.22 and 0.019 - 0.033 respectively. The values of unconfined compressive strength lie from 140 to 200 kN/m<sup>2</sup>, at corresponding moisture content of 30 to 34 %. With those values, it is possible to evaluate classification indices and estimate relevant engineering properties necessary for foundation design.

Key words: Natural soils; index and compressibility characteristics, shear strength properties, classification indices, foundation design

# 1. Introduction

## 1.1. General

Dangila, which is located 485 kms from Addis Ababa, is the area where the research is conducted. It is found 78 kms from the regional capital Bahirdar. Geographically; it is situated at [Coordinates: 11°6'N36°50'E 11.267°N 36.833°E](#). Dangilawas founded in 1935 as the main trade center of the area [1]. The topography of the town is mainly flat with some small hills on its North and South sides. The existing Master plan of Dangila dates back to 1990's. Since, then, revisions in the detail master plan haven't been made. The area of the town today is extended in all directions. The town has a potential for expansion in all directions.



Fig1.1 Location of Dangila on Ethiopian map (1)

The town has no sewerage system. No systematic soil investigation has been carried out prior to this work. A thorough and comprehensive geotechnical investigation is an essential requirement to the design and construction of civil engineering projects. The proper design of civil engineering structures like foundation of buildings, retaining walls, high ways, etc. requires adequate knowledge of sub surface conditions at the sites of the structures. Many damages to buildings, roads and other structures founded on soils are due to the lack of proper investigation of substructure condition.

## **1.2. Background information**

Unfortunately, very little information is obtained on the background information about engineering properties of Dangila soils. No systematic soil investigation has been carried out prior to this work. Many related researches conducted till now referred only in the other areas such as Addis Ababa, Bahirdar, etc. In all the above researches, the spatial variation of engineering properties of red clay soils has been clearly analyzed.

## **1.3 Objective**

### **General objective**

The general objectives of this thesis are to determine the engineering properties of soils in the a

### **Specific objectives**

- To determine Index properties of soils
- To determine compressibility characteristics of soils
- To determine unconfined shear strength of soils

## **1.4. Methodology**

### **1.4.1. Site investigation.**

A detailed field investigation was performed on the site. The area is relatively flat. Test pits are based on Soil type inspection. Subsurface exploration was performed digging test pits up to 3 depth beneath existing natural ground level. For sample preparation undisturbed samples are sealed and covered with plastic bags immediately to prevent moisture exchange with the atmosphere. 7 undisturbed samples were prepared for consolidation and unconfined compression tests, while disturbed samples were air dried and prepared for Index tests samples.

### **1.4.2. Laboratory tests**

Samples retrieved from the test pits were transported to Geotechnical Engineering laboratory Addis Ababa Institute of Technology. After sample preparation, following laboratory tests were carried out. These tests are index property testing, consolidation and unconfined compression tests. Index property tests performed on selected samples included grain size analyses, water contents, Atterberg Limits. All tests were conducted according to ASTM standard. Using laboratory results it is possible to evaluate classification indices and estimate relevant engineering properties necessary for foundation design.

## **1.5. Organization of thesis**

The thesis has been divided into six Chapters. In this introductory Chapter the background, objective and summary of the thesis work is presented. The second Chapter gives a brief literature review which discusses about clay mineralogy and origin and mineralogical composition of Ethiopian red clay soils. Sampling area description and in situ properties of the natural soil are dealt with in Chapter three. In the fourth Chapter the types of laboratory tests conducted with detail analysis of results are described. The fifth Chapter discusses interpretation and summary of laboratory test results. The last Chapter consists of conclusions and recommendations drawn from the research.

## **2. Literature review**

### **2.1. Soil**

Soil includes all earth materials, organic and inorganic, occurring in the zone overlying the rock crust. Soil is defined as a natural aggregate of mineral grains, with or without organic constituents. The behavior of a structure depends upon the properties of the soil materials on which the structure rests [20]. The study of engineering behavior of the different types of soil is extremely important to civil engineers, because every engineering structure such as a building, a road, a bridge, a deposit is founded on soil.

#### **2.1.1. Formation of soils**

Soils are formed by the process of weathering of the parent rock. The weathering of the rocks might be by mechanical disintegration, and/or chemical decomposition. The properties of the soil materials depend upon the properties of the rocks from which they are derived.

##### **I. Mechanical Weathering**

Mechanical weathering of rocks to smaller particles is due to the action of such agents as the expansive forces of freezing water in fissures, due to sudden changes of temperature or due to the abrasion of rock by moving water or glaciers. Temperature changes of sufficient amplitude and frequency bring about changes in the volume of the rocks in the superficial layers of the earth's crust in terms of expansion and contraction. Such a volume change sets up tensile and shear stresses in the rock ultimately leading to the fracture of even large rocks. This type of rock weathering takes place in a very significant manner in arid climates where free, extreme atmospheric radiation brings about considerable variation in temperature at sunrise and sunset. Erosion by wind and rain is a very important factor and a continuing event. Cracking forces by growing plants and roots in voids and crevasses of rock can force fragments apart.

## **II. Chemical Weathering**

Chemical weathering (decomposition) can transform hard rock minerals into soft, easily erodible matter. The principal types of decomposition are hydration, oxidation, carbonation, desilication and leaching. Oxygen and carbon dioxide which are always present in the air readily combine with the elements of rock in the presence of water.

### **2.1.2. General types of soils**

The individual size of the constituent parts of even the weathered rock might range from the smallest state (colloidal) to the largest possible (boulders).

#### **2.1.2.1. Residual and Transported Soils**

On the basis of origin of their constituents, soils can be divided into two large groups:

1. Residual soils, and
2. Transported soils.

Residual soils are those that remain at the place of their formation as a result of the weathering of parent rocks. The depth of residual soils depends primarily on climatic conditions and the time of exposure. In some areas, this depth might be considerable. In temperate zones residual soils are commonly stiff and stable. An important characteristic of residual soil is that the sizes of grains are indefinite. For example, when a residual sample is sieved, the amount passing any given sieve size depends greatly on the time and energy expended in shaking, because of the partially disintegrated condition.

Transported soils are soils that are found at locations far removed from their place of formation. The transporting agencies of such soils are glaciers, wind and water. The soils are named according to the mode of transportation. Alluvial soils are those that have been transported by running water. The soils that have been deposited in quiet lakes are lacustrine soils. Marine soils are those deposited in sea water. The soils transported and deposited by wind are aeolian soils. Those deposited primarily through the action of gravitational force, as in landslides, are colluvial soils. Glacial soils are those deposited by glaciers. Many of these transported soils are loose and

soft to a depth of several hundred feet. Therefore, difficulties with foundations and other types of construction are generally associated with transported soils.

### **2.1.2.2. Organic and Inorganic Soils**

Soils in general are further classified as organic or inorganic. Soils of organic origin are chiefly formed either by growth and subsequent decay of plants such as peat, or by the accumulation of fragments of the inorganic skeletons or shells of organisms. Hence a soil of organic origin can be either organic or inorganic. The term organic soil ordinarily refers to a transported soil consisting of the products of rock weathering with a more or less conspicuous admixture of decayed vegetable matter.

### **2.1.2.3. Classification based on Grain size**

According to their grain size, soil particles are classified as cobbles, gravel, sand, silt and clay. Grains having diameters in the range of 4.75 to 76.2 mm are called gravel. If the grains are visible to the naked eye, but are less than about 4.75 mm in size the soil is described as sand. The lower limit of visibility of grains for the naked eyes is about 0.075 mm. Soil grains ranging from 0.075 to 0.002 mm are termed as silt and those that are finer than 0.002 mm as clay. This classification is purely based on size which does not indicate the properties of fine grained materials.

### **2.1.3. Clays**

The term clay is applied to the fraction of grains whose equivalent diameter is less than 0.002mm. The individual grains are fragments of a single mineral i.e. a solid compound with a definite chemical composition and unique crystalline structure.

#### **2.1.3.1. Investigation on Clay mineralogy**

The minerals of clays are formed by the weathering of rocks. Most clay minerals of interest to geotechnical engineers are composed of oxygen and silicon- two of the most abundant elements on earth. Silicates are a group of minerals with a structural unit called the silica tetrahedron. A central silica cation is surrounded by four oxygen anions, one at each corner of the tetrahedron (Fig 2.1a). Silica tetrahedrons combine to form sheets, called silicate sheets (Fig 2.1b). Silicate

sheets may contain other structural units such as alumina sheets. Alumina sheets are formed by a combination of alumina minerals, which consist of an aluminum ion surrounded by six oxygen or hydroxyl atoms in an octahedron (Fig 2.1d) [8]. The main groups of clay crystalline materials that make up clays are the minerals Kaolinite, illite and montmorillonite.

### **I. Kaolinite**

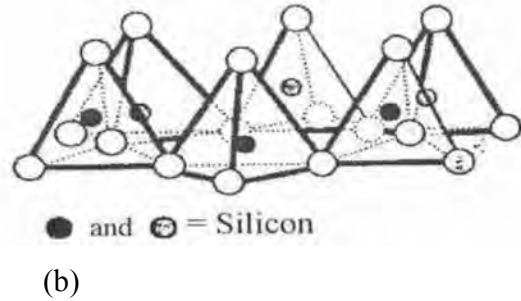
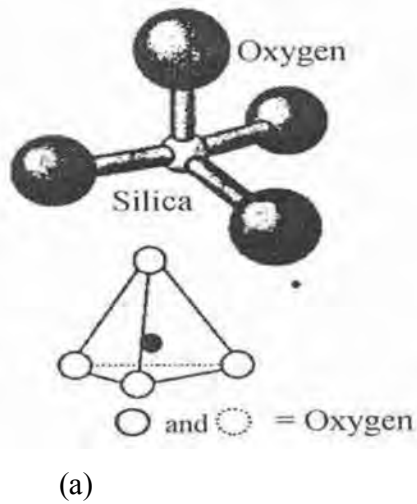
Kaolinite has a structure that consists of one silica sheet and one alumina sheet bonded together in to a layer about 0.72nm (nm = 10<sup>-9</sup>m) thick and stacked repeatedly (Fig 2.2a). The layers are held together by hydrogen bonds. Kaolinite has a few or no exchangeable cation, and the interlayer bonds are relatively strong preventing any hydration between layers and allowing many layers to build up [10]. Kaolinite is relatively stable and water is unable to penetrate between the layers. Consequently Kaolinite shows little swelling on wetting [11]. Kaolinites are found in soils that have undergone considerable weathering in warm, moist climates. They have low liquid limit and a low activity. Another member of the Kaolinite group appearing in some tropical soils is called halloysite, in which water molecules separate the layers. The halloysites are distinguished by one additional water molecule to the basic kaolinite. In contrast to most other clays, which are flaky, halloysite particles are tabular or rod like.

### **II. Montmorillonites**

Montmorillonites are made up of sheet like unit comprising an alumina octahedral sheet between two silica tetrahedral sheets, as shown in Fig. 2.2(c). As the electrons rotate around the nucleus of an atom there will be times when there are more electrons on one side of the atom than the other, giving rise to a weak instantaneous dipole. Weak Vander Waals forces hold layers together and the bonding of these sheets is rather weak, resulting in a rather unstable mineral, especially when wet. In fact, montmorillonite display a significant affinity for water, with subsequent swelling and expansion. Its excessive swelling capacity may seriously endanger the stability of overlying structures and road pavements. Bentonite is part of the montmorillonite clay family, usually formed from the weathering of volcanic ash [8].

### III. Illite

The illites are somewhat similar to montmorillonites in the structural units, but are different in their chemical composition. In illite, the layers are separated by potassium ion, where as in montmorillonite the layers are separated by loosely held water and exchangeable metallic ions (Fig 2.2 (b)). Unlike montmorillonite particles, which are extremely small and have a great affinity for water, the illite particles will normally aggregate and there by develop less affinity for water than montmorillonites. Correspondingly, their expansion properties are less. The cation exchange capacity of illite is less than that of montmorillonite. The inner layer bonding by the potassium ions is sufficiently strong. Illites usually occur as a very small, flaky particles mixed with other clay and non-clay materials [6].



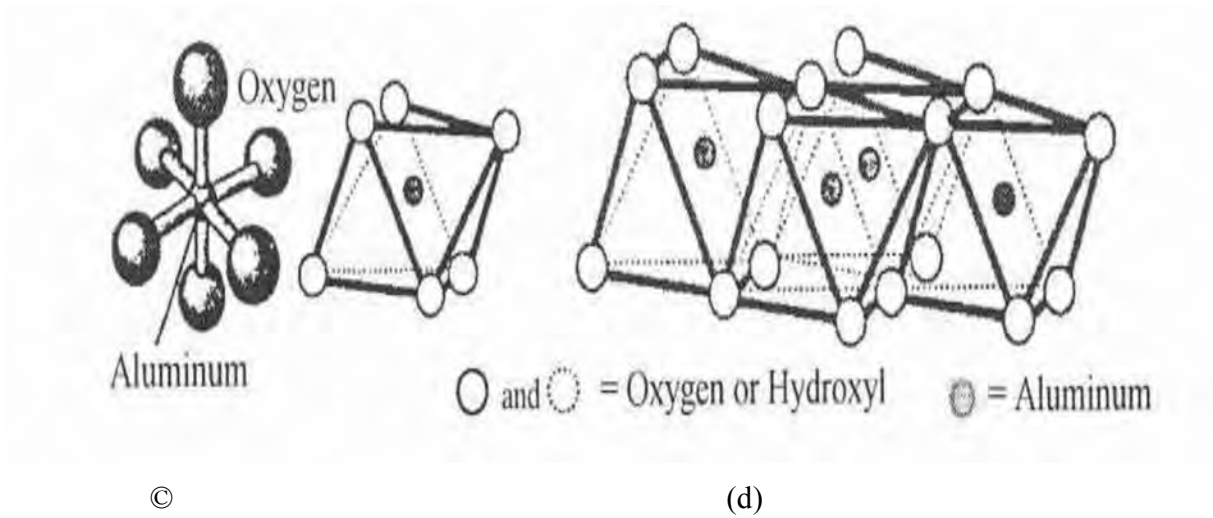


Fig. 2.1(a) A silica tetrahedron, (b) Silica sheets, (c) An Aluminumoctahedron, and (d) Aluminum sheets.

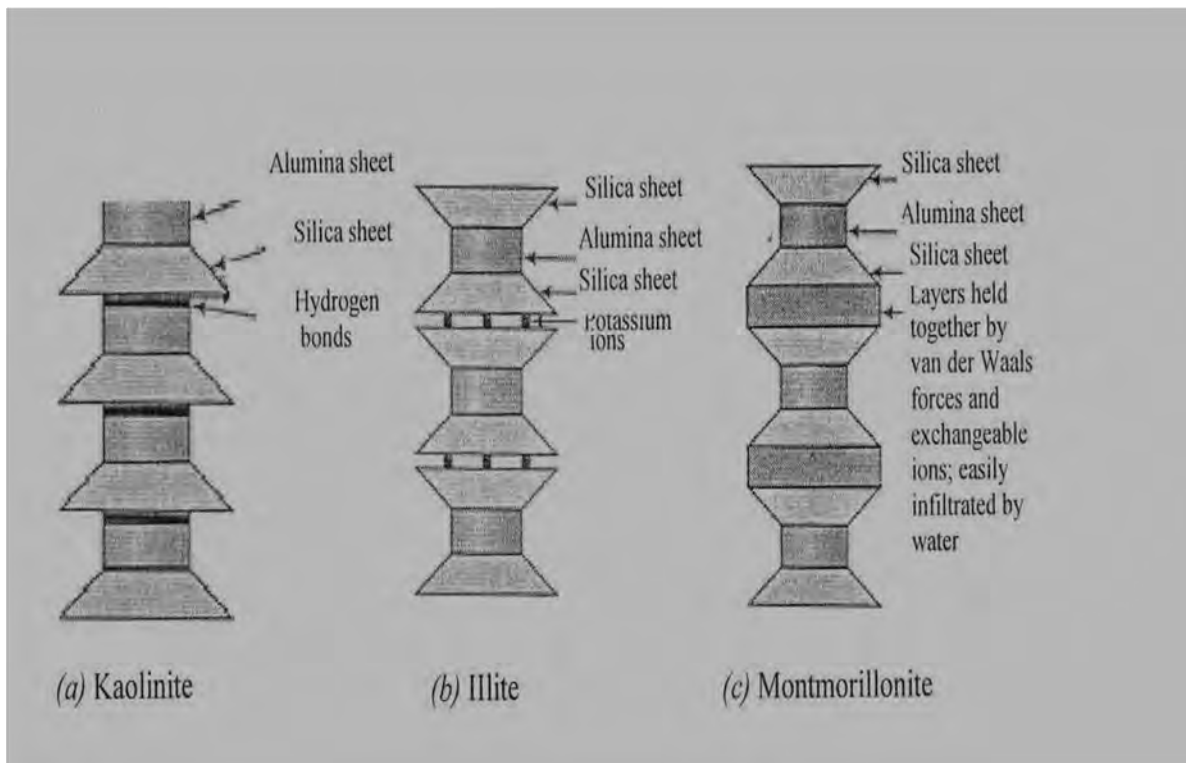


Fig. 2.2 Structure of Kaolinite, illite, and montmorillonite

## 2.2. Ethiopian clay soils

Soil mineralogy can be assessed in various ways. Very specialized techniques have been developed for practical purposes, but the most common approaches are x-ray diffraction, thermo gravimetry, and optical microscopy combined with some form of spectral element identifications. Mineralogical identification using these techniques requires specialized training and procedures. The X- ray diffraction (XRD) technique is by far the most widely used, but is only appropriate for minerals with distinctive crystallography [3]. Since 1923, X -ray analysis became the most widely used method for the investigation of clay minerals [23]. Kaolinite and halloysite are the predominant clay minerals of red soils in Ethiopia [9].

Table 2.1 Ranges in properties of Ethiopian soils according to Morin and Parry [9]

	<b>Black clays</b>	<b>Red clays</b>	<b>Other clays</b>	<b>Lacustrine soils</b>
Parent rock	Olivine basalt, basalt, trachyte	Olivine basalt, basalt, trachyte	Olivinebasalt, basalttrachyte, ignimbrite, etc	
Rainfall (in/year)	9-53	48 - 92	0.4 - 48	8-41
Temperature ( <sup>0</sup> F)	54 - 68	57 - 68	62 - 82	66 - 80
Drainage	Poor to good	Fair to good	Poor to good	Poor
Principal clay minerals	Montmorillonite,Kaolinite	Kaolinite	Montmorillonite	Montmorillonite
	Halloysite	Halloysite	Kaolinite	
		Montmorillonite		
PH Value	7.2 - 8.4	5.1 - 6.8	6.3 - 0.1	8.0 - 9.1
Principal Cations	Ca, Mg, K	Ca, Mg, K	Ca	Ca
Cations exchange Capacity (me/100g)	42 - 95)	30 - 77	22 - 90	61
Clay ( 2μ), %	13 - 75	34 - 76	5-51	3-41
Liquid limit (%)	37 - 88	44 - 66	NP - 57	NP – 70
Plasticity index (%)	11-48	14 - 30	NP - 35	NP – 35
Shrinkage limit (%)	7-28	10-30	11-19	15 – 20
Specific gravity	2.62 - 2.94	2.61 - 2.90	2.61 - 2.90	2.61 - 2.92
Organic content (%)	2 - 7+	1 - 4+	2 - 7+	4+
Compaction test Max density (1b/ft3)	69 - 101	74 - 106	73 - 104	76 – 90
Optimum Moisture (%)	40 - 17	38 - 29	28 - 21	30 – 27
CBR test values	2-8	6-9	-	-
Unconfined compressiveStrength (kN/m <sup>2</sup> )	95-265	143-246	61-156	63-188
Expansion pressure (kN/m <sup>2</sup> )	95-96	20-96	5-141	19-113

## **2.3. Laboratory tests of soils**

In nature, soils occur in a large variety. However, soils exhibiting similar behavior can be grouped together to form a particular group. Engineers are continually searching for simplified tests that will increase their knowledge of soils beyond that which can't be gained from visual examination without having to resort to the expense, detail, and precision required with engineering properties tests. These simplified tests provide indirect information about the engineering properties of soils and are, therefore, called index tests.

### **2.3.1. Index property tests**

Index property is a property, which helps in distinguishing the characteristics of a soil. Soil grain property and soil aggregate property are two main categories under this term. Soil grain property is based on the individual grains and depends on size, shape and mineralogical characteristics. Soil aggregate property, on the other hand is based on the property of the soil mass as a whole. Basic soil properties and parameters can be subdivided into physical, index, and engineering categories. Physical soil properties include particle size and distribution, specific gravity, and water content. Index parameters of cohesive soils include liquid limit, plastic limit, shrinkage limit, and activity. Such parameters are useful to classify cohesive soils and provide correlations with engineering soil properties [7]

#### **2.3.1.1. Particle size and distribution**

Particle size and distribution is carried out by using a set of standard sieves. Sieves are made by weaving two sets of wires at right angles to one another. The square holes thus formed between the wires provide the limit which determines the size of the particles retained on a particular sieve. The sieve sizes are given in terms of the number of openings per inch. The number of openings per inch varies according to different standards.

### 2.3.1.2. Atterberg limits

In soils containing largely of fine grains, the amount of water present in the voids has a pronounced effect on such engineering properties as shear strength & compressibility. Consistency is used as the basis for their classification. The consistency of a soil is its physical state characteristics at given moisture content.

Atterberg proposed five status of soil consistency [5]. The liquid and plastic limits are used internationally for soil identification and classification and for strength correlations. The shrinkage limit is useful in certain geographical areas where soils undergo large volume changes when going through wet and dry cycles. The collision and sticky limits are not generally used in geotechnical engineering work [5].

**Liquid limit:** To determine the liquid limit of the soils 15 disturbed representative samples were taken from six open test pits by direct excavation and the apparatus used for this test is cone penetrometer.

**Plastic limit:** Samples for plastic limit test were prepared by the same method followed in liquid limit test. **Shrinkage Limit:** It is useful for the determination of the swelling and shrinkage capacity of the soil [8].

**Linear Shrinkage:** Linear Shrinkage can be used for the evaluation of the swelling potential of clayey soil.

For soils with very small clay content the liquid and plastic limit may not produce reliable results to determine the plasticity index. An approximation of the plasticity index may be obtained in such cases by measuring the linear shrinkage (LS) and using the following expression [12].

$$IP = 2.13 * LS \dots \dots \dots 2.1$$

### 2.3.1.3. Specific gravity

Specific gravity of a soil refers to the mass of solid matter of a given soil sample as compared to an equal volume of water.

$$SG = M_s / V_s \gamma_w \dots \dots \dots 2.2$$

#### General Ranges of SG for Soils (19)

- Gravel 2.65 – 2.68
- Sand 2.65 – 2.68
- Silty sands 2.66--2.70
- Silt 2.66-2.7
- Inorganic clays 2.67-2.80
- Organic Soils variable ,but<2.0

### 2.3.2.Consolidation tests

The two most important soil properties required in the design of foundations are bearing capacity and settlement characteristics. The settlement characteristics of a foundation will depend not only on the soil and stress range to which the soil is subjected but also to relative size and depth of thickness of compressible soil strata [16].

The ultimate change in volume of soil occurring under a change in applied stress depends on the compressibility of the soil skeleton of the soil particles. However, the water in the voids of a saturated soil is relatively incompressible and, if no drainage takes place, change in applied stress results in a corresponding change in pore pressure, and the volume change is negligible. As the drainage takes place, water flows from zone of high excess pore pressure to zone of less or zero excess pore pressure, and thus the excess pore pressure dissipate. The applied stress is then transferred to the soil skeleton and volume change takes place. It is this time dependent volume change resulting from dissipation of excess pore pressure, which is known as consolidation.

The rate at which the water can escape depends upon the permeability of the soil. In a free drained cohesion less soil, such as medium and coarse sand or gravel, the change in volume is generally slight and almost instantaneous so that very soon after the application of load is carried by soil skeleton. However, with clay soils the permeability is low and there is a time lag between the application and change in volume.

The consolidation of a soil may be considered to consist of two stages: the primary consolidation stage during which the applied pressure increment is transferred from the pore water to the skeleton, and the secondary consolidation stage that follows the end of primary phase. A study of

consolidation requires knowledge of the compressibility of the soil skeleton and the rate at which excess pores pressure dissipates which is related to the permeability [2]. The consolidation test measures the magnitude and the rate of settlement with the application of load using an apparatus called Oedometer [5].

### I. Coefficients of consolidation

The coefficient of consolidation ( $C_v$ ) signifies the rate at which a saturated clay undergoes 1-dimensional consolidation when subjected to an increase in pressure. Measured in  $\text{cm}^2/\text{s}$ ,  $C_v$  is a function of hydraulic conductivity,  $k$  ( $\text{cm}/\text{s}$ ), coefficient of volume change,  $m_v$  ( $\text{cm}^2/\text{kg}$ ), and unit weight of pore fluid,  $\gamma_w$  ( $\text{kg}/\text{cm}^3$ ), and is expressed as follows:

$$C_v = \frac{k}{m_v \gamma_w} \dots \dots \dots 2.3$$

Terzaghi and peck (1967) anticipated that, with decreasing void ratio (increasing consolidation pressure) both  $k$  and  $m_v$  decreases rapidly so that the ratio ( $k/m_v$ ) and hence  $C_v$ , is fairly constant over a wide range of consolidation pressure. The variation of  $C_v$  with pressure has not received much attention in the past. Review of some available data in the literature review on this aspect, however shows that  $C_v$  is not constant but varies with consolidation pressure. Retianomy G. Robinson and Mehter M. Allam (Effect of clay mineralogy on coefficient of consolidation, Indian institute of science, Bangalore-560 012, INDIA) discussed that  $C_v$  increases with consolidation pressure for kaolinite but decreases with consolidation pressure for montmorillonite (bentonite). Both increasing and decreasing trends observed for soils of unknown clay mineralogy.

For a given load increment, the coefficient of consolidation,  $C_v$ , can be determined from the laboratory observations of time versus dial reading. There are several procedures presently available to estimate the coefficient of consolidation, some of which are described below.

#### A. Logarithm-of-time method

The logarithm of time method was originally proposed by Casagrande and Fadum (1940) and can be explained by referring to Figure 2.3.

1. Plot the dial readings for specimen deformation for a given load increment against time on semi log graph paper as shown in Figure 2.3.

2. Plot two points, P and Q, on the upper portion of the consolidation curve, which correspond to time  $t_1$  and  $t_2$ , respectively. Note that  $t_2 = 4t_1$ .
  3. The difference of dial readings between P and Q is equal to  $x$ . Locate point R, which is at a distance  $x$  above point P.
  4. Draw the horizontal line RS. The dial reading corresponding to this line is  $d_0$ , which corresponds to 0% consolidation.
  5. Project the straight-line portions of the primary consolidation and the secondary consolidation to intersect at T. The dial reading corresponding to T is  $d_{100}$ , i.e., 100% primary consolidation.
  6. Determine the point V on the consolidation curve that corresponds to a dial reading of  $d_{50} = \frac{d_0 + d_{100}}{2}$ . The time corresponding to point V is  $t_{50}$ , i.e., time for 50% consolidation.
  7. Determine  $C_v$  from the equation  $T_v = C_v t/H^2$ . The value of  $T_v$  for  $U_{av} = 50\%$  is 0.197
- So,

$$C_v = \frac{0.197 H^2}{t_{50}} \dots \dots \dots 2.4$$

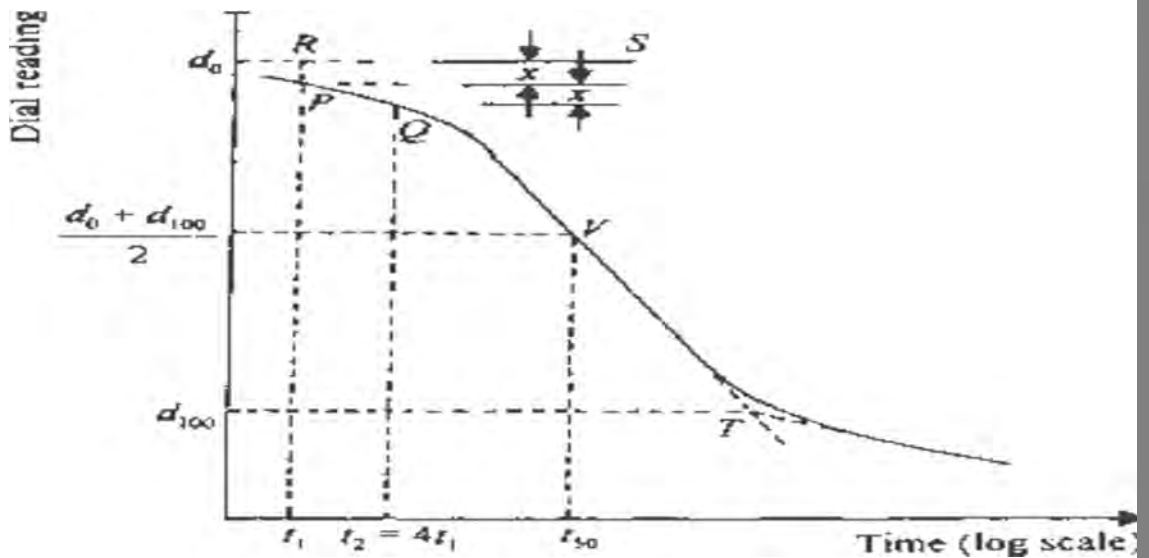


Figure 2.3 Logarithm-of-time methods for determination of  $C_v$

### B. Square-root-of-time method

The steps for the square-root-of-time method (Taylor, 1942) are

1. Plot the dial reading and the corresponding square-root-of-time  $\sqrt{t}$  as shown in Figure 2.4.
2. Draw the tangent PQ to the early portion of the plot.
3. Draw a line PR such that  $OR = 1.15(OQ)$ .
4. The abscissa of the point S (i.e., the intersection of PR and the consolidation Curve) will give  $\sqrt{t_{90}}$  (i.e., the square root of time for 90% consolidation).
5. The value of  $T_v$  for  $U_{av} = 90\%$  is 0.848. So,

$$Cv = \frac{0.848 H^2}{t_{90}} \dots\dots\dots 2.5$$

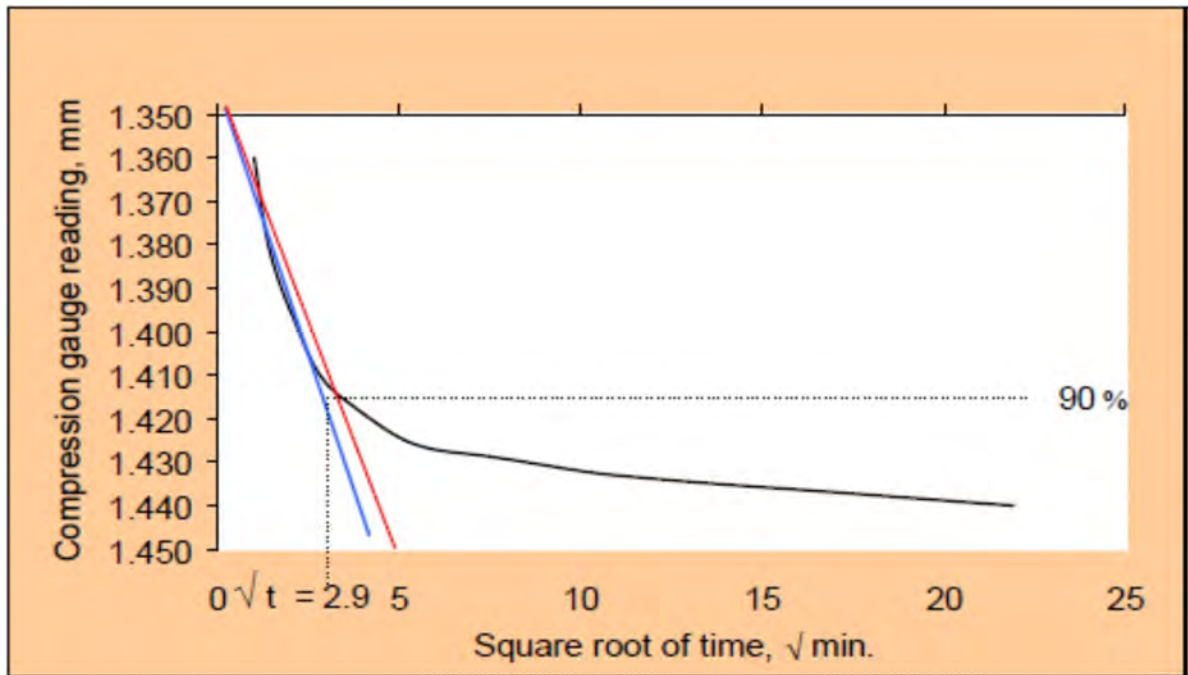


Fig.2.4 Typical curve (Square root of time method)

### II. Compression index (Cc) and Swelling index (Cs)

The compression index (Cc) and Swelling index (Cs) are intended to relate to how much consolidation or settlement will take place. The swelling index (Cs) relates to how much swelling will take place. The slope of the straight line portion of the  $e$  vs.  $\log p$  plot on loading side gives the compression index (Cc) whereas on unloading side swelling index (Cs) is obtained.

### III. Pre consolidation Pressure

In testing undisturbed specimens from field it is essential to obtain the field void ratio (generally obtained from the moisture content for an assumed fully saturated specimen) and the field effective past pressure, known as the (effective) pre consolidation pressure ( $\sigma'_c$ ). The pre consolidation pressure is always measured as an effective stress. Many soils have been subjected to greater pressure than the existing effective overburden pressure ( $\sigma'_o$ ) and then allowed to soil (known as over consolidated soil). The ratio of pre consolidation to that of the effective overburden pressure is known as the over consolidation ratio. The over consolidation ratio is thus expressed as,

$$\text{Over consolidation ratio} = \text{OCR} = \frac{\sigma'_c}{\sigma'_o} \dots\dots\dots 2.6$$

Where  $\sigma'_c$  is pre consolidation pressure

$\sigma'_o$  is effective overburden pressure

Soils with OCR=1, are known as normally consolidated soil. An estimate of the pre compression pressure can be obtained from the oedometer test results. Pre compression ( $P_c$ ) was determined from the curves of void ratio versus log pressure curves using an empirical graphical method proposed by Casagrande or Schmertman [16]

#### A. Casagrande method

One method commonly used to obtain the maximum pre consolidation pressure is that suggested by Casagrande (1936) and illustrated in figure 2.5. The technique consists of plotting the void ratio corresponding to the end of primary void ratio against  $\log_{10}$  of effective pressure ( $e$  vs.  $\log \sigma'$ ) as shown in fig 2.5. As the pressure continues to increase the graph tends to toward a straight steeper inclined line. The straight steepest portion of the slope, represented by symbol  $C_c$ , is termed the virgin (compression) index of the soil. In order to obtain the most probable value of the pre consolidation pressure, estimate the point of maximum curvature (i.e. minimum radius) of the graph. At this point draw a tangent line and horizontal (at constant void ratio) through the point.

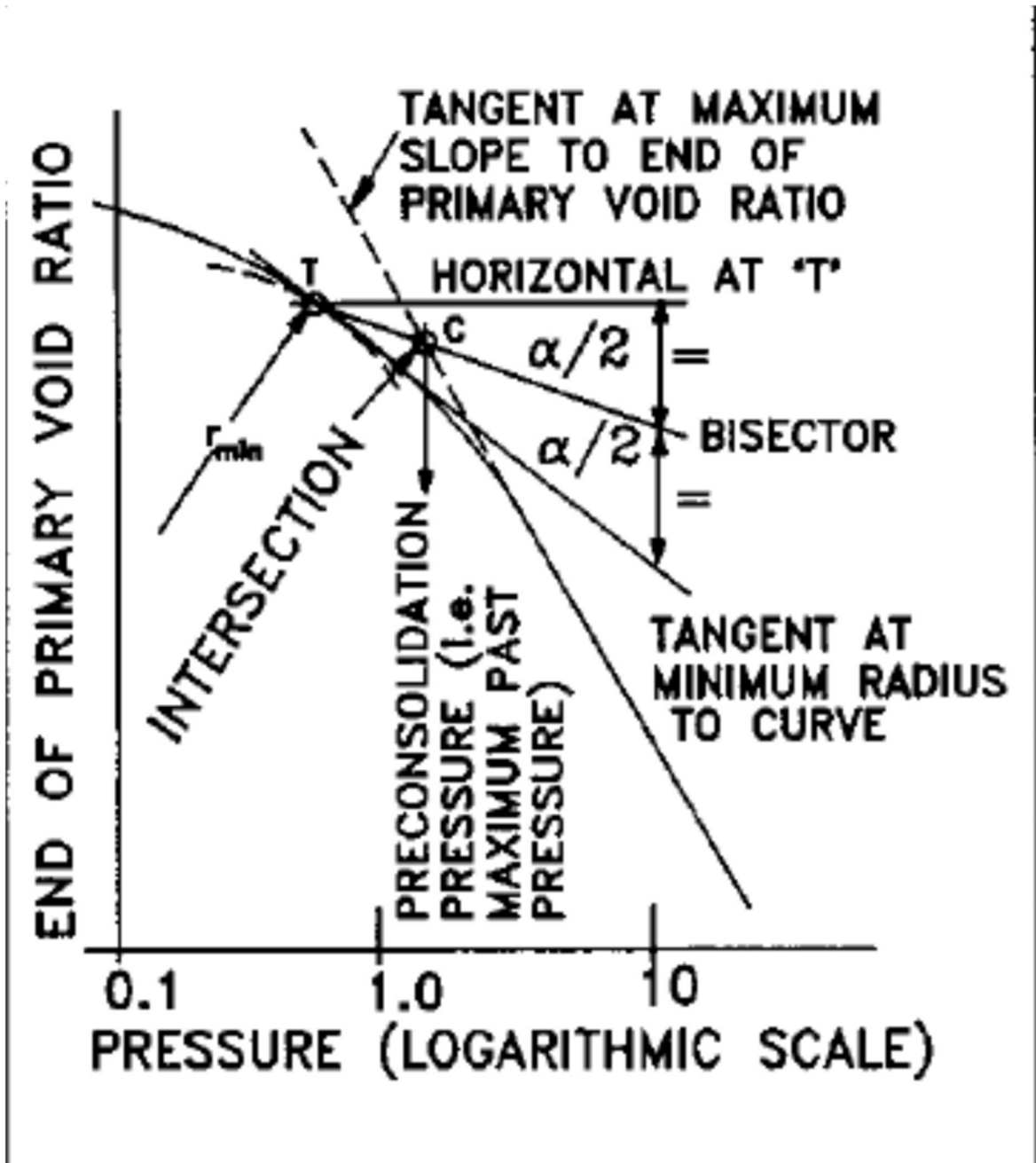


Figure 2.5. Casagrande (1936) construction to determine pre consolidation pressure

Now draw a line that bisects the angle made by two lines and locate the intersection of this bisector and an extension of the straight portion of the virgin compression (i.e. the steepest portion) of the graph. The pressure corresponding to this point is the estimated pre consolidation

pressure. If the pre consolidation pressure lies close to or below the overburden pressure, the soil is normally consolidated (i.e. the existing pressure is the maximum pressure ever exerted on the soil in its present condition). If the pre consolidation pressure is higher than the overburden pressure, the soil is over consolidated. If the soil is over consolidated, oedometer expansion tests should be performed at the end of consolidation stage. The expansion curve of the graph is generally used to estimate the recompression of the field of the soil to its pre consolidation pressure. If the overburden pressure is higher than pre consolidation pressure, the soil may be under consolidated (i.e. consolidation in the field under the overburden pressure is still progressing) or the specimen under the tests has been badly disturbed during sampling. [17]

## **B. Schmertman Method**

As often considered better method (by many engineers) of estimating the pre consolidation pressure of over consolidated soils is the Schmertman Method. The method is illustrated in fig 2.6. If the soil is over consolidated a line is drawn through the point representing the overburden pressure and the field void ratio parallel to the initial expansion portion of the void ratio parallel to the initial expansion portion of void ratio vs. logarithm of pressure ( $e \log p$ ) plot. Then join a guess / estimate of the overburden point to a point on the laboratory straight line portion of the plot. (Extended, if necessary at 0.42 field void ratio (42 %  $e_f$ ). If the best estimate has chosen the difference in void ratio between the estimate field curve and the laboratory  $e \log \sigma'$  curve should be symmetrical about the estimated pre consolidation pressure. Continue this selection until this symmetrical plot is achieved.[17]

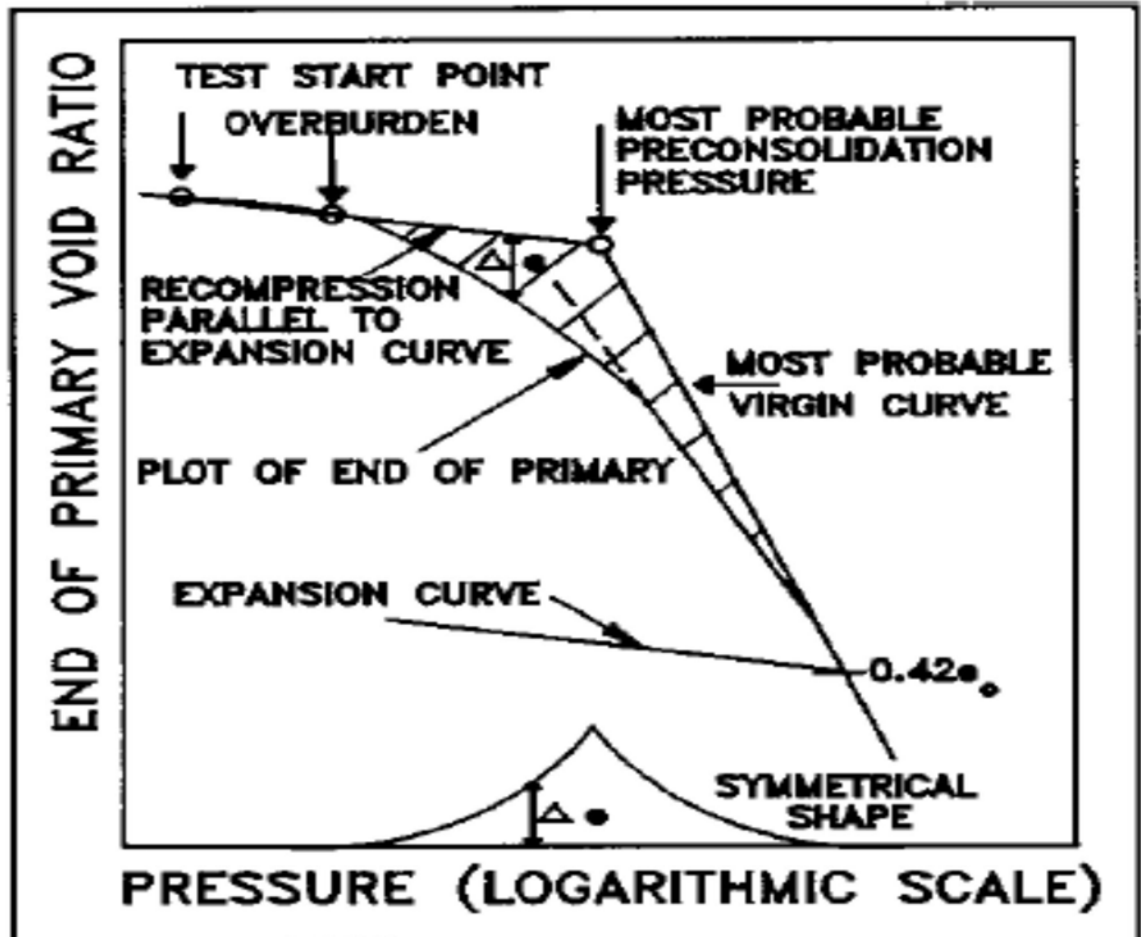


Figure 2.6. Scmertman (1954) construction to determine pre consolidation pressure

#### IV. Determination of the Coefficient of Permeability

The coefficient of permeability depends on several factors, most of which are listed below.

1. Shape and size of the soil particles.
2. Void ratio. Permeability increases with increase in void ratio.
3. Degree of saturation. Permeability increases with increase in degree of saturation.

The coefficient of permeability can be measured using field tests, or tests conducted in the laboratory. Permeability is also estimated from one dimensional consolidation test. The coefficient of permeability can be obtained from the following relationship [4].

$$K = \frac{C_{va}\gamma_w}{1+e} \dots\dots 2.7$$

Where  $C_v$  = coefficient of consolidation

$$a_v = \text{coefficient of compressibility} = \frac{\Delta e}{\Delta P}$$

$\gamma_w$  = unit weight of water

$e$  = void ratio

Using the above equation, the coefficient of permeability as the function of void ratio can be calculated from the consolidation test results. It is noted that, the ratio of change in void ratio to change in pressure, was also obtained. As shown in Table 2.2, the ranges of values of coefficient of permeability for various soils are indicated.

Table 2.2 Typical values of coefficient of permeability for various soils [14 ]

Material	Coefficient of permeability (mm/s)
Coarse	$10^{-10}$ - $10^{-3}$
Fine gravel, coarse and medium sand	$10^{-2}$ - $10^{-1}$
Fine sand, loose silt	$10^{-4}$ - $10^{-2}$
Dense silt, clay silt	$10^{-5}$ - $10^{-4}$
Silt clay, clay	$10^{-8}$ - $10^{-5}$

### 2.3.3. Shear strength tests

The shear strength of soils is an important aspect in many foundation-engineering problems related to stability such as the bearing capacity of shallow foundations and piles, the stability of slopes of dams and embankments, and lateral earth pressure on retaining walls. The purpose of shear strength testing is to establish representative values for the shear strength parameters. The drainage conditions during the test influence the measured values considerably. The most common laboratory methods employed to obtain shear strength parameters are direct shear test, triaxial compression test and unconfined compression test. In this document the unconfined compression test will be reviewed.

### **2.3.3.1. Unconfined compression (UC) test**

The unconfined compression test is a special case of the unconsolidated undrained triaxial test. In this case, no confining pressure to the specimen is applied. The UC test is one of the easiest and simplest tests for determining a quick estimate of the shear strength of cohesive soils. The test provides an immediate approximate value of the compressive strength of the soil, either in the undisturbed or the remolded condition. It is also widely used to determine the consistency of saturated clays and other cohesive soils [23].

### 3. Sampling Area Description

#### 3.1. General

.Geographically;-Dangilla is situated in Amhara regional state. It was founded in 1935 as the main trade center of the area (1).

#### 3.2. Topography and climate

##### 3.2.1. Topography

The town is found at an elevation of 2137 meters above sea level. Most of the town stretches over a predominantly flat land with imperceptible slope changes except for small rises in its north and south peripheries. There are also scattered and slightly depressed areas within the town's boundary which form temporary swamps where rain water settles for short to prolonged period of the year. [1]

##### 3.2.2. Climate

From the data collected for many years the monthly Rainfall and Temperature variation of Dangila town is tabulated as, [1]

Table 3.1 climate of Dangila town

Month	Jan	Feb	March	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec
Average T <sup>0</sup> c	16.2	17.2	17.3	18.4	18	17.1	16.2	16.6	17.3	16.8	16.8	15.9
Rainfall(mm)	3	24	35	33	85	227	345	347	237	91	63	12
Min T <sup>0</sup> c	3.2	4.4	4.1	9	9.8	9.9	10	10	10	9	7.5	4
Max T <sup>0</sup> c	29.2	28	27	27.8	26.6	23.1	22.4	23.2	24.6	25.2	26.1	28.1

### 3.3. Site investigation

A total of 20 samples were collected with 15 disturbed samples and 5 undisturbed samples at five locations. The sampling was done within an area of 2 m<sup>2</sup> at each location. The undisturbed samples were collected through use of sampling tubes of two types, 80 mm dia with 100 mm length and 80 mm dia. with 50 mm length. The tubes are driven by power to advance up to 3 m depth. The undisturbed samples are collected after collection of disturbed samples. For sample preparation, undisturbed samples are sealed and covered with plastic bags immediately after sampling to prevent moisture exchange with the atmosphere. The undisturbed samples were prepared for consolidation and unconfined compression tests, while the disturbed samples were air dried and prepared for Index tests.

For this Research, Five Test pits have been taken to analyze the whole data. This is because of:-

1. The area of town is around 9km<sup>2</sup>. So, its narrow area, guides the amount of test pits and samples to collect.
2. During Pitting & Sample Collection processes, it has been observed that most Samples have slightly similar Characteristics. So that targeting more Test pits not necessarily considered.
3. Laboratory results also confirm those uniformity of index & shear strength properties through samples.

**Table 3.2 location of test pits**

<b>Test pit</b>	<b>Location</b>
TP1	Near Amen School
TP2	Around St Michael church at kebele 02
TP3	Near the main market area, kebele 03
TP4	near to Agawmidir hall
TP5	Around Agunta ,St Mary church

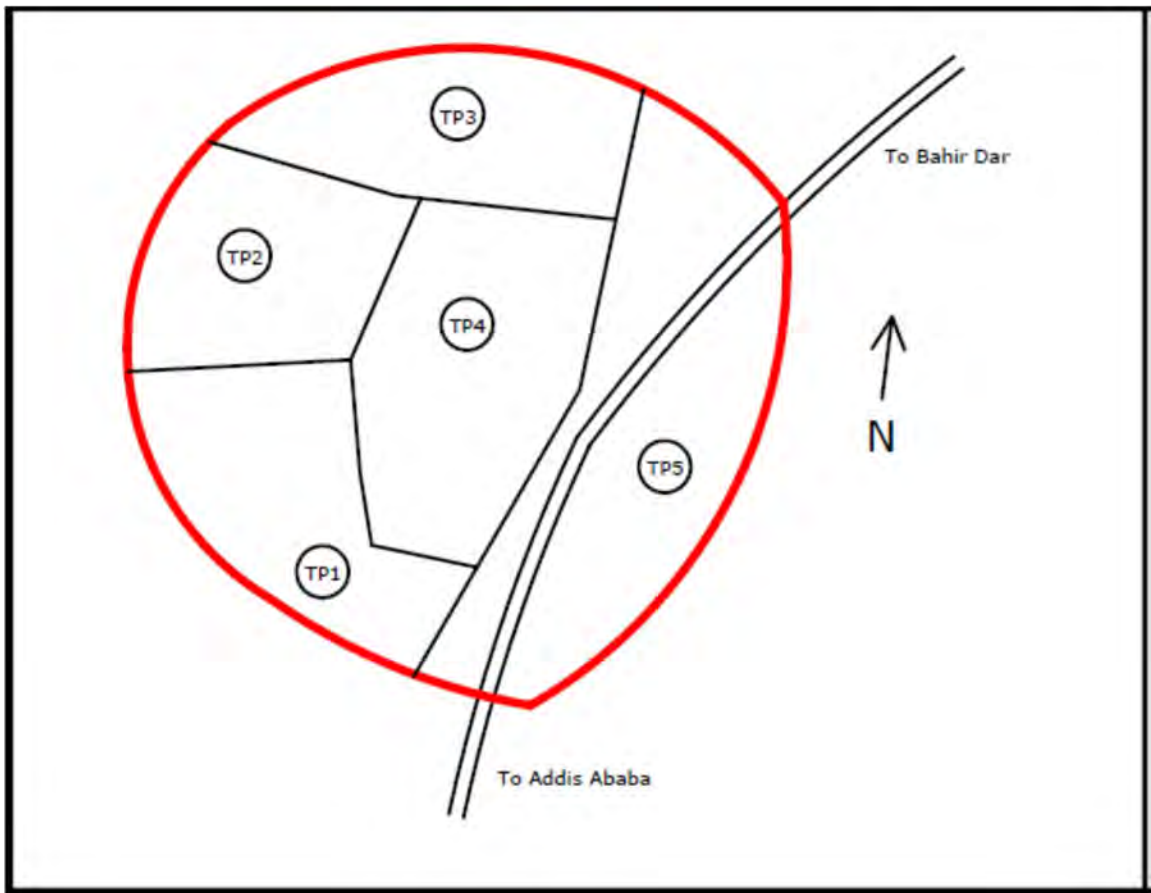


Fig 3.1 Map of Dangia

### 3.3.1. In-situ Properties of soils

The soil samples collected were soft, cohesive and wet in nature. Insitu properties of soils are visually identified as follows:-

Table 3.3 In-situ Properties of Dangila soils

TP3

<b><i>Item no</i></b>	<b><i>Color of the soil</i></b>	<b><i>Red, pink, yellowish red</i></b>
1	Depth of organic soil ( <i>observed from of test pits</i> )	25 – 30(cm)
2	Depth of water table ( <i>observed from wells for home water use</i> )	5 - 13(m)
3	Depth of Bed rock ( <i>observed from wells and deep pit for home water use and dry latrines</i> )	8 - 11(m)

0.25m	Brown organic soil
1m	Light red silty clay
2m	Fine red, highly plastic clay
3m	Stiff Yellowish red, highly plastic clay

a) TP2

0.25m	Brown organic soil
1m	Light red silty clay
2m	Fine red clay
3m	Pink coloured Plastic silty clay

b) TP3

0.25m	Brown organic soil
1m	Light red silty clay
2m	Stiff Clay
3m	Pink Firm clay

c)TP4

0.25m	Organic soil
1m	Light red
2m	Fine, red Clay
3m	Pink colored

d) TP5

0.25m	Organic soil
1m	Light red
2m	Fine, red
3m	Pink colored

e) TP1

Table 3.4 Soil profile for Different Test Pits

## **4. Laboratory test results**

Samples retrieved from the test pits were transported to Geotechnical Engineering laboratory, in Addis Ababa Institute of Technology, for index property testing, consolidation and unconfined compression testing. Index property tests performed on selected samples included grain size analyses, water contents, and Atterberg Limits.

### **4.1. Index property tests**

Particle size distribution, Atterberg limit test, hydrometer analysis and specific gravity tests are among the tests which show the index property of a soil.

#### **4.1.1. Particle-size distribution**

The common laboratory method to determine the size distribution of fine-grained soils is a hydrometer test. Hydrometer test was carried in ASTM D 422. The experimental results are tabulated in Table 4.1 and the particle size distribution curves are shown in Fig.4.1.

#### **4.1.2. Atterberg limits**

To determine the liquid limit and plastic limits of the soils 15 disturbed representative samples were taken from five open test pits by direct excavation and the apparatus used for this test is cone penetrometer. The details of the procedures are given in ASTM D 4318. The results of liquid limit tests are shown in Table 4.2.

Table 4.1 grain size analysis of test results

Location	Test pit No	Depth of sampling (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
Dangila Town	Tp1	1	1.25	21.31	27.04	50.4
		2	0	1.04	28.96	70
		3	0	1.96	22	76.04
	Tp2	1	0.63	18.8	40.57	40
		2	3.68	20.22	37.8	38.3
		3	2.0	15.2	25.8	57.0
	Tp3	1	3.06	20.44	20.11	56.4
		2	1.2	2.04	25.96	37.5
		3	5.82	8.18	48.5	70.8
	Tp4	1	23.9	22.68	13.07	40.85
		2	1.2	8.38	38.2	52.2
		3	3.2	6.38	34.2	56.22
	Tp5	1	1.25	21.31	27.04	49
		2	2.8	1.04	28.96	67.05
		3	2	1.6	22	74.4

### 4.1.1. Specific gravity

Specific gravity of a soil refers to the mass of solid matter of a given soil sample as compared to an equal volume of water. The soil samples obtained from the field were prepared as specified in ASTM D 854-00. The specific gravity test was conducted by the small pycnometer method as described in ASTM D 854. The results are shown on table 4.3.

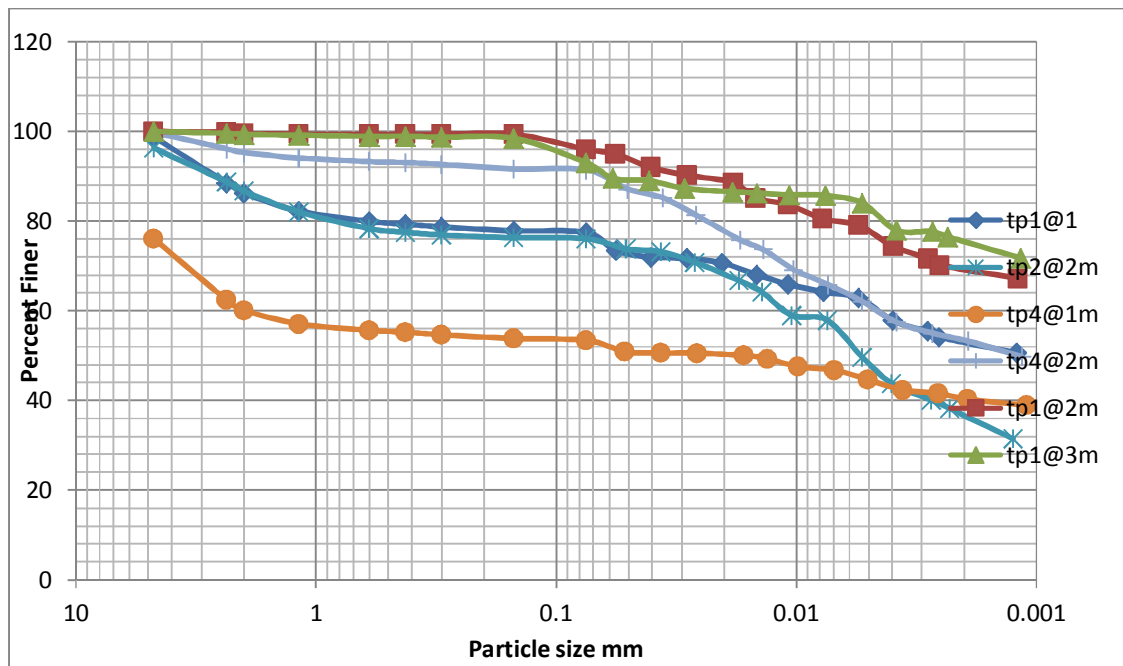


Fig 4.1 Gradation curve for different test pits

Table 4.2 Test Results of Atterberg limits

Location	Test pit No	Depth of sampling (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)
Dangila Town	Tp1	1	61.3	36.5	24.8
		2	62.4	40	22.4
		3	71.2	36.9	34.32
	Tp2	1	56.2	28.3	27.9
		2	55.6	23.5	32.15
		3	54.1	24.1	30
	Tp3	1	70	43.6	26.4
		2	77	35.8	32
		3	69.3	37.3	41.2
	Tp4	1	68.4	37.4	30.98
		2	74.6	32.36	42.24
		3	75.1	30.2	44.9
	TP5	2	68	34.7	33.3

Table 4.3. Specific Gravity Results

Location	Test pit No	Depth of sampling (m)	Gs
<b>Dangila Town</b>	Tp1	1	2.61
		2	2.65
		3	2.64
	Tp2	1	2.72
		2	2.72
		3	2.75
	Tp3	1	2.80
		2	2.78
		3	2.90
	Tp4	1	2.89
		2	2.83
		3	2.90

Note: - Higher values of specific gravity indicate existence of heavy minerals, like iron.

Table 4.4. Atterberg Limit Results of TP4

% of water content	No of drops
61.12	35
66.67	27
70.35	22
72.85	19

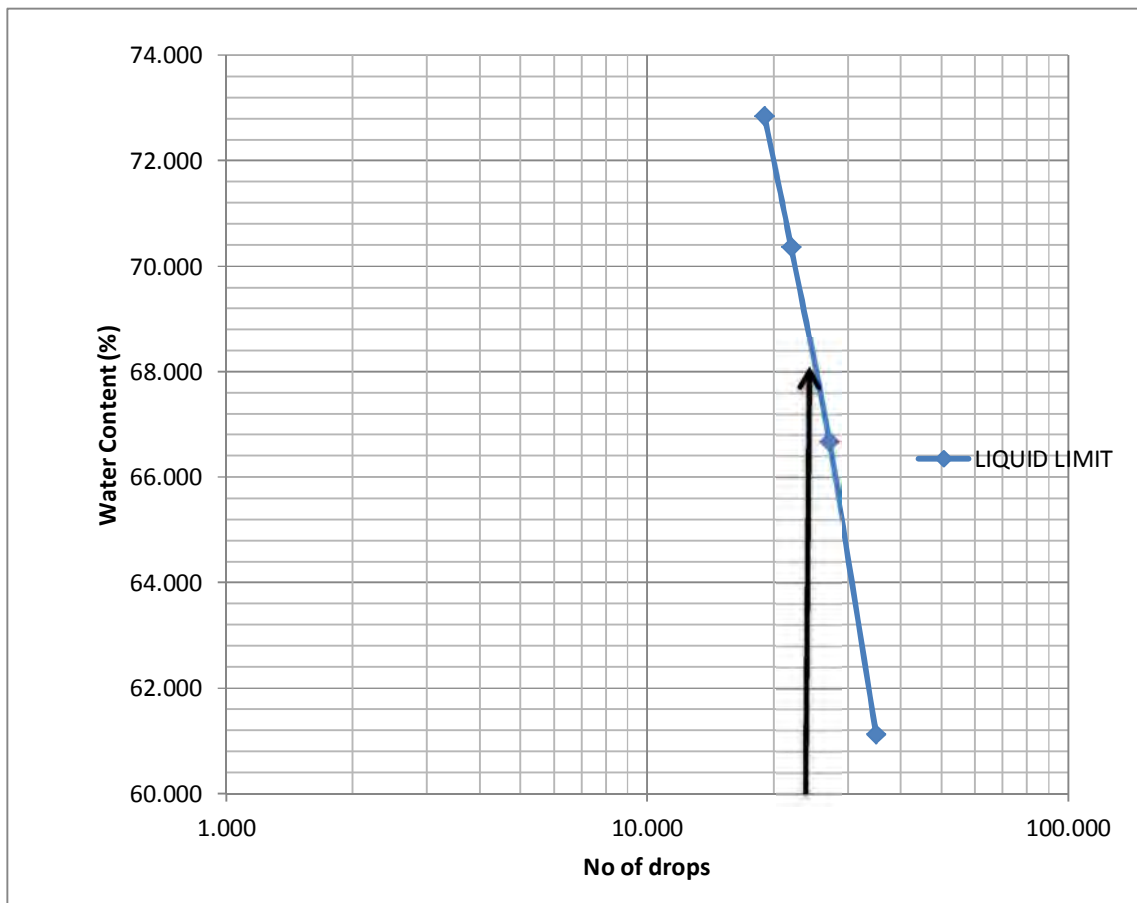


Fig 4.2 Determination of Liquid limits

## 4.2. Consolidation tests

Undisturbed samples were collected from the field. Detail procedures were followed as described in ASTM D 2435.

### 4.2.1. Coefficients of consolidation

For a given load increment, the coefficient of consolidation  $C_v$  can be determined from the laboratory observations of time versus dial reading. There are several procedures presently available to estimate the coefficient of consolidation, some of which are described below. In this work, Logarithm-of-time method is used to determine pre consolidation pressure and the results are shown in fig 4.2 and fig 4.3.

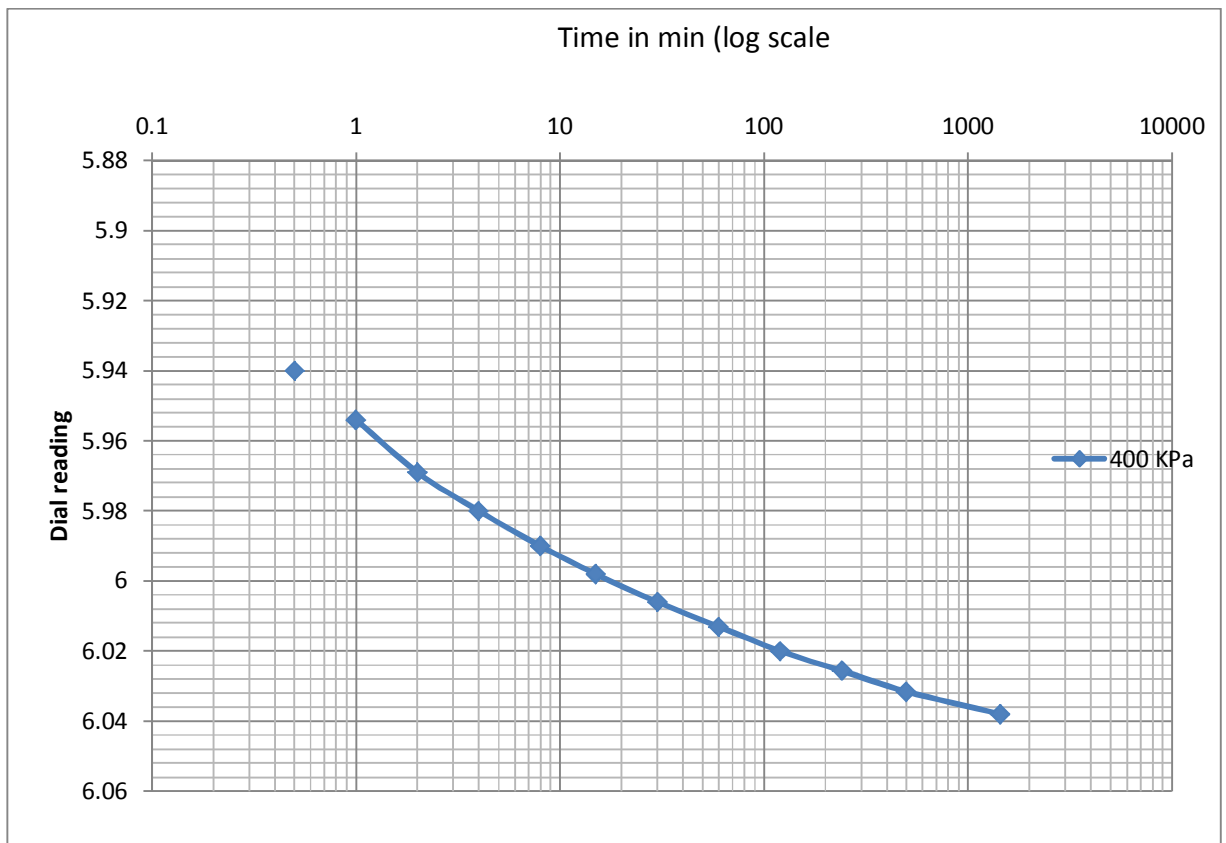


Fig4.3 log -time method for tp2 at 2m depth

#### 4.2.2 Compression index (Cc) and Swelling index (Cs)

The slope of the straight line portion of the  $e$  vs.  $\log p$  plot on loading side gives the compression index (Cc) and recompression index (Cr) whereas on unloading side swelling index (Cs) is obtained.

Table 4.5 Cc and Cs values of different samples

Samples from Test pits	Cc	Cs	Cr
Tp1	0.13	0.028	0.019
Tp2	0.42	0.019	0.025
Tp3	0.32	0.027	0.02
Tp4	0.23	0.03	0.02
Tp5	0.15	0.032	0.015

The test data were plotted as void ratio against log Pressure and Cc was determined from Fig 4.4.

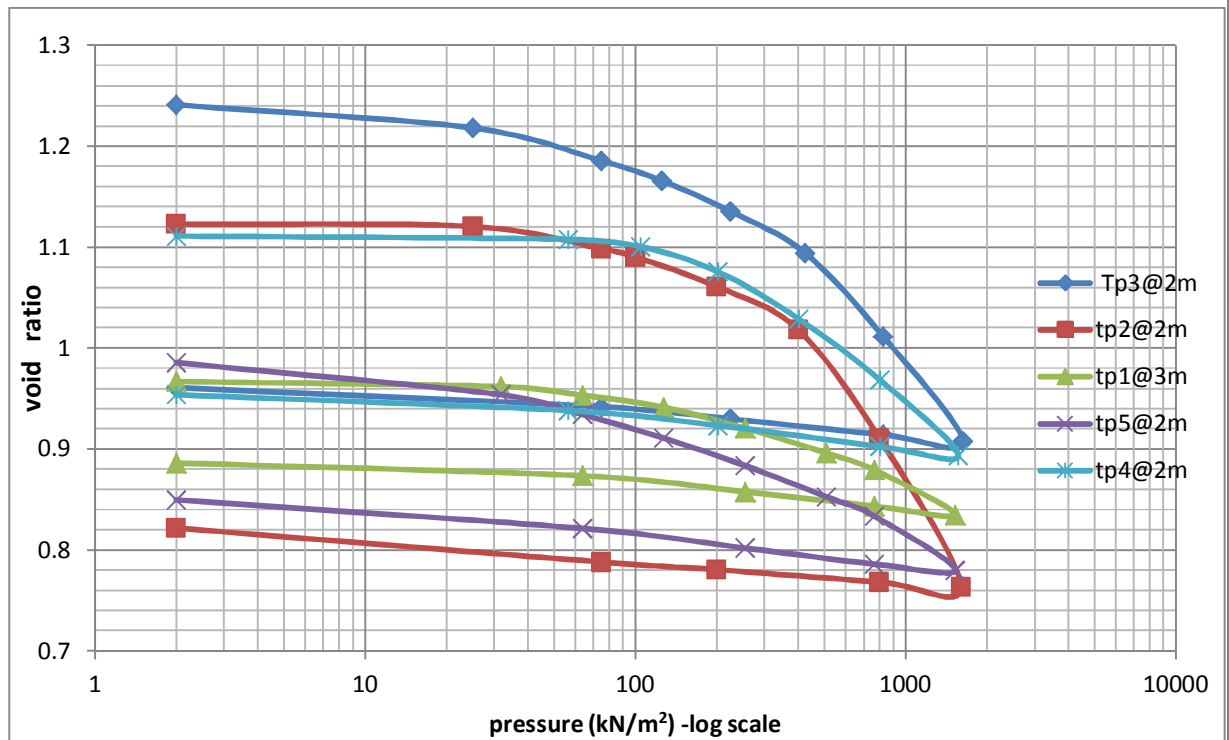


Fig 4.4 Void ratio vs. pressure

### 4.2.3. Pre consolidation Pressure

Pre compression ( $P_c$ ) was determined from the curves of void ratio versus log pressure using an empirical graphical method proposed by Casagrande or Schmertman [16]

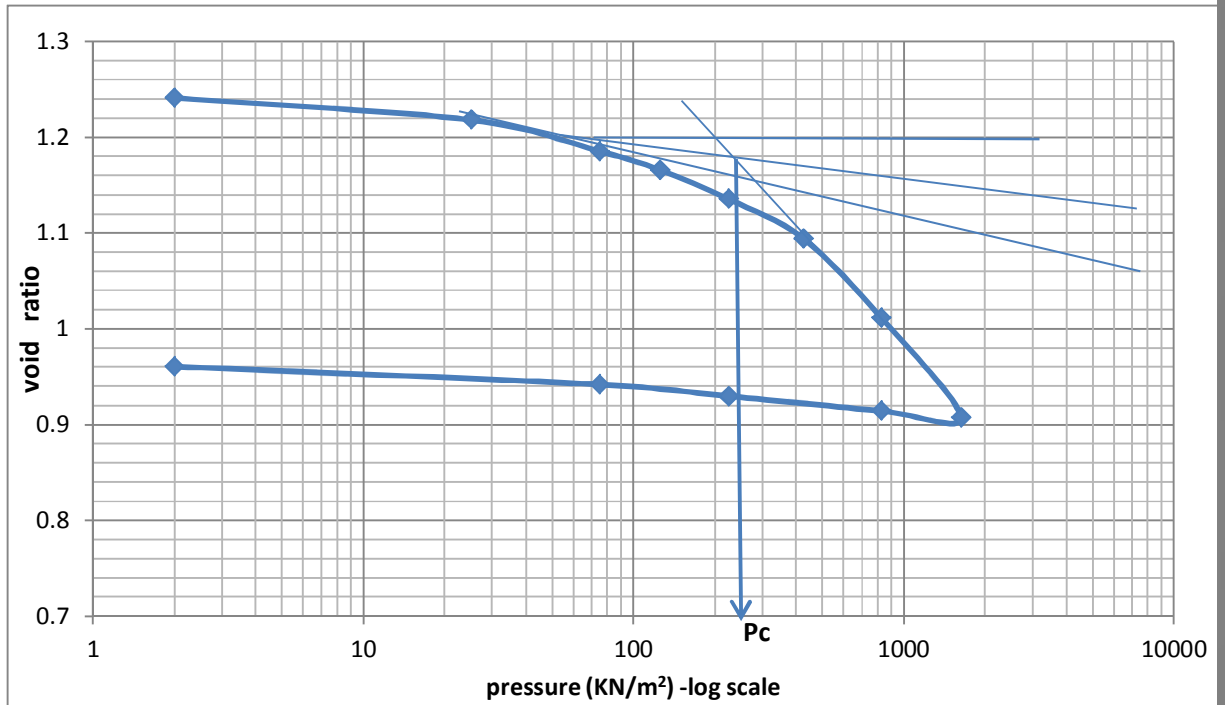


Fig 4.5 Pre consolidation pressure determination for TP1

In this work, casagrande method is used to determine pre consolidation pressure and the results are expressed in table 4.6.

Table 4.6 over consolidation ratios for different test pits

Test pits	Pre consolidation Pressure (kPa)	Over burden pressure(kPa)	OCR
Tp1	280	40	7
Tp2	370	30	12.33
Tp3	300	35	8.57
Tp4	400	40	10
Tp5	350	35	10

From the results obtained, the soils are found to be highly over-consolidated in their natural state

#### 4.2.4. Coefficient of Permeability

Permeability is estimated from one dimensional consolidation test. The coefficient of permeability can be obtained from the following relationship [4].

$$K = \frac{C_v a_v \gamma_w}{1+e} \dots\dots\dots 4.1$$

Where:-

$C_v$  = coefficient of consolidation

$a_v$  = coefficient of compressibility

$$= \frac{\Delta e}{\Delta P}$$

$\gamma_w$  = unit weight of water

$e$  = void ratio

Using the above equation, the coefficient of permeability as the function of void ratio was calculated from the consolidation test results and shown in Table 4.2. It is noted that, the ratio of change in void ratio to change in pressure, was obtained from Fig 4.2. As shown in Table 4.2, the range of values of coefficient of permeability lies between  $10^{-9}$  and  $10^{-8}$  cm/sec, this indicates that the soils are practically impervious. In general, void ratio versus log coefficient of permeability is close to a straight line for nearly all soils [24]. As shown in Fig 4.5, all the soil samples taken from the field conform to this observation.

Table 4.7 Relationship between Void ratio and coefficient of permeability

Test pit no	Depth	Pressure P kPa	Void ratio $e_r$	Coefficient of consolidation $C_v * 10^{-3}$ (Cm <sup>2</sup> /sec)	Coefficient of compressibility $* 10^{-5}$ (kPa)	Coefficient of permeability $* 10^{-9}$ (cm/sec)
Tp1	2m	0	1.123			
		31.875	1.12	2.05	69.01	44.41
		63.75	1.09	3.19	42.19	34.65
		127.5	1.09	1.81	22.82	26.14
		255	1.06	2.45	19.26	21.94
		510	1.01	1.85	16.74	19.83
		765	0.91	1.75	12.54	10.30
		1530	0.76	1.54	9.41	0.23

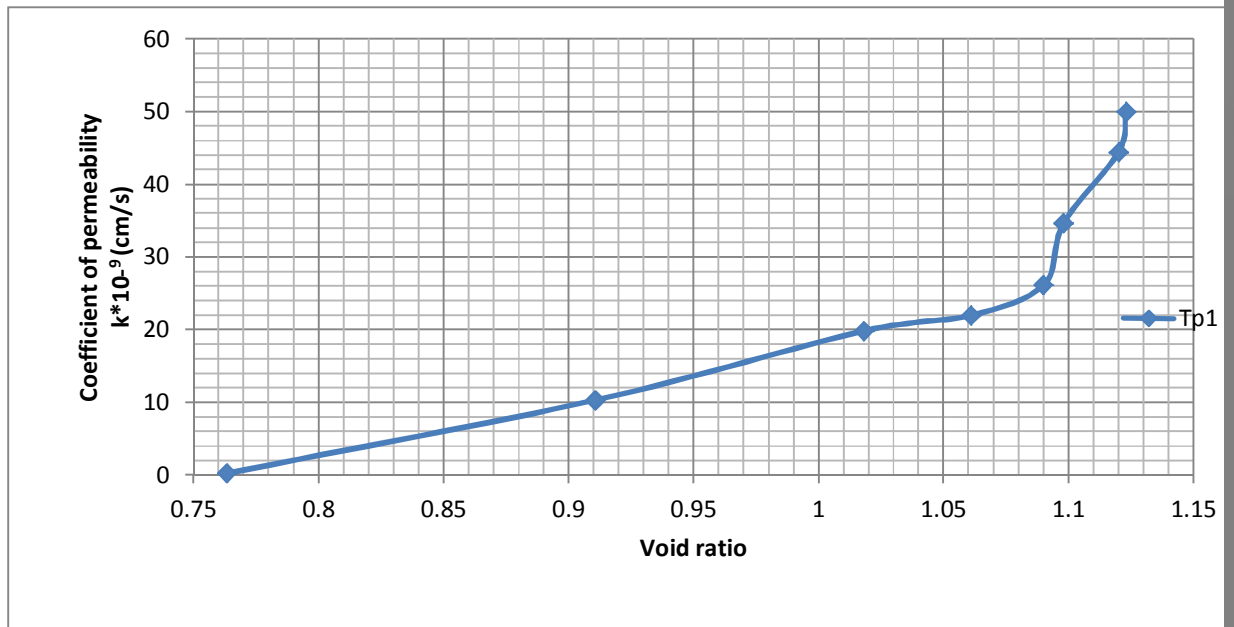


Fig 4.6 coefficient of permeability vs. void ratio for TP1

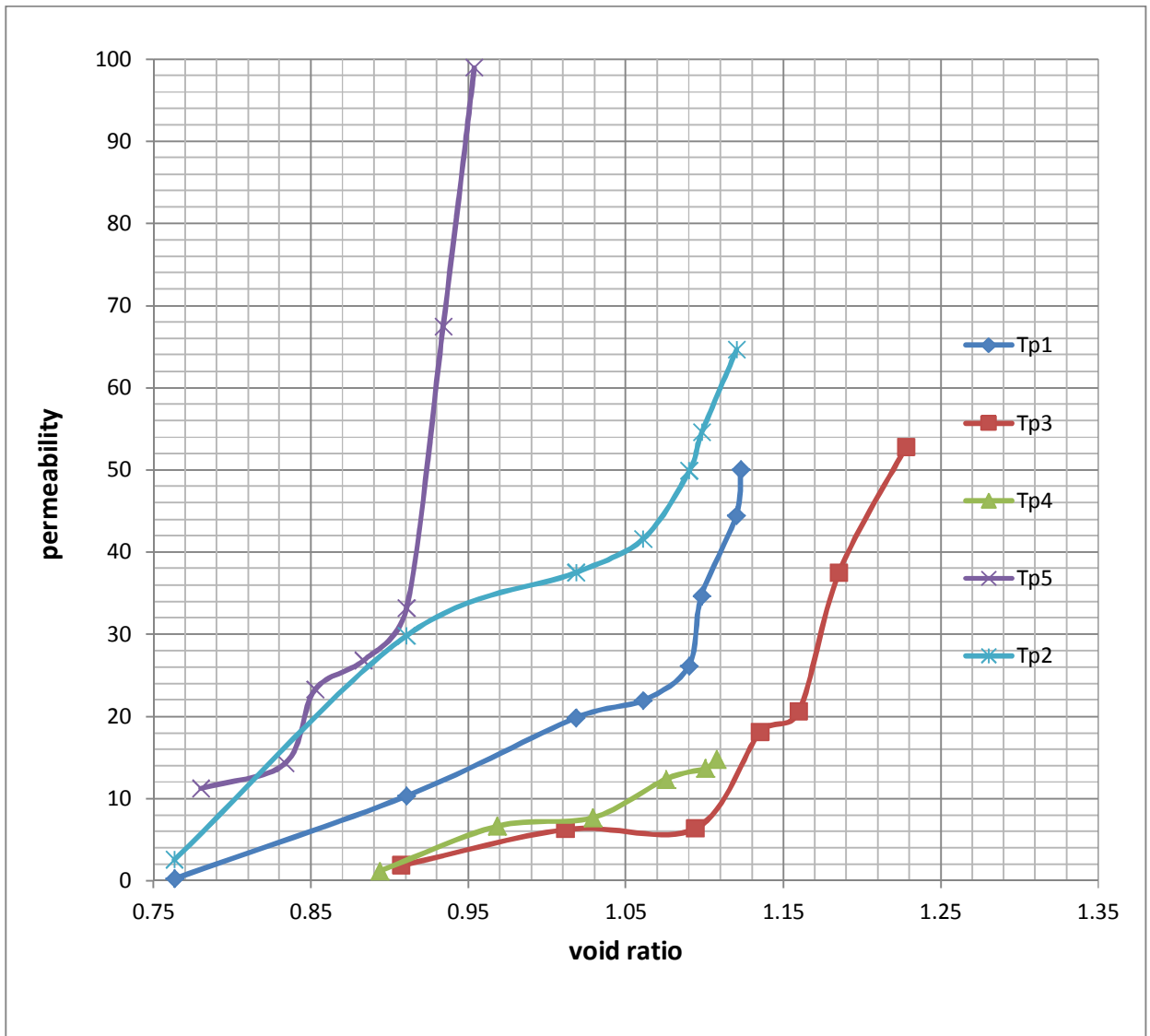


Fig 4.7 cumulative graph of coefficient of permeability vs. void ratio

### 4.2.5 Modulus of compressibility

For soils, whose behavior is typically non-linear, modulus of compressibility ( $E_s$ ), is not constant. The compressibility curve is obtained either from the plot of relative settlement versus effective stress or the void ratio against effective stress. The curve may be expressed by the following equation [11].

$$E_s = \frac{d\sigma'}{ds'} = v(\sigma')^w \dots\dots\dots 4.1$$

$$s' = \frac{\Delta H}{H} \dots\dots\dots 4.2.$$

In order to make the component dimensionless, it is advisable to make  $\sigma'$  also dimensionless by dividing it by unit stress  $\sigma_e$ . Then Eq 1 becomes

$$E_s = \frac{d\sigma'}{ds'} = v(\sigma_e')^w \dots\dots\dots 4.3$$

$v$  and  $w$  are coefficients;  $v$  has a unit of  $\text{kN/m}^2$

$w$  is dimensionless

$\sigma' =$  effective normal stress,  $\text{kN/m}^2$

$\sigma_e =$  Unit normal stress,  $\text{kN/m}^2$

$$\sigma_e' = \frac{\sigma'}{\sigma_e}$$

$s' =$  relative settlement

$v$  depends on the void ratio, water content and consistency of the sample, it could have values ranging from 50 to 30000  $\text{kN/m}^2$  [11].  $w$  depends on the soil type. It could assume values ranging from 0 to 1 [11].

$$\frac{ds'}{d\sigma'} = \frac{1}{v(\sigma')^w} \dots\dots\dots 4.4$$

$$ds' = \frac{1}{v} \dots\dots\dots 4.5.$$

$$s' = \frac{1}{v(1-w)} (\sigma')^{-w} + c \dots\dots\dots 4.6$$

Where  $c$  is integration constant

For  $\sigma' = 0, s' = 0$  then  $C = 0$

Equation 4.6 becomes

$$s' = \frac{1}{v(1-w)} (\sigma')^{1-w} \dots\dots\dots 4.7$$

$$s' = a(\sigma')^K \dots\dots\dots 4.8$$

Taking common logarithm, equation (4.8) becomes;

$$\text{Log}(s)' = k\text{Log}(\sigma)' + \text{Log}(a) \dots\dots\dots 4.9$$

From a plot of  $s'$  versus  $\sigma'$  one obtains a straight-line relationship for some cohesive soils. Other soils give a straight line relationship when results are plotted on double log scale [11]. Using equation (4.9), on the data obtained from one-dimensional consolidation test, relative settlement versus pressure (effective stress) was plotted in figures 4.9. The values of the coefficients,  $v$  and  $w$ , were calculated using those figures and tabulated.

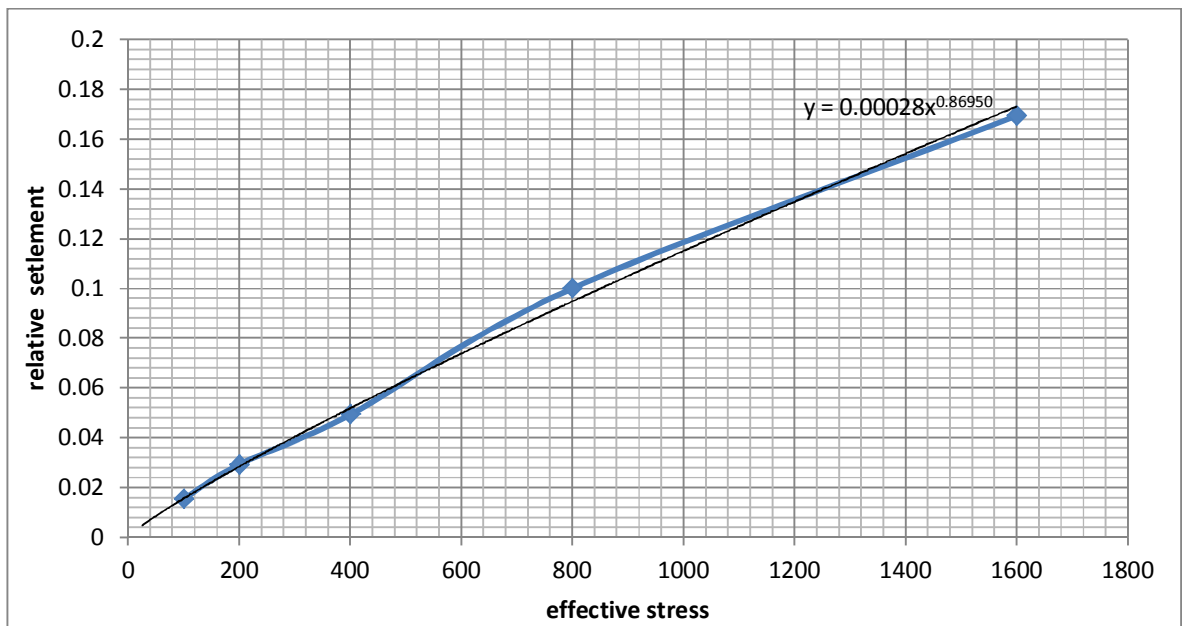


Fig 4.8. Relative settlement Vs Effective Stress (tp2 @ 2m)

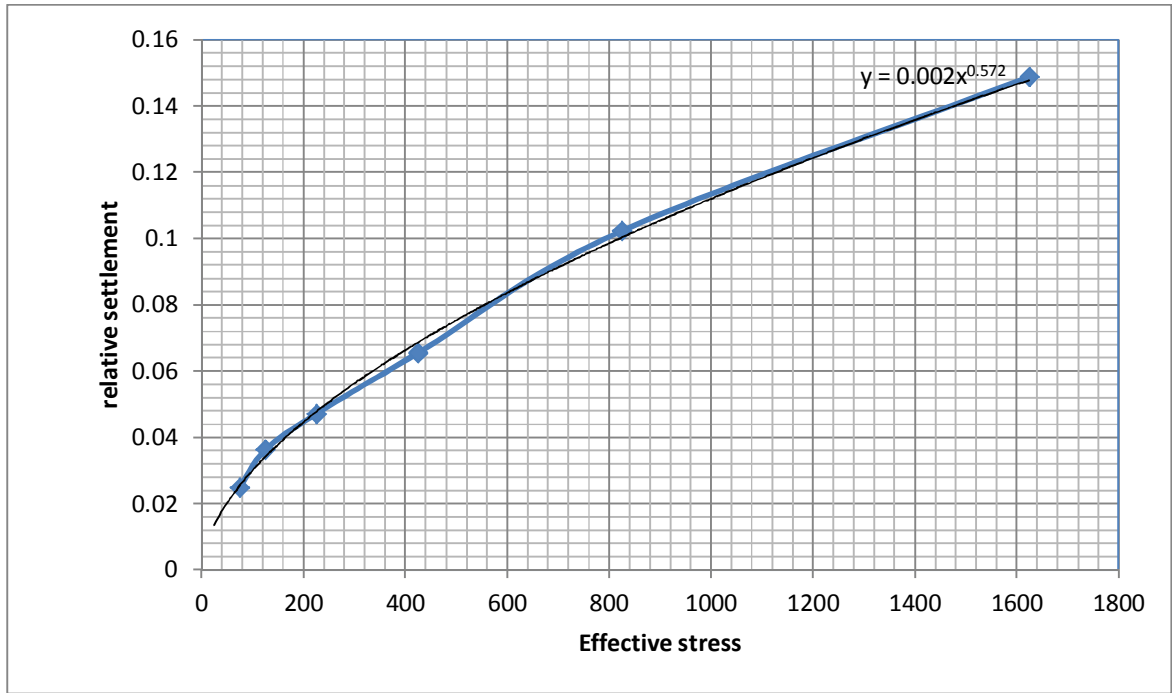


Fig 4.9 Relative settlement Vs Effective Stress (tp3 @ 2m)

Table 4.8 Effective stress and relative settlements for Tp2

<a href="#">tp2@2</a>	Applied pressure	Final dial reading	Total compression, $\Delta H$ , cm	Relative settlement $s' = \Delta H/H$	w	V (kN/m <sup>2</sup> )	Modulus of compressibility
Tp2at 2m depth	0	5.05	0	0	0.131	3835.8	$E_s = 3835.83$ $(\sigma_e')^{0.131}$
	25	5.07	0.023	0.013			
	75	5.28	0.23	0.014			
	100	5.36	0.30	0.01			
	200	5.63	0.58	0.029			
	400	6.03	0.98	0.049			
	800	7.052	1.99	0.099			
	1600	8.44	3.38	0.16			

Table 4.9 Effective stress and relative settlements for Tp3

<a href="#">tp3@2</a>	Applied pressure	Final dial reading	Total compression, $\Delta H$ , cm	Relative settlement $s'$ = $\Delta H/H$	w	V (kN/m <sup>2</sup> )	Modulus of compressibility
Tp3 at 2m depth	0	5	0	0	0.4281	2081.434	Es=2081.4 ( $\sigma_e'$ ) <sup>0.42805</sup>
	25	5.116	0.116				
	75	5.4968	0.49	0.024			
	125	5.724	0.72	0.036			
	225	5.942	0.94	0.047			
	425	6.31	1.31	0.065			
	825	7.04	2.04	0.102			
	1625	7.97	2.97	0.148			

Table 4.10 effective stress values

Tp4@2m depth $E_s=2795.25(\sigma_e')^{0.284}$		Tp1@2m depth $E_s=2573.61(\sigma_e')^{0.35}$		Tp2@2m depth, $E_s=3835.83(\sigma_e')^{0.13}$		Tp3@2m depth, $E_s=2573.61(\sigma_e')^{0.35}$		Tp5@2m depth, $E_s=500(\sigma_e')^{0.4249}$	
Effective stress, $\sigma'$	$E_s$ (kN/m <sup>2</sup> )	Effective stress, $\sigma_e'$	$E_s$ (kN/m <sup>2</sup> )	Effective stress, $\sigma'$	$E_s$ (kN/m <sup>2</sup> )	Effective stress, $\sigma'$	$E_s$ (kN/m <sup>2</sup> )	Effective stress, $\sigma_e'$	$E_s$ (kN/m <sup>2</sup> )
0	0	0	0	0	0	0	0	0	0
56.58	8811.56	31.875	8716.83	25	5847.76	25	8001.59	31.9	2176.63
104.385	10488.7	63.75	11128.6	75	6752.93	75	11784.5	63.8	2922.09
202.47	12664.2	127.5	14207.7	100	7012.28	125	14108.9	128	3922.85
402.7	15400.6	255	18138.7	200	7678.82	225	17356.1	255	5266.34
803.1	18742.5	510	23157.4	400	8408.71	425	21716.3	510	7069.95
1567.81	22671.5	765	26714.3	800	9207.98	825	27434.7	765	8399.19
		1530	34105.7	1600	10083.2	1625	34837.5	1530	11275.8

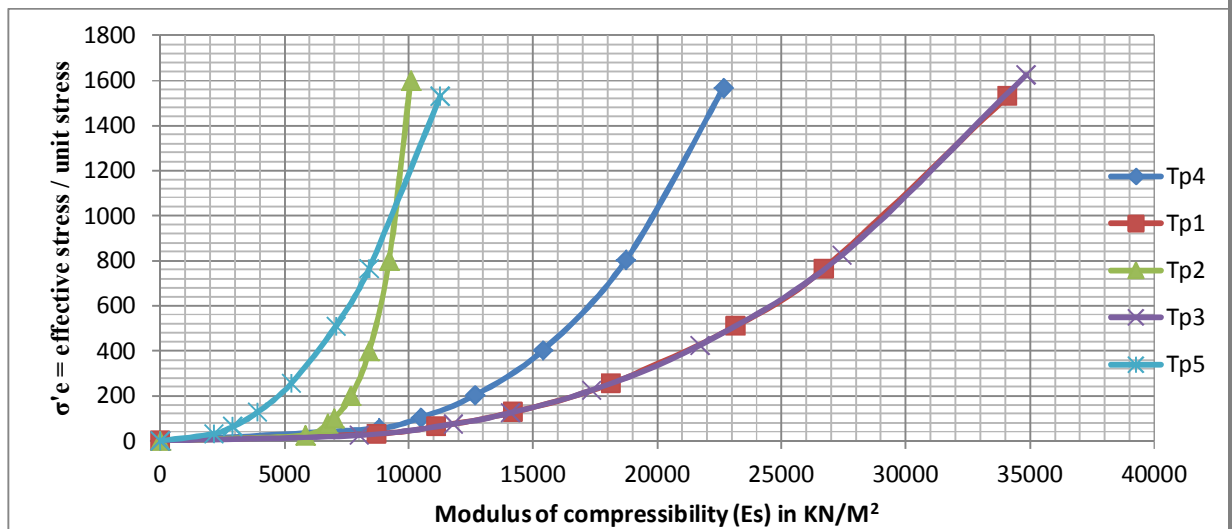


Fig 4.10 graph of modulus of compressibility vs effective stress

It can be observed that the modulus of compressibility increase with stress for a uniform soil formation.

### **4.3. Shear strength tests**

#### **4.2.5 General**

The purpose of shear strength testing is to establish representative values for the shear strength parameters. The drainage conditions during the test influence the measured values considerably. The most common laboratory methods employed to obtain shear strength parameters are direct shear test, triaxial compression test and unconfined compression test. For this research work, unconfined compression tests were conducted.

In this research, for the shear strength property, Only UCS test was conducted; this is because of following.

1. The soil is highly cohesive so that the UC strength can smoothly define its shear strength property.
2. As the Laboratory and the research area is too far, and only vehicle transportation available it's hard to get enough as well as good quality of undisturbed sample for other shear tests.
3. Inadequacy of Laboratory test equipment.

#### **4.2.5 Unconfined compression (UC) test**

The UC tests were carried out on 5 undisturbed samples obtained, by tube sampling, from the field. The undisturbed soil samples were gradually loaded at the two ends at a rate of 1.6mm/sec until it was destroyed by brittle or plastic failure. The vertical compression of the sample was measured throughout the loading process, while the sample may deform laterally without confinement. Details of testing procedures are given in BS 1377: Part 7: 1990. The results of UC tests are shown in Table 4.10. The high UCS values may be due to the cementing effect of particles by sesquioxides

Table 4.11 Unconfined compression test result for Tp2

Deformation	Load Dial	Sample	Strain	%	Corrected	Load	Stress
		DL (mm)					
0	0	0	0	0	12.56	0.00	0.00
20	72	0.2	0.0025	0.25	12.59	0.05	39.46
40	99	0.4	0.005	0.5	12.62	0.07	54.12
60	126	0.6	0.0075	0.75	12.66	0.09	68.70
80	150	0.8	0.01	1	12.69	0.10	81.58
100	168	1	0.0125	1.25	12.72	0.12	91.14
120	177	1.2	0.015	1.5	12.75	0.12	99.78
140	184.5	1.4	0.0175	1.75	12.78	0.13	103.59
160	198	1.6	0.02	2	12.82	0.14	110.47
180	213	1.8	0.0225	2.25	12.85	0.15	114.38
200	217.5	2	0.025	2.55	12.88	0.15	116.50
240	219	2.4	0.026	2.6374	12.90	0.15	117.14
260	208.5	2.6	0.028	2.8571	12.93	0.14	111.27
280	195	2.8	0.030	3.0769	12.96	0.13	103.83

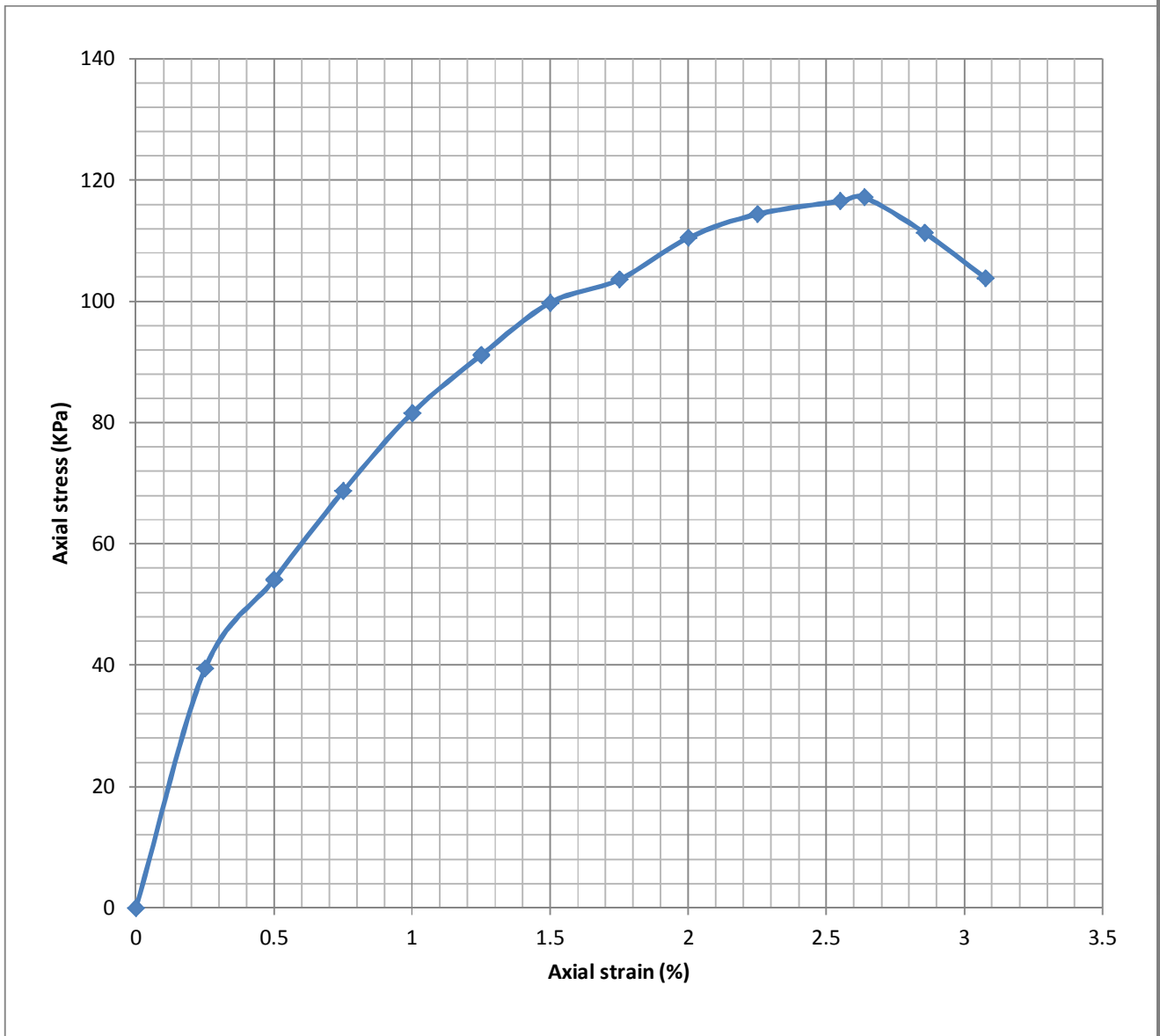


Fig 4.11. Unconfined compression test for Tp2

From the graph,

Unconfined compressive strength ( $q_u$ ) = 120kPa

The undrained shear strength ( $s_u$ ) =  $\frac{q_u}{2} = \frac{120}{2} = 60 \text{ Kpa}$

Table 4.12 UCS test results

<b>Test pits</b>	<b>Depth</b>	<b>Unconfined compression, <math>Q_u</math>, kPa</b>	<b>Undrained shear strength, <math>S_u</math>, kPa</b>
Tp1	2	151	75.5
Tp2	2	120	60
Tp3	2	138.5	69.25
Tp4	2	145	72.5
Tp5	2	226	113

## 5. Discussion and Analysis of test results

### 5.1. Index property test results

#### A. Particle size distribution

The particle size distribution of Dangila soils from various depths are shown in Figure 5.1. For TP1, the particle size distribution showed high percentage of fine fraction of 80 % at 0 -1 m depth after which the value increased to 100 % at 3 m depth. At TP4, the fine-grained fraction is 50 % at depth of 1m; increases to 90 % at 2m depth. The proportion of clay size particles also increase from 56% at 1 m depth to 70% at 2 m depth and decreases to 40 % at 3 m depth. The Dangila Soils are classified into two groups according to USCS (Unified Soils Classification System); an inorganic silt with high plasticity (MH) or inorganic clay with high plasticity (CH).

Table 5.1 Classification of Dangila soils according to USCS classification

Location	Test pit No	Depth of sampling (M)	Symbol
Dangila Town	Tp1	1	MH
		2	MH
		3	MH
	Tp2	1	CH
		2	CH
		3	CH
	Tp3	1	MH
		2	CH
		3	MH
	Tp4	1	MH
		2	CH
		3	CH

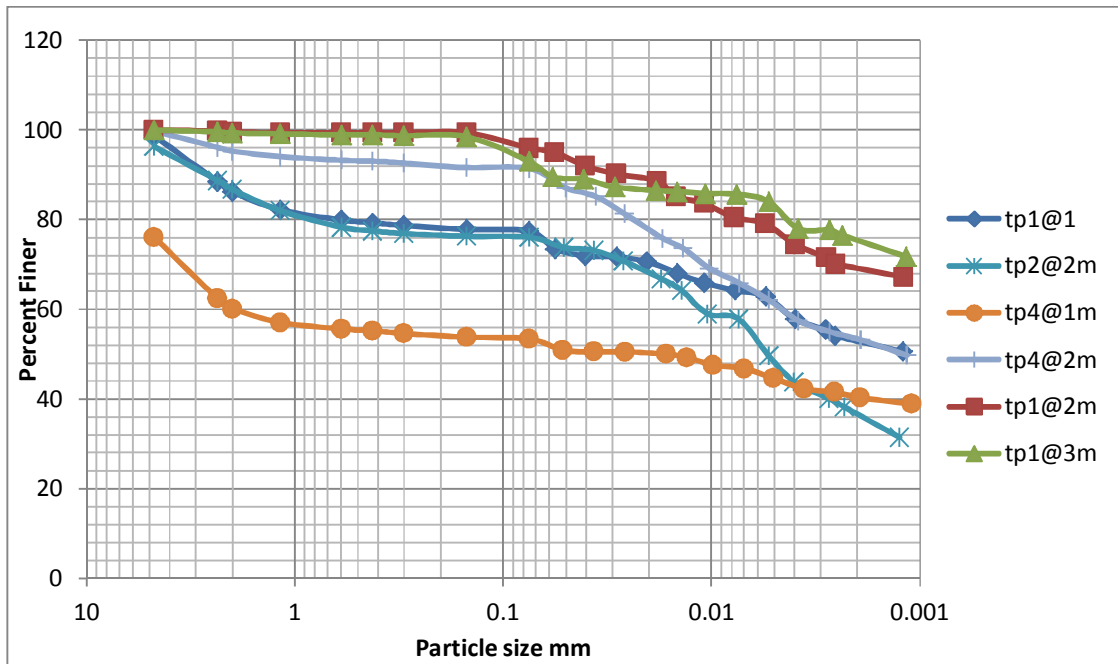


Fig 5.1 particle size distribution for different samples

B. Water content

Water content (w) tends to be low on the surface, and then increase with the soil depth. At TP2, the water content of samples was 38%, but 34 % for TP5 at 2m depth. Ground water table exists from 5-13 m depth as shown in table 5.2.

Table 5.2 depth of Ground water table

Areas around	Depth of Water table
Tp1	11-13m
Tp2	3-5m
Tp3	10-11m
Tp4	6-8m
Tp5	11-13m

### C. Specific gravity

The soil's specific gravity depends on the soil mineralogy and can show the history of the soil's weathering [9]. For TP1, The specific gravity increased from 2.61 at 1 m depth to 2.65 at 2 m depth, after which the values decreased to 2.64 at 3 m depth. The specific gravity lies in between 2.72 to 2.75, 2.8 to 2.9, and 2.80 to 2.90 for TP2, TP3 and TP4 respectively. But for TP5, it is 2.72 on average. The range of specific gravities is similar to the results obtained by Morin [9] and a previous study carried out in Addis Ababa [13] and Bahirdar. The variation of specific gravity with depth is shown in Fig.

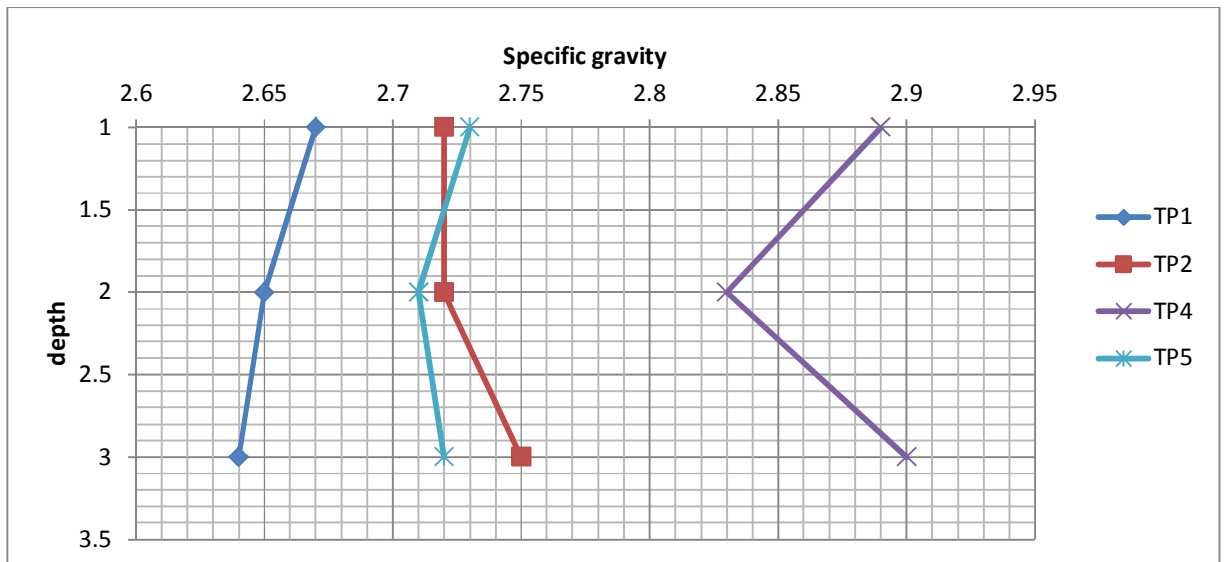


Fig 5.2. Variation of specific gravity with depth

### D. Atterberg Limits

The liquid limits vary from 54.1 % at 3m depth at TP2 to 77 % at TP3 at depth of 2m which the values decreased to 58 % at 10 m depth and 48 % at 25 m depth. The highest liquid limit, 77% recorded at 2 m depth is probably due to accumulation of clay size particles. The liquid limit of soils at TP2 is relatively low; it may be due to increased coarse-grained fraction at the soils. Classification of soils according to their plasticity index is described in the following table

Table 5.3. Soil classification according to plasticity index [7]

Plasticity index	Plasticity
0	Non-plastic
<7	Low plastic
7-17	Medium plastic
>17	Highly plastic

According to the classification, the PI values of Dangila soils lie in highly plastic zone. The moisture content indicates that the soils are in plastic state at insitu.

Table 5.4. Atterberg limits of soils

Test pit No	Depth of sampling (m)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)
Tp1	1	61.3	36.5	24.8
	2	62.4	40	22.4
	3	71.2	36.9	34.32
Tp2	1	56.2	28.3	27.9
	2	55.6	23.5	32.15
	3	54.1	24.1	30
Tp3	1	70	43.6	26.4
	2	77	35.8	41.2
	3	69.3	37.3	32
Tp4	1	68.4	37.4	30.98
	2	74.6	32.36	42.24
	3	75.1	30.2	44.9
TP5	2	68	34.7	33.3

## **5.2. Consolidation test results**

### **A. Coefficient of consolidation $C_v$ :-**

These values range between  $0.94 \times 10^{-03}$  and  $3.6 \times 10^{-03} \text{ cm}^2/\text{sec}$  for the soil analysis in study area with an average value of  $2.0 \times 10^{-03} \text{ cm}^2/\text{sec}$ . For all values,  $C_v$  is not constant but varies with consolidation pressure but show increasing and decreasing trends.

### **B. Coefficient of Permeability**

The coefficient of permeability estimated from one-dimensional consolidation test lies between  $1.8 \times 10^{-09}$  and  $9.89 \times 10^{-08} \text{ cm/sec}$  and shows that the soil is relatively impervious.

### **C. Modulus of Compressibility**

From one dimensional consolidation test result, the values of the modulus compressibility are obtained for the given effective stress. However, these values are a bit higher than the normal values. There was an attention to consider an average value of  $E_s$  but due to the variation in the coefficients  $v$  and  $w$  with each other it is better to consider as they are. For a given value of effective stress the corresponding value of  $E_s$  varies from site to site. This is because non uniformity of stiffness of soil sample.

### **D. Compression index and Swelling index**

The compression indexes of undisturbed samples are calculated. From the results, values of  $C_c$  and  $C_s$  are in the range from 0.13 to 0.22 and 0.019 to 0.033 respectively.

## **5.3. Unconfined compression test results**

The values of unconfined compressive strength lie from 140 to 200 kN/m<sup>2</sup>, at corresponding moisture content of 30 to 34 %.

Table 5.5. Unconfined compression test results

Test pits	Depth	Unconfined compression, $q_u$	Undrained shear strength, $s_u$
Tp1	2	151	75.5
Tp2	2	120	60
Tp3	2	138.5	69.25
Tp4	2	145	72.5
Tp5	2	226	113

The consistency of soil can be defined by the following table (19).

Table.5.6. Consistency in terms of Unconfined Compressive strength

S.No	Unconfined compressive strength	Consistency
1	<25	Very soft
2	25-50	Soft
3	50-100	Medium (firm)
4	100-200	Stiff
5	200-400	Very stiff
6	>400	Hard

According to the table, Dangila soils lie in a consistency range of stiff to very stiff.

#### 5.4. Comparison of the test results

The laboratory test results of this investigation can be compared with the other research as shown in the table.

Table 5.7 Comparison of Red clay soil

	<b>Previous researches</b>		<b>Present research</b>
Location	Addis Ababa (13)	Bahirdar (18)	Dangila
Soil type	Red clay	Red clay	Red clay
Liquid limit (%)	56-75	61-68	54-77
Plasticity index (%)	29-47	24-31	22-44
Specific gravity	2.66-2.77	2.75-2.83	2.61-2.90
Unconfined compressive Strength (kN /m <sup>2</sup> )	-	148-220	140-220
Expansion pressure (kN /m <sup>2</sup> )	-	-	20
Coefficient of Permeability (*10 <sup>-9</sup> cm/sec)	-	1.41-16.7	1.17-98.9
Compression index	-	0.266-0.41	0.13-0.42

From the above, Dangila soils are more plastic. They have also higher values of specific gravity and Stiffness compared to Bahirdar and Addis Ababa clay soils. From these properties, it can be justified that Dangila soils have heavy weight minerals like iron. From the Unconfined Compression and Consolidation results, it is also indicated that these soils are highly cemented lateritic soils. They are old in geologic time scales and have undergone many changes. This may be because of erosion, desiccation, groundwater fluctuations, aging, freeze-thaw cycles, wet-dry cycles, glaciation, and cementation in the past.

## 6. Conclusions and Recommendations

### 6.1. Conclusions

The following conclusions can be drawn based on the results.

1. According to the Unified Soil Classification System, Dangila soils are classified as either inorganic clays of high plasticity (CH) or high plastic silts (MH). The values of specific gravity range from 2.61 to 2.90.
2. The coefficient of permeability estimated from one-dimensional consolidation test lies between  $10^{-9}$  and  $10^{-8}$  cm/sec and shows that the soil is relatively impervious. The coefficient of consolidation also ranges in between  $0.94 \times 10^{-03}$  and  $3.6 \times 10^{-03}$  cm<sup>2</sup>/sec. Low permeability means a small amount of drainage and settlement as well. Soils in whole study area are suspected to be good foundation materials due to low values of rate of consolidation observed.
3. Dangila soils are found over-consolidated in nature with ratio of some degree because they are old in geologic time scales and have undergone many changes. This may be because of erosion, desiccation, groundwater fluctuations, aging, freeze-thaw cycles, wet-dry cycles, glaciation, and cementation in the past. Thus it is more advantageous as over consolidated soil compress less than Normal one.
4. The values of  $C_c$  and  $C_s$  are in the range from 0.13 to 0.22 and 0.019 to 0.033 respectively.
5. The values of unconfined compressive strength range from 140 to 220 kN/m<sup>2</sup>, whereas the undrained shear strength varies b/n 70 to 110 kPa. These values indicate that Dangila soils are naturally stiff.

## **6.2. Recommendations for further work**

1. The maximum sampling depths of the soil under investigations were 3m for the test samples. Detailed investigations have to be made along more samples of profiles.
2. The samples were collected from 485 km from the laboratory. Moreover, the sample was found sensitive to the energy applied to it. Hence, fresh soil representing the actual field condition has to be further investigated for Dangila soil in order to determine the actual soil behaviors.
3. The scope of this report did not include any environmental assessment or evaluation regarding the presence or absence of wetlands or hazardous or toxic materials in the soil

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## Appendix

### Preliminary foundation Evaluation

#### Strip Foundation Analysis

Soil Profile for Tp3

clay

$\gamma = 18.5 \text{ kN/m}^3$

$C_c = 0.3$

$C_r = 0.02$

$C_v = 1.6 \times 10^{-3} \text{ cm}^2/\text{s}$  (Maxamoung values calculated)

$W = 32.74\%$  - natural water content

$q_u = 69.25 \text{ kPa}$

#### Bearing Capacity Analysis

Using Terzaghi bearing capacity equations

When  $\phi = 0$ ,  $N_c = 5.7$  and  $N_q = 1.0$ .

$q_{ult} = C N_c S_{dc} + N_q Y D_f$

Hence,  $q_{ult} = 69.25 \times 5.7 + 18.5 D_f$

When designing a foundation on the basis of ultimate bearing capacity, a suitable factor of safety should be used to determine the allowable pressure so that the foundation system may be safe against shear failure. For footing foundations, a factor of safety of 2 to 3 is commonly used under normal loading conditions. Thus, for our condition we have taken a factor of safety 2.5 .

Proposed strip Footing	Depth (m)					
	1.5	2.0	2.5	3.0	3.5	4.0
	Allowable Bearing Pressures (kPa)					
	168.9	172.69	176.39	180	183.7	187.4

From the above analysis, allowable bearing pressure determined from UCS test result varies from 162 to 187.4 kPa.

## Settlement analysis

### Consolidation settlement

Since clay layer is considered as a foundation bearing layer, settlement analysis is conducted in addition to the shear failure criteria to determine the safe bearing pressure.

The amount of consolidation settlement is computed using the compression index  $C_c$ , recompression index ( $C_r$ ) obtained from the empirical equation. More commonly  $C_c$  is used in computing  $\Delta H$  as follows:

$$\Delta H = \frac{C_{rH}}{1+e_o} \left[ \log\left(\frac{P'_o + \Delta P}{P'_o}\right) \right] \text{ if } (P'_o + \Delta P) < P_c$$

or

$$\Delta H = \frac{C_{rH}}{1+e_o} \left[ \log\left(\frac{P'_o + \Delta P}{P'_o}\right) \right] + \frac{C_{cH}}{1+e_o} \left[ \log\left(\frac{P'_o + \Delta P}{P'_o}\right) \right] \text{ if } (P'_o + \Delta P) > P_c$$

Where,

$C_c$  = compression index from  $e$  vs.  $\log p$  plot or ( $C_c = 0.009[LL - 10]$ )

$C_r$  = recompression index from  $e$  vs.  $\log p$  plot

$e_o$  = average in-situ void ratio in the stratum for which  $C_c$  applies ( $e_o = w/G_s$ )=1.24

$H$  = thickness of stratum.

$p'_o$  = effective overburden pressure at mid-height of  $H$

$\Delta p$  = average increase in pressure from foundation in layer  $H$  in same unit as  $p'_o$

$P_c$  = Pre consolidation Pressure

The following data are used for calculating the settlement:

- Let the foundation be placed at a depth of 3 meters below NGL
- The maximum pressures on the foundation layer are assumed as 187kPa
- Compression index ,  $C_c$  and  $C_r$  from consolidation test results =0.3 &.02
- Initial void ratio,  $e_o = 1.24$
- $P_c=300$  Kpa
- Average Unit weight of the silty clay soil,  $\gamma_t = 18.5$  kPa

The settlement is checked for 4m thick silty clay layer stretched from the base of the foundation to the average level of bedrock.

$P_c = 300 \text{ kPa}$   
 $p'_o = 92.5 \text{ kPa}$   
 $\Delta p = 187 \text{ kPa}$

$P_c > p'_o + \Delta p$ , therefore the using appropriate formulas will be used.

Settlement Calculation using 180kPa as the maximum allowable Pressure, proposed depth of footing at 3m below NGL at TP3.

H (m)	H <sub>i</sub> (m)	p' <sub>o</sub> (kPa)	e <sub>oi</sub>	Δp (kPa)	p' <sub>o</sub> +Δp (kPa)	Cr log[(p' <sub>o</sub> +Δp)/ p' <sub>o</sub> ]	(1 + e <sub>oi</sub> )	S(cm)
	4	37	1.24	180	217	0.01536516	2.24	13.8

S=138 cm

Based on the above calculations the total primary consolidation settlement is 138mm .These values are above the limit of acceptable settlement. For isolated footings, the recommended maximum settlement value for clays is 75 mm (Table 5.7 of Bowles, 1997). Therefore, based on the analysis shown above it is not recommended to use the allowable bearing pressures shown in Table for the clay soil layer at foundation level of 3m below the natural ground level.

The allowable bearing capacity

H (m)	H <sub>i</sub> (m)	p' <sub>o</sub> (kPa)	e <sub>oi</sub>	Δp (kPa)	p' <sub>o</sub> +Δp (kPa)	Cr log[(p' <sub>o</sub> +Δp)/ p' <sub>o</sub> ]	(1 + e <sub>oi</sub> )	S(cm)
	4	37	1.24	60	97	0.0083714	2.24	7.50

Therefore, to govern to be safe for allowable settlements, the recommended bearing capacity should be 60kPa.

Item No	Depth footing (m)	q <sub>all</sub> (Kpa)
1	4	75
2	3	60
3	2	54
4	1	50

From the above analysis, it is recommended to use Shallow mat or Deep foundation for Heavy structures with greater effect than above bearing capacities

## Appendix A.

### A. Calculations on Consolidation test results

#### 1. Samples from TP1

##### Before test

Sample type = Undisturbed

Ring Area, cm<sup>2</sup> = 38.4845

Height of sample, mm = 20

Seating Load, kPa = 14

Initial moisture content, % = 34.8

Specific Gravity: = 2.71

Wet density, g/cm<sup>3</sup> = 1.845

##### *At the end of the test*

Final Moisture Content, % = 35.85%

Dry specimen wt (m<sub>s</sub>), gm = 106

### Calculations

$$\text{Height of solids, } H_s = \frac{M_s}{A \times G_s \times p_w} = \frac{106}{38.48 \times 2.71 \times 1} = 10.168 \text{ mm}$$

$$\text{Initial void ratio, } e_0 = \frac{H - H_s}{H_s} = \frac{20 - 10.16}{10.16} = 0.96$$

### A. Loading

First loading = 31.87 kPa

Initial dial reading = 1.93

Final dial reading=1.981

Change in thickness of sample=0.051mm

Height of specimen= $H_i - \Delta h = 20 - 0.051 = 19.949$  mm

Void ratio after consolidation

$$e_1 = \frac{H - \Delta h}{H_s} = \frac{19.949 - 0.051}{10.168} = 0.9618$$

Second loading=63.75

Initial dial reading=1.981

Final dial reading=2.067

Change in thickness of sample=  $2.067 - 1.981 = 0.086$ mm

Height of specimen= $H_i - \Delta h = 19.949 - 0.086 = 19.863$ mm

Void ratio after consolidation

$$e = \frac{H - \Delta h}{H_s} = \frac{19.863 - 0.086}{10.168} = 0.9533$$

Third loading=127.5

Initial dial reading=2.067

Final dial reading=2.191

Change in thickness of sample=  $2.191 - 2.067 = 0.124$ mm

Height of specimen= $H_i - \Delta h = 19.863 - 0.124 = 19.739$ mm

Void ratio after consolidation

$$e = \frac{H - \Delta h}{H_s} = \frac{19.739 - 0.124}{10.168} = 0.941$$

Fourth loading=255 kPa

Initial dial reading=2.191

Final dial reading=2.4

Change in thickness of sample= 2.4-2.191=0.209mm

Height of specimen= $H_i - \Delta h = 19.739 - 0.209 = 19.53$ mm

Void ratio after consolidation

$$e = \frac{H - H_s}{H_s} = \frac{19.53 - 10.168}{10.168} = 0.920$$

Fifth loading=510 kPa

Initial dial reading=2.4

Final dial reading=2.651

Change in thickness of sample= 2.651-2.4=0.251 mm

Height of specimen= $H_i - \Delta h = 19.53 - 0.251 = 19.279$ mm

Void ratio after consolidation

$$e = \frac{H - H_s}{H_s} = \frac{19.279 - 10.168}{10.168} = 0.896$$

Sixth loading=765 kPa

Initial dial reading=2.651

Final dial reading=2.82

Change in thickness of sample= 2.82-2.651=0.169mm

Height of specimen= $H_i - \Delta h = 19.279 - 0.169 = 19.11$ mm

Void ratio after consolidation

$$e = \frac{H - H_s}{H_s} = \frac{19.11 - 10.168}{10.168} = 0.879$$

Seventh loading=1530 kPa

Initial dial reading=2.82

Final dial reading=3.27

Change in thickness of sample= 3.27-2.81=0.45mm

Height of specimen= $H_i - \Delta h = 19.11 - 0.45 = 18.66$ mm

Void ratio after consolidation

$$e = \frac{H - H_s}{H_s} = \frac{18.66 - 10.168}{10.168} = 0.835$$

### *B. Unloading*

First un loading=765 kPa

Initial dial reading=3.27

Final dial reading=3.185

Change in thickness of sample= 3.27-3.185=0.085mm

Height of specimen= $H_i - \Delta h = 18.66 + 0.085 = 18.745$ mm

Void ratio after consolidation

$$e = \frac{H - H_s}{H_s} = \frac{18.745 - 10.168}{10.168} = 0.843$$

Second un loading, P=255 kPa

Initial dial reading=3.185

Final dial reading=3.04

Change in thickness of sample= 3.185-3.04=0.145mm

Height of specimen= $H_i - \Delta h = 18.745 + 0.085 = 18.89$ mm

Void ratio after consolidation

$$e = \frac{H - H_s}{H_s} = \frac{18.89 - 10.168}{10.168} = 0.857$$

third un loading, P=63.75 kPa

Initial dial reading=2.88

Final dial reading=3.04

Change in thickness of sample= 3.04-2.88=0.16mm

Height of specimen=Hi-Δh=18.89 +0.16=19.05mm

Void ratio after consolidation

$$e = \frac{H - \Delta h}{H_s} = \frac{19.05 - 0.168}{10.168} = 0.873$$

Final un loading, P=Seating load

Initial dial reading=2.88

Final dial reading=2.75

Change in thickness of sample= 2.88-2.75=0.13mm

Height of specimen=Hi-Δh=19.05 +0.13=19.18mm

Void ratio after consolidation

$$e = \frac{H - \Delta h}{H_s} = \frac{19.18 - 0.168}{10.168} = 0.886$$

### *Summary of calculations*

Table A1 void ratio for different pressure values

Applied Pressre	0	31.87	63.75	127.5	255	510	765	1530	765	255	63.75	0
E	0.96	0.961	0.953	0.941	0.92	0.89	0.8	0.84	0.843	0.858	0.873	0.886

The above values plotted and give Cc and Cs

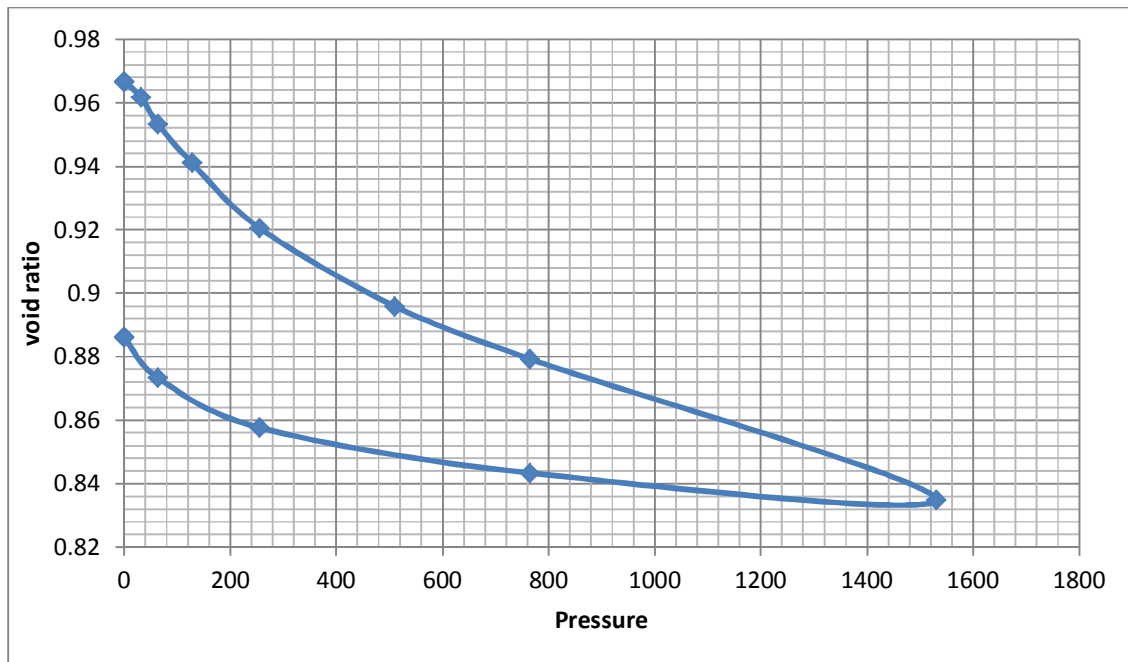


Fig A1 void ratio VS pressure

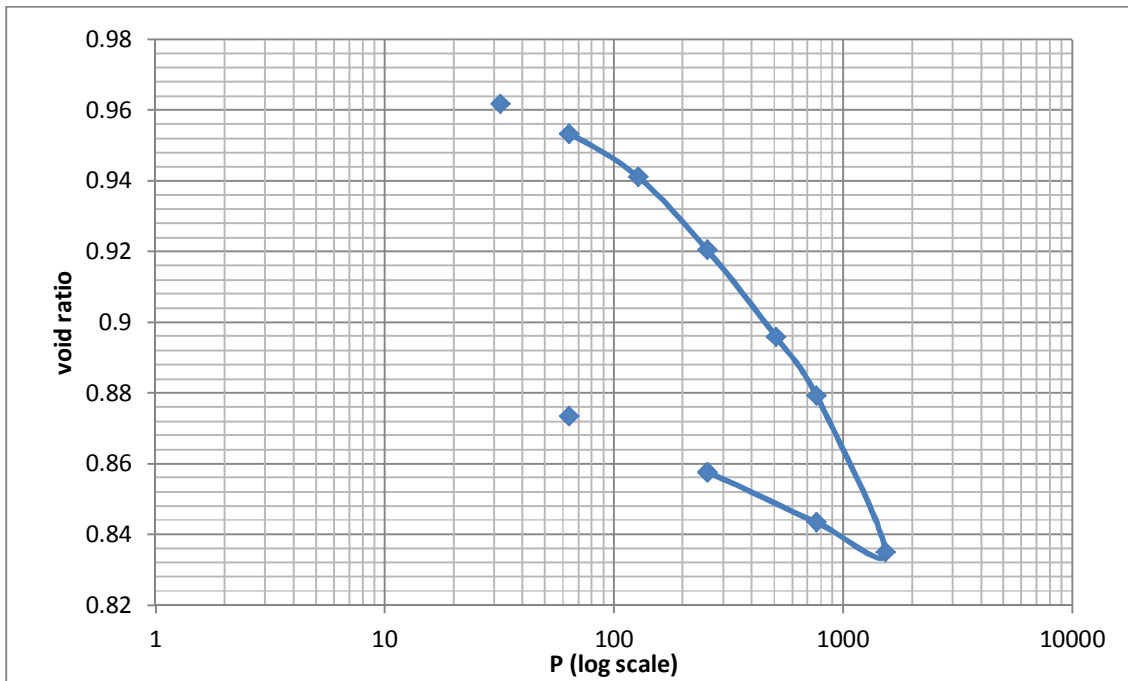


Fig A2 void ratio VS pressure

$$C_c = \frac{\Delta e}{\Delta \log P} = \frac{0.896 - 0.835}{\log_{10} 1530 - \log_{10} 510} = 0.1275$$

$$C_s = \frac{0.873 - 0.843}{\log_{10} 765 - \log_{10} 63.75} = 0.0277$$

Where ;

$C_c$  is compression index

$C_s$  is swelling index

C. Coefficient of consolidation and coefficient of permeability

Using log time method

$$C_v = \frac{0.197 d^2}{t_{50}}$$

Where  $d$ =drainage height= $\frac{\text{Average Height of specimen}}{2}$  (for double drainage)

$t_{50}$ =time for 50% degree of consolidation

1. First loading,  $P=31.875$  kPa

From the above graph

$D_{100}=1.98$

$D_0=1.95$

$$D_{50} = \frac{d_{100} + d_0}{2} = \frac{1.98 + 1.95}{2} = 1.965$$

$t_{50}=6$  min

For this loading,  $H_i=20$  mm

$H_f=19.949$  mm

$$H_{av} = \frac{H_i + H_f}{2} = \frac{20 + 19.949}{2} = 19.974 \text{ mm}$$

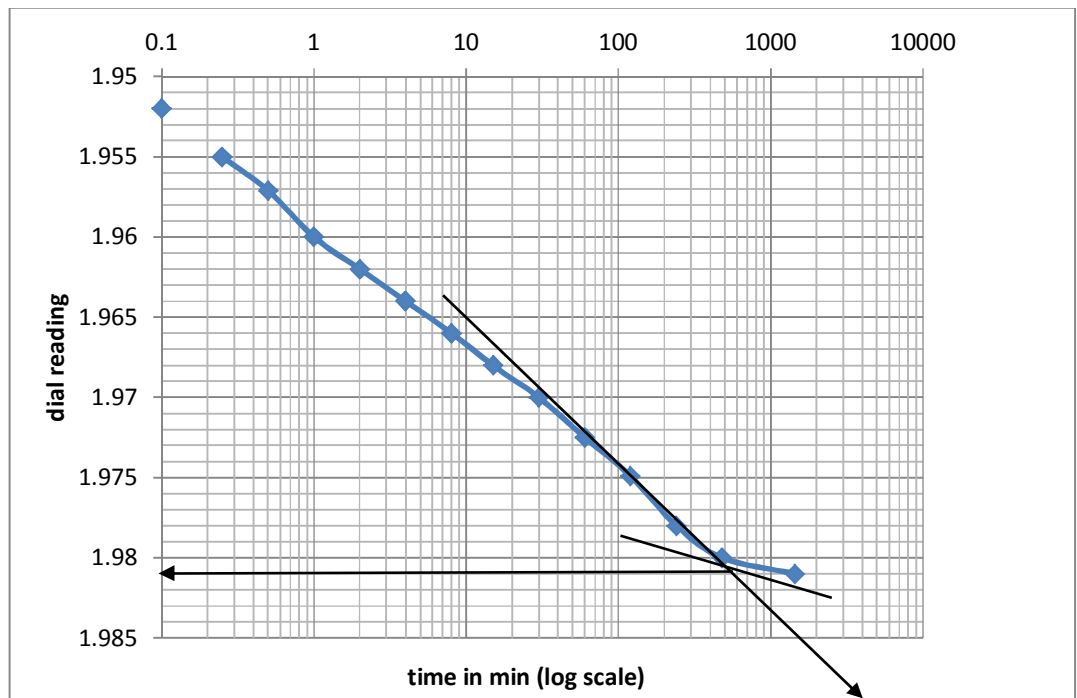


Fig A3 settlement vs. time for P=32 kPa

$$d = \frac{H_{av}}{2} = 9.978 \text{ mm}$$

$$Cv = \frac{0.197 d^2}{t_{50}} = \frac{0.197 \times 9.978^2}{6} = 3.268 \text{ mm}^2/\text{min}$$

$$av = \frac{\Delta e}{\Delta p} = \frac{0.966 - 0.9618}{31.875 - 0} = 0.000132 \text{ m}^2/\text{kN}$$

$$k = \frac{Cv av \gamma_w}{1 + e} = \frac{3.268 \times 10^{-6} \times 0.000132 \times 9.81}{1 + 0.9618} = 2.158 \text{ E-9 m/min}$$

$$= 3.596 \text{ E-9 cm/sec}$$

Where ; –

$Cv$  is coefficient of consolidation

$av$  is coefficient of compressibility

$k$  is coefficient of permeability

2. 2<sup>nd</sup> loading, P=63.75 kPa

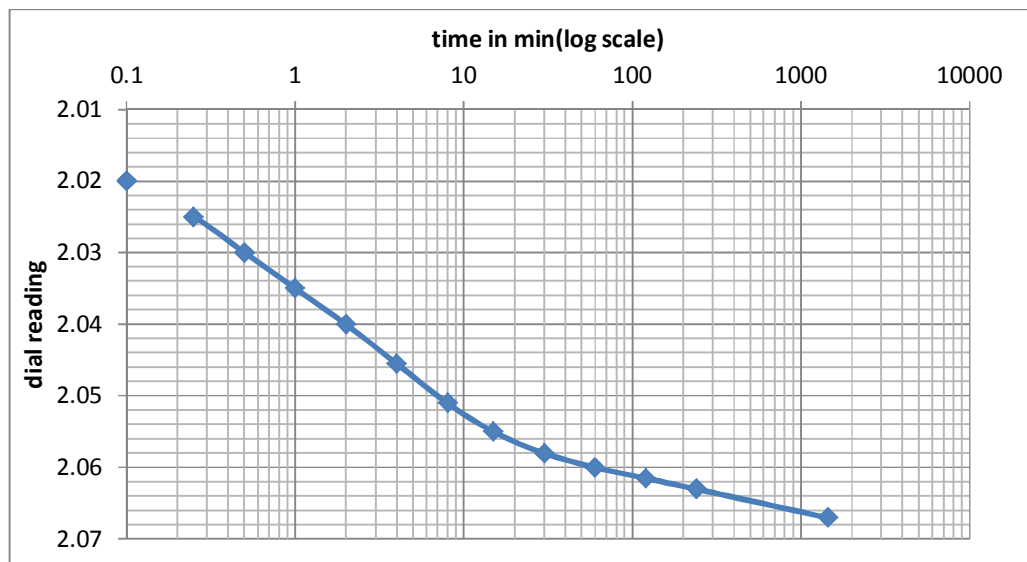


Fig A4 settlement vs. time for P=32 kPa

From the above graph

$$D_{100} = 2.058$$

$$D_0 = 2.015$$

$$D_{50} = \frac{d_{100} + d_0}{2} = \frac{2.058 + 2.015}{2} = 2.0365$$

$$t_{50} = 1.5 \text{ min}$$

For this loading,  $H_i = 19.949 \text{ mm}$

$$H_f = 19.86 \text{ mm}$$

$$H_{av} = \frac{19.949 + 19.86}{2} = 19.9065 \text{ mm}$$

$$d = \frac{H_{av}}{2} = 9.953 \text{ mm}$$

$$Cv = \frac{0.197 d^2}{t_{50}} = \frac{0.197 \times 9.953^2}{1.5} = 13.01 \text{ mm}^2/\text{min}$$

$$av = \frac{\Delta e}{\Delta p} = \frac{0.9618 - 0.9533}{63.7 \times 0.875} = 0.000267 \text{ m}^2/\text{kN}$$

$$k = \frac{Cv \cdot \gamma_w}{1 + e} = \frac{13.01 \times 10^{-6} \times 0.000267 \times 9.81}{1 + 0.9533} = 1.7445 \text{ E-}8 \text{ m/min}$$

$$= 2.908 \text{ E-}8 \text{ cm/sec}$$

3. 3<sup>rd</sup> loading,  $P = 127.5 \text{ kPa}$

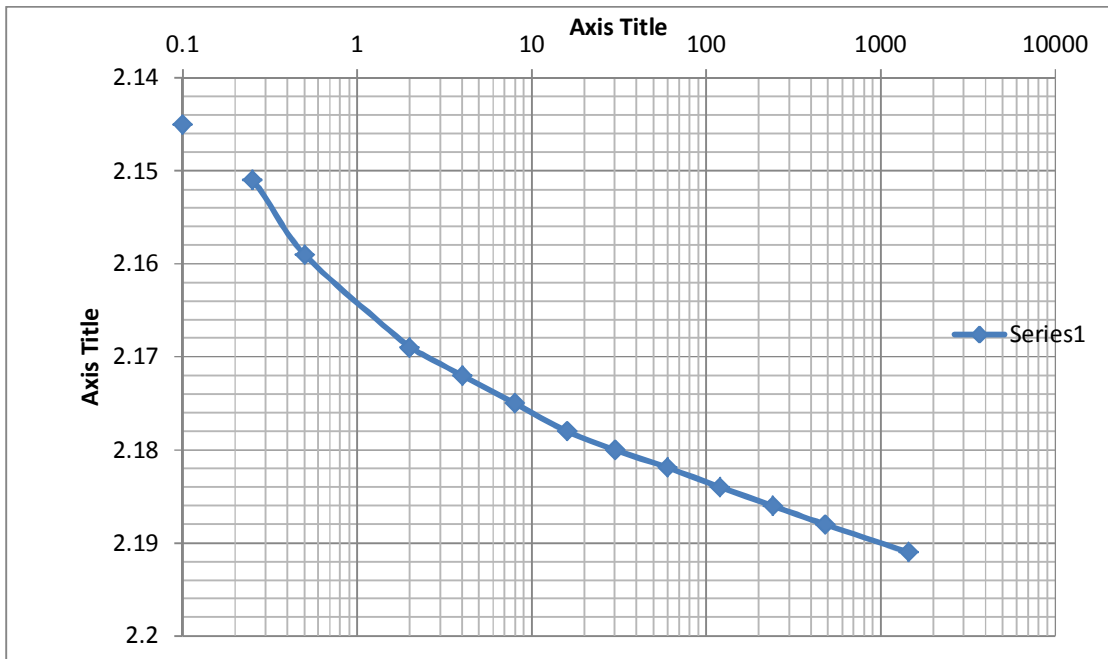


Fig A5 settlement vs. time for P=32 kPa

From the above graph

$$D_{100} = 2.17$$

$$D_0 = 2.133$$

$$D_{50} = \frac{d_{100} + d_0}{2} = \frac{2.133 + 2.17}{2} = 2.1515$$

$$t_{50} = 1 \text{ min}$$

$$\text{For this loading, } H_i = 19.864 \text{ mm}$$

$$H_f = 19.74 \text{ mm}$$

$$H_{av} = \frac{19.74 + 19.86}{2} = 19.8 \text{ mm}$$

$$d = \frac{H_{av}}{2} = 9.901 \text{ mm}$$

$$Cv = \frac{0.197 d^2}{t_{50}} = \frac{0.197 \times 9.953^2}{1} = 19.31 \text{ mm}^2/\text{min}$$

$$av = -\frac{\Delta e}{\Delta p} = \frac{0.9533 - 0.941}{127.5 - 75} = 0.000193 \text{ m}^2/\text{KN}$$

$$k = \frac{Cv \cdot \gamma_w}{1+e} = \frac{18.62 \cdot 10^{-6} \cdot 0.000193 \cdot 9.81}{1+0.941} = 1.8835 \text{E-}8 \text{m/min}$$

$$= 3.146 \text{E-}8 \text{cm/sec}$$

4. 4<sup>th</sup> loading, P=255 KPa

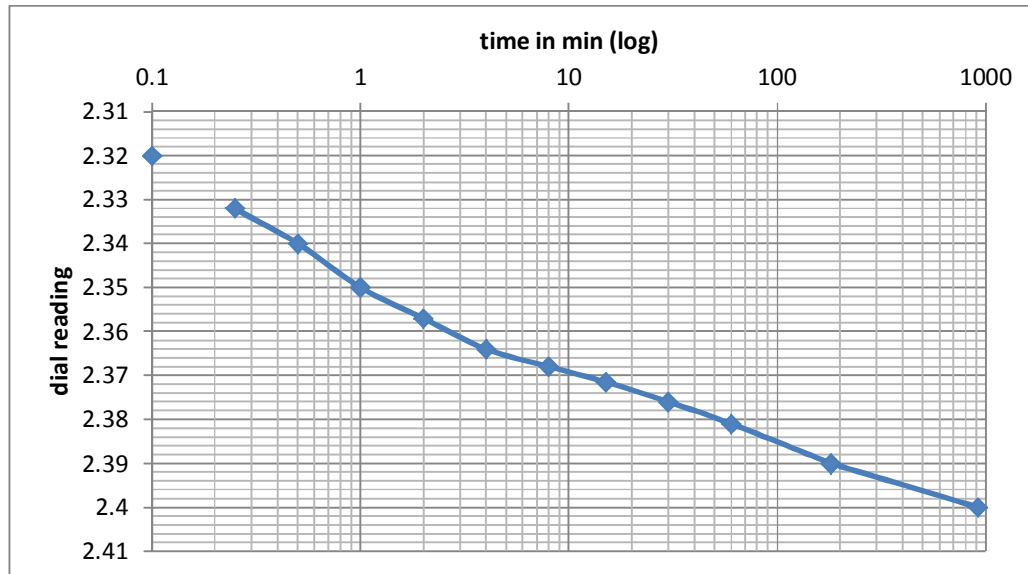


Fig A6 settlement vs. time for P=32 kPa

From the above graph

$$D_{100} = 2.363$$

$$D_0 = 2.32$$

$$D_{50} = \frac{D_{100} + D_0}{2} = \frac{2.363 + 2.32}{2} = 2.3415$$

$$t_{50} = 1 \text{ min}$$

For this loading,  $H_i = 19.74 \text{ mm}$

$$H_f = 19.53 \text{ mm}$$

$$H_{av} = \frac{H_i + H_f}{2} = \frac{19.74 + 19.53}{2} = 19.635 \text{ mm}$$

$$d = \frac{H_{av}}{2} = 9.8175 \text{ mm}$$

$$Cv = \frac{0.197 d^2}{t_{50}} = \frac{0.197 \times 9.8175^2}{1.5} = 13.086 \text{ mm}^2/\text{min}$$

$$av = -\frac{\Delta e}{\Delta p} = \frac{0.941-0.920}{2 \cdot 0.5} = 0.000165 \text{ m}^2/\text{kN}$$

$$k = \frac{Cv \cdot \gamma_w}{1+e} = \frac{13.086 \cdot 10^{-6} \cdot 0.000165 \cdot 9.81}{1+0.920} = 1.1032 \text{E-8 m/min}$$

$$= 1.839 \text{ E-8 cm/sec}$$

5. 5<sup>th</sup> loading, P=510 kPa

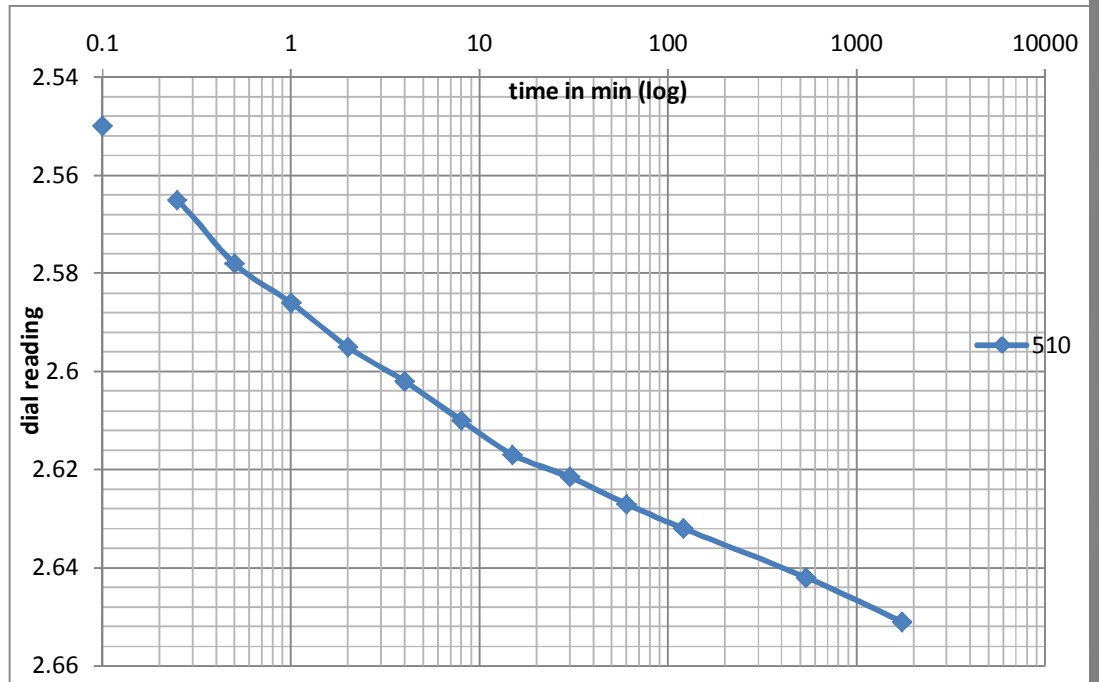


Fig A7 settlement vs. time for P=32 kPa

From the above graph

$$D_{100} = 2.603$$

$$D_0 = 2.544$$

$$D_{50} = \frac{D_0 + D_{100}}{2} = \frac{2.544 + 2.603}{2} = 2.592$$

$$t_{50} = 1.5 \text{ min}$$

For this loading,  $H_i = 19.53 \text{ mm}$

$$H_f = 19.28 \text{ mm}$$

$$H_{av} = \frac{19.28 + 19.53}{2} = 19.405 \text{ mm}$$

$$d = \frac{H_{av}}{2} = 9.705 \text{ mm}$$

$$Cv = \frac{0.197 d^2}{t_{50}} = \frac{0.197 \times 9.705^2}{1.5} = 12.36 \text{ mm}^2/\text{min}$$

$$av = -\frac{\Delta e}{\Delta p} = \frac{0.920 - 0.896}{510 - 2} = 0.0000941 \text{ m}^2/\text{kN}$$

$$k = \frac{Cv av \gamma_w}{1 + e} = \frac{12.36 \times 10^{-6} \times 0.0000941 \times 9.81}{1 + 0.896} = 6.022 \text{ E-9 m/min}$$

$$= 1.004 \text{ E-8 cm/sec}$$

6. 6<sup>th</sup> loading, P=765 kPa

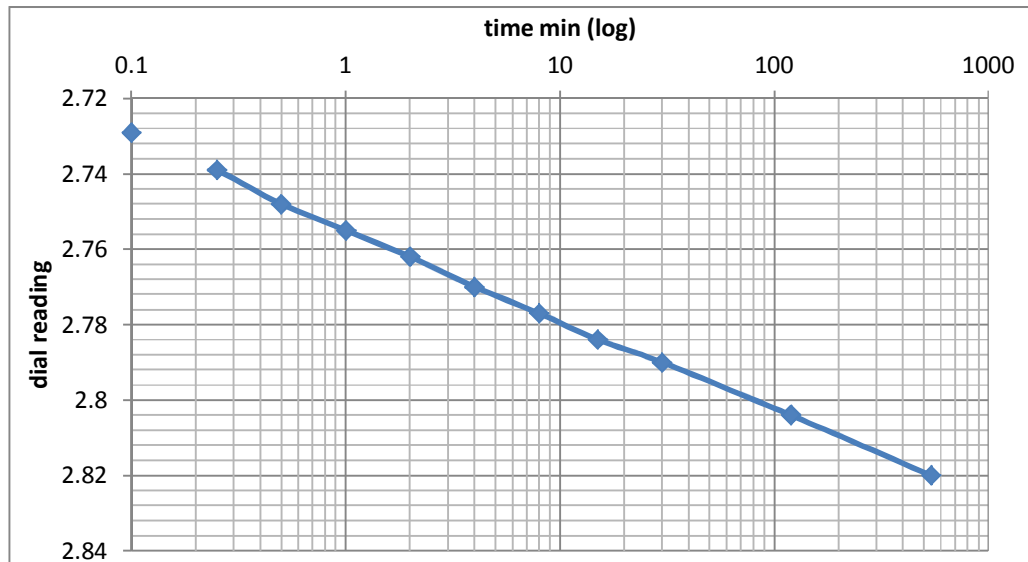


Fig A8 settlement vs. time for P=32 kPa

From the above graph

$$D_{100} = 2.785$$

$$D_0 = 2.723$$

$$D_{50} = \frac{D_{100} + D_0}{2} = 2.754$$

$$t_{50} = 1.5 \text{ min}$$

For this loading.  $H_i = 19.28 \text{ mm}$

$H_f = 19.11 \text{ mm}$

$$H_{av} = \frac{19.28 + 19.11}{2} = 19.195 \text{ mm}$$

$$d = \frac{H_{av}}{2} = 9.5975 \text{ mm}$$

$$Cv = \frac{0.197 d^2}{t_{50}} = \frac{0.197 \times 9.5975^2}{1.5} = 12.09 \text{ mm}^2/\text{min}$$

$$av = -\frac{\Delta e}{\Delta p} = \frac{0.896 - 0.879}{70} = 0.0000666 \text{ m}^2/\text{kN}$$

$$k = \frac{Cv a_v \gamma_w}{1 + e} = \frac{12.09 \times 10^{-6} \times 0.0000666 \times 9.81}{1 + 0.879} = 4.186 \text{ E-9 m/min}$$

$$= 6.977456 \text{ E-9 cm/sec}$$

7. 7<sup>th</sup> loading,  $P = 1530 \text{ kPa}$

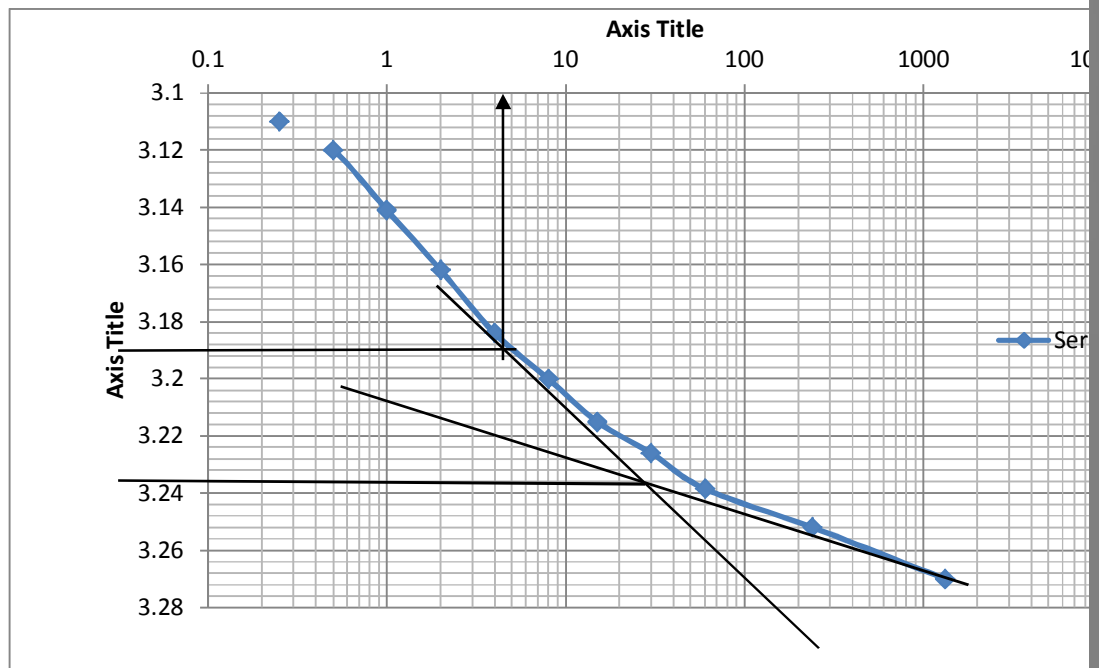


Fig A9 settlement vs. time for  $P = 32 \text{ kPa}$

From the above graph

$D_{100} = 3.225$

$$D_0 = 3.079$$

$$D_{50} = \frac{d_{100} + d_0}{2} = 3.152$$

$$t_{50} = 2.1 \text{ min}$$

For this loading,  $H_i = 19.11 \text{ mm}$

$$H_f = 18.661 \text{ mm}$$

$$H_{av} = \frac{18.661 + 19.11}{2} = 18.886 \text{ mm}$$

$$d = \frac{H_{av}}{2} = 9.44 \text{ mm}$$

$$Cv = \frac{0.197 d^2}{t_{50}} = \frac{0.197 \times 9.44^2}{2.1} = 8.36 \text{ mm}^2/\text{min}$$

$$av = -\frac{\Delta e}{\Delta p} = \frac{0.879 - 0.835}{1530} = 0.0000575 \text{ m}^2/\text{kN}$$

$$k = \frac{Cv a_v \gamma_w}{1 + e} = \frac{8.38 \times 10^{-6} \times 0.0000575 \times 9.81}{1 + 0.835} = 2.576 \text{ E-9 m/min}$$

$$= 4.2945 \text{ E-9 cm/sec}$$

## Appendix B.

### I. Grain size distribution Table and curves

1ED1	grain size(mm)	% finer passing
1	2	6
1	4.75	98.75
2	2.36	88.51
3	2	86.21
4	1.18	82.24
5	0.6	79.94
6	0.425	79.32
7	0.3	78.69
8	0.15	77.85
9	0.075	77.44
10	0.056446	73.43
11	0.040436	71.96
12	0.028575	71.7
13	0.020401	70.65
14	0.014596	68.03
15	0.010844	65.94
16	0.007712	64.21
17	0.005499	62.9
18	0.003977	57.82
19	0.002838	55.46
20	0.002551	54.15
21	<0.001205	0

a)

1ED2	grain size(mm)	% finer passing
1	2	6
1	4.75	100
2	2.36	99.79
3	2	99.58
4	1.18	99.38
5	0.6	99.38
6	0.425	99.38
7	0.3	99.38
8	0.15	99.38
9	0.075	98.96
10	0.056824	90.82
11	0.040431	90.52
12	0.028607	90.2
13	0.01834	88.62
14	0.014773	85.16
15	0.010825	83.65
16	0.007777	80.48
17	0.005517	79.07
18	0.003956	74.42
19	0.002816	71.61
20	0.002533	70.17
21	<0.00120	0

b)

1ED3	grain size(mm)	% finer passing
1	2	6
1	4.75	100
2	2.36	99.57
3	2	99.35
4	1.18	99.14
5	0.6	98.92
6	0.425	98.92
7	0.3	98.7
8	0.15	98.49
9	0.075	98.06
10	0.057965	89.03
11	0.040987	89.03
12	0.029154	87.28
13	0.01848	86.54
14	0.014628	86.28
15	0.010694	85.87
16	0.007571	85.7
17	0.005313	84.06
18	0.003831	77.96
19	0.002712	77.75
20	0.002351	76.55
21	<0.00120	0

c)

Table B1. Grain size distribution values for TP1 at depth of a) 1m, b) 2m and c) 3m

2ED1	Dia of openingsieve(mm)	% finer passing
	2	8
1	4.75	99.4
2	2.36	94.8
3	2	93.2
4	1.18	89.2
5	0.6	85.6
6	0.425	84.6
7	0.3	83.4
8	0.15	81.8
9	0.075	81.4
10	0.051239	76.1
11	0.036753	73.2
12	0.026517	71.13
13	0.01742	69.88
14	0.013939	64.9
15	0.010478	64.21
16	0.007452	59.01
17	0.005313	57.92
18	0.004017	49.65
19	0.002741	43.89
20	<0.0012057	0

a)

2ED2	grain size(mm)	% finer passing
	2	6
1	4.75	96.32
2	2.36	88.76
3	2	86.72
4	1.18	82.02
5	0.6	78.34
6	0.425	77.53
7	0.3	76.91
8	0.15	76.3
9	0.075	76.1
10	0.051239	73.8
11	0.036753	73.18
12	0.026517	70.82
13	0.01742	66.8
14	0.013939	64.21
15	0.010478	59.01
16	0.007452	57.92
17	0.005313	49.65
18	0.004017	43.89
19	0.002741	40.11
20	<0.0012057	0

b)

Table B2. Grain size distribution values for TP2 at depth of a) 1m, and b) 2m

TP4@1	grain size(mm)	% finer passing 100-(5)
1	2	6
1	4.75	76.1
2	2.36	62.41
3	2	60.16
4	1.18	57.09
5	0.6	55.66
6	0.425	55.26
7	0.3	54.64
8	0.15	53.82
9	0.075	53.42
10	0.051747	50.9
11	0.036718	50.58
12	0.025998	50.54
13	0.016556	50.05
14	0.013188	49.3
15	0.00983	47.56
16	0.006977	46.81
17	0.005047	44.65
18	0.003615	42.33
19	0.002563	41.57
20	0.001938	40.33
21	<0.001205	0

a)

TP4@2	grain size(mm)	% finer passing 100-(5)
1	2	6
1	4.75	99.8
2	2.36	96.12
3	2	95.3
4	1.18	94.07
5	0.6	93.26
6	0.425	93.05
7	0.3	92.64
8	0.15	91.62
9	0.075	91.42
10	0.050202	87.12
11	0.036002	85.19
12	0.026295	81.38
13	0.017169	75.86
14	0.01374	73.65
15	0.010247	69.18
16	0.007369	65.86
17	0.005303	62.05
18	0.003832	57.47
19	0.002708	55.01
20	0.001921	53.22
21	<0.001205	0

b)

Table B3. Grain size distribution values for TP4 at depth of a) 1m, and b) 2

## II. Consolidation test

### 1. Void ratio vs. Pressure

tp1@3m depth		
No	Applied pressure	e
1	0	0.966795
2	31.875	0.96178
3	63.75	0.953323
4	127.5	0.941129
5	255	0.920576
6	510	0.895892
7	765	0.879273
8	1530	0.83502
9	765	0.843379
10	255	0.857638
11	63.75	0.873373
12	0	0.886157

a)

Tp 2@ 2m		
No	Applied pressure	e
1	0	1.122892
2	25	1.120347
3	75	1.098269
4	100	1.090202
5	200	1.060906
6	400	1.018235
7	800	0.910605
8	1600	0.763276
9	800	0.768159
10	200	0.780472
11	75	0.787997
12	0	0.821656

b)

Table B4 void ratio values for different values of pressure a) for TP1 at 3m depth and

b) for TP2 at 2m depth

tp4@2m		
<u>No</u>	Applied pressure	e
1	0	1.110928
2	56.59	1.107655
3	104.38	1.100479
4	202.4	1.075464
5	402.68	1.028707
6	803.087	0.96844
7	1567.81	0.893608
8	803.087	0.902474
9	202.473	0.923161
10	56.589	0.937938
11	0	0.95378

a)

tp5@2m		
<u>No</u>	Applied pressure	e
1	0	0.985527
2	31.875	0.953758
3	63.75	0.933903
4	127.5	0.910672
5	255	0.883471
6	510	0.852496
7	765	0.833634
8	1530	0.780025
9	765	0.786021
10	255	0.801865
11	63.75	0.821224
12	0	0.849518

b)

Table B5 void ratio values for different values of pressure a) for TP4 at 2m depth and

b) for TP5 at 2m depth

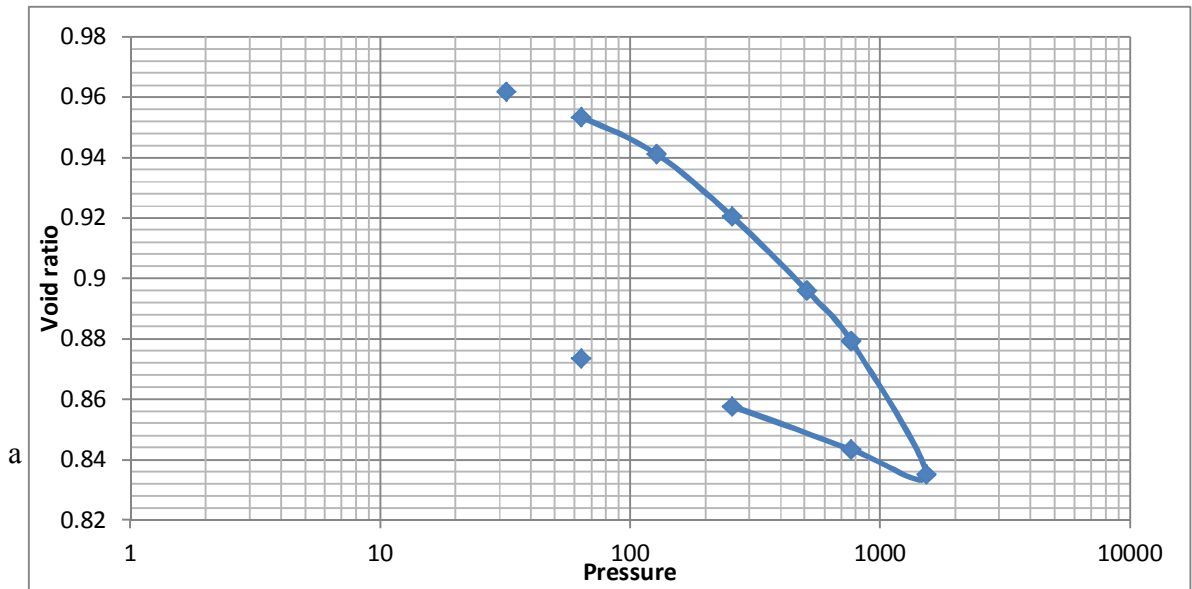


Fig B1 Void ratio vs. pressure for TP1 at 3m depth

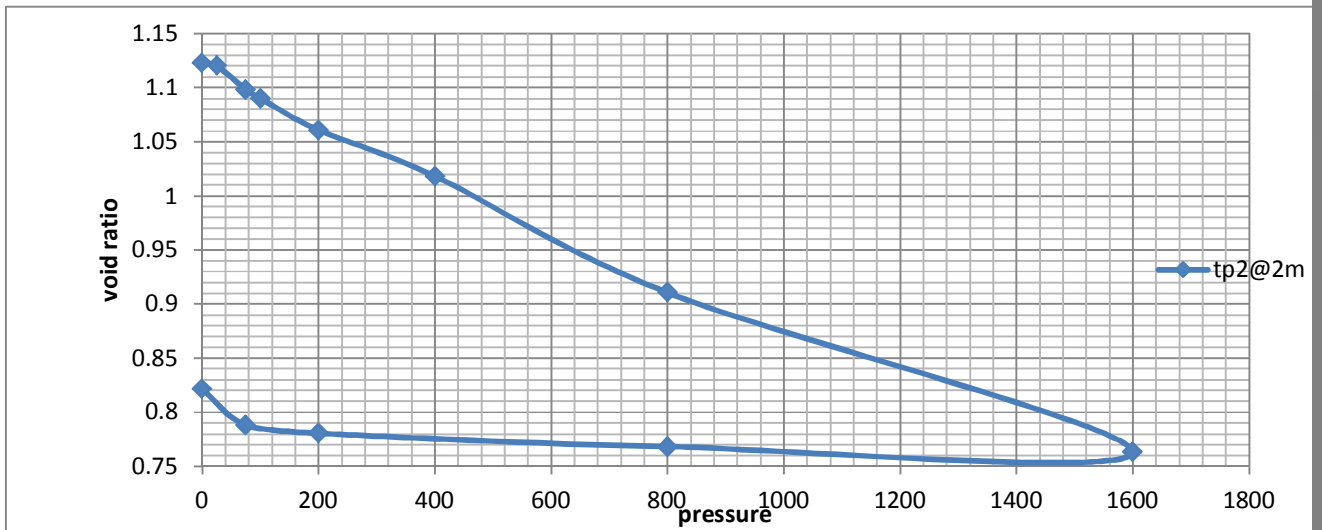
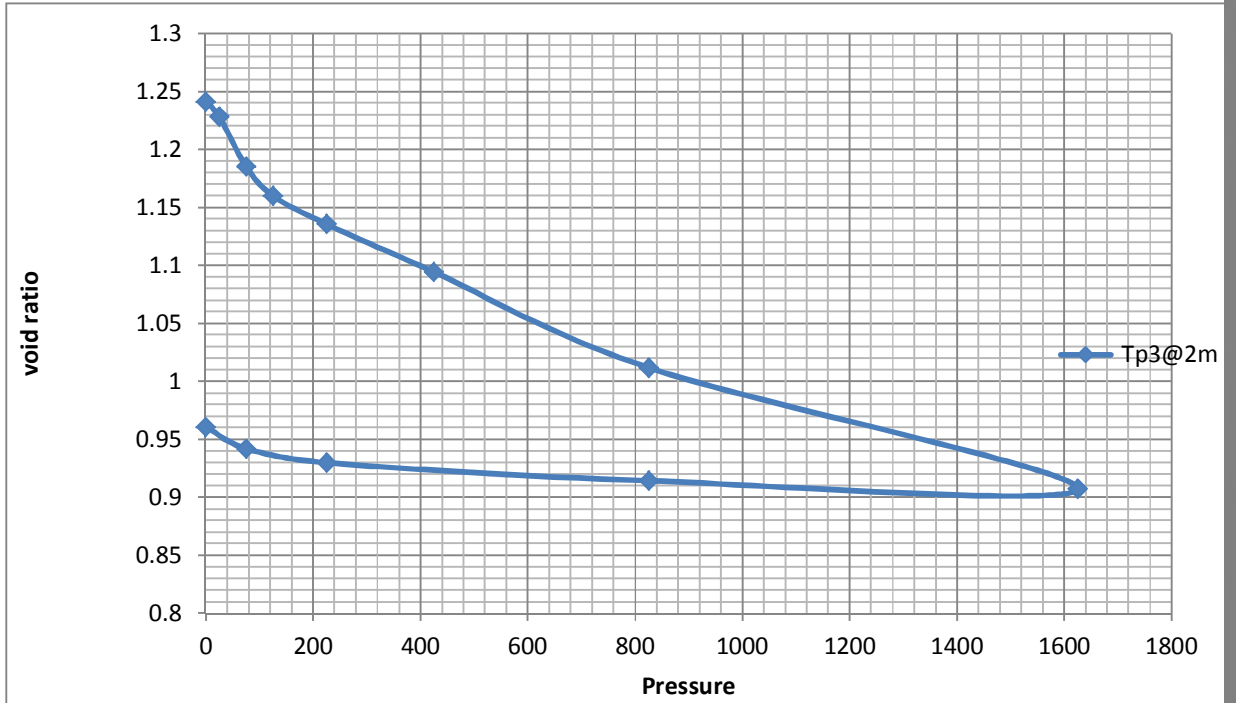
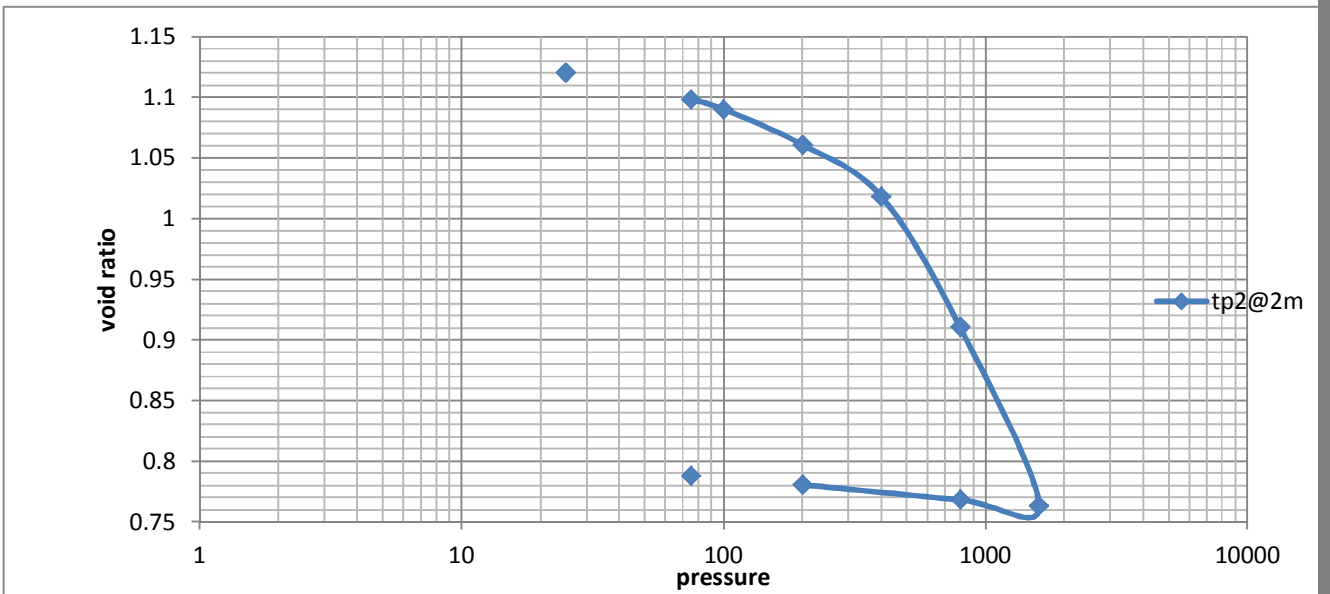
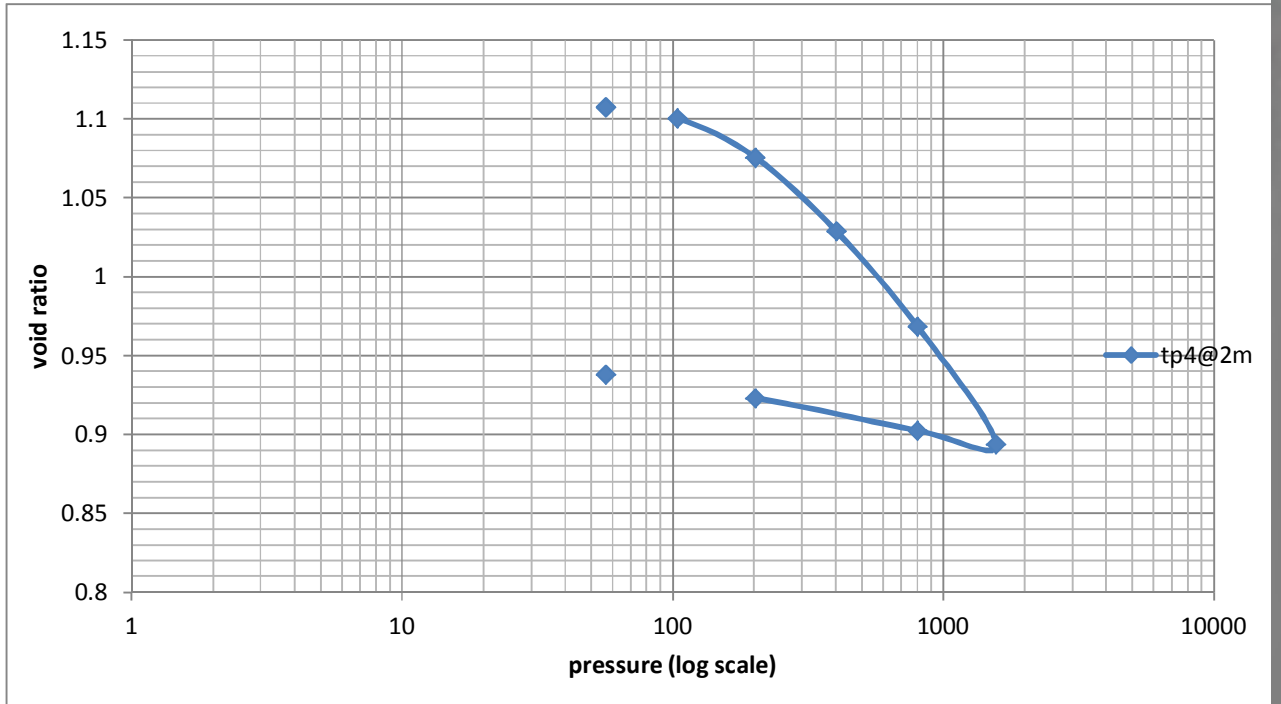


Fig B2 Void ratio vs. pressure for TP2 at 2m depth

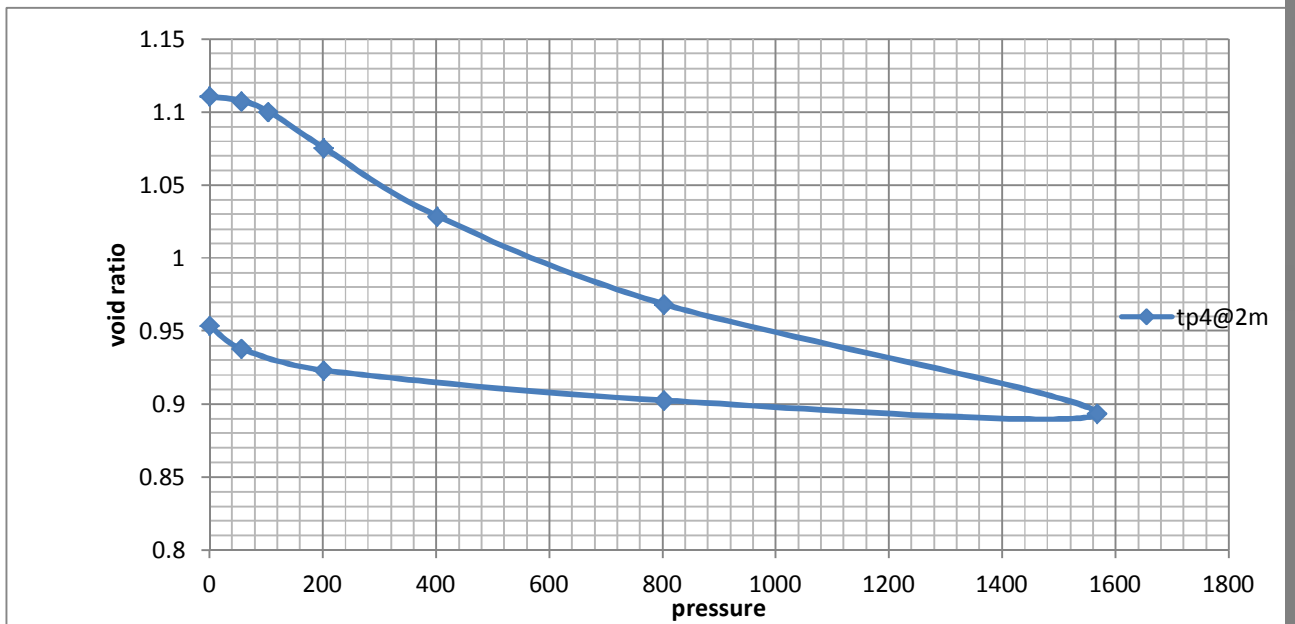


a)



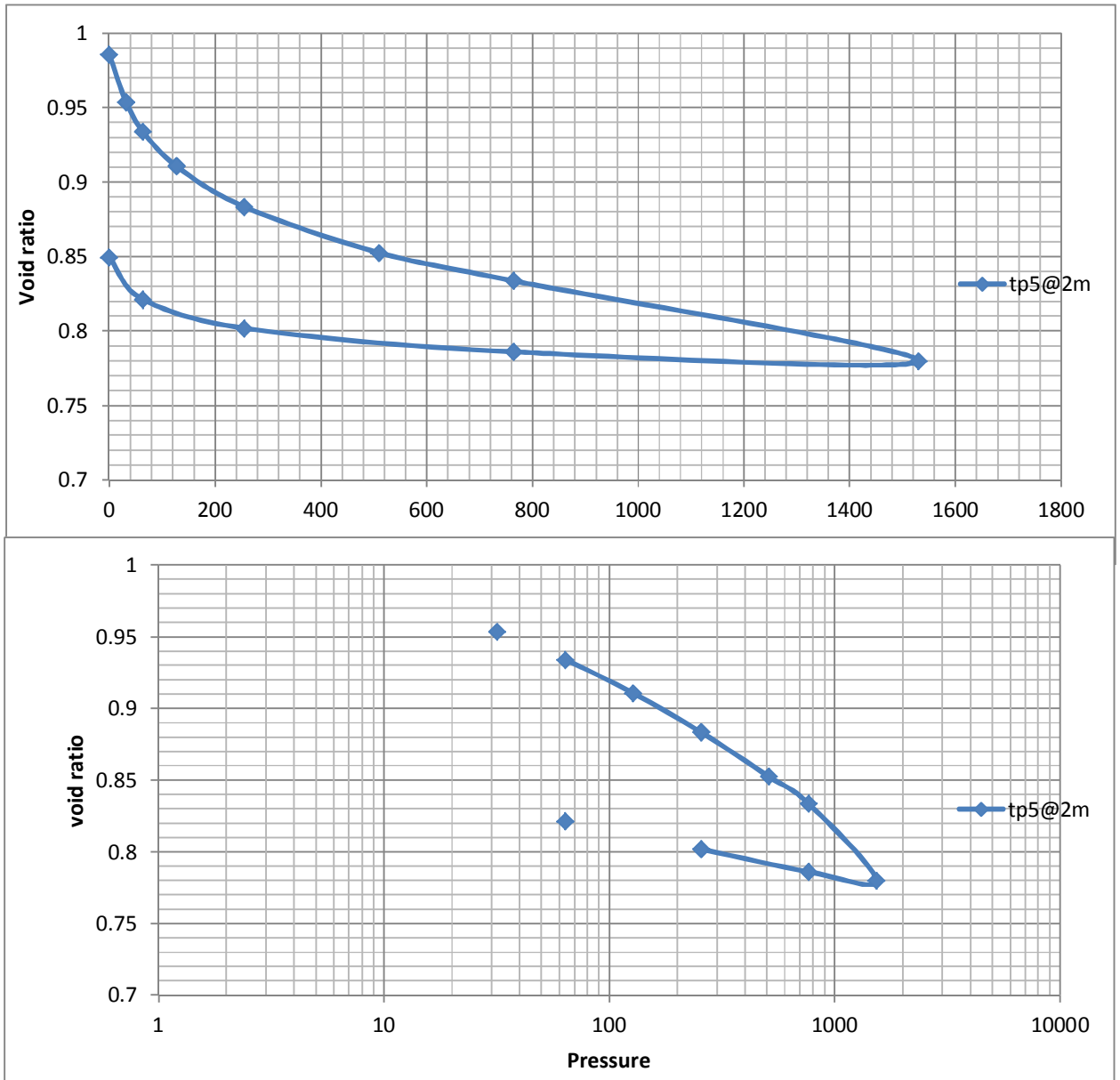


a)



b)

Fig B3 Void ratio vs. pressure for TP4 at 2m depth



b)

Fig B4 Void ratio vs. pressure for TP5 at 2m depth

Test pit no	Depth	Pressure P kN/m <sup>2</sup>	Void ratio ef	Coefficient of consolidation Cv *10-3 (Cm2/sec)	Coefficient of compressibility *10-5 ( m2/kN)	Coefficient of permeability *10-9 ( cm/sec)
Tp3	2	0	1.2409		0	
		25	1.2279	1.162664	85.336	52.76111
		75	1.1853	1.081616	51.992	37.45628
		125	1.1598	1.619723	50.914	20.5735
		225	1.1354	1.612088	24.427	18.09028
		425	1.0941	1.241951	20.6395	6.34548
		825	1.0116	1.130292	20.6165	6.231356
		1625	0.9074	0.94753	13.02138	1.868399

Table B 6. Relationship between Void ratio and coefficient of permeability

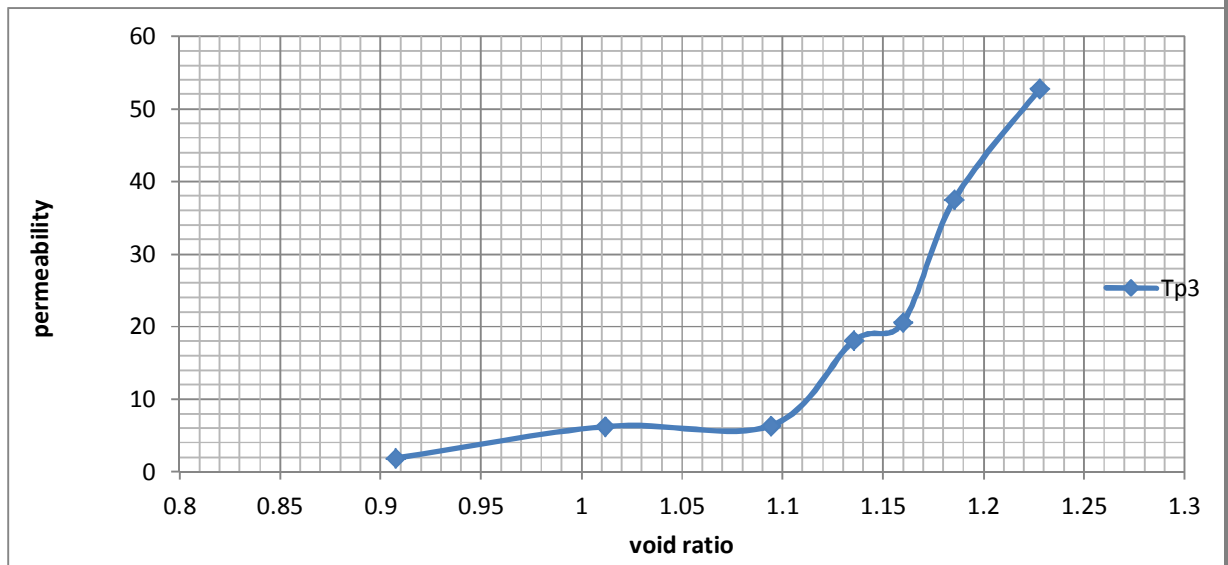


Fig B5 coefficient of permeability vs. void ratio for TP3

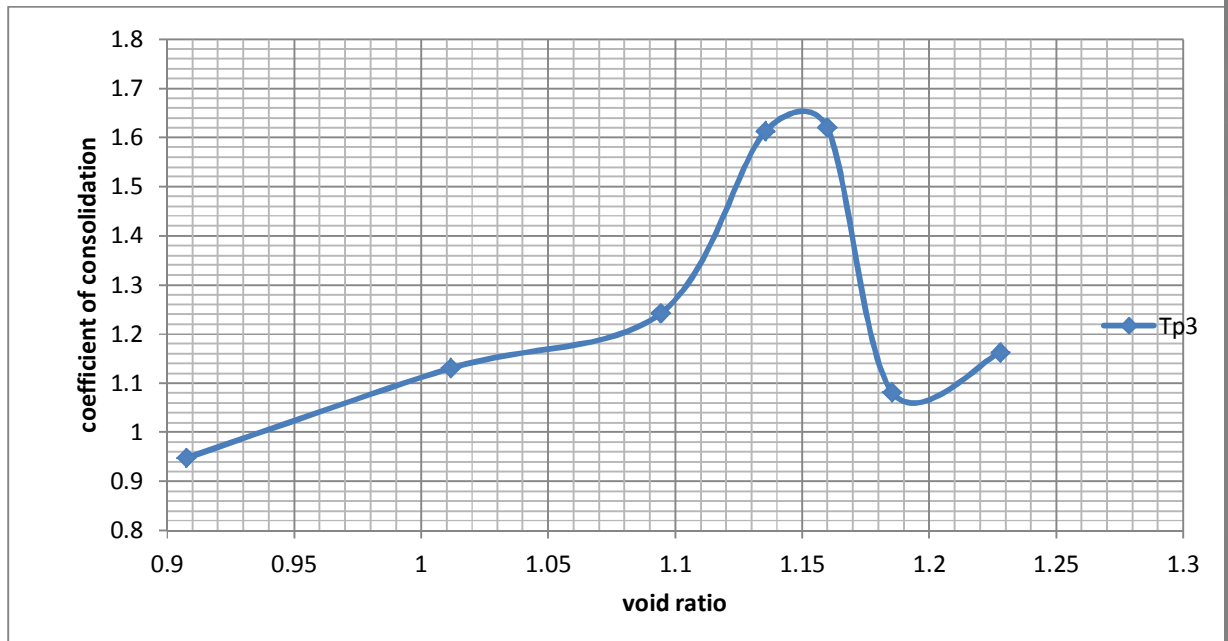


Fig B6 coefficient of consolidation vs. void ratio for TP3

Test pit no	Depth	Pressure P kN/m <sup>2</sup>	Void ratio ef	Coefficient of consolidation Cv *10 <sup>-3</sup> (Cm <sup>2</sup> /sec)	Coefficient of compressibility *10 <sup>-5</sup> (m <sup>2</sup> /kN)	Coefficient of permeability *10 <sup>-9</sup> (cm/sec)
Tp4	2	0	1.11			
		56.59	1.107	1.025	25.519	14.81
		104.38	1.100	1.312	23.34	13.66
		202.4	1.075	1.0943	15.05	12.37
		402.68	1.028	1.82	15.01	7.67
		803.087	0.96	1.31	9.78	6.64
		1567.81	0.893	1.437	5.78	1.17

Table B7 Relationship between Void ratio and coefficient of permeability

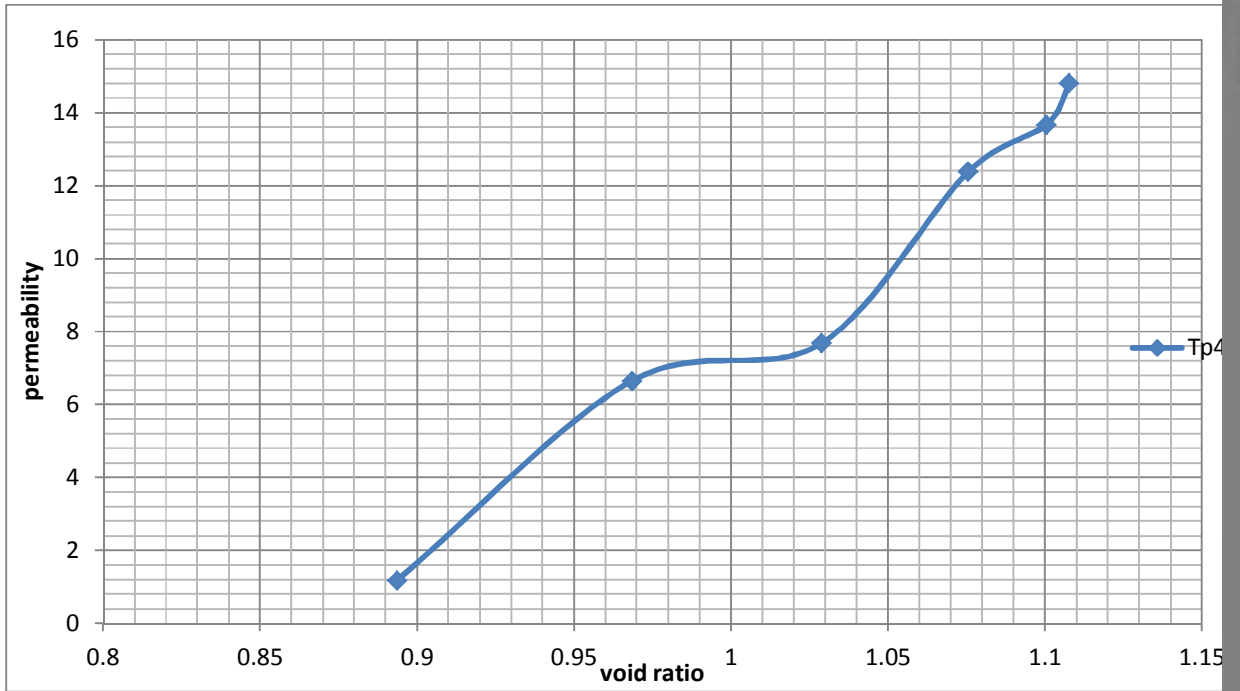


Fig B7 coefficient of permeability vs. void ratio for TP4

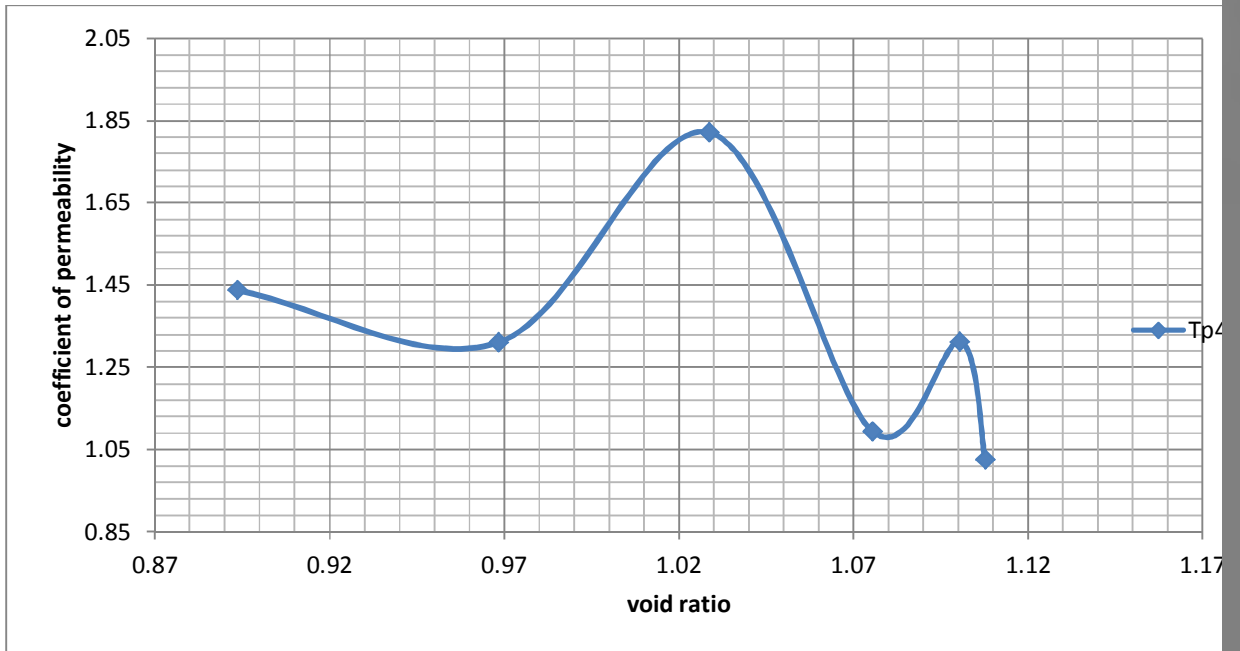


Fig B8 coefficient of consolidation vs. void ratio for TP4

Test pit no	Depth	Pressure P kN/m <sup>2</sup>	Void ratio ef	Coefficient of consolidation Cv *10 <sup>-3</sup> (Cm <sup>2</sup> /sec)	Coefficient of compressibility *10 <sup>-5</sup> (m <sup>2</sup> /kN)	Coefficient of permeability *10 <sup>-9</sup> (cm/sec)
Tp5	2	0	0.98			
		31.875	0.95	2.53	99.66	98.92
		63.75	0.93	3.130	62.290	67.39
		127.5	0.910	3.60	36.44	33.13
		255	0.88	2.98	21.33	26.81
		510	0.85	3.61	12.147	23.25
		765	0.83	3.609	7.39	14.28
		1530	0.780	2.90	7.007	11.23

Table B8 Relationship between Void ratio and coefficient of permeability

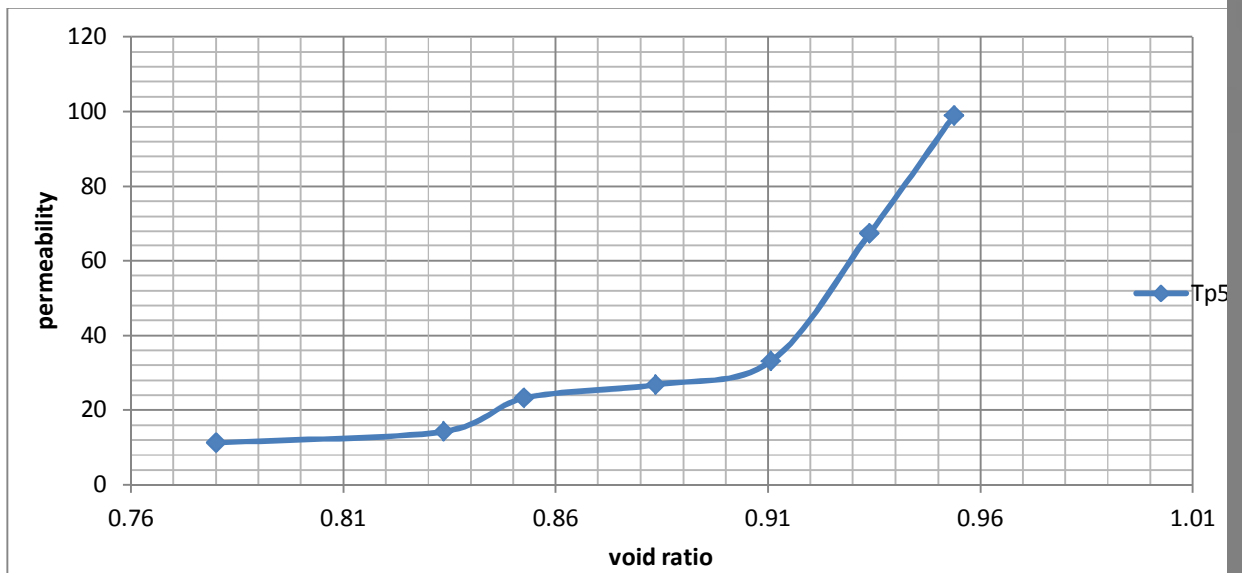


Fig B9 coefficient of permeability vs. void ratio for TP5

Location	Test pit no	Depth	Pressure P kN/m <sup>2</sup>	Void ratio ef	Coefficient of consolidation Cv *10 <sup>-3</sup> (Cm <sup>2</sup> /sec)	Coefficient of compressibility *10 <sup>-5</sup> (m <sup>2</sup> /kN)	Coefficient of permeability *10 <sup>-9</sup> (cm/sec)
Dangila	Tp2	2	0	1.1228			
			25	1.1203	3.1309	44.156	64.6
			75	1.0982	3.60	32.26	54.3
			100	1.0902	2.98	29.29	49.3
			200	1.0609	3.60	26.90	41.3
			400	1.0182	3.61	21.33	37.3
			800	0.9106	2.909	18.41	29.3
			1600	0.7632	2.53	10.18	2.3

Table B9 Relationship between Void ratio and coefficient of permeability

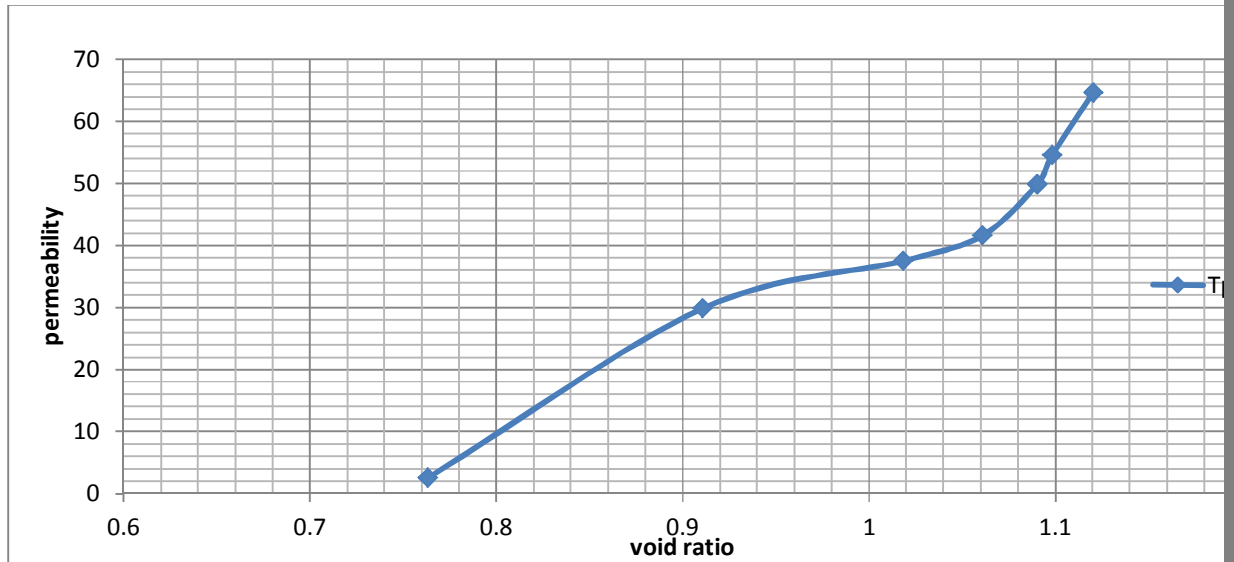


Fig B9 coefficient of permeability vs. void ratio for TP2

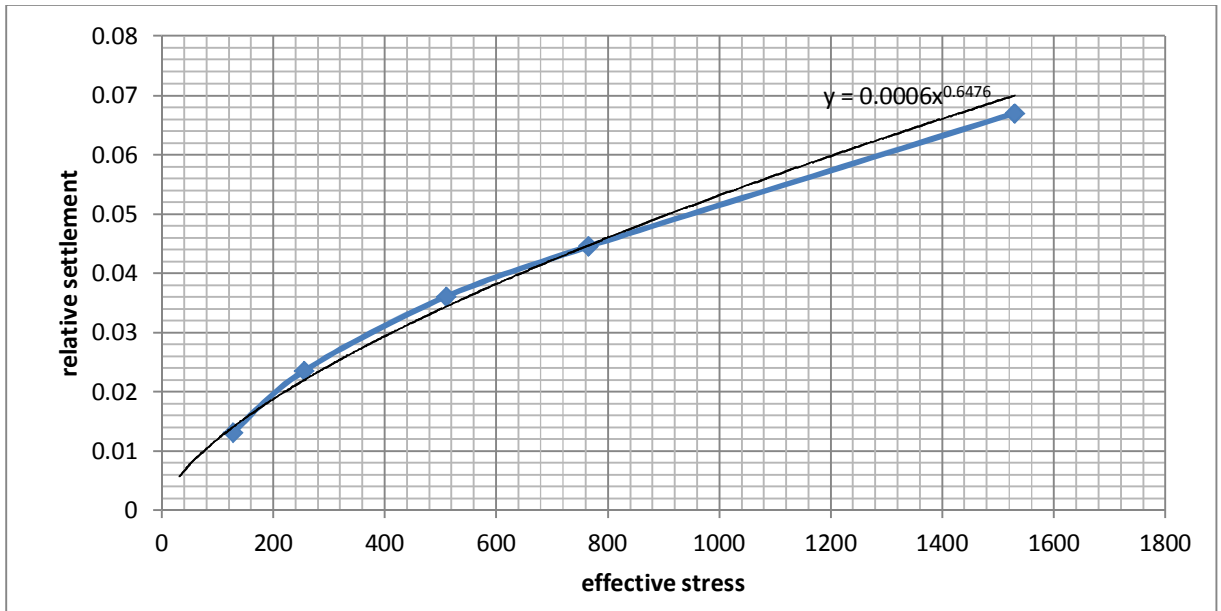


Fig B11 Relative settlement Vs Effective Stress (tp1 @ 2m)

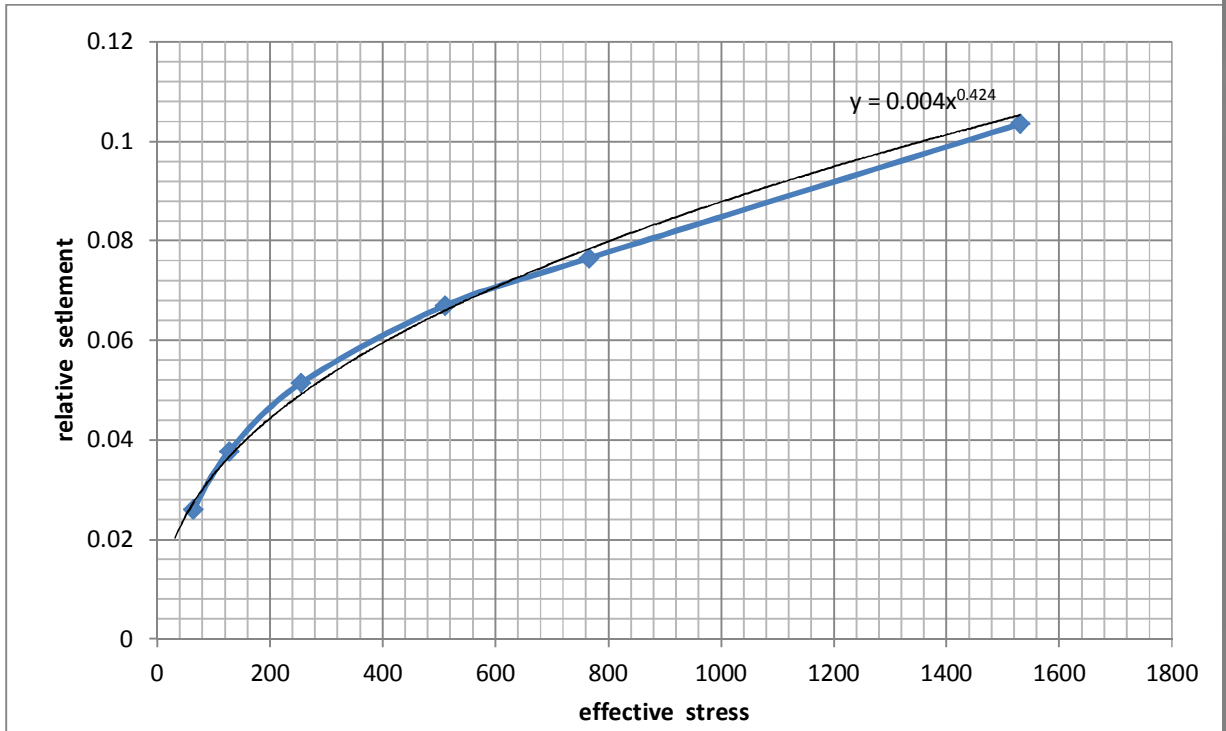


Fig B12 Relative settlement Vs Effective Stress (tp5 @ 2m)

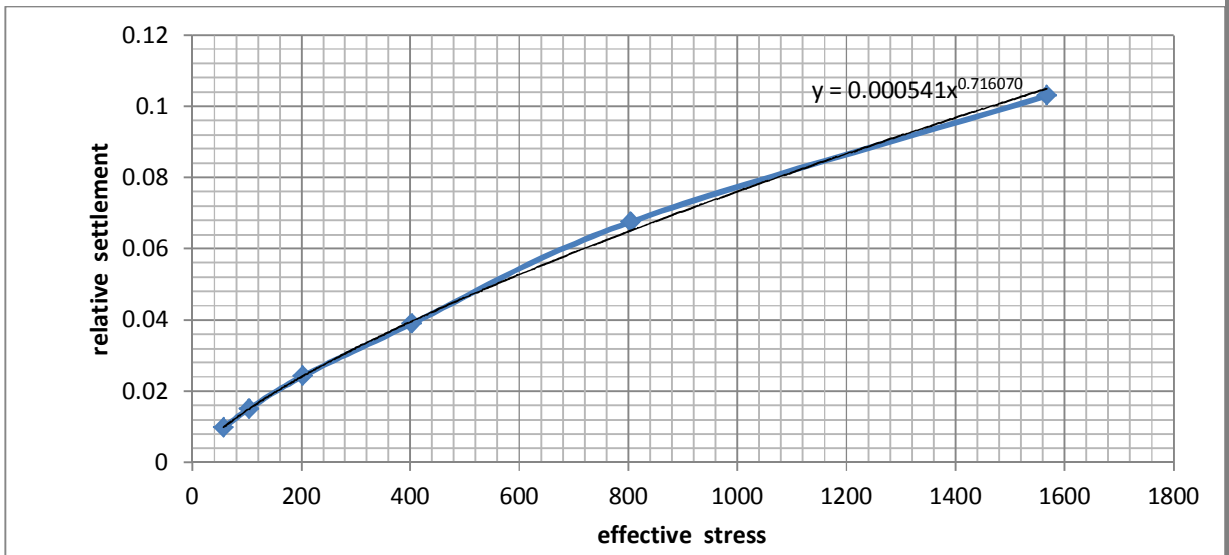


Fig B13 Relative settlement Vs Effective Stress (tp4 @ 2m)

Table B10 Effective stress and relative settlements for Tp1

<a href="#">tp1@3m</a>	Applied pressure	Final dial reading	Total compression, $\Delta H$ , cm	Relative settlement $s' = \Delta H/H$	w	V (kN/m <sup>2</sup> )	Modulus of compressibility
Tp1 at 3m depth	0	1.93	0	0	0.3524	2573.605	Es=2573.61 ( $\sigma_e'$ ) <sup>0.3524</sup>
	31.875	1.981	0.051				
	63.75	2.067	0.137				
	127.5	2.191	0.261	0.01305			
	255	2.4	0.47	0.0235			
	510	2.651	0.721	0.03605			
	765	2.82	0.89	0.0445			
	1530	3.27	1.34	0.067			

Table B11 Effective stress and relative settlements for Tp1

sample taken	Applied pressure	Final dial reading	Total compression, $\Delta H$ , cm	Relative settlement $s' = \Delta H/H$	W	V (kN/m <sup>2</sup> )	Modulus of compressibility
Dangila town	0	1.95	0	0	0.575 1	500.744	Es=500 ( $\sigma_e'$ ) <sup>0.4249</sup>
	31.875	2.27	0.32				
	63.75	2.47	0.52	0.026			
	127.5	2.704	0.754	0.0377			
	255	2.978	1.028	0.0514			
	510	3.29	1.34	0.067			
	765	3.48	1.53	0.0765			
	1530	4.02	2.07	0.1035			

Table B12 Effective stress and relative settlements for Tp4

sample taken	Effective stress, $\sigma'$	Dial reading	Total compression, $\Delta H$ , cm	Relative settlement $s'$ = $\Delta H/H$	w	V(kN/m <sup>2</sup> )	Modulus of compressibility
Tp4 at 2m depth	0	4.021			0.28	2795.24	Es=2795.25 ( $\sigma_e'$ ) <sup>0.2845</sup>
	56.58	4.0520	0.0310	0.0097			
	104.385	4.12	0.099	0.0151			
	202.47	4.357	0.336	0.0242			
	402.7	4.8	0.779	0.0389			
	803.1	5.371	1.35	0.0675			
	1567.81	6.08	2.059	0.1029			

Table B13 Effective stress and relative settlements for Tp4

Deformation Dial Reading	Load Dial Reading	Sample Deformation DL (mm)	Strain (e)	% Strain	Corrected Area A' (cm <sup>2</sup> )	Load (KN)	Stress (kPa)
0	0	0	0	0	12.56	0	0
20	31.5	0.2	0.0021978	0.2198	12.588	0.021735	17.26690
40	48.6	0.4	0.0043956	0.4396	12.615	0.033534	26.58168
60	66	0.6	0.00659341	0.6593	12.643	0.04554	36.01889
80	87	0.8	0.00879121	0.8791	12.671	0.06003	47.37441
100	105	1	0.01098901	1.0989	12.7	0.07245	57.04924
120	129	1.2	0.01318681	1.3187	12.728	0.08901	69.93331
140	151.5	1.4	0.01538462	1.5385	12.756	0.104535	81.94806
160	171	1.6	0.01758242	1.7582	12.785	0.11799	92.28937
180	192	1.8	0.01978022	1.978	12.813	0.13248	103.3913
200	206.4	2	0.02197802	2.1978	12.842	0.142416	110.8964
220	216	2.2	0.02417582	2.4176	12.871	0.14904	115.7936
240	231	2.4	0.02637363	2.6374	12.9	0.15939	123.5559
260	241.5	2.6	0.02857143	2.8571	12.929	0.166635	128.8805
280	247.5	2.8	0.03076923	3.0769	12.959	0.170775	131.7837
300	256.5	3	0.03296703	3.2967	12.988	0.176985	136.2661
320	261	3.2	0.03516484	3.5165	13.018	0.18009	138.3416
340	246	3.4	0.03736264	3.7363	13.047	0.16974	135.3678
360	222	3.6	0.03956044	3.956	13.077	0.15318	117.1338
380	195	3.8	0.04175824	4.1758	13.107	0.13455	102.6524
400	174	4	0.04395604	4.3956	13.137	0.12006	91.38

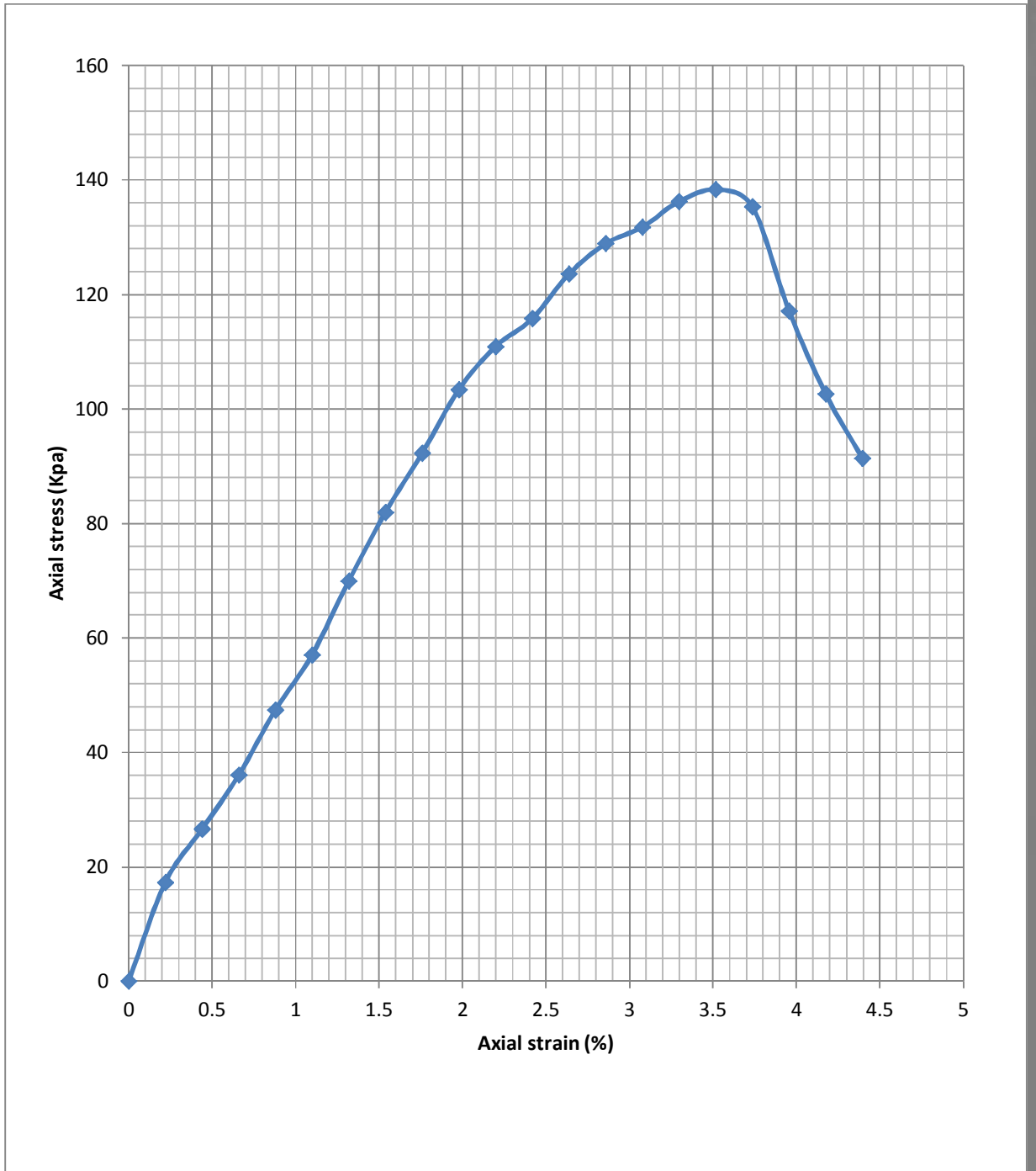


Fig B14 Unconfined compression test results for Tp3

Unconfined compressive strength ( $q_u$ ) = 138.5kPa

The undrained shear strength ( $s_u$ ) =  $\frac{q_u}{2} = \frac{138.5}{2} = 69.25$  kPa

Table B14 Unconfined compression test results for Tp5

Deformation Dial Reading	Load Dial Reading	Sample Deformation DL (mm)	Strain (e)	% Strain	Corrected Area A' (cm <sup>2</sup> )	Load (kN)	Stre (kPa)
0	0	0	0	0	12.56	0	
20	50	0.2	0.0025	0.25	12.591	0.0345	27.39948
40	99	0.4	0.005	0.5	12.623	0.06831	54.11500
60	154	0.6	0.0075	0.75	12.655	0.10626	83.96739
80	203	0.8	0.01	1	12.687	0.14007	110.4054
100	270	1	0.0125	1.25	12.719	0.1863	146.4739
120	315	1.2	0.015	1.5	12.751	0.21735	170.4536
140	360	1.4	0.0175	1.75	12.784	0.2484	194.3097
160	400	1.6	0.02	2	12.816	0.276	215.3503
180	420	1.8	0.0225	2.25	12.849	0.2898	225.5410
200	415	2	0.025	2.5	12.882	0.28635	222.2860
250	380	2.5	0.03125	3.125	12.965	0.2622	202.2342

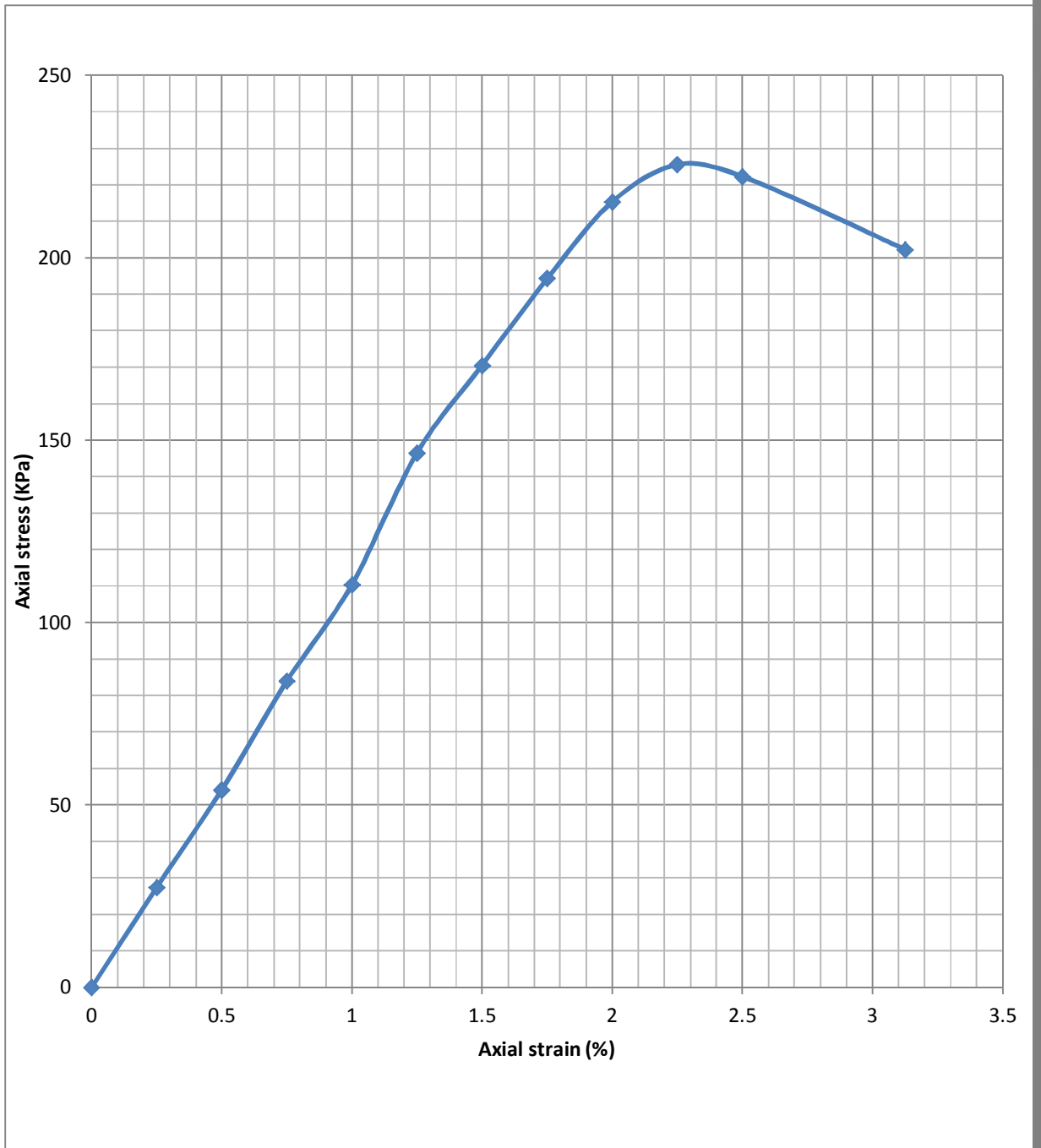


Fig B15 Unconfined compression test results for Tp5

Unconfined compressive strength ( $q_u$ ) = 226kPa

The undrained shear strength ( $s_u$ ) =  $\frac{q_u}{2} = \frac{226}{2} = 113$  kPa

Deformation	Load Dial	Sample	Strain	%	Corrected	Load	Stress
Dial Reading	Reading	Deformation	(e)	Strain	Area A' (cm <sup>2</sup> )	(kN)	(kPa)
		DL (mm)					
0	0	0	0	0	12.56	0	0
20	50	0.2	0.00243902	0.2439	12.591	0.0345	27.4011
40	110	0.4	0.00487805	0.4878	12.622	0.0759	60.1351
60	190	0.6	0.00731707	0.7317	12.653	0.1311	103.615
80	224	0.8	0.0097561	0.9756	12.684	0.15456	121.856
100	247	1	0.01219512	1.2195	12.715	0.17043	134.037
120	260	1.2	0.01463415	1.4634	12.747	0.1794	140.744
140	272	1.4	0.01707317	1.7073	12.778	0.18768	146.875
160	281	1.6	0.01951	1.9512	12.81	0.1938	151.35
180	280	1.8	0.02195	2.1951	12.842	0.1932	150.44
200	264	2	0.02439024	2.439	12.874	0.18216	141.494
220	255	2.2	0.0268	2.6829	12.906	0.1759	136.32

Table B15. Unconfined compression test results for Tp1

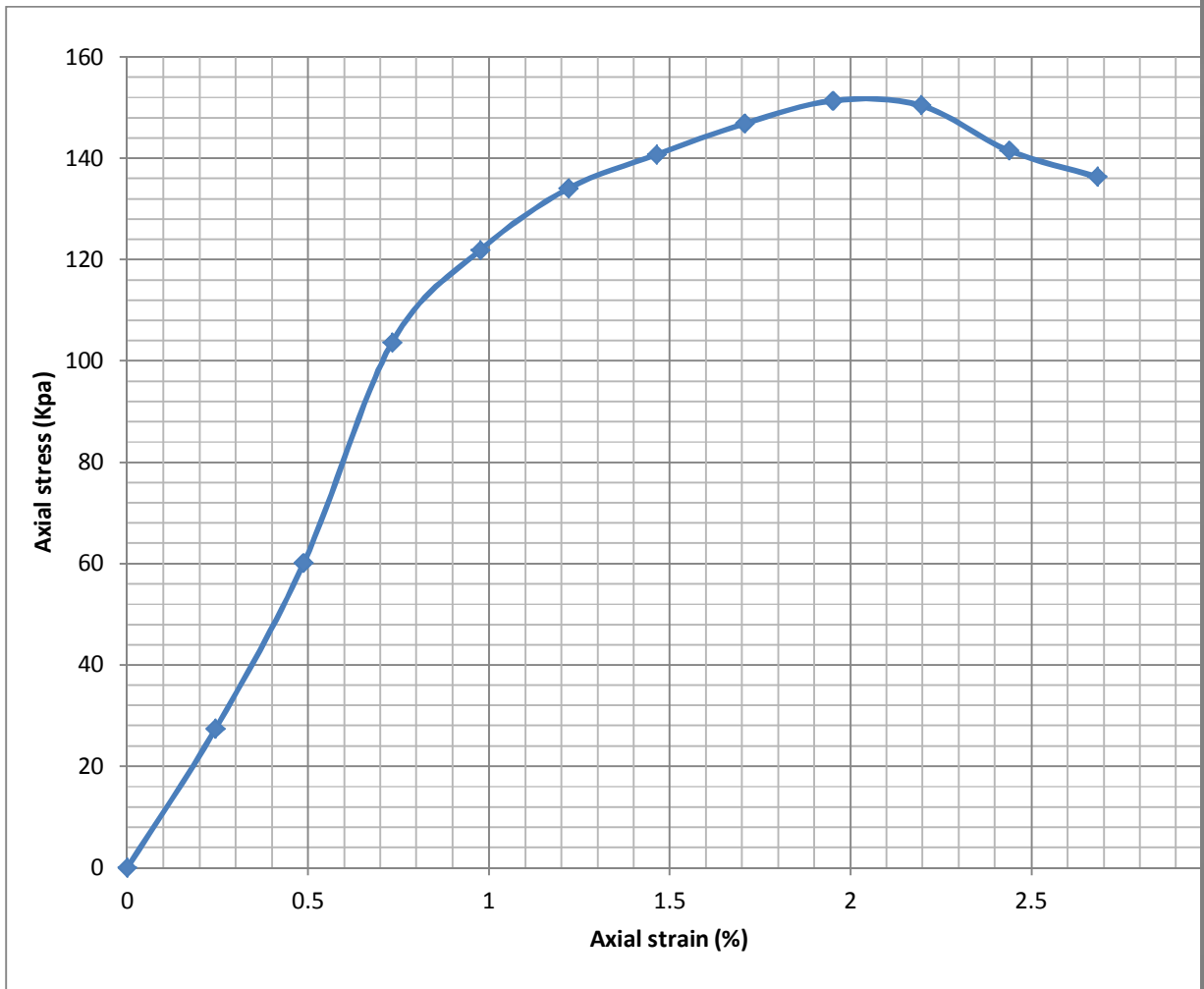


Fig B16 Unconfined compression test results for Tp1

Unconfined compressive strength ( $q_u$ ) = 151 kPa

The undrained shear strength ( $s_u$ ) =  $\frac{q_u}{2} = \frac{151}{2} = 75.5$  kPa

Deformation Dial Reading	Load Dial Reading	Sample Deformation DL (mm)	Strain (e)	% Strain	Corrected Area A' (cm <sup>2</sup> )	Load (KN)	Stress (kPa)
0	0	0	0	0	12.56	0	0
20	61	0.2	0.00210526	0.2105	12.586	0.04209	33.440596
40	115	0.4	0.00421053	0.4211	12.613	0.07935	62.910744
60	162	0.6	0.00631579	0.6316	12.64	0.11178	88.434730
80	195	0.8	0.00842105	0.8421	12.667	0.13455	106.22368
100	221	1	0.01052632	1.0526	12.694	0.15249	120.13124
120	245	1.2	0.01263158	1.2632	12.721	0.16905	132.89381
140	265	1.4	0.01473684	1.4737	12.748	0.18285	143.43580
160	262	1.6	0.01684211	1.6842	12.775	0.18078	144.32667
180	265	1.8	0.01894737	1.8947	12.803	0.18285	142.82282
200	264	2	0.02105263	2.1053	12.83	0.18216	141.97854
220	260	2.2	0.02315789	2.3158	12.858	0.1794	139.52665
240	252	2.4	0.02526316	2.5263	12.886	0.17388	134.94207
260	240	2.6	0.02736842	2.7368	12.913	0.1656	128.23868
280	225	2.8	0.02947368	2.9474	12.941	0.15525	119.96354
260	210	2.6	0.02736842	2.7368	12.913	0.1449	112.20885

Table B16 Unconfined compression test results for Tp4

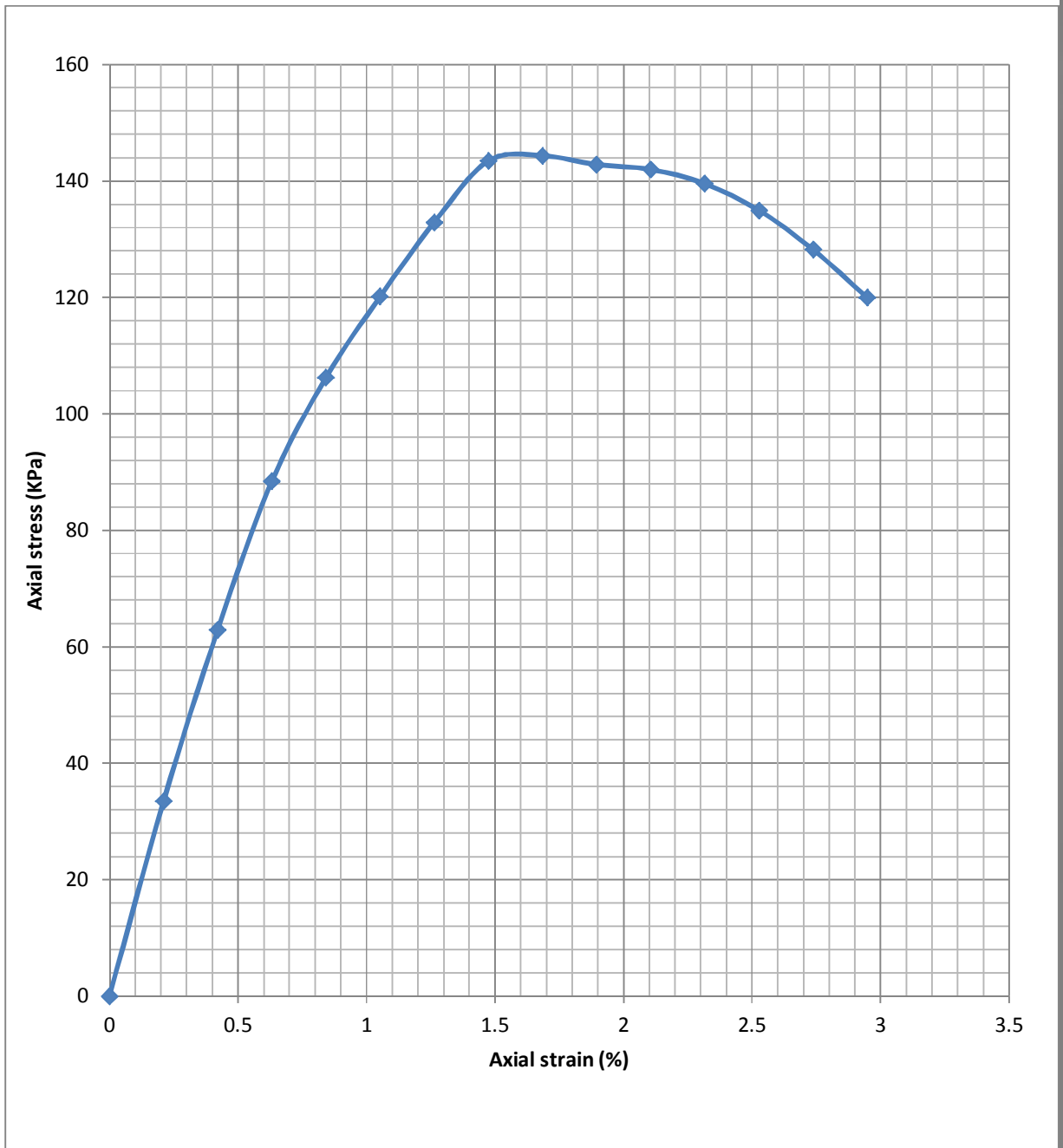


Fig B17 Unconfined compression test results for Tp4

Unconfined compressive strength ( $q_u$ ) = 145kPa

The undrained shear strength ( $s_u$ ) =  $\frac{q_u}{2} = \frac{145}{2} = 72.5$  kPa