ADAMA SCIENCE AND TECHNOLOGY UNIVERSITY

SCHOOL OF CIVIL ENGINEERING AND ARCHITECTURE

DETERMINATION OF PARAMETERS FOR THE TANGENT MODULUS

APPROACH OF SOIL FOUND IN ASELA TOWN

By: Bekele Kushu

August 2016

DETERMINATION OF PARAMETERS FOR THE TANGENT MODULUS APPROACH OF SOIL FOUND IN ASELA TOWN

A Thesis Submitted to School of School of Graduate Studies of Adama Science and Technology University

In Partial Fulfillment of the Requirement for the Degree of

Master of Science in Geotechnical Engineering

By: BEKELE KUSHU **Advisor: HENOK FIKRE (Dr.ING)**

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A Thesis Submitted to School of Graduate Studies of Adama Science and Technology University in Partial Fulfillment of the Requirement for

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(Geotechnical Engineering)

Approved By the Board of Examiners

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DECLARATION

I declare that the work in the project entitled Determination of Parameters for the Tangent Modulus Approach of Soil found in Asela Town has been performed by me in the Department of Civil Engineering, School of Civil and Architectural Engineering, under the supervision of Dr.-Ing. Henok Fikre. The information derived from literature has been duly acknowledged in the text and list of references provided. No part of this project was previously presented for another degree at any university.

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Name of Student	Sign	Date

ACKNOWLEDGMENT

Foremost the glory goes to the ALMIGHTY GOD; through him all things are possible. In him, I put my trust for protection and guidance.

I would like to express my heartiest gratitude and thanks to my advisor Dr.-Ing.Henok Fikre for his supervision to carry out this research work. His encouragement, guidance and helpful suggestions throughout the work inspired me in many ways and made it possible for me to complete the work. The efforts of my advisor are gratefully acknowledged.

My appreciation goes to Shumi and Selemon the Geotechnical Engineering laboratory technicians and for their continuous support during laboratory work. I am thankful to them.

I would like to thank the Municipality of Asela town in providing facility for collecting of the soil samples from different parts of the town.

ABSTRACT

Prediction of settlement by classical approach has drawbacks since soil parameters obtained from laboratory test by semi-log plot attributed to scale transformation. The tangent modulus approach, based on arithmetic plot of ε vs. σ' and M vs. σ', can overcome this drawbacks, and can be used in analyzing nonlinear soil settlement. In this paper, application of tangent modulus approach is verified by introducing and determining parameter for Asela soils. In addition, limitation of the classical approach is justified by examining the outcomes of one dimensional consolidation test results. Results of modulus number ranges from 10 – 20 for normally consolidated portion and 20-50 for over-consolidated portion. Stress exponent is found to be zero for both normally consolidated and over-consolidated portion. The outcomes of analysis justify the limitation of classical approach and prove that the tangent modulus approach has better accuracy of nonlinear settlement computation. Also, it is feasible to obtain tangent modulus parameters from arithmetic plot of M vs. σ', which provides an easier way in computing settlement of the soil.

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SYOMBBOLS AND ABBREVIATION

a	Stress exponent
a_v	Coefficient of compressibility
AASHTO	American Association of Highway and Transportation Officials
ASTM	American Society for Testing Materials standard
C_c	Compression index
C_r	Recompression index
C_v	Coefficient of consolidation
e_o	Initial void ratio
e	Void ratio
k	Permeability
k’	Earth pressure coefficient
M	Tangent modulus
m	Normally consolidated modulus number
m_r	Over consolidated modulus number
m_v	Coefficient of volume compressibility
OCR	Over consolidation range
NCR	Normally consolidated range
u	Pore water pressure
U	Degree of consolidation
V	Volume
ε	Strain
γ_w	Unit weight of water
σ	Total stress

σ'	Effective stress
σ_{ov}	Over burden stress
σ_{pc}	Pre-consolidation stress
p	Atmospheric pressure
TP	Test Pit
CRS	Constant Rate of Strain
U	Degree of Consolidation
T	Time Factor

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1. INTRODUCTION

1.1 Background of the Problem

Infrastructures like skyscrapers, dams, reservoirs, bridges and tunnels are required due to rapid growth of cities, industry and commerce. These heavily loaded structures generally induce high stresses on the soil underneath which undergoes compressive strain resulting in the settlement of structures.

Excessive or unequal settlement may prevent the structure from functioning properly, which results in unnecessary cost of maintenance and even in extreme case collapse due to instability.

Settlement determination has to be given due consideration by engineers as it affects engineering structures from functioning properly and has to be also managed before causing serious damage.

Different countries use different approaches to estimate settlement of structures. In our country the classical approach is used in order to estimate settlement; but this approach has many limitations, among others, the following are important:

- the logarithmic scale in the e vs. log σ' hides away the details of the soil behaviors
- the determination of the pre-consolidation pressure σ'_{pc} using the Cassagrande approach is highly dependent on the human element and attributed to scale transformation
- the determination of initial void ratio, e_o is both tedious and subjected to many variables, for instance, it requires knowledge of the specific gravity of the soil and end of test water content. Hence, it is common knowledge that it is very sensitive to both water content and specific gravity
- the approach is limited to cohesive soils only

The other method which is used in Scandinavian countries, the tangent modulus approach on the other hand, has many advantages [4], [13]. These are:

- this method deals with the use of a coherent definition of compression modulus leading to a unified procedure of practical settlement calculations for different types of soil

- the method also showed that the calculation of the time rate of consolidation of clays has to be based on strain distribution, instead of pore pressure distribution, to avoid fundamental misunderstanding
- the concept simply and clearly relates the observed physical changes to the mathematical formulations; raw data is used to obtain desired answers without recourse to assumptions

The simplicity and rationality inherent in the resistance concept made it the preferred method for analysis of the compressibility of soils.

Previous works show that computation of soil compressibility using tangent modulus approach is applicable for Addis Ababa soils [17], [18]. Hence, it is essential to further look at the applicability of these methods for different soils in our country and to determine parameters for this method.

Asela is a town found in central Ethiopia Arsi zone of the Oromia region. Based on the federal census of 2007, currently, the population size of the town found to be 86,441. As the town is accommodating all these people, the public infrastructures development and service delivery of the town are at increasing rate of supply. Hence, geotechnical data are very essaincial to design safe and economical structures. This scenario has therefore prompted the need of this research work in Asela town.

1.2 Objective

- to present tangent modulus approach concept to our country
- to compare tangent modulus approach with classical approach
- to determine modulus number 'm' for typical soils in Asela town

1.3 Methodology

To achieve the objective of the thesis, index properties and a series of one dimensional consolidation tests were performed on soil samples collected from different location of Asela town. Table 1.1 shows the different locations of test samples.

Table 1.1 Location of test samples

Location	Designation
Near Burkitu Kebele	TP1
Chilalo School	TP2
Behind halila kebele	TP3
Andinet School	TP4
Sid of welkesa kebele	TP5
Police Station	TP6
Infront of Bufeta kebele	TP7
Kenenisa TVET	TP8
Hospital	TP9
Medhanialem	TP10

Index property tests including particle size distribution, specific gravity; Atterberge limits, free swell, and moisture content were conducted on disturbed soil samples collected from each 10 test pits at two different (1.5m and 3m) depths. The methodology adopted for these tests include extensive review of standards, literatures, publications, and journals on the index properties of the soils.

Undisturbed samples were taken from each test pits at 3m depth to carry out one dimensional consolidation test. ASTM D2425-96 procedures were followed while conducting consolidation tests. From the data obtained, effective stress vs. void ratio in semi-log scale and effective stress vs. strain in arithmetic scale were plotted. These plots were used to compute the compressibility and resistance to deformation (M) of the soil. Effective stress vs. resistance to deformation were plotted to obtain modulus number 'm', for all samples.

1.4 Structure of the Thesis

This thesis work is divided in to six chapters, each covering a specific topic of the research work. In this introductory chapter the background, objective and methodology of the thesis work was presented. The second chapter gives a brief literature review which discusses about classical and tangent modulus approach of settlement analysis. In the third chapter laboratory test results, interpretation in accordance with the classical and tangent modulus approach and determination of parameters for local soil is presented. Laboratory test results discussion are presented in chapter four. Chapter five includes the conclusions and recommendations drawn from the research works.

2. LITERATURE RIVIEW

2.1 Classical Approach of Settlement Analysis

The pioneering work on stress-strain behavior during one-dimensional consolidation was done by Terzaghi [21]. He published a theory for one-dimensional consolidation and today it is regarded as the classic consolidation theory.

2.1.1 Mathematical Modeling of One-Dimensional Consolidation

The natural loading and unloading of a soil stratum during deposition and erosion of overlying material takes place under condition of one-dimensional consolidation. In the derivation of the formulae the following points are considered:

- The change in volume of the soil is equal to the change in volume of pore water expelled which is equal to the change in the volume of the voids. Since the area of the soil is constant, the change in volume is directly proportional to the change in height.
- at any depth the change in vertical effective stress is equal to the change in excess pore water pressure
- the change in void ratio is assumed proportional with effective stress increase
- as water is incompressible, the change in volume is determined from the principles of continuity
- Darcy's law is valid

Based on the above consideration, general time rate of one-dimensional consolidation equation is derived as show in Equation 2.1.

$$\frac{\partial u}{\partial t} = \left(\frac{k}{\gamma_w m_v}\right)\left(\frac{\partial^2 u}{\partial^2 z}\right) = c_v\left(\frac{\partial^2 u}{\partial^2 z}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.1)$$

where : c_v = coefficient of consolidation = k/ (γ_w m_v)

γ_w = unit weight of water

m_v = modulus of volume compressibility

k = permeability

This equation describes the spatial variation of excess pore water pressure with time and depth. In the derivation of Equation 2.1, k and m_v are assumed constant. This is usually not the case because as the soil consolidates void spaces are reduced and k decreases. Also, m_v is not linearly related with effective stress. The consequence of k and m_v not being constant is that c_v is not constant. In practice, c_v is assumed to be a constant and this assumption is reasonable only if the stress changes are small enough such that k and m_v do not change significantly.

Equation 2.1 is the basic differential equation of Terzaghi's [21] consolidation theory and can be solved with proper boundary conditions.

2.1.2 Settlement Computation

2.1.2.1 Evaluation of Void Ratio-Pressure Relationships

An estimate of consolidation settlement following complete dissipation of hydrostatic excess pressure requires determination of the relationship between the in situ void ratio and vertical effective stress distribution in the soil. The loading history of a test specimen taken from an undisturbed and saturated soil sample, for example, may be characterized by a void ratio versus logarithm pressure diagram [22].

Removal of soil sample from its field location will reduce the confining pressure, but tendency of the sample to expand is restricted by the decrease in pore water pressure. The void ratio will remain constant at constant water content because the decrease in confinement is approximately balanced by the decrease in water pressure, therefore, the effective stress remains constant by the above equation, and the void-ratio should not change. Classical consolidation assumes that elastic expansion is negligible and the effective stress is constant during release of the in-situ confining pressure after the sample is taken from the field. Sample disturbance reduced by pushing undisturbed sample into metal shell tubes and testing in the consolidometer without removing the horizontal restraint. This helps maintain the in situ horizontal confining pressure, and reduce any potential volume change following removal from the field, and helps the correction for sample disturbance [22].

A normally consolidated soil in-situ will be at void ratio e_o and effective over burden pressure σ'_{ov} equal to the pre-consolidated stress σ_{pc}. e_o may be estimated as the initial void ratio prior to the test, if the water content of the sample had not changed during storage and soil expansion is negligible. In-situ settlement from applied loads is determined from the field virgin consolidation curve. Reconstruction of the field virgin consolidation with slope c_c shown on Figure 2.1 may be estimated by procedures involving the Casagrande [9] construction procedure which requires care and judgment for determining the greatest curvature for evaluation of pre-consolidated stress.

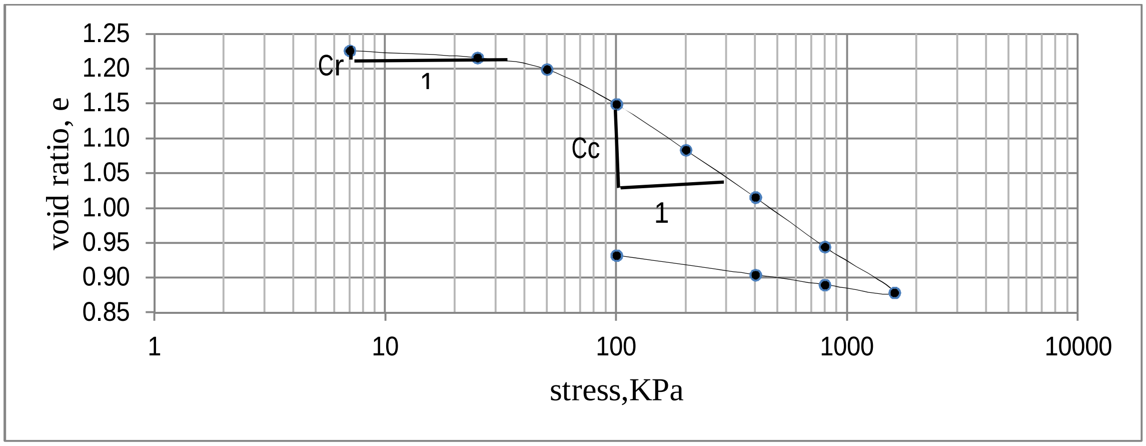


Figure 2.1 Compression and recompression indices

In this theory it is recommended to take high quality of undisturbed specimens for reducing the probable range of the pre-consolidated $\sigma_{cp.}$ Besides to this the loge scale of the plot may have also some influence on the evaluation of the parameters. Having these all parameters the ultimate consolidation settlement may be estimated by;

$$\mathbf{S_{cj}} = \frac{\Delta \mathbf{e_j}}{\mathbf{1 + \Delta e_{oj}}} * \mathbf{H_j} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots . \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad \mathbf{(2.2)}$$

where: S_{cj} = Consolidation settlement of stratum j
Δe_j = Change in void ratio of stratum j = e_{fj} - e_{oj}

e_{oj}= Initial void ratio of stratum j at initial pressure σ_{oj}

e_{fj} = Final void ration of stratum j at final pressure σ_{fj}

H_j= Height of stratum j

The change in void ratio may be calculated by:

$$\Delta e_j = c_c \log\left(\frac{\sigma'_{fj}}{\sigma'_{ozj}}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.3)$$

where: c_c is the slope of the field virgin consolidation curve or compression index

σ'_{ozj} is overburden effective stress of stratum j = $\gamma_j * z_j$

σ'_{fj} is final effective stress of stratum j = $\sigma'_{ozj} + \Delta\sigma'_j$

$\Delta\sigma'_j$ is change in effective stress of stratum j due to applied surface load

γ_j is soil unite weight of stratum j

z_j is thickness of stratum j

Determine one-dimensional consolidation settlement of stratum j with thickness H_j, from

$$S_{cj} = \frac{\Delta e_j}{1 + e_{oj}} * H_j \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.4)$$

Determine the total consolidation Sc of the entire profile of compressible soil from the sum of the settlement of each stratum,

$$S_c = \sum_{j=1}^{n} s_{cj} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.5)$$

The in-situ settlement for an applied load will be the sum of recompression settlement and virgin consolidation.

Hence, the settlement S_{cj} of stratum j may be estimated as the sum of recompression and virgin consolidation settlements.

$$Sc_j = c_r \log\left(\frac{\sigma'_{pj}}{\sigma'_{ovj}}\right) + c_c \log\left(\frac{\sigma'_{fj}}{\sigma'_{pj}}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.6)$$

2.1.2.2 Pre- Consolidation Pressure

In the typical e vs. log σ' plot, it can be seen that the upper part is curved; however at higher pressure, e vs. log σ' bear a linear relation.

When the soil specimen was obtained from the field, it was subjected to certain maximum effective pressure. During the process of soil exploration, the pressure is released.

In the laboratory, when the soil specimen is loaded, it will show relatively small decrease of void ratio with load up to maximum effective stress to which the soil was subjected in the past [22]. This is represented by the upper portion of e vs. log σ' plot. If the effective stress on the soil specimen is increased further, the decrease of void ratio with stress level will be larger. This is presented by the straight line portion of e vs. log σ′ plot.

It is based on this stress history that most clay soils are characterized as normally consolidated or over-consolidated ones. Therefore it is mandatory for geotechnical engineer to know the value of pre-consolidated pressure which represents the highest level of stress to which the soil has historically been subjected prior to the current application of load. It is determined from the same e vs. log σ′ graph using the most popular graphical procedure suggested by Casagrande [9].

2.1.2.2.1 Casagrande Construction

The nonlinearity of the arithmetic compression curve must be dealt with by breaking down the curve into a series of small linear increments. For each load increment, a calculation is required to compute the total settlement. Recognizing these difficulties, Terzaghi [21] made numerous investigations in the laboratory and the field to establish general relationship between applied effective stress and deformation in his one dimensional consolidation theory. Later, Terzaghi hypothesized that the relationship between stress and strain is logarithmic.

Founded on the logarithmic relationship developed by Terzaghi, Casagrande [9] developed a graphical tool that allowed engineers to quickly solve settlement problems.

The assumption of the Casagrande [9] construction is that in finding the pre-consolidation pressure, the slope of the bilinear portion of the curves can be determined. This, in turn, allows the engineer to predict settlements using a simple equation that links volume change to the logarithm of effective stress.

The procedure begins with the selection of the point of maximum curvature on the semi-log plot. Next, both horizontal and tangent lines are drawn through this point. The angle between the horizontal and tangent lines is bisected. Finally, after extending the virgin compression branch of the curve through the bisector line, the pre-consolidation pressure is determined as the point of intersection of these two lines as given in Figure 2.2.

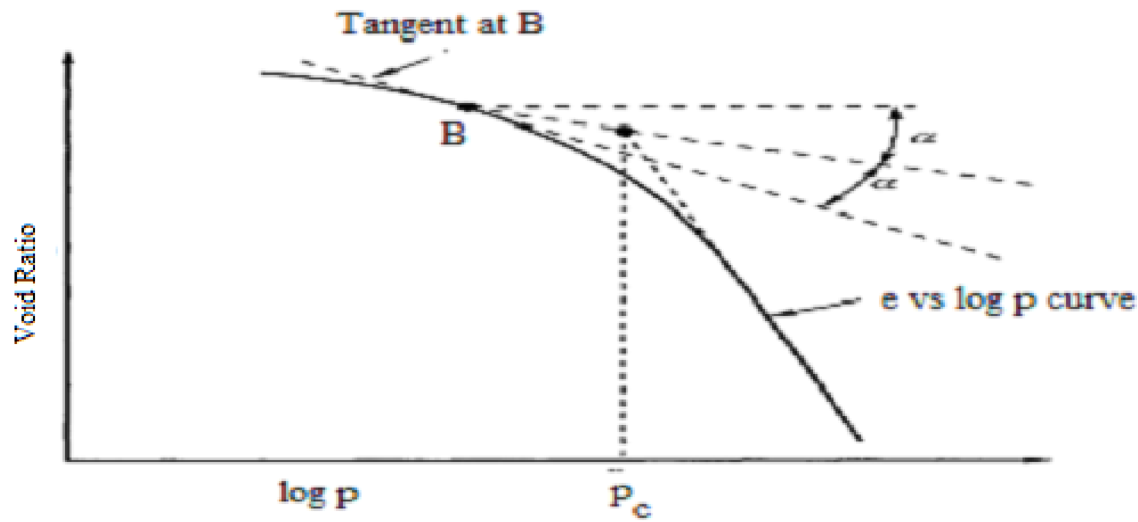


Figure 2.2 Typical e vs. log p plot showing procedures for determination of p_c

2.1.3 Problems with the Classical Approach

The classical approach works well for typical over-consolidated clays, but some difficulty is encountered in applying it to normally consolidated clays. The problem is the effect of scale transformation. By using a semi-log transformation of the actual stress-strain behavior to predict settlements, the lower stress levels are expanded and the higher stress levels are compressed. The result is that more weight is placed on deformation behavior at low stresses while less importance is given to the deformation behavior at high stresses. Banerjee and Sribalaskandarajah [4] examined the effects of scale transformation and concluded that the distortion of the true consolidation behavior of the clay manifests itself in the following four shortcomings of the classical approach: apparent over-consolidation, pre-consolidation pressure, maximum volumetric strain, and parameter estimation.

Apparent over-consolidation is a result of the expansion of the lower stress range on the semi-log plot. What is observed as over-consolidation in semi-log compression curves may not be representative of the true volume change behavior of the soil. This brings to question the validity of the bilinear model as a basis for finding the pre-consolidation pressure and compression indices of a soil and the predictions based on this method.

The same argument can be extended to question the validity of the point selected as the pre-consolidation pressure. The main problem in selection of the pre-consolidation pressure from the semi-log plot is the subjectivity involved in the process. Figure 2.3 shows that the pronounced break in the curve on the semi-log plot may be nothing more than the effects of scale transformation. The question then arises whether the point selected as the pre-consolidation pressure has anything to do with the properties of the soil or whether the break in the curve is simply attributable to scale transformation.

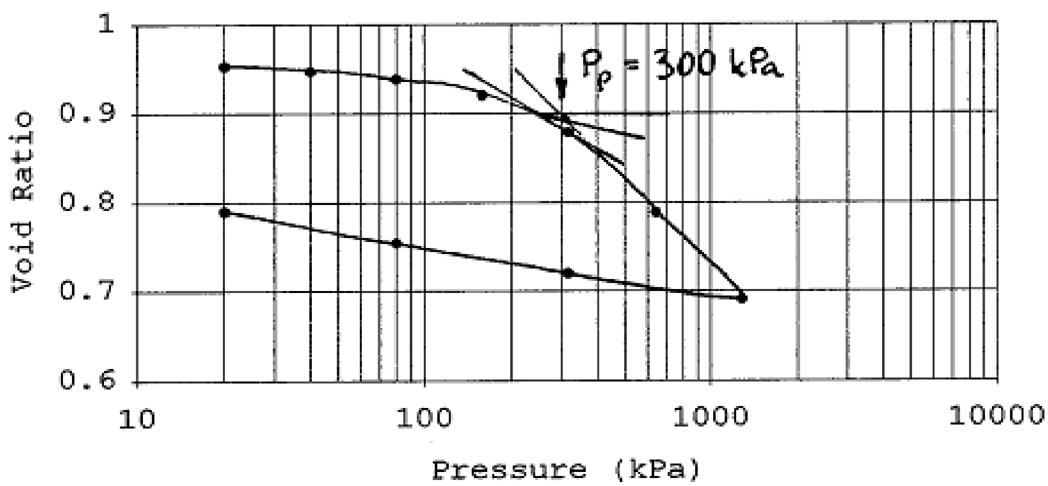


Figure 2.3 Casagrande constructions

The issue of maximum volumetric strain is also an effect of the scale transformation. On an arithmetic plot of volumetric strain vs. stress, it is clear that the curve approaches a limit state of maximum volumetric strain or minimum void ratio. As shown in Figure 2.4 there is a limit state at which each and every clay particle and peds has adopted a parallel or dispersed alignment [25]. While it is not a physically realistic state to achieve, it is clearly a limiting condition.

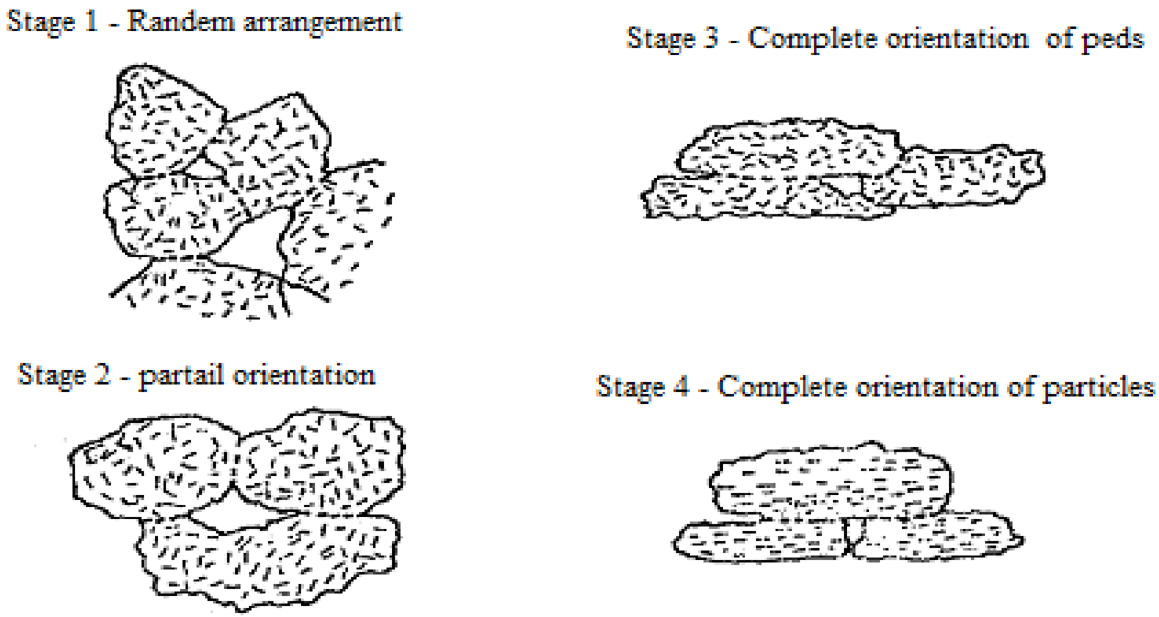


Figure 2.4 Effects of loading on clay structure [25]

As is clear from examination of Figure 2.3 the bilinear shape of the logarithmic compression curve implies that there is no minimum void ratio or maximum volumetric strain. Alternatively, as shown in Figure 2.5, the natural scale plot shows the asymptotic nature of the compression curve, indicating a limit state of maximum volumetric strain.

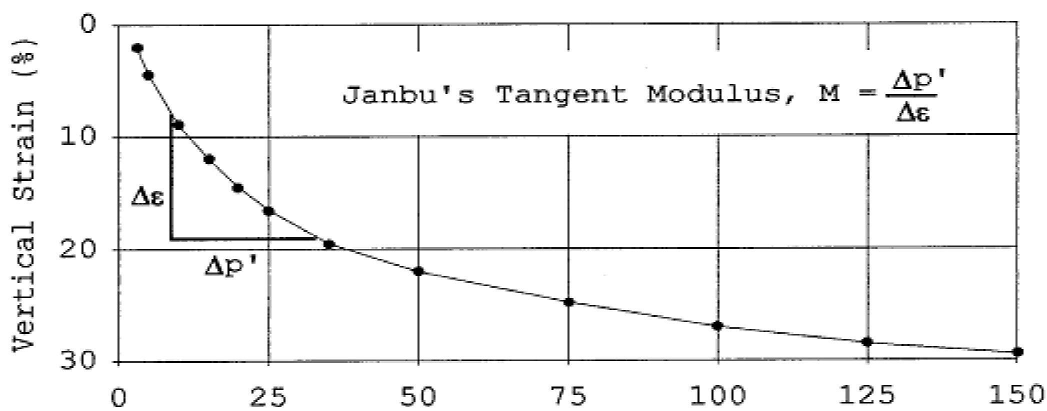
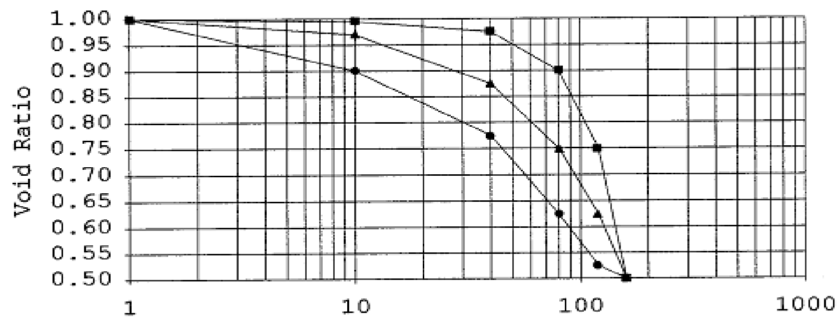


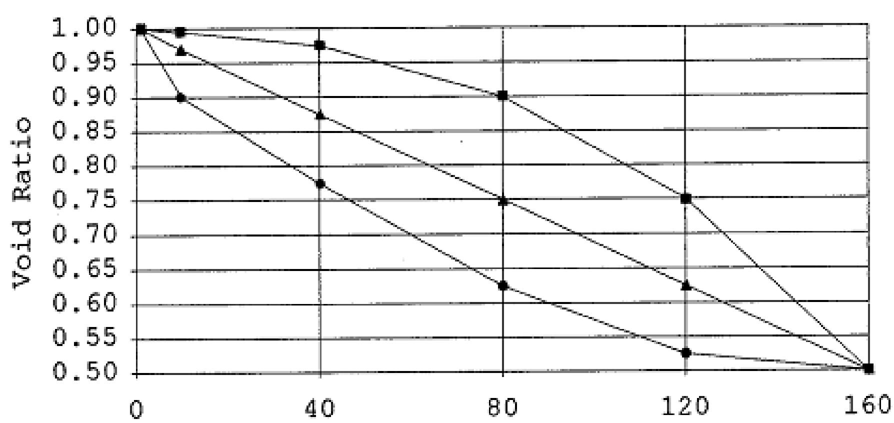
Figure 2.5 Typical arithmetic compression curves

Lastly, parameter estimation is difficult on the semi-log plot, especially for soft or sensitive clays. In order to apply the Casagrande construction, a subjective determination must be made and linearity must be imposed where linearity does not naturally exist.

Figure 2.6a and 2.6b shows semi-log and arithmetic scale plots on different materials. Figure 2.6a shows that the semi-log plot results in all samples a similar bilinear curve. Plotting similar data using arithmetic scale, displays in different manner as shown in Figure 2.6b. The middle curve reveals linear elastic material. The upper curve shows increase in compressibility which is the case in strain softening typical of non-dense soils with high liquidity index, while the lower curve shows decrease in compressibility which is the case in Strain hardening typical of dense soils with low liquidity index. Thus, those inferred from the semi- log plot are not soil properties; they are purely the product of the way the data are plotted [23].



a. Void ratio mean effective stress curve in log scale



b. Void ratio mean effective stress curve in arithmetic scale

Figure 2.6 Semi-log and arithmetic scale plots [23].

It is not clear why e vs. log P plots and the incomplete coefficient c_c determined from a constructed line have been in practice [13], [23] for many years. This has been criticized by authors like Janbu [13], [14] using data from sedimentary soils where the problem is much greater with residual or lateritic soils.

One can understand from literatures, the determination procedure using the classical approach attributed to scale transformation and is not obvious manifestations of the collapse of the clay structure [13], [14], [15]. The validity of any information from the semi-log compression curve is therefore put into question. Hence, invalidity of classical approach had initiated scholars to investigate more reliable method of compressibility parameter determination.

2.2 Tangent Modulus Approach of Settlement Analysis

2.2.1 General

The compressibility curve obtained from the consolidation test result may be expressed with sufficient accuracy by the equation of Ohde [20] given by:

$$E_S = \frac{d\boldsymbol{\sigma}'}{ds'} = \boldsymbol{\upsilon}(\boldsymbol{\sigma}')^{\omega} \quad \ldots\ldots \quad (2.7)$$

In the above equation s‘ is the relative settlement, υ and ω are coefficient where υ has a unit of kPa. It depends on the void ratio, water content and consistency of the sample. It could have values ranging between 50 and 30000 kPa [20].

The coefficient ω is a dimensionless quantity which depends on the soil type. It could have values ranging from 0 to 1[20].

The tangent of the compressibility, which is a function of σ′ gives the modulus of compressibility as shown in Figure 2.7**.**

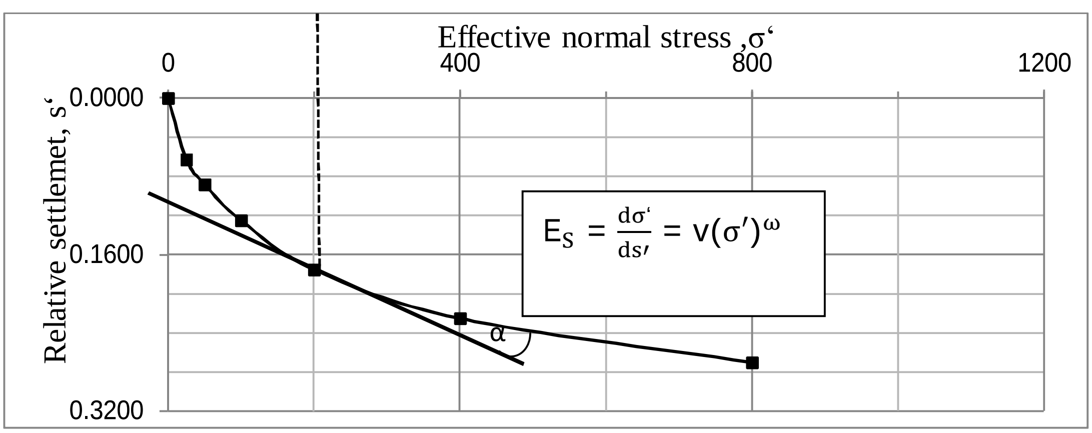


Figure 2.7 Compressibility curve (linear scale)

From equation 2.7

$$\frac{ds'}{d\sigma'} = \frac{1}{v(\sigma')^{\omega}} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (2.8)$$

$$s' = \frac{1}{v}\int (\boldsymbol{\sigma'})^{-\omega}\, d\boldsymbol{\sigma'} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots . (2.9)$$

For the case $\omega \neq 1$

$$s' = \frac{1}{\boldsymbol{\upsilon}(1-\omega)}(\boldsymbol{\sigma'})^{1-\omega} + C \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots . (2.10)$$

These equations given by Ohde were later on developed and interpreted in the resistance concept, tangent modulus, by Janbu[13] as it is presented hereafter.

Any media possess a resistance against a forced change of existing equilibrium condition. This resistance of the media, or of an isolated part of it, can be determined by measuring the incremental effect of a given incremental cause, where after

$$\text{Resistance} = \frac{\text{inccremental cuase (given)}}{\text{inccremental effect (measured)}} \ldots \ldots \ldots \ldots \ldots \ldots (2.11)$$

According to the outcome of a research work at the Technical University of Norway [14] the common term resistant can as well be applied to soil mechanics. The research revealed that the tangent to the stress - strain curve was found to be an appropriate and practical measure of deformation characteristics for all soils regardless of their type. The mechanical meaning of the Tangent Modulus, M, can therefore be defined as a resistance against deformation, expressed as

$$\text{Deformation Resistance,} \qquad M = \frac{\text{stress change (given)}}{\text{strain change (measured)}} \ldots \ldots \ldots (2.12)$$

This definition can be shown by stress - strain curve as obtained from an Oedometer or Triaxial test as seen in Figure 2.8. The common case is represented by a non - linear stress – strain relationship. The tangent modulus, M, is given by

$$\mathbf{M} = \frac{\mathbf{d\sigma'}}{\mathbf{d\varepsilon}} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots . (\mathbf{2.13})$$

where: M = Deformation modulus (Deformation Resistance)

σ′ = Change in effective stress (in given direction)

ε = Change in strain (in that direction)

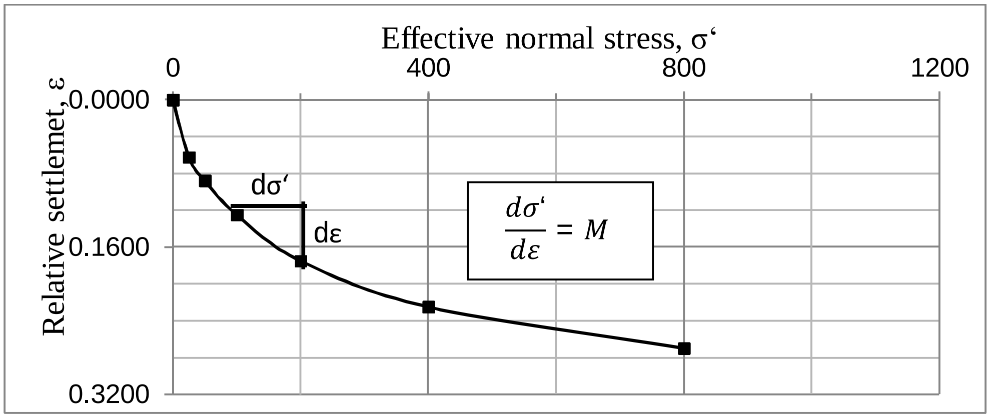


Figure 2.8 Typical stresses - strain curve

The tangent modulus, M, and the stress, σ′ can be shown by a graph of M vs. σ′ as shown in Figure 2.9.

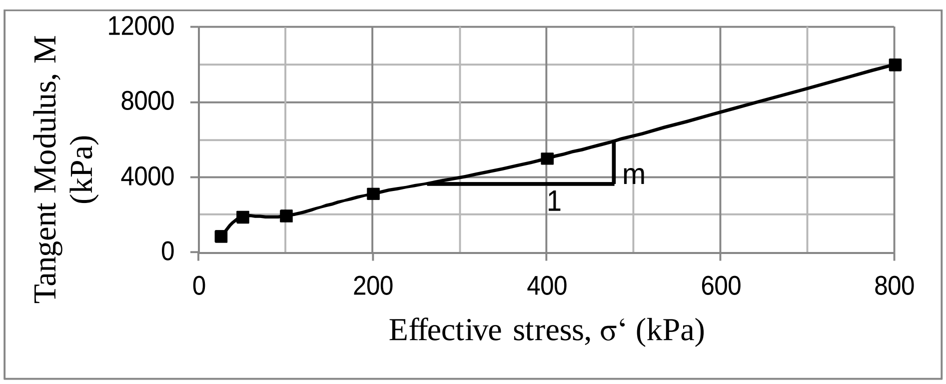


Figure 2.9 Typical tangent modulus vs. stress curve

Since deformation tests on soils are carried out as three - dimensional it is evident that any definition of a deformation modulus is meaningless unless the state of stress or strain is described more specifically [14].

Besides to this the branches of loading - unloading and reloading are entirely different. It is necessary to be able to distinguish between the corresponding modules. It is therefore suggested that the following terms should be used:

m = Compression modulus

m_s = Swelling modulus

m_r = Recompression modulus.

2.2.2 Application of the Resistance Concept in the Field of Soil Compressibility

The field condition for the deformation test in the laboratory is usually carried out either by Oedometer test or Triaxial test. By using either of these tests, the soil response is determined by subjecting representative undisturbed soil sample to an external action (e.g. changing external stress). The response of the sample to this external action is measured (either as strain and/ or pore pressure). The boundary conditions of the test can be explained by means of Figure 2.10.

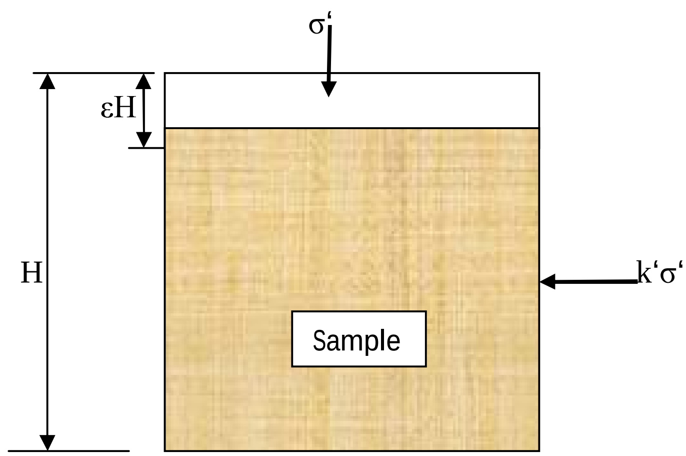


Figure 2.10 Test model

The Oedometer tests correspond to no lateral yield, or plain strain. Hence;

$$k = ko$$

where ko = effective earth pressure coefficient at rest such test corresponds to the field conditions.

Large number of Oedometer and Triaxial tests on different types of soils in Norway by N. Janbu [14] were conducted. He was able to deduce that the tangent modulus, derived from the vertical stress-strain curve, is suitable for direct measurement of soil compressibility. The plots of stress-strain and tangent modulus- stress on a linear scale have a different shape. This shape holds for soils of variable type and origin: clay, silt, sand, organic soils, sedimentary soils and residual soils etc. It appears, therefore, to constitute a general model of soil behavior to which reference can be made when evaluating the test result [20].

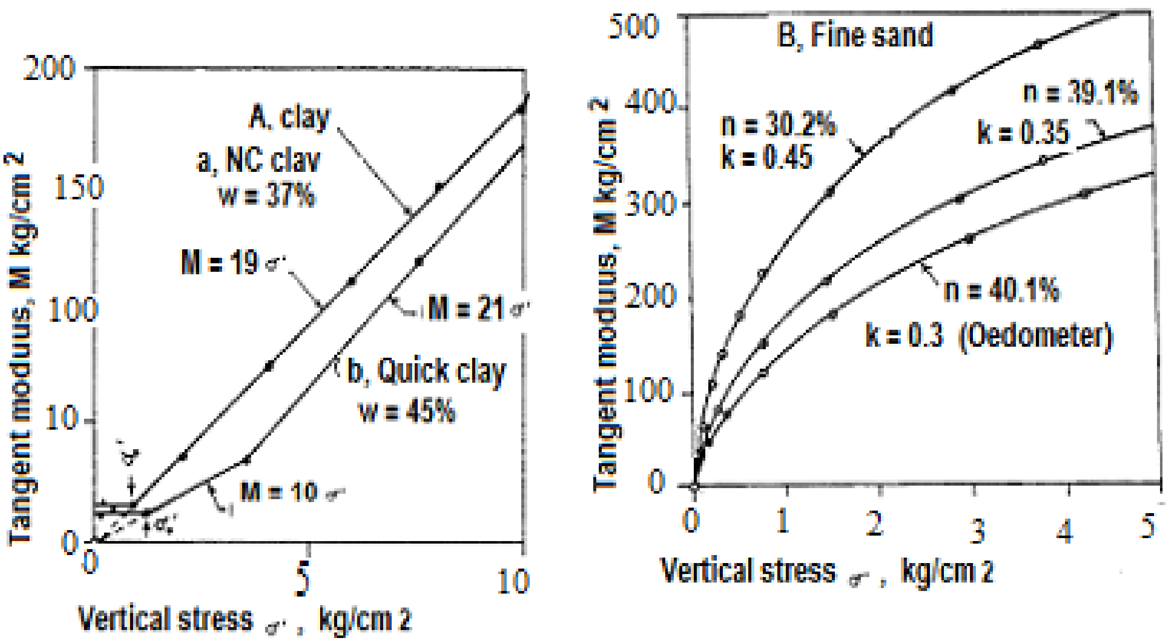


Figure 2.11 Plot of tangent modulus Vs stress for different types of soil [14]

Figure 2.11 show that the tangent modulus representing the behavior of soil is dependent on the nature of the soil. For low stress level on the loading branch the tangent modulus, M, against deformation is large while the stress increases this high resistance eventually decreases appreciably owing to the partial collapse of the grain skeleton. This break down of the resistance occurs around the pre-consolidation stress level σ_{pc}. This leads to a very practical and reasonable way of determining the pre-consolidated pressure from a linear scale plot [12], [13].

The variation of tangent modulus, M, with effective stress, σ′ for a wide range of material type is plotted in Figure 2.12.The general shapes of these plots are typical of the soil types.

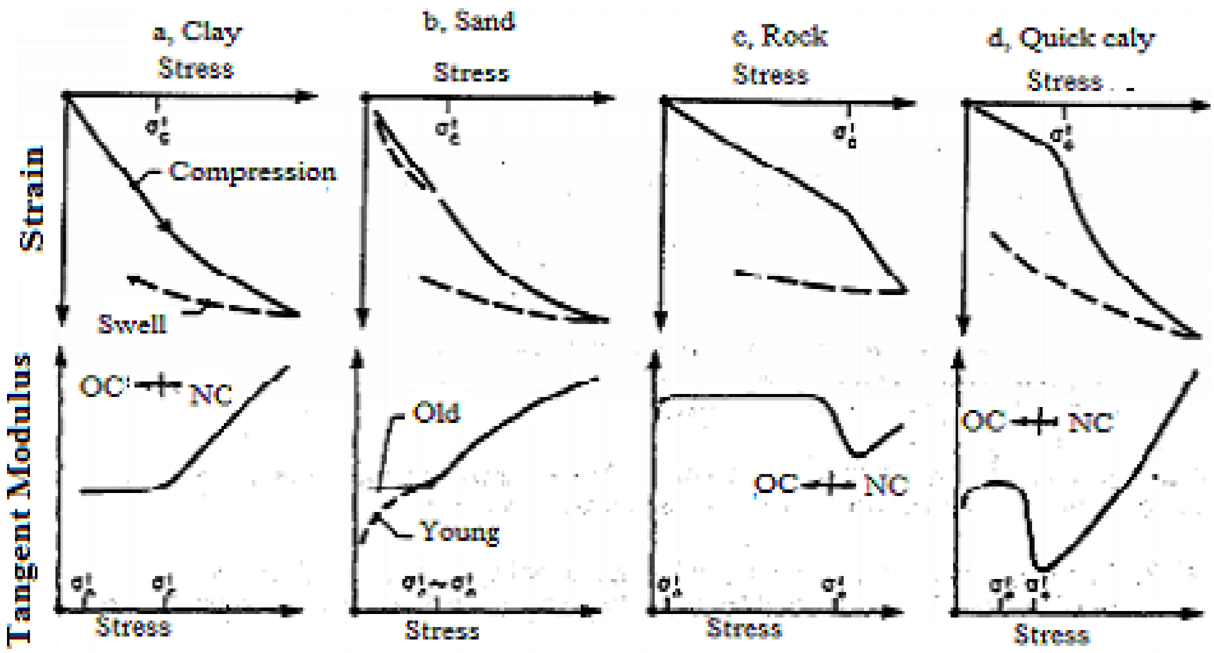


Figure 2.12 typical σ –ε and M - σ curves obtained by Oedometer Test

As it can be seen from Figure 2.12a, for over consolidated clays (OC), the modulus M is almost independent of the stress change $\Delta\sigma'$ between the present effective over burden σ'_{ov} and the pre-consolidated pressure σ'_{pc}, for stress above σ'_{pc} the modulus increases. But for normally consolidated Clays (NC), the modulus increases linearly with the effective stress change $\Delta\sigma'$. For sands and silts, the modulus varies with the effective stress raised to some power with an average of nearly 0.5 for practical purposes [13], [15].

Figure 2.12 generally show us a comprehensive behavior of soils ranging from clay to silt and sand. The general behavior and characteristics of soils which are hidden in the strain- stress curves are clearly shown.

From the study of a large number of stress - strain curves for different types of soils, for engineering purposes, one can adequately cover the variation in compressibility by means of single definition, namely tangent modulus [14]. Compression module for individual soil type is generally depend on effective stress, hence,

$$M = f(\sigma'). \quad (2.14)$$

However, the function f (σ′) is different for each new type of soil. Janbu [13] established that the variations of M with effective stress for the different soil types are adequately represented by a simple formula of the type:

$$\mathbf{M = mp\left(\frac{\sigma'}{p}\right)^{1-a}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \mathbf{(2.15)}$$

where: M = Modulus

m = modulus number (dimensionless)

a = stress exponent (dimensionless)

σ′ = Vertical stress (effective)

P = reference pressure = 1 atmosphere = 100kpa

The reference pressure is introduced solely for the purpose of obtaining a dimensionally correct expression.

The governing parameters m and a have been the subject of experimental investigation over a period of about 10 years and the general trend of variation is demonstrated in the Figure 2.12 and 2.13. The soil type considered ranges from sound rock with low porosity up to the softest clays with porosity nearly 90% [14].

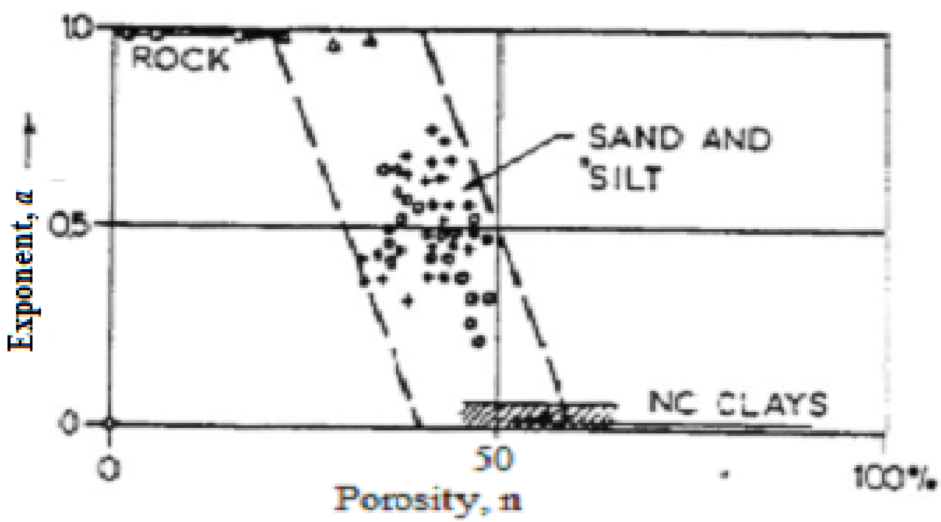


Figure 2.13 Variation of stress exponent with porosity [14]

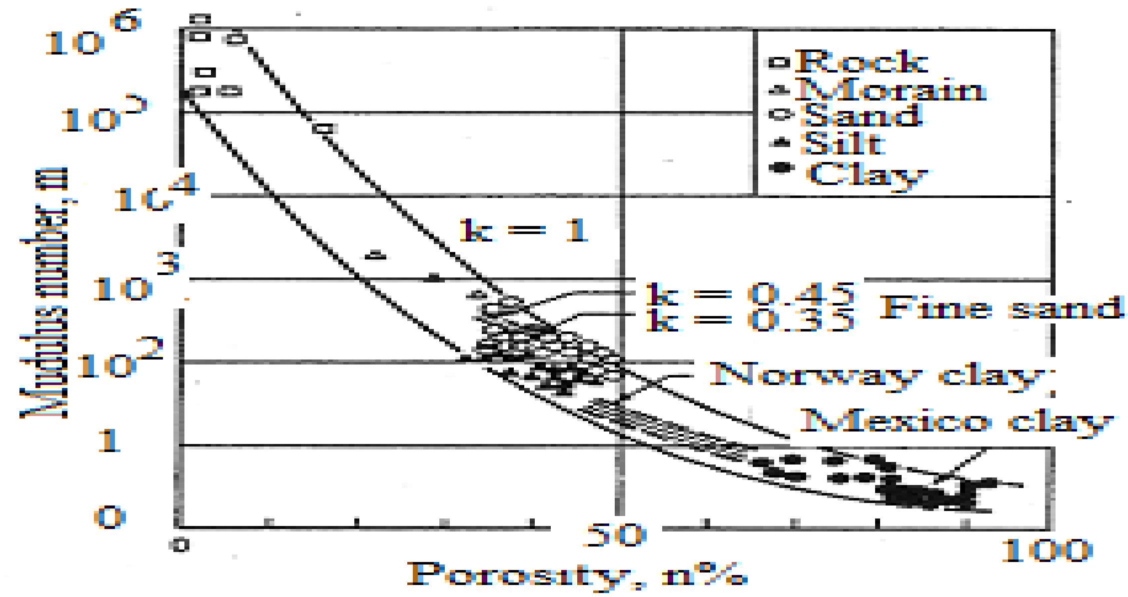


Figure 2.14 Variation of modulus number with porosity [14]

Figure 2.14 shows that for rock the stress exponent "a" is equal to one corresponds to a tangent modulus M is constant i.e. independent of stress variation. For such soils the modulus number m is generally high as we can see in the Figure 2.14.

For normally consolidated clays the stress exponent a is equal to zero, corresponding to a linear variation of modulus versus stress ($M = m\sigma'$). The modulus number is generally low. For soils of intermediate porosity (sand) the stress exponent, a, varies around 0.5 as an average. This corresponds to a modulus which is about proportional to the square root of the stress. The corresponding modulus number m is generally located between the low values assigned to clay on one hand and up to those rocks on the other hand.

Figure 2.13 and Figure 2.14 are not sufficiently detailed for direct practical application. This is because that it is only meant as illustration of the general tendencies for variation of "a" and "m" with porosity [13], [14].

The main impression one gets from these figures is that it is the modulus number that actually determines the magnitude of M while the value of a is of less importunate as long as it is approximately known, say with in ± 25%. This fortunate circumstance make it possible to assemble several soil types into a small number of soil groups, each of which can be studied in greater detail with respect to modulus number [13], [14].

From the test results of Norwegian institute, soils are basically grouped in three principal groups according to the value that shown in the Figure 2.15.

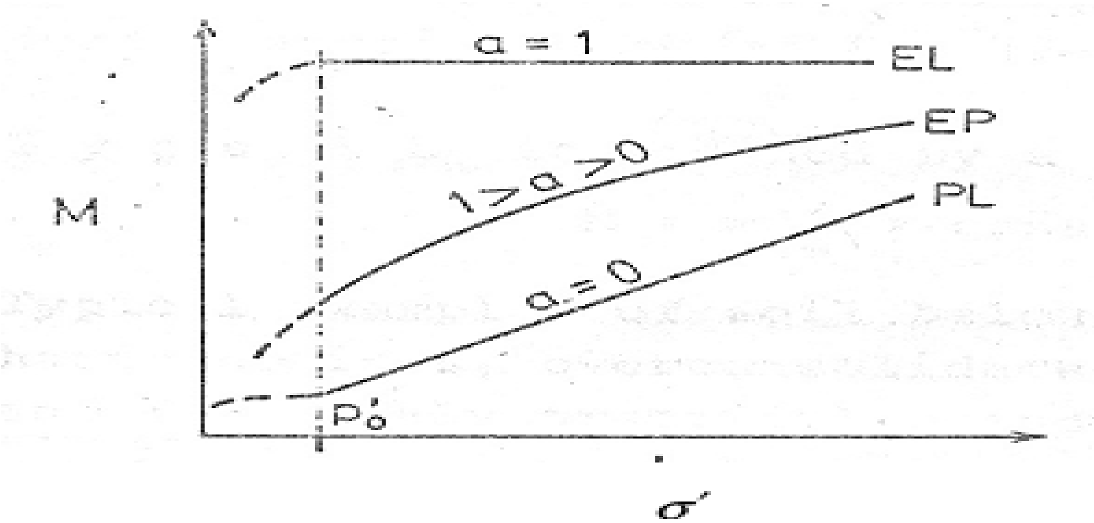


Figure 2.15 Principal Types of Soil

For practical purposes the institute has found it adequate to assemble the various soil types in to the following three main categories with respect to modulus variation and specific value of a.

Type EL = elastic (a=1)

Type EP = elastic - plastic (a = 0.5)

Type PL = Plastic (a=0)

Each of these principal groups corresponds to a specific value of the exponent, a, and for each group the modulus formula is therefore a special case of

$$M = mp\left(\frac{\sigma'}{p}\right)^{1-a} \quad \text{(2.16)}$$

Rocky hard moraines, over-consolidated clays, undrained modulus of clay are typical examples of soil belonging to type EL defined by a = 1 corresponding to

$$M = mpa \quad \text{(2.17)}$$

Type Ep are used for normally consolidated sand (NCS) where a = 0.5. In silty sandy soils (NC) the a - value may range typically between 0.4 to 0.65, so 0.5 should be considered as an average choice for practical purpose only. The formula for M is there for

$$M = m\sqrt{\sigma' * p} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.18)$$

Type PL where a = 0 corresponds to normally consolidated clays, NCC, for which

$$M = m\sigma' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.19)$$

Hence, it is only the modulus number that is required as a result of compression tests. Classifying the vast character and soil type in to three main categories by a single non dimensional number will make the tangent modulus approach more preferable over the classical approaches.

2.2.3 Strain and Settlement Computation

Independent of soil type, it is possible to establish a common definition for the compression modulus. Thus, the various steps of the settlement analysis can be derived from the same basic principle.

Besides, since the definition of the compression modulus utilizes familiar classical concepts, such as incremental stress and strain, the mechanical meaning of each step in the analysis will be familiar in the engineering practice [14] . In utilizing the concept, resistance concept, for settlement analysis it is obvious that the vertical stress profile has to be determined. This profile consists of the effective overburden σ'_{vo} and net stress distribution with depth. An infinitesimal element having thickness dz at an arbitrary depth below the foundation level undergoes deformation $d\delta$ due to an additional stress $d\sigma'$ and the effective overburden pressure prior to load application σ'_{ov} , as strain is generally dependent on both these stresses.

The infinitesimal vertical compression $d\delta$ of the layer dz by definition is the stain, ε

$$\mathbf{d\delta = \varepsilon . dz} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.20)$$

The total compression of the entire sediment of thickness H is then found by summing up the compression of each individual layer, hence

$$\boldsymbol{\delta} = \int_0^H \boldsymbol{\varepsilon}.\,dz \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (2.21)$$

According to this equation the settlement is equal to the area of the ε - z diagram. This elementary observation is of great practical importance for several reasons. Firstly, it may reduce to a minimum the number of ε - values that need to be calculated. Secondly, by plotting ε versus z, the integration Equation 2.21 is reduced to a simple area determination. Finally, the ε distribution with depth is of most importance in calculating the time rate of settlement of clays. It is therefore believed that ε - z plot should be a must in all settlement calculations.

The principle of calculating vertical strain (at an arbitrary depth in an arbitrary soil deposit) is based on the definition of the deformation modulus itself.

$$d\varepsilon = \frac{d\boldsymbol{\sigma}'}{M} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots... (2.22)$$

Therefore, as the effective stress increases from the overburden pressure σ'_{ov} until its final value $\sigma'_{ov} + \Delta\sigma'$. The resulting strain is obtained by integrating equation (2.22) between these stresses limit.

Hence,

$$\boldsymbol{\varepsilon} = \int_{\boldsymbol{\sigma}'_{ov}}^{\boldsymbol{\sigma}'_{ov}+\boldsymbol{\Delta\sigma}'} \frac{d\boldsymbol{\sigma}'}{M} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.23$$

In order to obtain an explicit strain formula, it is necessary to express ‘M’ mathematically as a function of σ'.

The generalized experimental value of the compression modulus M is given as

$$\mathbf{M} = \mathbf{mp}\left(\frac{\boldsymbol{\sigma}'}{\mathbf{p}}\right)^{\mathbf{1-a}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (\mathbf{2.24})$$

Introducing equation 2.24 in to Equation 2.23 one gets the following generalized strain formula

$$\varepsilon = \frac{1}{ma}\left[\left(\frac{\sigma'_{ov}+\Delta\sigma'}{p}\right)^a - \left(\frac{\sigma'_{ov}}{p}\right)^a\right] \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (2.25)$$

This is directly applicable for the entire range of 'a' form 0 to 1, except a = 0, which represents especial boundary case.

As suggested above this formula includes the majority of stress - strain relationships besides satisfy the boundary condition $\varepsilon = 0$ for $\Delta\sigma' = 0$.

The Equation 2.25 is also found advantageous to deal with the three principal categories of soils called EL (elastic), EP (elastic plastic) and PL (plastic). Each category corresponds to a given value of 'a', equal to 1, 0.5 and 0, respectively. The strain formula corresponding to these types are as follows.

For soil types corresponding to type EL

Where: $a = 1$, $\varepsilon = \Delta\sigma'/M$

Where: $M = mp =$ constant. ……..……………………..…………………..... (2.26)

For soil type EP, when a = 0.5, that is $M = m\sqrt{\sigma' p}$ ' leads to:

$$\boldsymbol{\varepsilon = \frac{2}{m}\left[\sqrt{\frac{\sigma'_{ov} + \Delta\sigma'}{p}} - \sqrt{\frac{\sigma'_{ov}}{p}}\right] \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (2.27)}$$

Type PL, for which a = 0 and $M = m\sigma'$ is governed by

$$\varepsilon = \frac{1}{m}\ln\left(\frac{\sigma'_{ov} + \Delta\sigma'}{\sigma'_{ov}}\right) \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (2.28)$$

Equation (2.28) is obtained by separate limit consideration.

The above equations can be employed for our local soils by determining the parameter m.

2.2.4 Time Rate of Consolidation

In dealing with settlements not only the prediction of the settlements that will occur when stress changes act over a given period of time are important but the question of how settlements develop with time, which has also great practical applications.

For coarse - grained soils such as sand and gravel, the time - dependency is usually of little practical interest. This is because the pores are sufficiently large to allow drainage almost simultaneously with stress change. Consequently, the practical interest is focused on the total settlement alone. Almost the same situation exists for layers of coarse silt [14].

For rock and heavily over consolidated clays, the deformation behavior resembles somewhat elastic materials, and hence, in practice, the question of time rate is rarely considered necessary for these soils[14].

For non - saturated soils, and for organic soils, the time rate of settlement may very well be of considerable importance, but it is beyond the scope of this thesis.

For normally consolidated, fully saturated clays and very fine silts are the only soil types for which it is both practically necessary and theoretically possible to predict the time rate of settlement.

To define the degree of consolidation properly in the classical theory, which is discussed in the previous chapter, it was important to assume that the change of strain is proportional to the stress change, which is true only for the pre-consolidated soils. Hence, the degree of consolidation in the classical theory is only dependent on the shape of additional stress distribution diagram with no consideration given to stress existing prior to load application. Owing to this and practical experience the theory has frequently failed in predicting even roughly the settlement behavior of structures after completion [15].

Thus, the equation has to be modified so as to include the actual soil properties and at the same time it has to give values in good consent with already completed structures. However, international experience have clearly shown that the compression of saturated, normally consolidated clays depends on stress history and that there is generally no proportionality between stress change and strain change. The immediate consequence is that the classical equation for degree of consolidation is in general inapplicable [6], [20].

Thus, the degree of consolidation (U) – time factor (T) solutions that are available in the literature for different stress distributions with depth can, therefore in general, not be applied to normally consolidated clays, but only to soils with constant modulus M (e.g. to over consolidated clays). The stress distribution diagram is needed only as an intermediate step in order to calculate the strain.

The distribution of strain with depth bears no resemblance whatsoever with the stress distribution. Consequently, the shape of the stress distribution diagram is of no direct value for the time rate evaluation.

Since it is the area of the strain diagram that per definition gives the settlement, it is quite obvious that it is the ε - distribution that determines the time - rate of consolidation.

Instead of defining and using the pore water pressure distribution as a function of time and depth, it is rather realistic to define the variation of strain with respect to time and depth of the layer.

$$\varepsilon = f(t, z) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.29)$$

The first step in obtaining a practical solution of equation 2.29 must be derived a differential equation on the basis of strain, instead of pore water pressure. Such equations have been obtained by Janbu [14]. Short outline of the aspect is presented below.

For completely saturated clays, the continuity equation in terms of strain

$$\frac{\partial \varepsilon}{\partial t} = \frac{\partial v}{\partial t} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.30)$$

in which v = vertical velocity of percolating water. Moreover, Darcy's law in terms of strain takes the form γ_w

$$\mathbf{v = \frac{-k}{\gamma_w}\frac{\partial U}{\partial Z}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{(2.31)}$$

$$\mathbf{v = \frac{-k}{\gamma_w}\left(\frac{\partial}{\partial z}\left(\sigma'_{p-}\sigma'\right)\right)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{(2.32)}$$

Where: u =is the pore pressure

σ'_p = effective stre/ss at the end of primary consolidation

σ' =effective stress at any time t

But: $M = \frac{d\sigma'}{d\varepsilon}$ or $d\sigma' = Md\varepsilon$

Then**:**

$$v = \frac{-k}{\gamma_w} M \left(\frac{\partial}{\partial z} \left(\varepsilon_{p-} \varepsilon \right) \right) \ldots \ldots (2.33)$$

But also: $\varepsilon_p - \varepsilon = \bar{\varepsilon}$

Then**:**

$$\mathbf{v} = \frac{-\mathbf{k}}{\boldsymbol{\gamma}_\mathbf{w}} \mathbf{M} \frac{\partial}{\partial \mathbf{z}} \bar{\boldsymbol{\varepsilon}} \ldots \ldots (\mathbf{2.34})$$

Where: εp=strain at the end of primary consolidation

ε =strain at any time t

$\bar{\varepsilon}$ =Remaining strain

From $\varepsilon_p - \varepsilon = \bar{\varepsilon}$

$$\frac{\partial \boldsymbol{\varepsilon}}{\partial \mathbf{t}} = -\frac{\partial \bar{\boldsymbol{\varepsilon}}}{\partial \mathbf{t}} \ldots \ldots (\mathbf{2.35})$$

From continuity equation:

$$\frac{\partial \boldsymbol{\nu}}{\partial \mathbf{Z}} \mathbf{dzdt} = \frac{\partial \boldsymbol{\varepsilon}}{\partial \mathbf{t}} \mathbf{dzdt} \ldots \ldots (\mathbf{2.36})$$

$$\frac{\partial \boldsymbol{\nu}}{\partial \mathbf{Z}} = \frac{\partial \boldsymbol{\varepsilon}}{\partial \mathbf{t}} \ldots \ldots (\mathbf{2.37})$$

Substituting $\boldsymbol{\nu} = \frac{-\mathbf{k}}{\boldsymbol{\gamma}_\mathbf{w}} \mathbf{M} \frac{\partial}{\partial \mathbf{z}} \bar{\boldsymbol{\varepsilon}}$ **and** $\frac{\partial \varepsilon}{\partial t} = -\frac{\partial \bar{\varepsilon}}{\partial t}$

The above equation yields:

$$\frac{\mathbf{k}}{\boldsymbol{\gamma_w}} \mathbf{M} \frac{\partial^2 \bar{\boldsymbol{\varepsilon}}}{\partial \mathbf{Z}^2} = \frac{\partial \bar{\boldsymbol{\varepsilon}}}{\partial \mathbf{t}} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (\mathbf{2.38})$$

$$\mathbf{c_v} \frac{\partial^2 \bar{\boldsymbol{\varepsilon}}}{\partial \mathbf{Z}^2} = \frac{\partial \bar{\boldsymbol{\varepsilon}}}{\partial \mathbf{t}} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (\mathbf{2.39})$$

where: $c_v = \frac{k}{\gamma_w} M$

This equation is called one-dimensional consolidation equation in terms of ε in which:

c_v = coefficient of consolidation

M= tangent modulus

K=permeability and

γ_w = unit weight of water

This is the one dimensional consolidation in terms of the remaining strain. But as it has been discussed in section 2.1, the one dimensional consolidation in terms of the excess pore water pressure is:

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t}$$

The two are identical for constant modules and permeability. But in reality neither M nor K is constant. The difference of the results is due to the difference in the initial conditions.

3. LABORATORY TEST RESULTS AND ANALYSIS

3.1 Classification Tests

3.1.1 General

The previous work results proved that the soil under investigation is tropical lateritic soil [24]. These soils contain loosely bound water of hydration or molecular water which can be lost at 105^{o}c oven temperature results a change in the soil characteristics [2], [7], [25]. Hence, the test samples should not be over-dried prior to testing as the soils will not experience such high temperatures in the field [12], [16]. To address this problem, moisture content tests were conducted on air dried sample. The air dried soil samples were prepared by spreading the material out in trays in the laboratory and leaving it open to the air for at least 15 days or equivalently put inside oven at a temperature of 50^{o}C with maximum relative humidity 30% for at least 5 days [5], [11].

3.1.2 Natural Moisture Content

Moisture contents of the soil samples were determined in the laboratory according to ASTM D2216 - 98 procedures. Results of the natural moisture content test for all test samples are shown in Table 3.1.

Test results shows that moisture content of the soil increase with depth TP6 and TP10. Two reasons can be mentioned for the deviation of TP6 and TP10. First, it shows the complexity of soil due to moisture content variation with depth depends on variations of texture and structure, soil mineralogy, seasons and land uses. Thus, detail study is required to point out the influence factor mentioned above for deviation of TP6 and TP10. This is beyond the scope this thesis. Second, unsuccessful test results may encounter in these two samples.

Table 3.1 Natural moisture content results

Test pit	Depth	Natural Moisture Content, %
TP1	1.5	20.35
	3.0	26.69
TP2	1.5	21.56
	3.0	23.11
TP3	1.5	19.49
	3.0	24.88
TP4	1.5	22.65
	3.0	26.86
TP5	1.5	36.38
	3.0	38.51
TP6	1.5	25.22
	3.0	20.14
TP7	1.5	23.81
	3.0	28.30
TP8	1.5	19.02
	3.0	20.72
TP9	1.5	33.73
	3.0	35.09
TP10	1.5	34.05
	3.0	32.59

3.1.3 Atterberg Limit

For the determination of the Atterberg limit values, air dried soil samples were tested following the procedure given in ASTM D421-85, and D4318 – 98.

The Atterberg limit test results of the soils under investigation is summarized in Table 3.2. Typical graph of liquid limit determination is shown in Figure 3.1.

Table 3.2 Atterberg limit test results

Description	Depth (m)	Liquid limit (LL, %)	Plastic limit (PL, %)	Plastic index (PI, %)
TP1	1.5	46	21	25
	3.0	51	29	22
TP2	1.5	57	31	26
	3.0	49	28	21
TP3	1.5	45	23	22
	3.0	43	20	23
TP4	1.5	59	32	27
	3.0	65	33	32
TP5	1.5	63	29	34
	3.0	66	36	30
TP6	1.5	64	31	33
	3.0	66	35	31
TP7	1.5	55	28	27
	3.0	55	26	29
TP8	1.5	64	26	38
	3.0	71	35	36
TP9	1.5	73	36	37
	3.0	85	45	40
TP10	1.5	74	38	36
	3.0	70	35	35

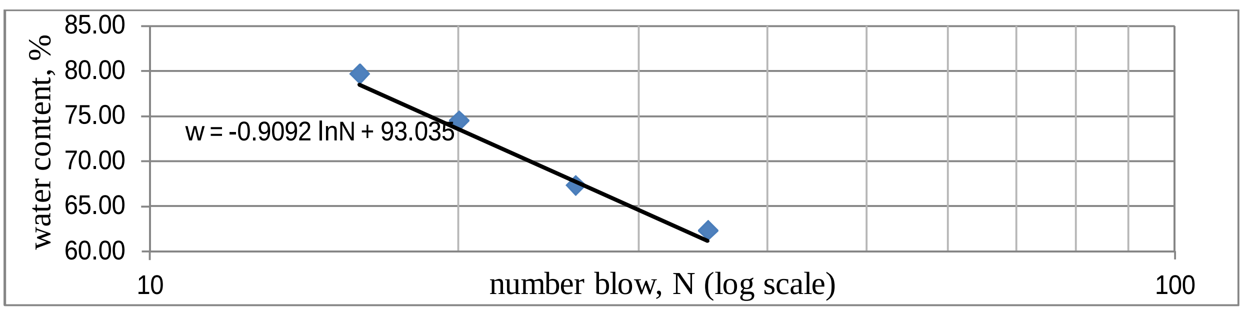


Figure 3.1 Typical liquid limit determination graph

From the above index property the consistency of the soil samples were computed as :

$$Ic = (LL - w)/PI*100 \quad \text{(3.1)}$$

where: w = water contents of the soil in natural condition

LL = liquid limit

PI = plastic index

The consistency index indicates the consistency (firmness) of the soil. It shows the nearness of the water content of the soil to its plastic limit. A soil with a consistency index of zero is at a liquid limit. It is extremely soft and has negligible shear strength. On the other hand, a soil at a water content equal to the plastic limit has a consistency index of 100%, indicating that the soil is relatively firm. A consistency index of greater than 100% shows that the soil is relatively strong, as it is semi solid state. A negative value of consistency index is also possible, which indicates that the water content is greater than the liquid limit" [14].

The tested samples had ranges of firmness from very stiff (hard) to stiff as shown in the Table 3.3. Appendix B Table B-3 shows consistency in terms of range of consistence index [14]. The objective of this thesis as indicated earlier was determining the range of value for the tangent modulus number, m. This range of value was tabulated in accordance with the range of firmness

Table 3.3 Consistency index results

Test pit	Depth, (m)	Natural moisture content, %	Plastic Index, PI (%)	Liquid limit, LL (%)	Consistency index (%)	Firmness
TP1	3	28.73	22	51	101.23	very stiff
TP2	3	28.64	21	49	97.90	Stiff
TP3	3	24.65	23	43	79.78	Stiff
TP4	3	25.56	32	65	123.25	very stiff
TP5	3	38.92	30	66	90.27	Stiff
TP6	3	22.86	31	66	139.16	very stiff
TP7	3	30.36	29	55	84.97	Stiff
TP8	3	23.72	36	71	131.33	very stiff
TP9	3	36.75	40	85	120.63	very stiff
TP10	3	35.82	34	69	97.59	Stiff

3.1.4 Specific Gravity

The specific gravity of samples was determined in accordance with ASTM test designation D 854 – 98 procedure method "B". Test results are presented in Table 3.4.

Table 3.4 Specific gravity test results

Description	Depth	Specific gravity
TP1	1.5	2.91
	3.0	2.85
TP2	1.5	2.76
	3.0	2.68
TP3	1.5	2.75
	3.0	2.72
TP4	1.5	2.80
	3.0	2.70
TP5	1.5	2.76
	3.0	2.79
TP6	1.5	2.73
	3.0	2.65
TP7	1.5	2.68
	3.0	2.72
TP8	1.5	2.64
	3.0	2.64
TP9	1.5	2.73
	3.0	2.78
TP10	1.5	2.75
	3.0	2.78

3.1.5 Grain Size Analysis

Sieving and sedimentation tests are conducted to determine particle size distribution within a soil sample. When preparing tropical residual soil samples for sieving, break down of individual particles by crushing must be avoided [5]. For this reason wet sieving is preferred to dry sieving. The detailed procedure of this analysis can be obtained from the ASTM Designation (ASTM D422-63) and (D2217-85) for sample preparation and laboratory test procedure respectively.

The percentages obtained from sedimentation and sieving are combined and plotted on a particle size distribution curve. The relative percentages of sand-silt-clay in the samples are tabulated in Table 3.5. The graphical presentations of typical samples are shown in Figure 3.2 and 3.3.

From the particle size distribution results, it is observed that there is a range of variation of the particle sizes. Silts and clays constitute over 80% of particles. Hence, the soil under investigation is fine.

The proportion of clay, in each test pit, generally decreased with increasing depth except TP9. This is an indication of the decrease in degree of weathering with depth.

Table 3.5 percentage amounts of particle-sizes distribution

Description	Depth, m	particle size distribution			
		Gravel,[%]	Sand,[%]	Silt,[%]	Clay,[%]
TP1	1.5	0.60	5.53	49.57	44.30
	3.0	1.76	18.11	47.53	32.60
TP2	1.5	0.24	6.75	51.69	41.32
	3.0	0.53	7.50	56.53	35.44
TP3	1.5	0.42	9.33	44.25	46.00
	3.0	1.01	17.05	50.94	31.00
TP4	1.5	0.00	9.61	50.84	39.55
	3.0	0.46	13.74	54.80	31.00
TP5	1.5	0.15	6.92	42.15	50.78
	3.0	0.83	14.79	53.23	31.15
TP6	1.5	0.95	7.73	51.32	40.00
	3.0	1.84	13.66	52.50	32.00
TP7	1.5	0.06	7.16	42.78	50.00
	3.0	0.21	7.34	47.45	45.00
TP8	1.5	0.40	4.28	44.08	51.24
	3.0	0.24	2.96	59.13	37.67
TP9	1.5	0.14	8.33	58.12	33.41
	3.0	0.35	13	50.37	36.28
TP10	1.5	0.00	5.91	52.09	42.00
	3.0	0.22	5.9	61.4	32.48

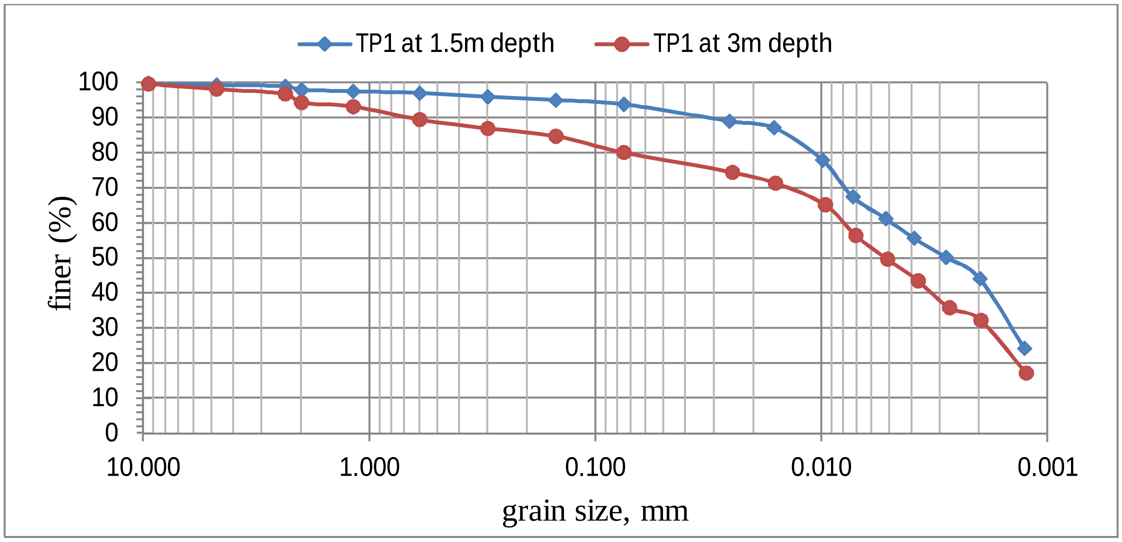


Figure 3.2 Typical gradation curves for variation in sample depths for TP1

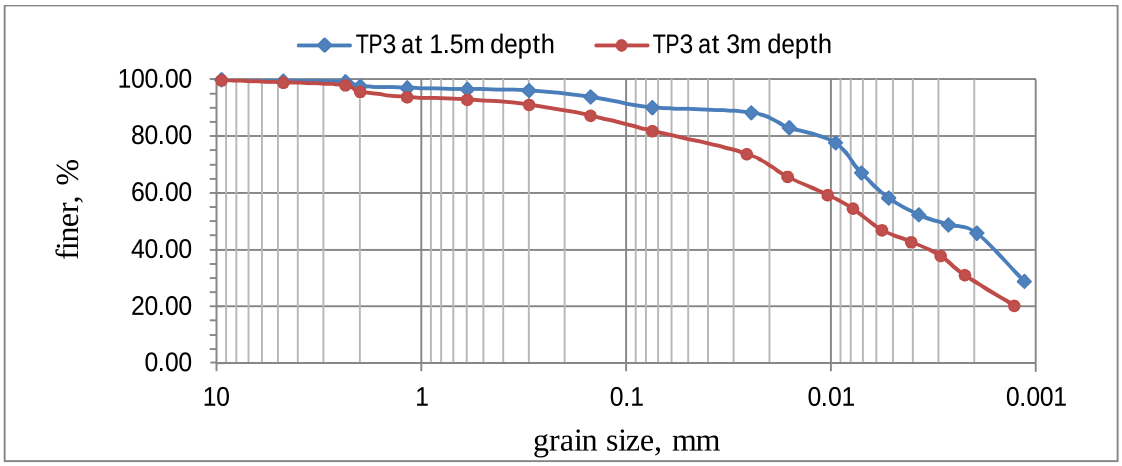


Figure 3.3 Typical gradation curves for variation in sample depths for TP3

3.1.6 Free Swell

Free swell test was conducted according to procedure IS: 2720 (Part 40) 1977. The results are presented in Table 3.6. As can be seen from the test results most samples shows less than 50% free swell. Generally, soils with free swell of less than 50% are considered to have low degree of expansion [16]. Accordingly, the soils found in Asela town are low in expansion.

High water holding capacity soils shrink and become very stiff when natural water content is below plastic limit. These soils are stressed during dry and tend to expand when in contact with water. Hence, soils that have high LL become high in swelling index. As shown from test results TP9 has 74% FS, 85% LL, 40% PI and 1.1 AC which shows greater in activity than all samples.

Table 3.6 Free Swell Test Results

Description	Depth	Free Swell, [%]
TP1	1.5	41
	3.0	39
TP2	1.5	38
	3.0	49
TP3	1.5	35
	3.0	45
TP4	1.5	44
	3.0	67
TP5	1.5	41
	3.0	56
TP6	1.5	63
	2.9	58
TP7	1.5	34
	3.0	39
TP8	1.5	44
	3.0	58
TP9	1.5	69
	3.0	74
TP10	1.5	55
	3.0	65

3.1.7 Activity Number

The activity of clay is expressed as:

Activity A = Plasticity Index/percent finer than 2μm ……………………….. (3.2)

Colloidal activity number values for the soils in research area were calculated and summarized in Table 3.7. Degree of colloidal clay activity is shown in Appendix B Table B-4. The clay soil which has an Activity value greater than 1.25 can be considered as belonging to the swelling type [20].

Table 3.7 Skempton colloidal activity number results

Description	Depth (m)	Plastic index (PI, %)	percent of clay	Colloidal activity, AC (%)
TP1	1.5	25	44.30	0.56
	3.0	22	32.60	0.67
TP2	1.5	26	41.32	0.63
	3.0	21	35.44	0.59
TP3	1.5	22	46.00	0.43
	3.0	23	31.00	0.71
TP4	1.5	27	39.55	0.68
	3.0	32	30.00	1.07
TP5	1.5	34	50.78	0.67
	3.0	30	31.15	0.96
TP6	1.5	33	40.00	1.03
	3.0	31	32.00	0.97
TP7	1.5	27	50.00	0.54
	3.0	29	45.00	0.64
TP8	1.5	37	51.24	0.72
	3.0	36	36.67	0.98
TP9	1.5	36	33.41	1.08
	3.0	40	36.28	1.10
TP10	1.5	36	42	0.86
	3.0	34	32.48	1.05

As mentioned by Morin and Todor, residual soils have low activity number. Because the predominant clay minerals in lateritic soils are of Kaolinite group. These soils are known to be inactive (activity less than 1.25). This is due to the mode of weathering involving the coating of the soil particles with the Sesquioxides. From Table 3.7, one can see that the soil under investigation has activity number <1.25 (Inactive and Normal), which agrees with the Morin' investigation of lateritic soils.

3.1.8 Soil Classification

Soil classification was conducted using both USCS and AASHTO Classification System, according to ASTM D 2487 – 06 and AASHTO M-145 respectively.

3.1.8.1 Unified Soil Classification System (USCS)

Based on determined Liquid Limit and Plasticity Indexes for a soil sample under investigation the proper group of the soil has been made by use of the plasticity chart as shown in Figure 3.4.

The line defined as “A-line” is expressed by Equation 3.3 is used to separate, inorganic clays of high or low plasticity and organic clays and silts.

$$IP = 0.73(LL - 20)\ \% \qquad (3.3)$$

The result of soil type under investigation is shown in Table 3.8 and Figure 3.4 on the plasticity chart. Most of the soil under investigation falls below the A-line and highly plastic silt (MH), except some sample as shown on the plastic chart, which are high to medium plastic clays. The values below the A-line show that there is a mineral content of Kaolinite, Chlorite and Volcanic ash soils (Andosols) with strong Allophanic influence. Accordingly most of soils of Asela were found to fall in this region, which is in consistency with the result of Yilma [24]. Thus, the soil under investigation is regarded as lateritic.

For the soil under investigation, the test results lie below “U-Line” defined by;

$$IP = 0.9(LL - 8)\ \% \qquad (3.4)$$

It is true that results above this line indicates erroneously execute hence repeating the test is usually recommended. As shown in figure 3.4 all samples are found below “U-Line” this show the test is conducted correctly.

Table 3.8 Classification of Soils According to USCS

Description	Depth (m)	Liquid limit (LL, %)	Plastic index (PI, %)	%age amount of particle sizes				Classification According to USCS
				Gravel	Sand	Silt	Clay	Group Symbol
TP1	1.5	46	25	0.60	5.53	52.55	41.32	CL
	3.0	51	22	1.76	18.11	46.38	33.75	MH
TP2	1.5	57	26	0.24	6.75	56.67	36.34	MH
	3.0	49.2	21	0.53	7.50	62.85	29.12	ML
TP3	1.5	45	20	0.42	9.33	41.75	48.50	CL
	3.0	43	22	1.01	17.05	51.94	30.00	CL
TP4	1.5	59	27	0.00	9.61	50.84	39.55	MH
	3.0	65	32	0.46	13.74	55.80	30.00	MH
TP5	1.5	63	34	0.15	6.92	42.15	50.78	CH
	3.0	66	30	0.83	14.79	53.23	31.15	MH
TP6	1.5	64	41	0.95	7.73	51.32	40.00	CH
	3.0	66	31	1.84	13.66	52.50	32.00	MH
TP7	1.5	55	28	0.06	7.16	42.78	50.00	CH
	3.0	55	29	0.21	7.34	47.45	45.00	CH
TP8	1.5	63	37	0.4	4.28	44.08	51.24	CH
	3.0	71	36	0.24	2.96	60.13	36.67	MH
TP9	1.5	72	36	0.14	8.33	58.12	33.41	MH
	3.0	85	40	0.35	13	50.37	36.28	MH
TP10	1.5	74	36	0	5.91	52.09	42.00	MH
	3.0	69	34	0.22	5.9	61.4	32.48	MH

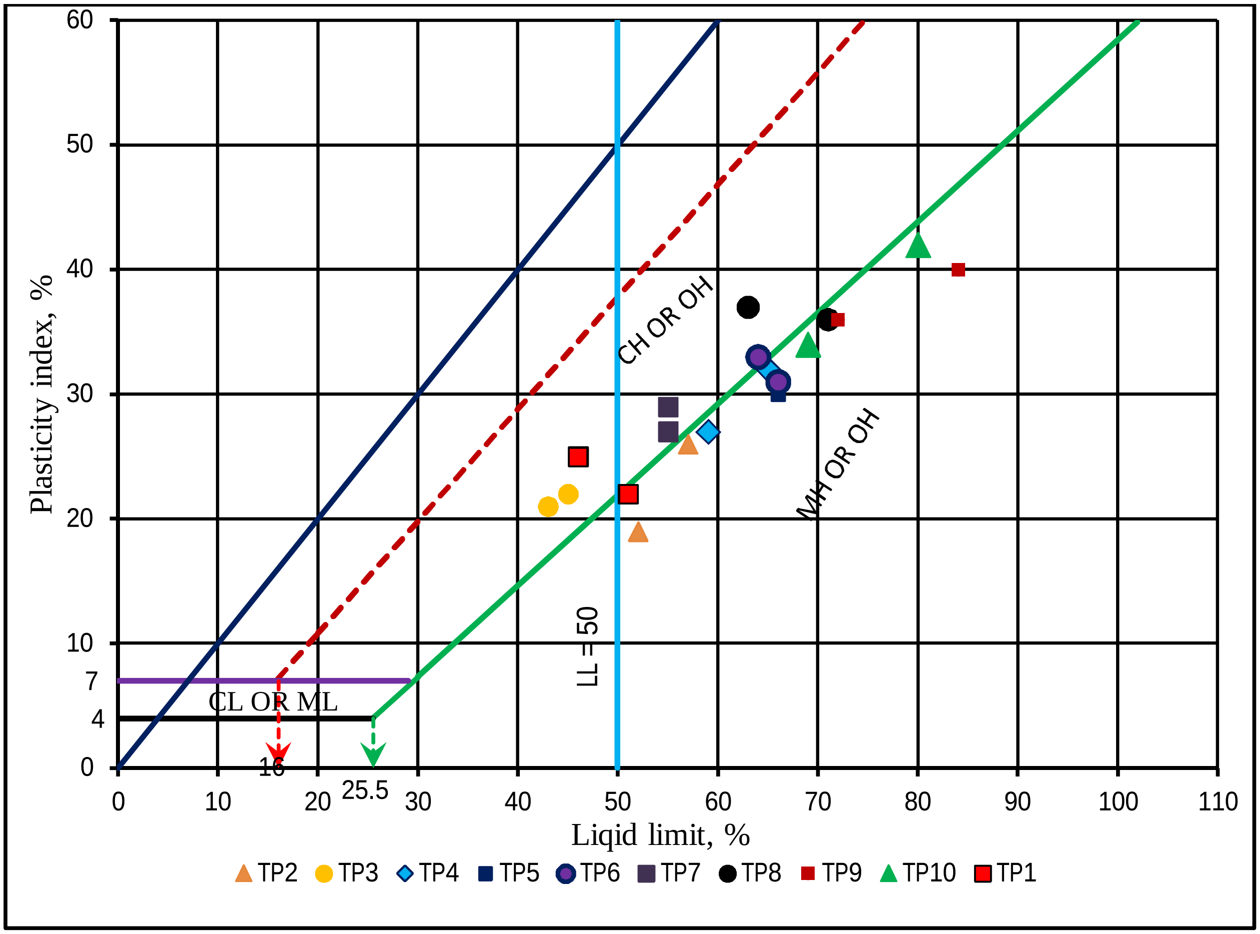


Figure 3.4 Plasticity chart for USCS system

3.1.8.2 AASHTO Classification System

Table 3.9 present results of soil classification according to AASHTO M-145. This classification is checked graphically using Figure 3.5.

All samples fall in A-7 group. A-7 materials are generally considered the poorest performers with regard to roadway construction, but can be utilized in sub-grade material. Furthermore, group index values of the samples was calculated and indicate that only few samples are below 20, which are good sources of sub-grade material, other samples may need some stabilization techniques to be used as source of material. The value of GI is greater than 20 for more samples; this indicates that percentage amount of fine particles is high; the soils are highly plastic.

Table 3.9 Classifications according to AASHTO

Description	Depth (m)	Liquid limit (LL, %)	Plastic index (PI, %)	%age amount of particle size			Classification according to AASHTO	
				2mm	0.425mm	0.075mm	Group Symbol	Group Index
TP1	1.5	46	25	97.97	96.64	93.87	A-7-6	15
	3.0	51	22	94.4	87.25	80.13	A-7-6	15
TP2	1.5	52	19	98.39	97.68	93.01	A-7-5	14
	3.0	49.2	21	97.8	96.97	91.97	A-7-6	14
TP3	1.5	45	20	97.72	96.93	90.25	A-7.6	13
	3.0	43	22	95.78	94.28	81.94	A-7-6	13
TP4	1.5	59	27	97.13	96.23	90.39	A-7-5	19
	3.0	65	32	96.21	95.16	85.8	A-7-5	20
TP5	1.5	61	27	99.25	97.47	92.93	A-7-6	19
	3.0	66	30	96.74	92.14	84.8	A-7-5	20
TP6	1.5	64	41	98.7	97.37	91.32	A-7-6	20
	3.0	66	31	96.18	95.25	84.5	A-7-5	20
TP7	1.5	51	21	99.23	98.97	92.78	A-7-6	15
	3.0	55	29	99.29	99.1	92.45	A-7-6	19
TP8	1.5	54	23	99.22	98.97	95.32	A-7-6	16
	3.0	71	36	99.44	99	96.8	A-7-5	20
TP9	1.5	72	36	99.21	98.54	91.53	A-7-5	20
	3.0	90	46	98.79	97.67	86.65	A-7-5	20
TP10	1.5	74	36	99.71	99.13	94.09	A-7-5	20
	3.0	73	38	99.31	98.99	93.88	A-7-5	20

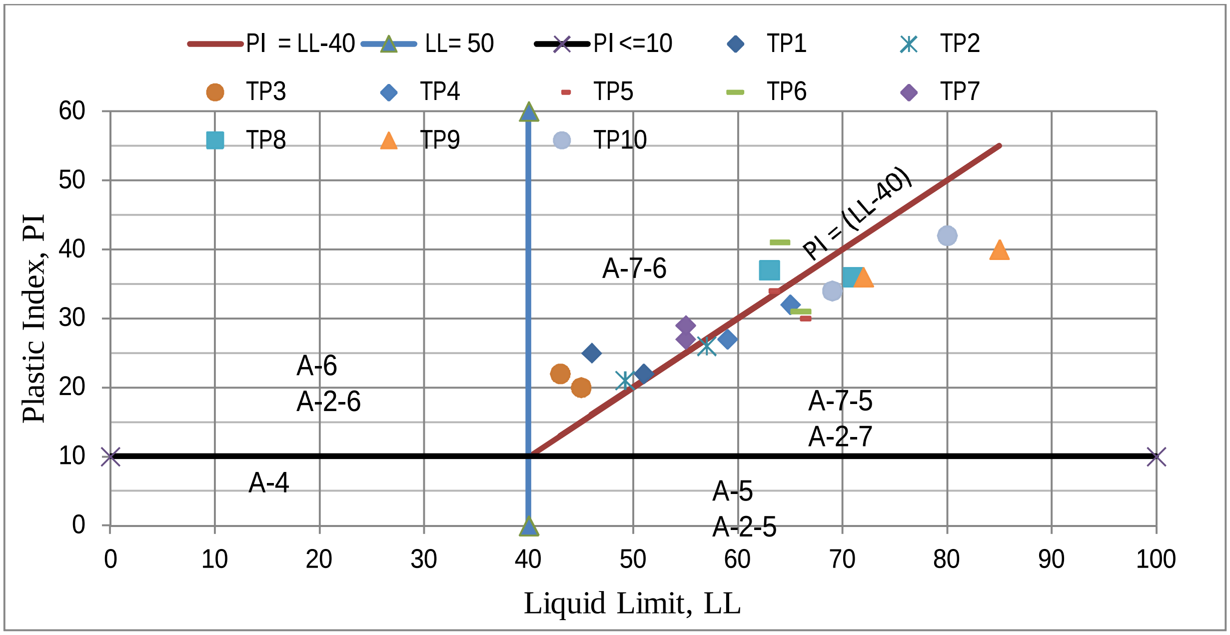


Figure 3.5 Relationships between liquid limit and plastic index for silt-clay groups

3.2 Consolidation Test

3.2.1 General

A one-dimensional consolidation test is widely used to obtain the settlement and rate of settlement parameters. For the vast majority of practical settlement problems, both seepage and strain were considered so as to take place in one direction, this is only being vertical. Hence, in this thesis soil samples were tested in one dimensional consolidation apparatus called Oedometer.

Oedometer test confines the soil laterally in a metal ring so that settlement and drainage can occur only in the vertical direction. Actually some radial displacement and lateral drainage probably occur, but based on experience; these appear to be small enough that a one dimensional analysis gives adequate accuracy in most cases.

As discussed in literature review, classical approach was failed to reveal actual soil behavior rather than attributed to scale transformation. Hence, invalidity of classical approach to describe actual soil behavior becomes the issue of scholars to investigate more reliable alternative.

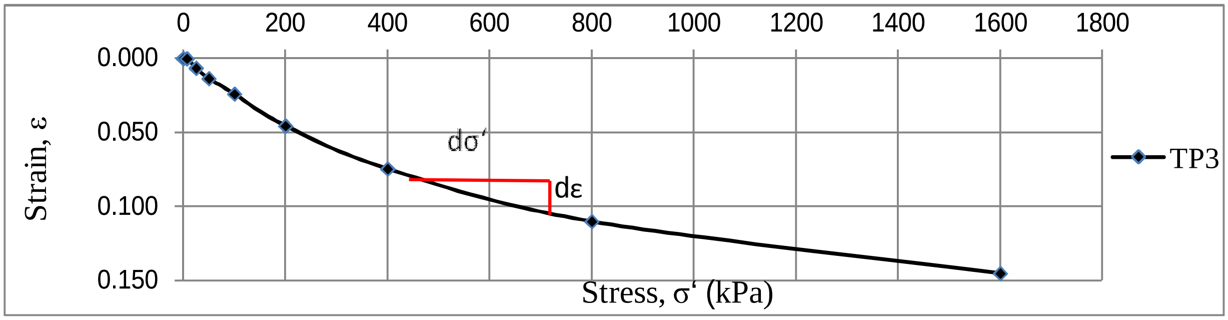
Tangent modulus approach, proposed by Janbu [13], [14], [15] and referenced by the Canadian Foundation Engineering Manual, CFEM [8], combines the basic principles of linear and non-linear stress-strain behavior. The method applies to all soils, clays as well as sands. By the Janbu method, the relation between stress and strain is simply a function of two non-dimensional parameters that are unique for any soil: a stress exponent, a, and a modulus number, m. Janbu has presented a comprehensive summary of his method [13]. The simplicity and rationality inherent in the resistance concept made it the preferred method of analysis of the compressible soils. In this paper tangent modulus approach is verified to our local soil and limitation of classical approach is examined. To achieve the objective both Casagrande and Janbu approaches were used to evaluate test data.

3.2.2 Test Result

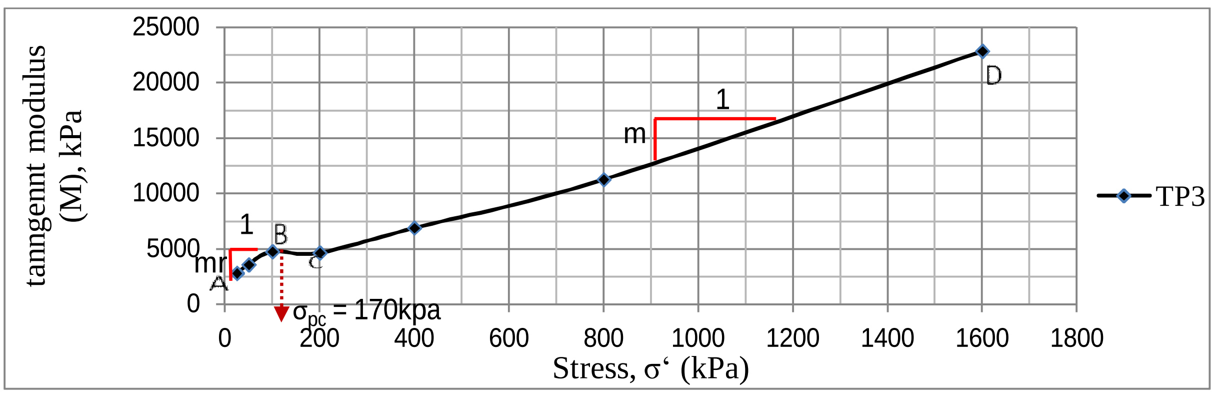
Oedometer test results are presented both in tangent modulus approach, ε vs. σ' with M vs. σ', and classical approach, e vs. log σ'. The same test data were used to evaluate both approaches. From the change in thickness recorded at the end of each normal load the strains and void ratios are calculated. σ' vs. ε curves are plotted in accordance with Janbu [13] recommendation as shown in typical Figure 3.6a and 3.7a. Tangent lines drawn to these curves are used to compute resistance to deformation 'M' of the soil. Variations of tangent modulus, M, obtained from tangent to σ' vs. ε curves are plotted against stress in arithmetic scale as shown in Figure 3.6b and 3.7b. The slope of this graph is the base to determine normally consolidated tangent modulus number (m) and over consolidated tangent modulus number (m_r). As stated earlier, determination of tangent modulus number is the main objective of this paper.

For the purpose of comparisons with Janbu method, test results were also presented according to Casagrande [9]. Void ratios vs. stress curves for loading and unloading were plotted in semi-log scale. Based on these plots, the compression index (c_c) and recompression index (c_r) is determined by taking any two points on the straight portions as shown in Figure 3.6c and 3.7c. Summary of test results are shown in Table 3.10.

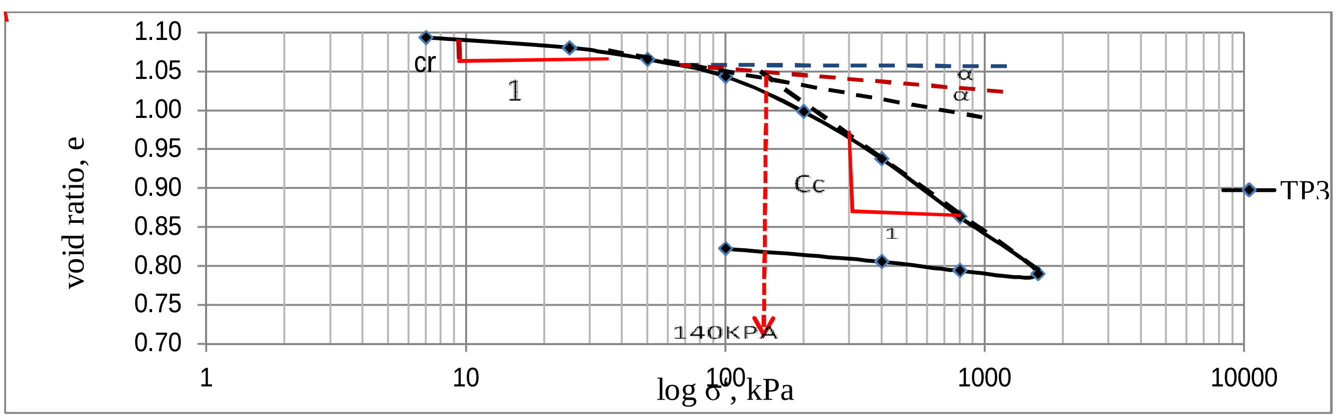
As seen in Table 3.10 all samples are shown as over-consolidated soil by both Casagrande and Janbu analysis except TP5 and TP10 which are normally consolidated soil by Janbu analysis. Hence, typical results of TP3 from over-consolidated soil and TP5 from normally consolidated soil are chosen for illustration purpose in both Janbu and Casagrande graphical methods as shown in Figure 3.6 and 3.7. Other results are attached in Appendix A-1.



a, Typical effective stress vs. strain curves in arithmetic scale

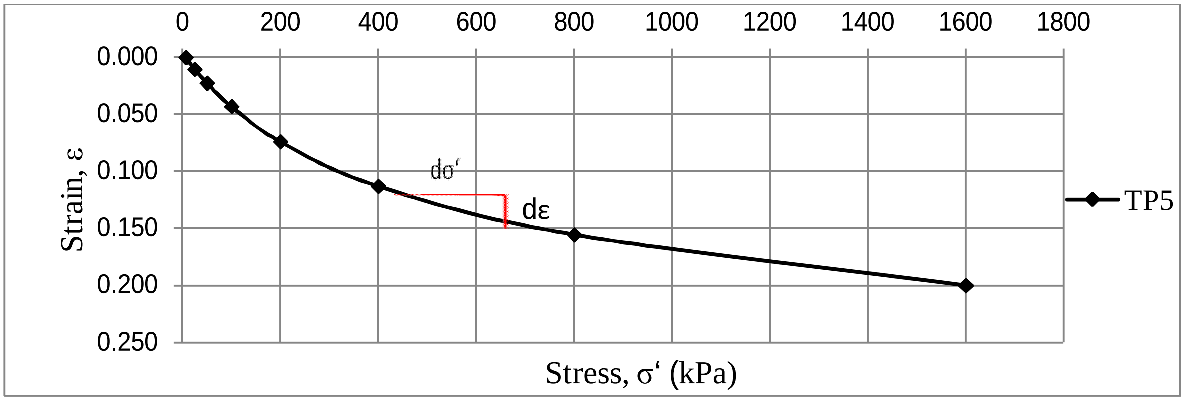


b, Typical effective stress vs. tangent modulus graphs in arithmetic scale

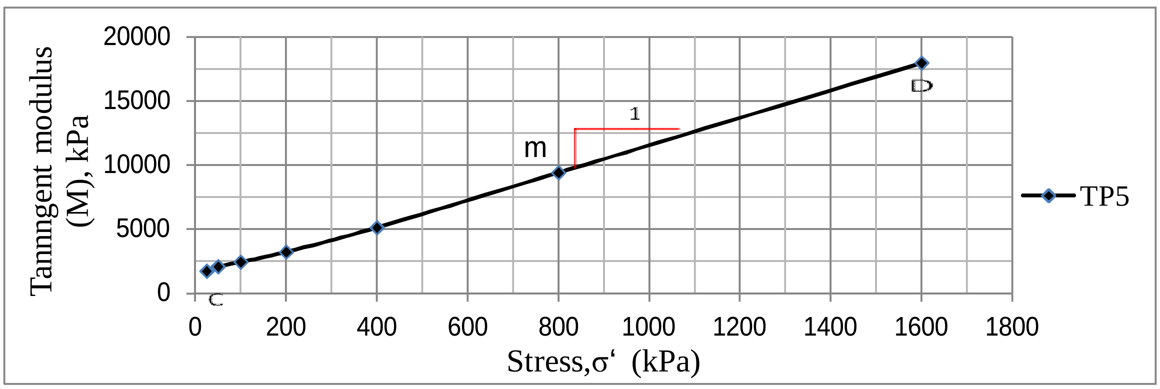


c. Typical effective stress vs. void ratio curves in semi log scale

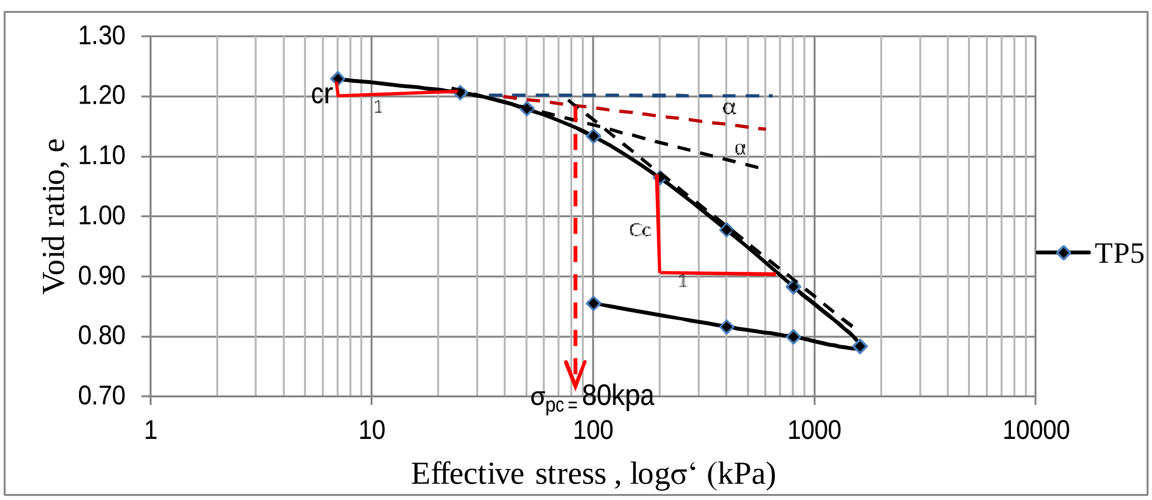
Figure 3.6 Casagrande and Janbu graphical presentations of Oedometer test results of TP3



a, Typical effective stress vs. strain curves in arithmetic scale



b, Typical effective stress vs. tangent modulus graphs in arithmetic Scale



c. Typical effective stress vs. void ratio curves in semi log Scale

Figure 3.7 Casagrande and Janbu graphical presentations of Oedometer test results of

Table 3.10 Summary of parameters determined in classical and tangent modulus approach

Test pit	Natural moisture content (%)	Consistency	Natural moisture content (%)	Wet unit weight (gm/cm^3)	Consolidation parameter results by classical approach			Consolidation parameter results by tangent modulus approach			Deference in σ_{pc} between both approaches
					c_c	c_r	σ_{pc}, kPa	m	m_r	σ_{pc}, kPa	
TP1	28.73	very stiff	28.73	1.67	0.222	0.056	155	17.69	48.84	200	45
TP2	28.64	stiff	28.64	1.71	0.217	0.035	180	12.76	40	190	10
TP3	24.65	stiff	24.65	1.77	0.25	0.023	150	14.45	25	170	20
TP4	25.56	very stiff	25.56	1.73	0.244	0.055	145	14.75	31	175	30
TP5	38.92	stiff	38.92	1.84	0.311	0.036	80	10.30	m_r not revealed	σ_{pc} not revealed	-
TP6	22.86	very stiff	22.86	1.52	0.21	0.044	115	18.17	37.7	175	60
TP7	30.36	stiff	30.36	1.73	0.344	0.057	150	10.10	22.69	175	25
TP8	23.72	very stiff	23.72	1.59	0.25	0.02	90	15.84	28.38	100	10
TP9	36.75	very stiff	36.75	1.79	0.3	0.069	155	11.69	22.97	160	5
TP10	35.82	stiff	35.82	1.67	0.32	0.078	98	10	m_r not revealed	σ_{pc} not revealed	-

3.2.3 Interpretation of Results Based on the Classical and Tangent Modulus Approach

The results of consolidation tests were examined with both classical and tangent modulus approach. The objective of applying both methods is to compare the performance of the methods to describe the best consolidation behavior of the soil.

One can simply understand the drawback of conventional semi-log scale, e vs. log σ‘; from results of typical samples, TP3 and TP5. Both samples in e vs. log σ‘ plots generally show similar bilinear shapes as seen in Figure 3.6c and 3.7c. Plots of these samples in tangent modulus approach; σ‘ vs. ε shows non linear stress - strain relations and M vs. σ‘ shows linear stress – tangent modulus relations in different fashion between TP3 andTP5 as seen in Figure 3.6b and 3.7b. TP3 break in to two linear portions of tangent modulus number at specific stress level. The slope prior to break is greater than the slope after break. TP5 reveals one continuous linear portion of tangent modulus number as seen in Figure 3.7b.

Observation of TP3 in M vs. σ‘ graph is interpreted as over consolidated and normally consolidated ranges. The stress at break is defined as pre-consolidated stress, σ_{pc}. The linear slope prior to break shows over consolidated portion from which, over consolidated tangent modulus number ‘m_r’ is determined. And the linear slope after break shows normally consolidated portion from which normally consolidated modulus number ‘m’ is determined. The slope prior to the break is greater than that after the break. Thus, over consolidation modulus number ‘m_r’ is greater than normally consolidated modulus number ‘m’. The single linear portion in TP5 shows normally consolidated soil; from which only normally consolidated modulus number ‘m’ is determined. These observed clear differences in soil state in tangent modulus approach cannot be seen in the classical approach since semi log graph shows similar bilinear shape for both normally consolidated and over-consolidated soil.

The plot of TP5 using e vs. log σ‘ as shown in Figure 3.7c suggests the existence of a pre-consolidation pressure at 80 kPa stress level, while the linear plot M vs. σ‘ shows no trace of this; in fact the behavior is almost linear as seen in Figure 3.7b.

Table 3.10 presents summary of Oedometer test results of both classical and tangent modulus approach with some effective factors. The difference between pre-consolidation stresses determined by tangent modulus and classical approach are 5kPa to 60kPa. This justifies that the point selected as the pre-consolidation pressure was not the true point of pre-consolidation pressure in semi-log plots. This observation confirms the report of Banerjee and Sribalaskandarajah (1994) saying “Validity of Casagrande graphical analysis failed to explain the true behavior of the soil rather than simply attribute to scale transformation”. The same argument can be extended to question the validity of the bilinear model as a basis for finding compression indices of a soil due to the effect of scale transformation.

Observed characteristics of Figure 3.6b and 3.7b provide valuable insight into the behavior of clay soil during consolidation. By analyzing characteristics of TP3 in Figure 3.6b in the framework of the theory of clay structure and stress history, point C can be defined as a pre-consolidated stress which is equal to 170kPa. This observation in test results confirms the theory “at over consolidated stress, clay soils attain low resistance to deformation and increase in compressibility”. This can mean large break dawn in soil structure at this stress level and entire soil matrix begins to pick up the burden. The linear slopes A to B and C to D clearly show over consolidated and normally consolidated ranges respectively. The higher slope in over consolidated range than normally consolidated range indicates higher resistance to deformation in over consolidated clay relative to normally consolidated clay soils. On other hand, plot of M vs. σ‘ is linearly continuous from C to D without break as shown in Figure 3.7b in TP5. The graph does not reveal point of pre-consolidated stress. This shows that the soil is normally consolidated; however e vs. log σ‘ indicates that the soil is over-consolidated with 80kpa pre-consolidated stress as seen in Figure 3.7c. This result confirms Banerjee and Sribalaskandarajah [4] investigation that drawback of classical approach in prediction of apparent over-consolidation and point of over-consolidation stress.

While TP3 and TP5 are chosen for illustration purpose, as shown in Figure 3.6 and 3.7, the same general behavior is found in the other samples.

However, it is common to assume a constant value of tangent modulus ‘M’ in over-consolidated range. It is clear from the result that the obtained M shows linear variation with effective stress. But, the tangent modulus in normally consolidated range shows similar variation of ‘M’ with effective stress, the same as the Scandinavian soil. As discussed in literature review, soils that show linear variation of ‘M’ with effective stress are grouped as plastic with stress exponent (a) equal to zero. This observation points out that the soil of the study area is different in over consolidated range from the soil of Scandinavian country. Thus, using one stress exponent (a) as in the Scandinavian countries for over consolidated range is incorrect. Hence, the stress exponent for the study area is zero for both ranges. Therefore in contrary to the classical approach one can determine settlement for both normally consolidated and over consolidated soils in one comprehensive formula.

3.2.4 Determination of Ranges of Values for the Test Results

According to the test results, it is clearly seen that the characteristics of soil of the study area is consistent with the soil of the Scandinavians where the tangent modulus approach was developed, except for over consolidated soils. In over-consolidated soil, tangent modulus varies with effective stress in both over-consolidated and normally consolidated ranges. Hence, test results were consistent with the type plastic (PL). Thus, the value of the stress exponent ‘a’ was equal to zero and the only soil parameter required is the modulus number m. One can therefore, apply the same formula as developed by the Norwegian Institute using the ranges of values of ‘m’ developed for our local soils. Table 3.11 shows ranges which are typical for soil of the study area in both over consolidated and normally consolidated ranges for stiff to very stiff consistency ranges.

Table 3.11 Typical ranges of modulus number and stress exponent results of Asela (red clay)

Firmness	Tangent modulus number, m		Stress Exponent, a
	NC	OC	
Stiff - very stiff (hard)	10 – 20	20 – 50	0

4. DISCUSSIONS OF TEST RESULTS AND COMPARISONS

4.1 Discussions of Test Results

One dimensional consolidation laboratory tests were conducted on 10 soil samples collected from different location of Asela town. All samples are shown as over-consolidated soils with both Janbu and Casagrande methods except TP5 and TP10 which are normally consolidated soil with Janbu method. The results of pre-consolidated stress predicted by Casagrande method was 5kPa – 60kPa less than those estimated using Janbu method. Reduction of pre-consolidation stress in Casagrande method shows the effect of scale transformation as discussed in literature review.

Test results of TP5 and TP10 using Casagrande method suggest existence of pre-consolidation stress which is not revealed in Janbu method. This observation in test result confirms Banerjee and Sribalaskandarajah reports that the distortion of the true consolidation behavior of the clay manifests itself in four shortcomings of the classical approach: apparent over-consolidation, pre-consolidation pressure, maximum volumetric strain, and parameter estimation. The results of Figure 3.6 and 3.7 are good agreement for scale transformation premises.

As observed from the test results, the tangent modulus approach clearly showed the over consolidated (OC) and normally consolidated (NC) ranges, and the point of pre-consolidated pressure, σ_{cp}. These are basic characteristics of the soil which requires a lot of efforts to obtain in the classical method.

Over -consolidated and normally consolidated ranges are equally important to determine the soil behavior. In normally consolidated range the values almost correspond to the values found in Scandinavian countries. But in the over consolidated range the response of local clay soils is different from the soils found in the literatures. The steep slope in the over consolidated range gives a greater value of modulus number than in normally consolidated range. Hence, there were two straight lines having different slopes for both ranges. One can therefore use these values in one strain formula for computation of settlements in both over consolidated and normally consolidated ranges.

The simplicity of the graph gives an appreciated strain computation without computing the void ratios as an intermediate step. The avoidance of computation of the void ratio gave a further step that it avoided the irrational concept of computing it with classical method.

The ease of finding the strain distribution throughout the soil strata also gave us a more and precise way of solving problems of the time - rate of consolidation. This avoids the inconsistency of the argument made by the classical method that the stress is proportional to strain and hence the time rate of settlement is dependent on the pore water pressure. In real sense this is not true as it can be understood from the test results. This means the stress and strains are not proportional and hence pore pressure distribution is different from strain distribution. Therefore, the U-T table that is found in most of the literatures, based on pore water pressure, is not applied to normally consolidated clays, but to soils with constant 'M', in the over consolidated range. This is not applicable to our local soils as M shows variation with effective stress in over consolidation range.

4.2 Comparison of Test Results with Previously Done Researches

As shown from comparison Table 4.1, the result of Asela and Addis Ababa red clay soil reveals similar ranges of compression modulus number 'm', but re-compression modulus number 'm_r' of Asela shows a greater range than Addis Ababa red clay soil for stiff to very stiff consistency.

As one can see from Table 4.1, modulus number for silty clay soil with stiff to very stiff consistency of Scandinavian countries ranges from 20-60. This shows greater ranges of modulus number than modulus number of current study area. Stress exponent, 'a' of Addis Ababa and Asela is zero for both over- consolidated and normally consolidated soil as shown in Table 4.1, but stress exponent of Scandinavian soil is zero for normally consolidated soil and one for over-consolidated soil. Typical modulus number and stress exponent of Addis Ababa and Scandinavian soils for different soil consistency is shown in Appendix B.

Table 4.1 Comparison of test results with previously done research work

	Previous research (Nerea, 2004)	Previous research (Fellinius, 2015)	Current research
Types of soil	Red clay	Silty clay	Red clay
location	Addis Ababa	Scandinavian countries	Asela
Consistency	Stiff-very stiff	Stiff-very stiff	Stiff-very stiff
Compression modulus number, m	10-20	20-60	10-20
Re-compression modulus number, m_r	20-40		20-50
Compression stress exponent, a	0	0	0
Re-compression stress exponent, a	0	1	0

5. CONCLUSION AND RECOMMENDATION

5.1 Conclusion

In this thesis an investigation has been made to assess limitation of classical approach and applicability of the tangent modulus concept for the study area and to establish the range of values of the soil parameters. In order to meet the objectives, undisturbed and disturbed soil samples were collected from different parts of Asela town and laboratory tests were conducted to determine the physical properties, and consolidation behavior of the soils.

Based on the test results the following conclusions were drawn.

1. Index properties such as grain size distribution, Atterberg limit, and consistency index were tested and showed almost the same ranges of values to the previously studied clay soils of Asela town.
2. It has been shown herein that the tangent modulus concept which has a good scientific explanation is applicable for local clay soil to compute compressibility.
3. For clay soils under area of investigation, the modulus number, m, is predicted as 10 - 20 in normally consolidated range and 20 - 50 in over consolidated range, while stress exponent, a, is equal to zero in both ranges for stiff to very stiff consistency.
4. The test results of local clay soils show nonlinear stress - strain relation which is bilinear in void ratio versus stress relation.
5. The value of over-consolidated modulus number, $m_{r,}$ is greater than the normally consolidated modulus number, m. This change in resistance between the two ranges break the graph of tangent modulus vs. stress to have a sharp and clear point of demarcation called the pre-consolidated pressure, a pressure where major structural break down of the soil takes place.
6. The linear relationship between tangent modulus and stress in the over consolidated range made soil of the study area different from other clay soils found in Scandinavian countries.
7. The tangent modulus approach can be used to compute the strain distribution throughout the soil strata for different modulus numbers in both ranges of consolidated soil in one comprehensive equation.

8. Non-linear relationships between stresses and strains in both over consolidated and normally consolidated ranges enable us to conclude that the time rate of settlement is independent of pore water pressure distribution, but it is on strain distribution.

5.2 Recommendation

The present work has attempted to verify the applicability of the tangent modulus concept. However, this investigation has not covered all areas of the concepts. In view of this, it would be desirable to extend the present work in the following areas.

1. Even though tangent modulus approach is applicable for all types of soil, here in this thesis, as the test result indicated, one type of soil, i.e. clay soils found in Asela town is considered with stiff to very stiff consistency. One can develop reference data base for different soil types and consistency index with only one parameter, i.e., the modulus number.
2. As it can be understood from the test results, stress- strain are not proportional and hence pore pressure distribution is different from strain distribution. Therefore, the U-T table developed based on pore water pressure is not applicable. A U-T reference depending on strain distribution can be developed in order to determine the rate of settlements.
3. To improve the quality of the data provided by conventional Oedometer, future research programs are recommended by Constant Rate of Strain (CRS) Oedometer to increase the level of accuracy of test data, so that of consolidation parameters.
4. Further application of the method for preparing the “iso m “values for the town of Assela with more sampling and data.

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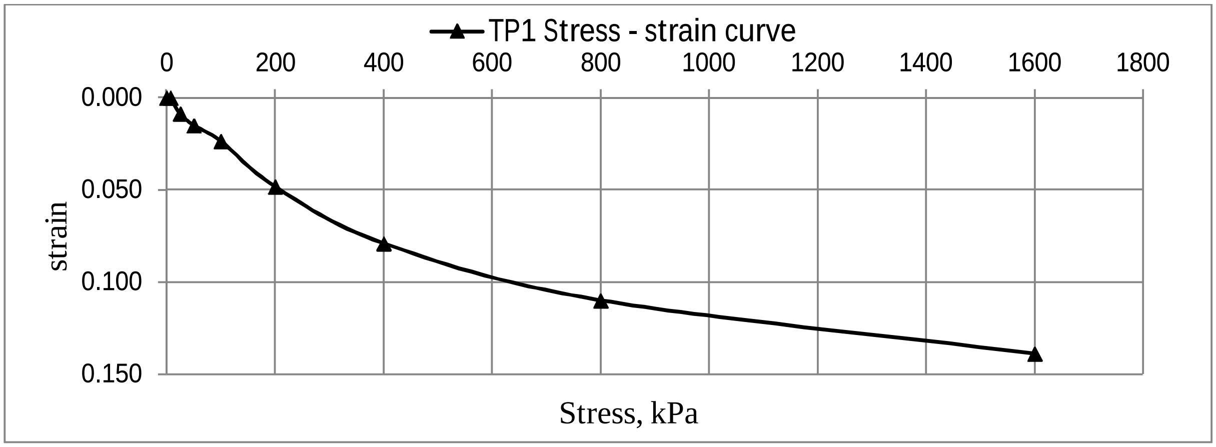
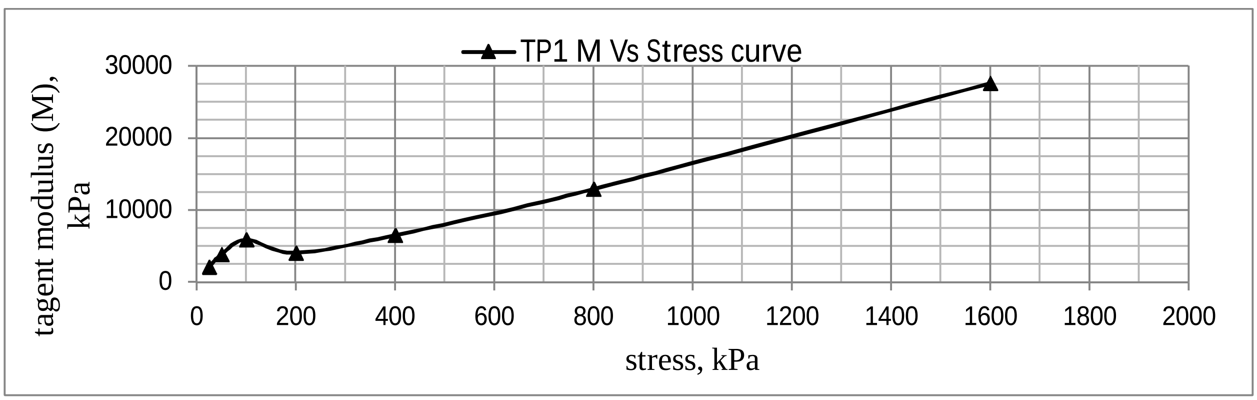
APPENDIX A:

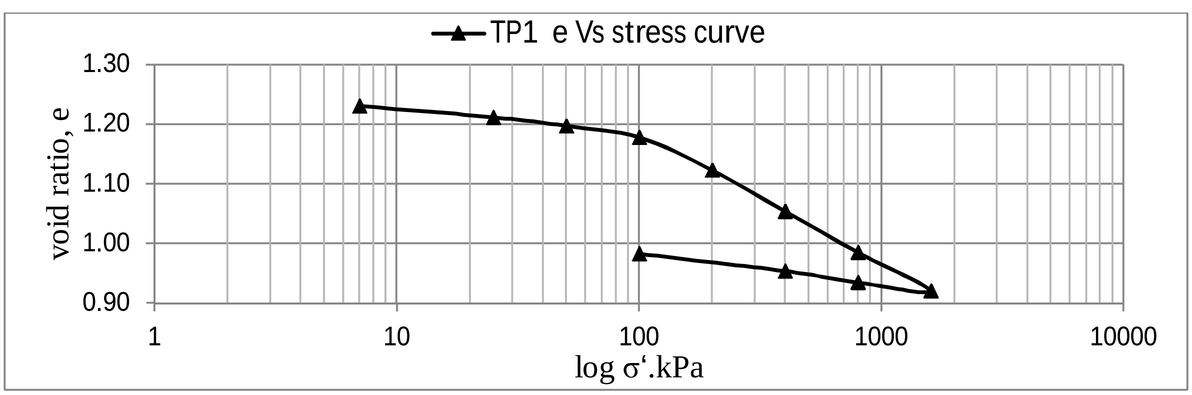
A -1. Janbu and Casagrande Graphical Presentation of Test Results



a, Stress – strain curve

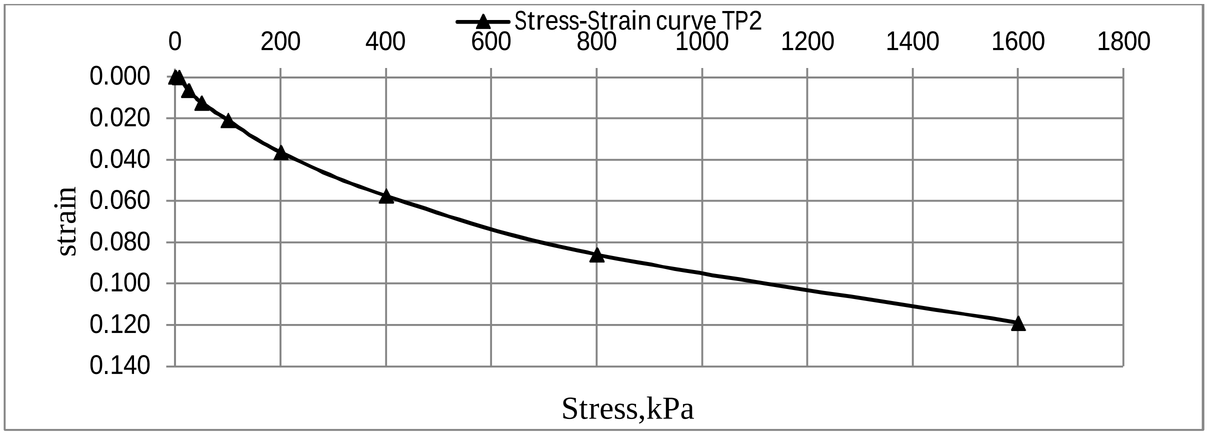


b, tangent modulus – stress curve

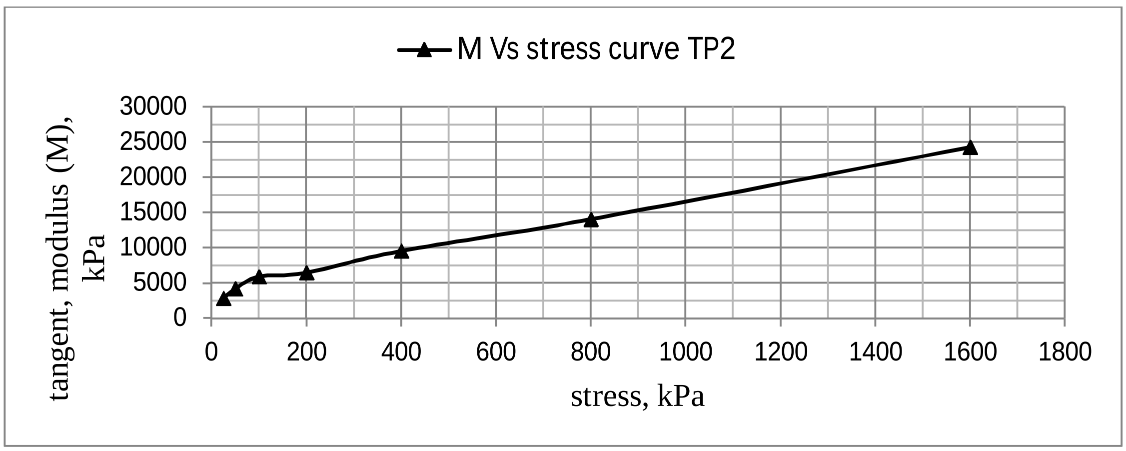


c, void ratio – stress curve

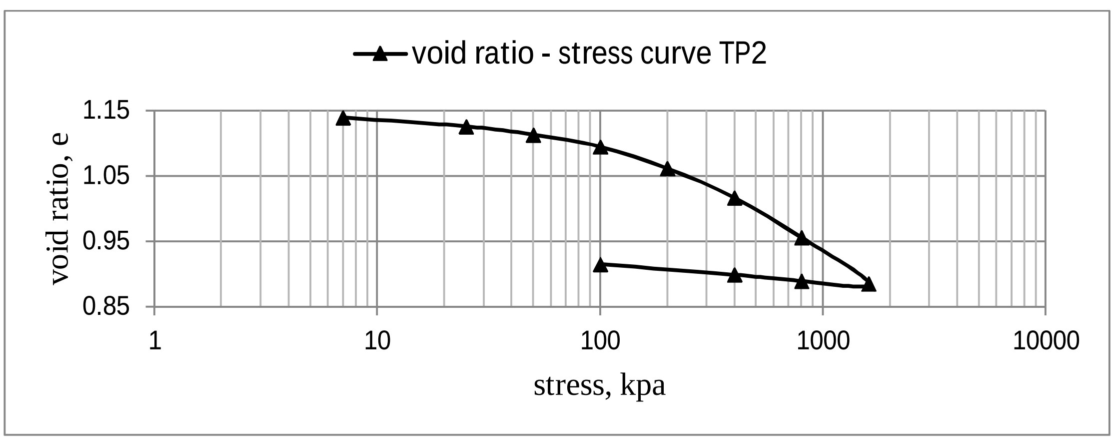
Figure A- 1 Oedometer Test results of TP1



a, stress – strain curve

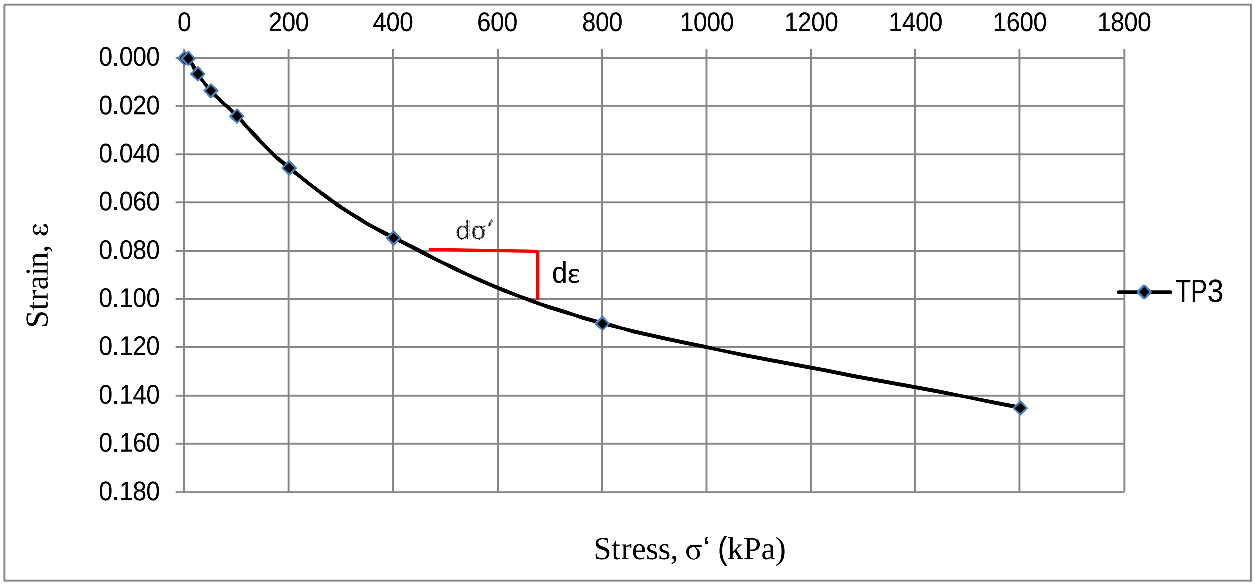


b, tangent modulus - stress curve

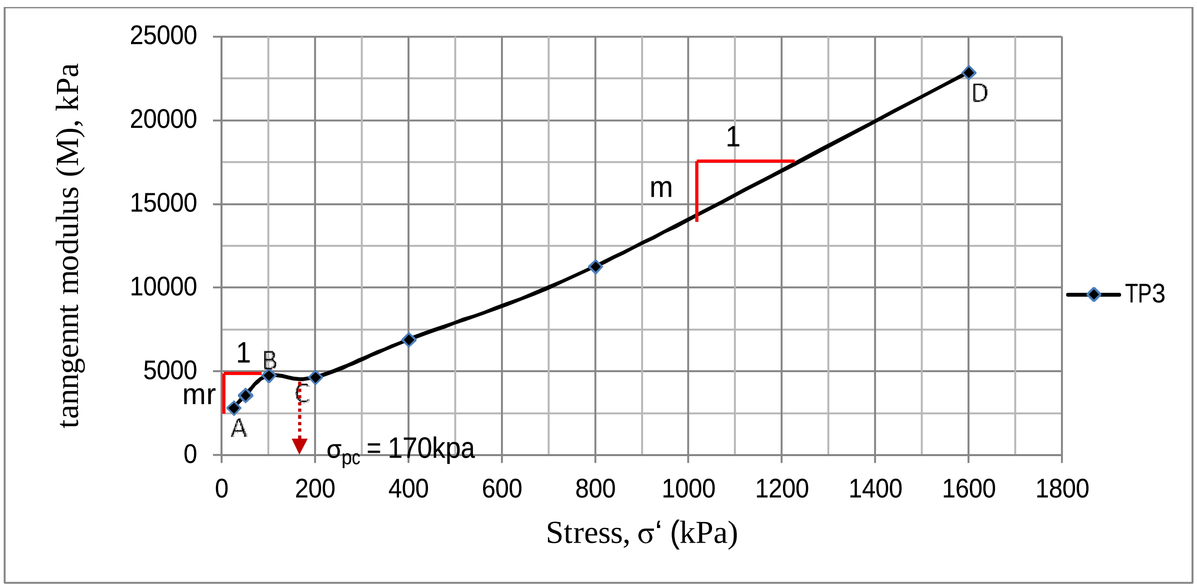


c, void – ratio stress curve

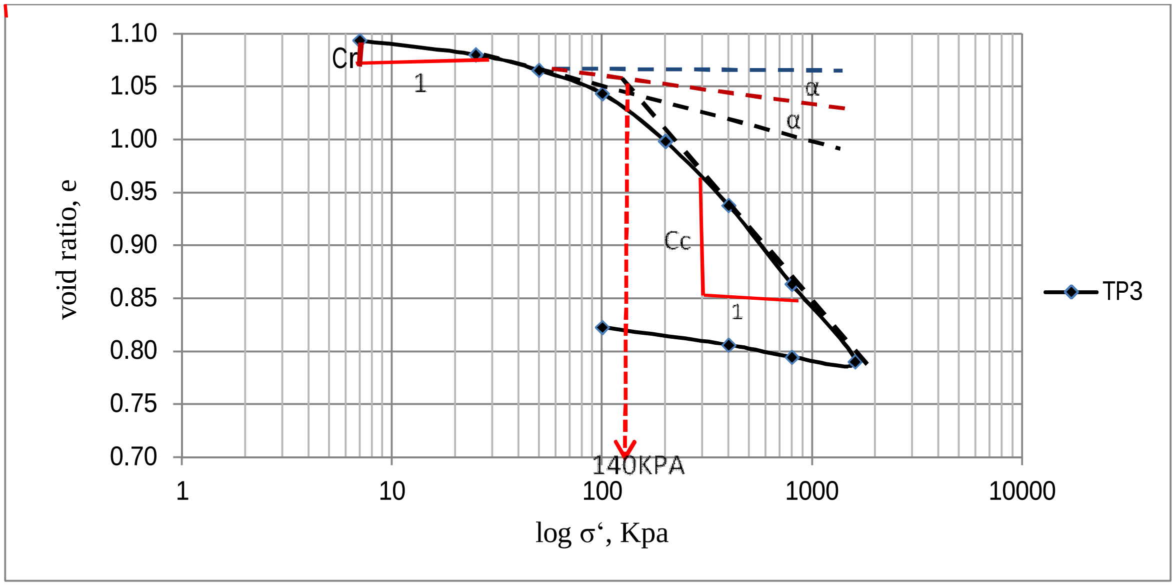
Figure A – 2 Oedometer test results of TP2



a, stress – strain curve

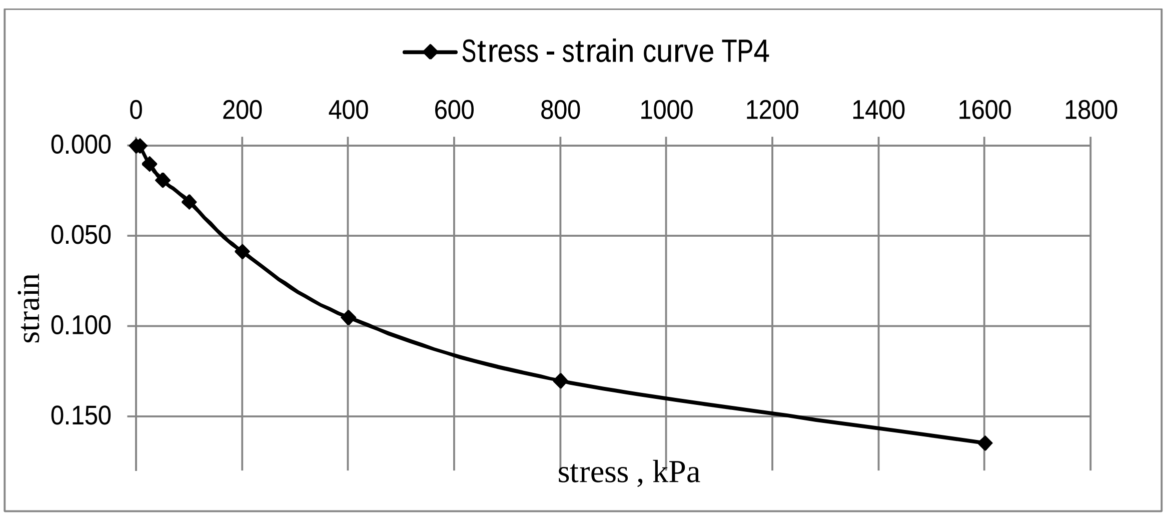


b, tangent modulus - stress curve

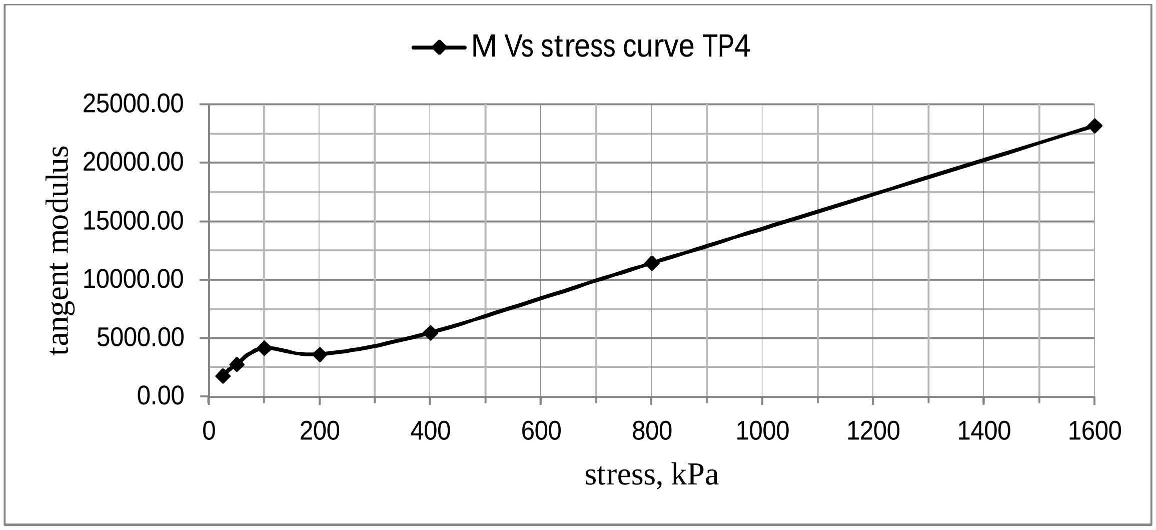


c. void ratio - stress curve

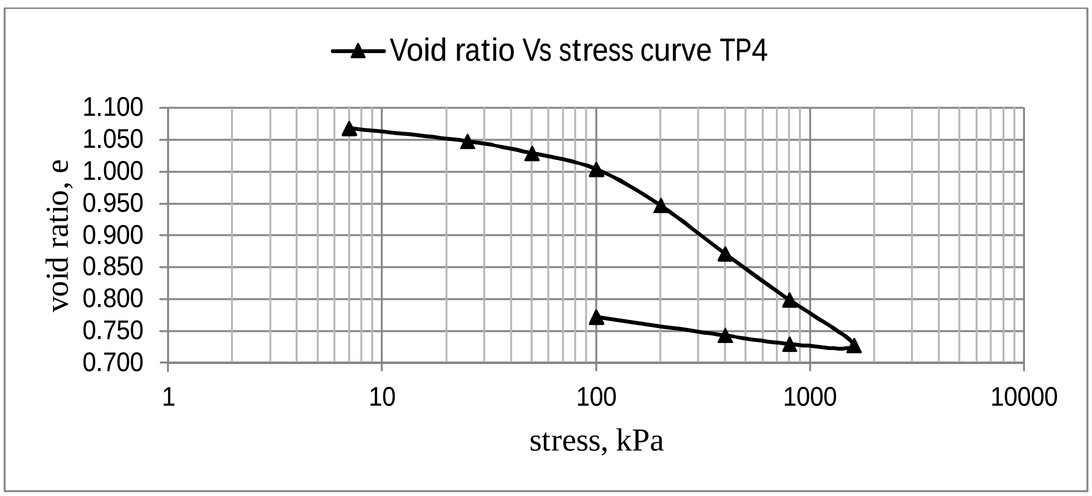
Figure A -3 Oedometer test results of TP3



a, stress strain curve

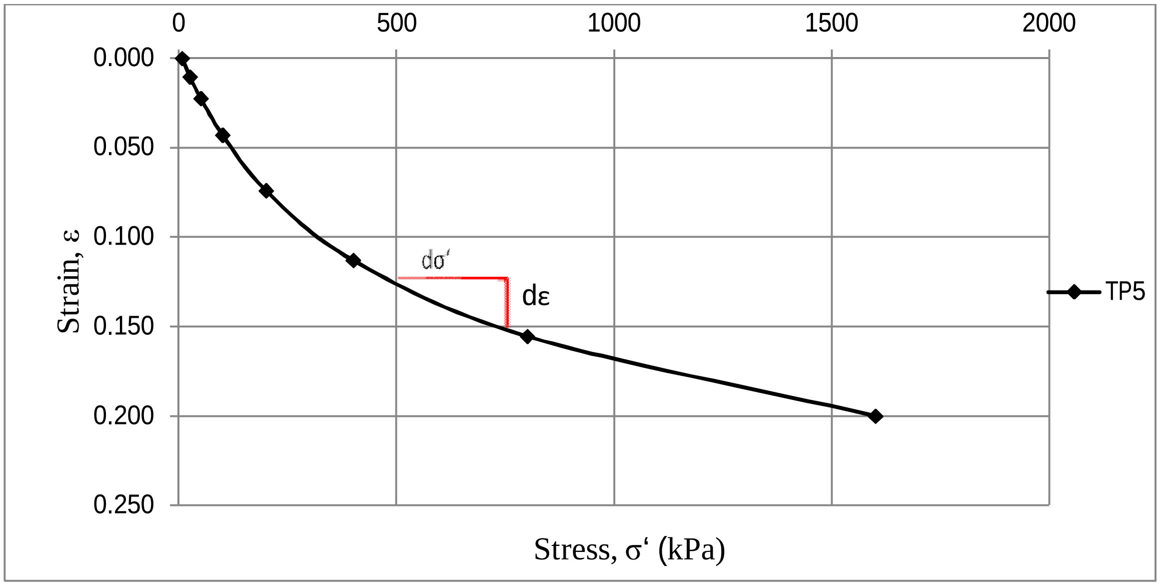


b, tangent modulus – stress curve TP4

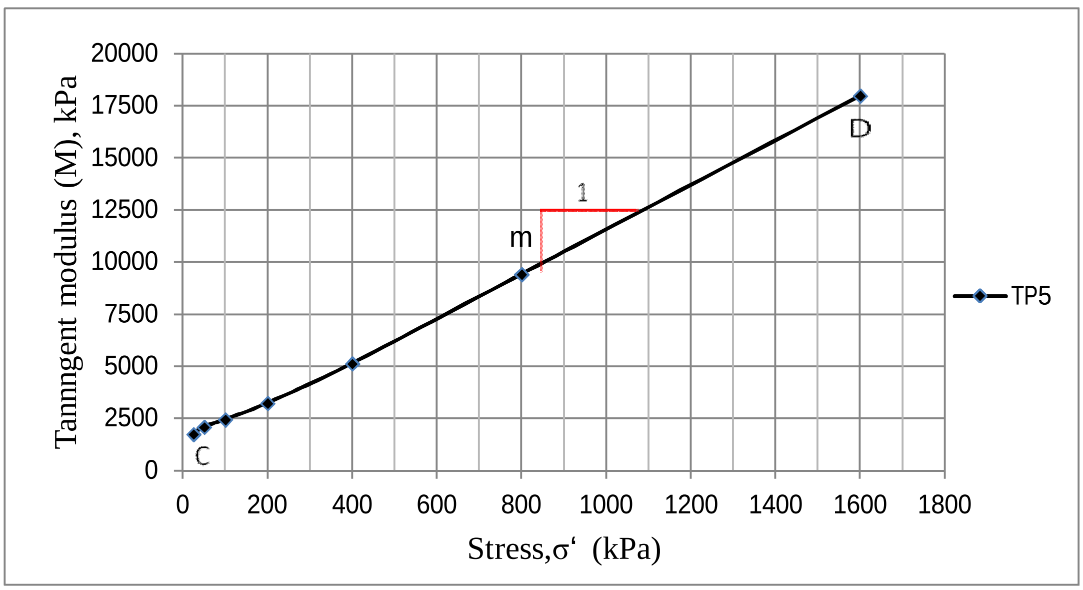


c, void ratio – stress curve

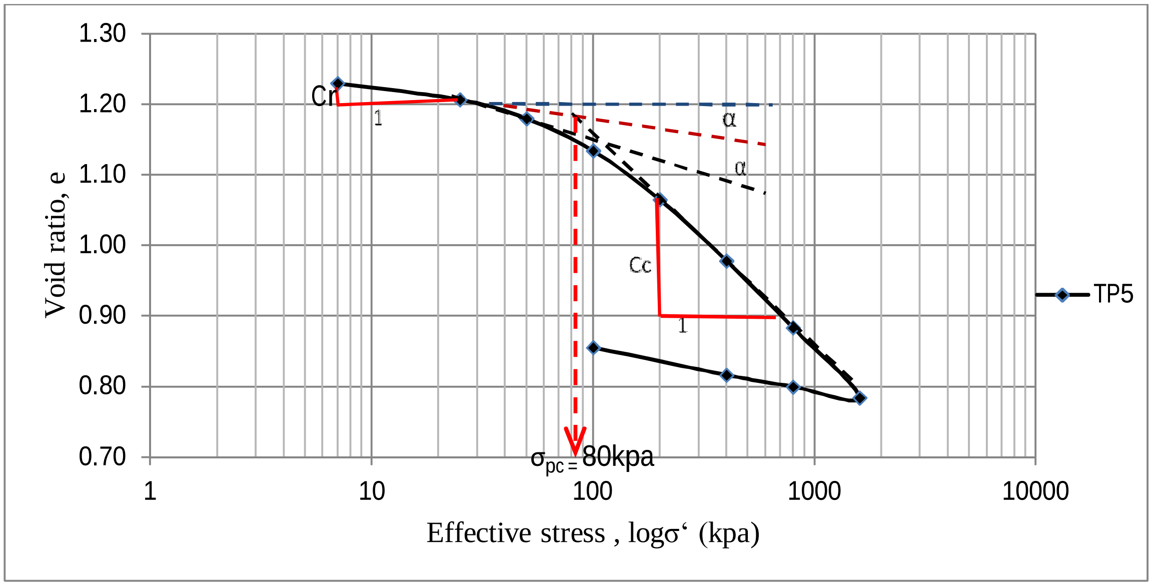
Figure A -4 Oedometer test results of TP4



a, stress – strain curve

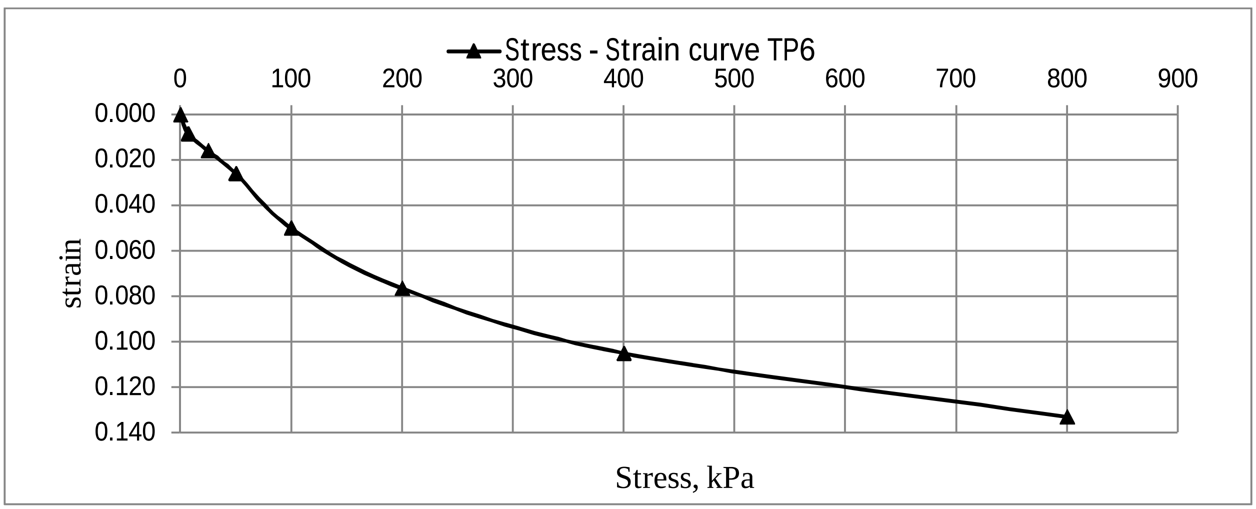


b, tangent modulus - stress curve

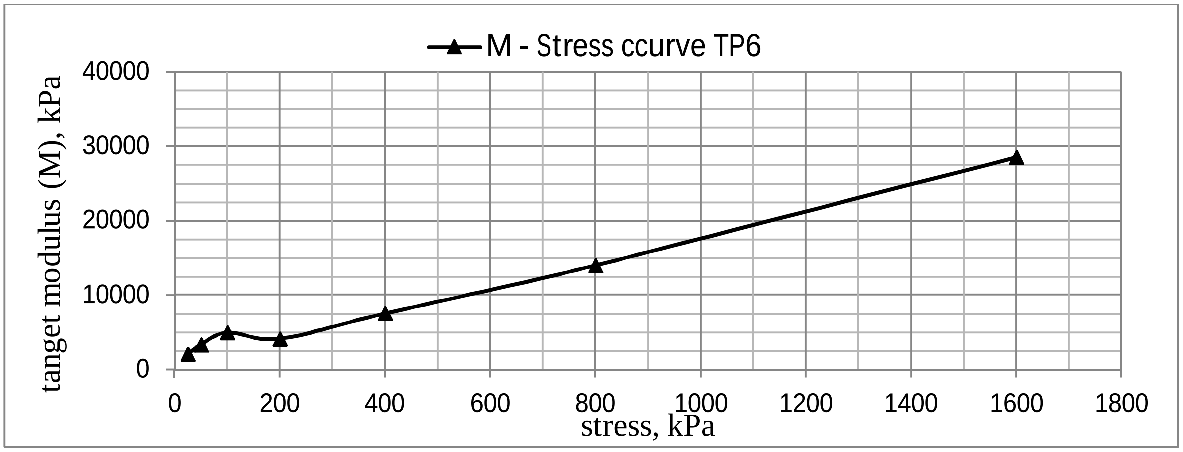


c. void ratio - stress curve

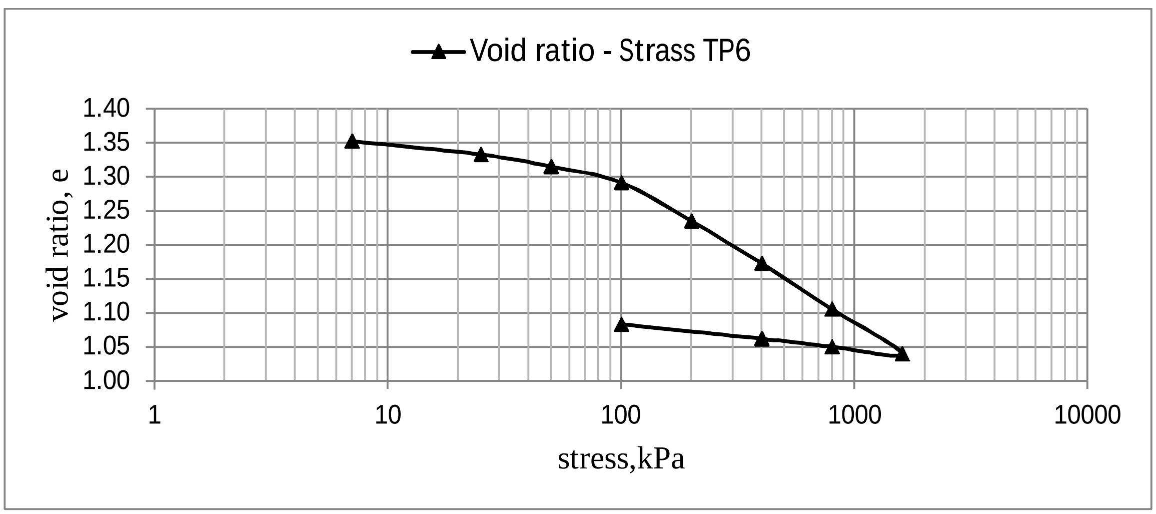
Figure A – 5 Oedometer test results of TP5



a, stress – strain curve

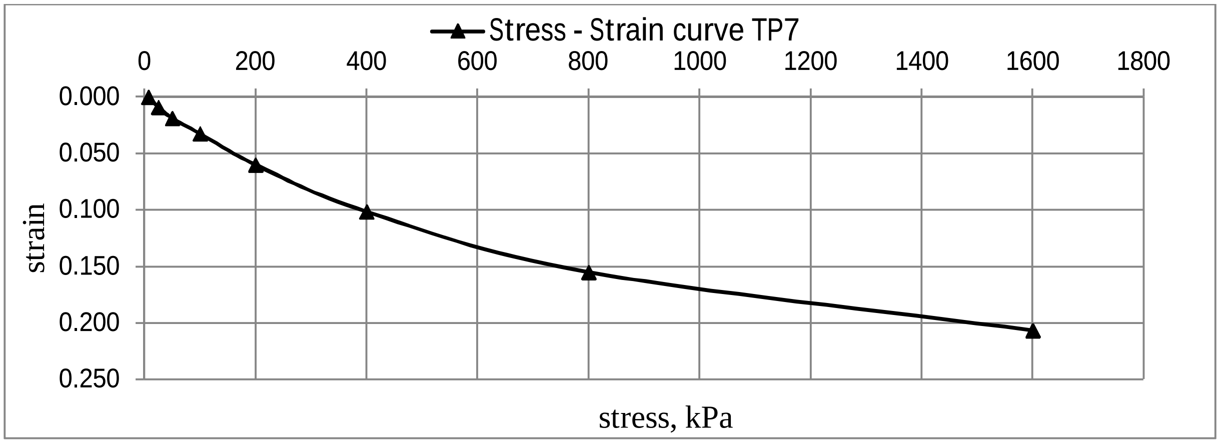


b, Tangent Modulus – Stress

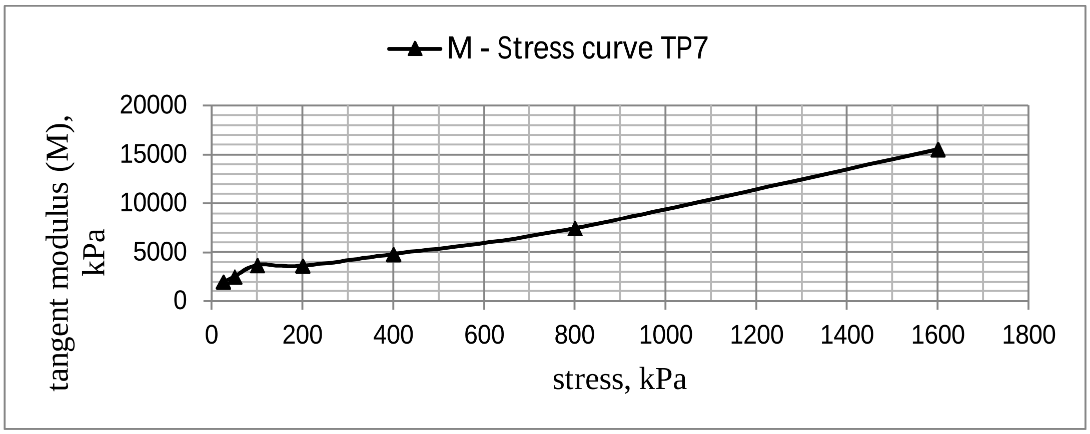


c, Void ratio – Stress curve

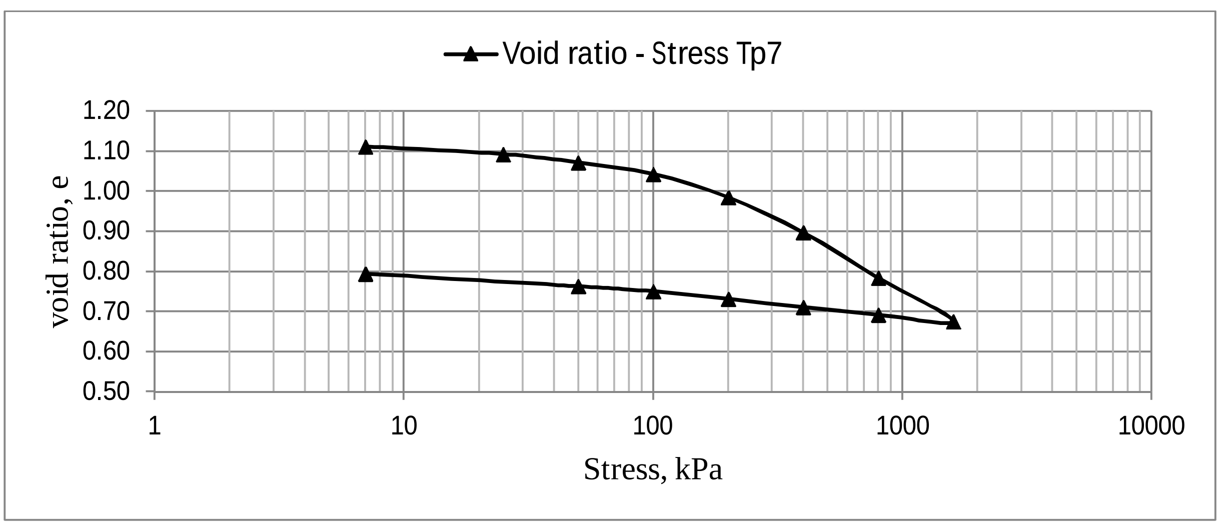
Figure A –6 Oedometer test results of TP6



a, Stress – Strain curve

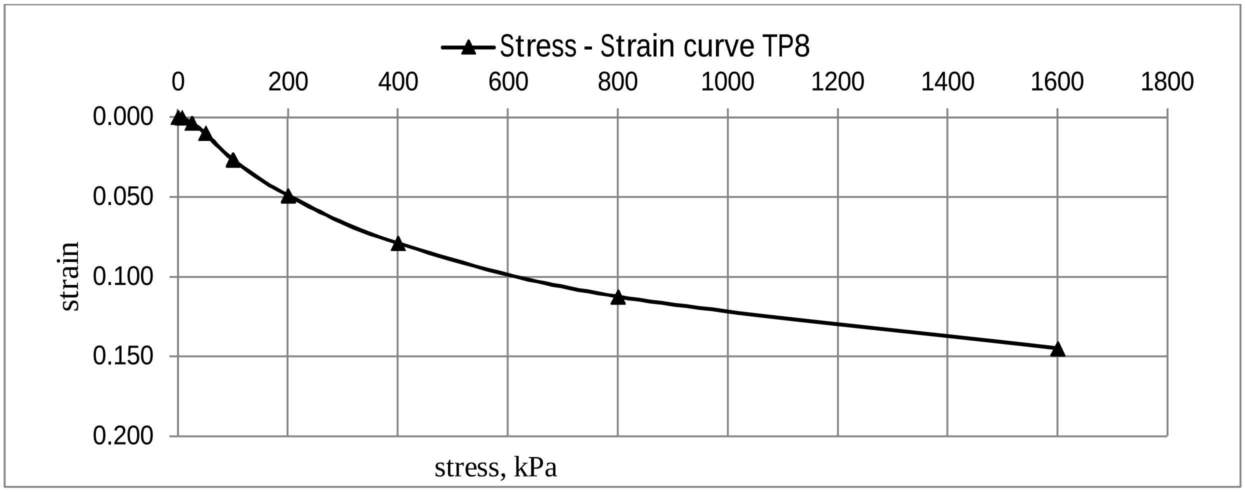


b, Tangent Modulus – Stress curve

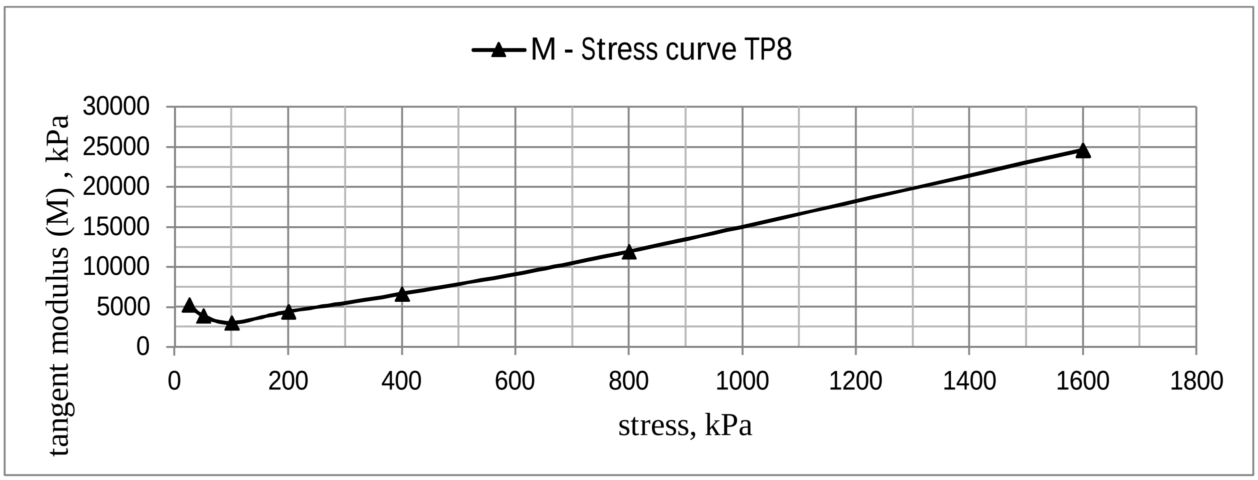


c, Void ratio – Stress curve

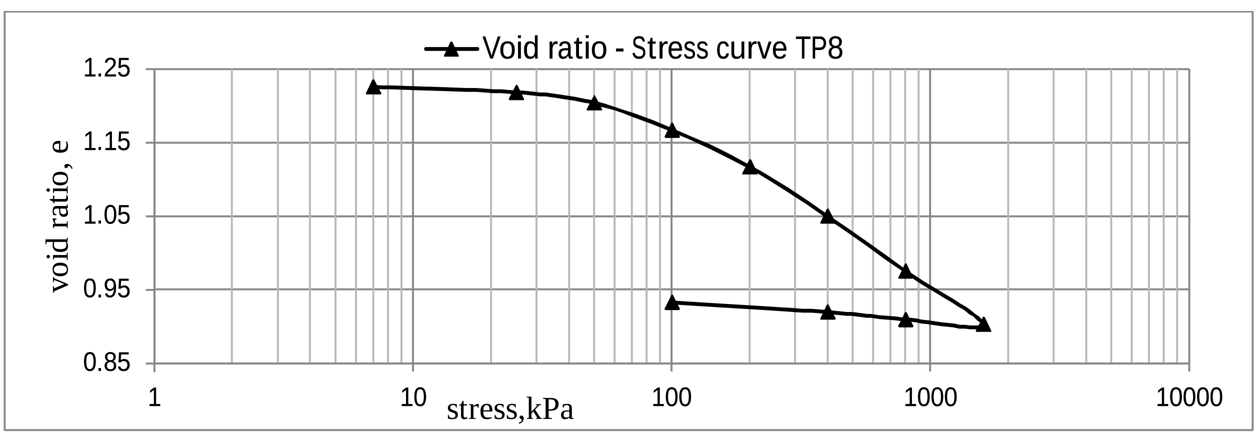
Figure A –7 Oedometer test results of TP7



a, Stress – Strain curve

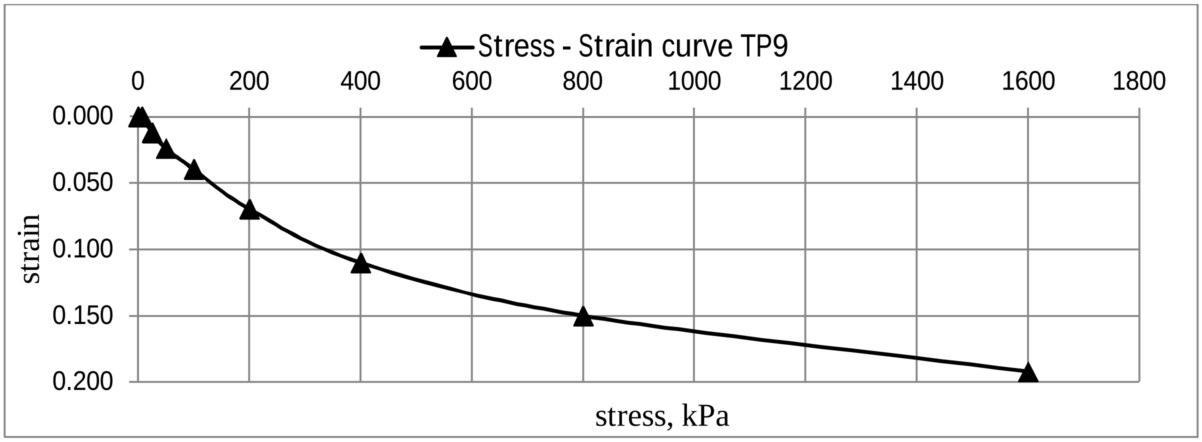


b, Tangent Modulus – Stress curve

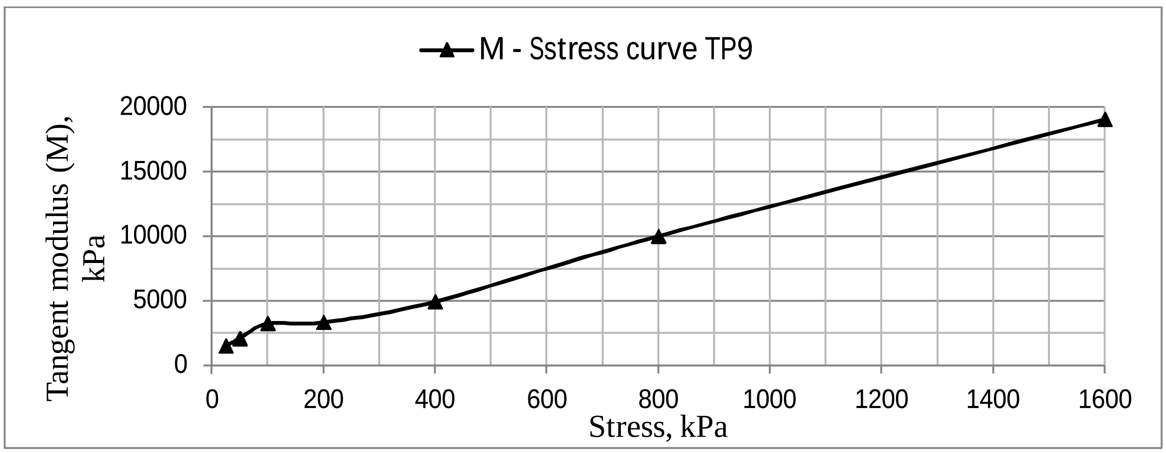


c, Void ratio – Stress curve

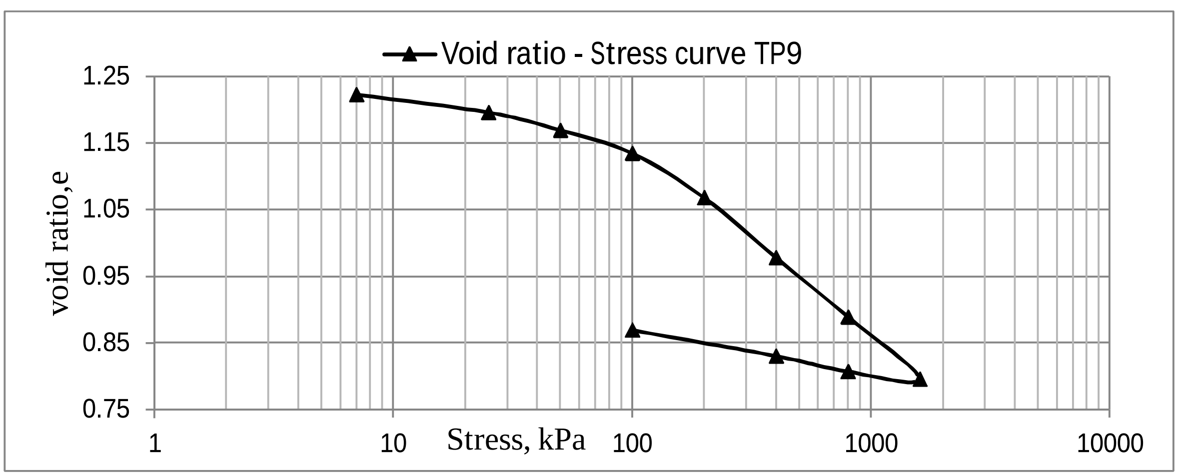
Figure A –8 Oedometer test results of TP8



a, Stress – Strain curve

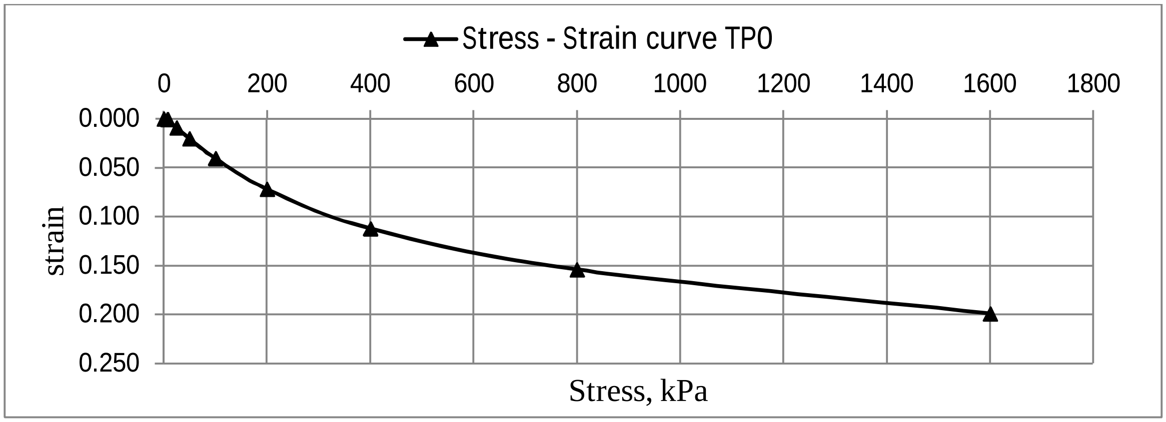


b, Tangent Modulus – Stress curve

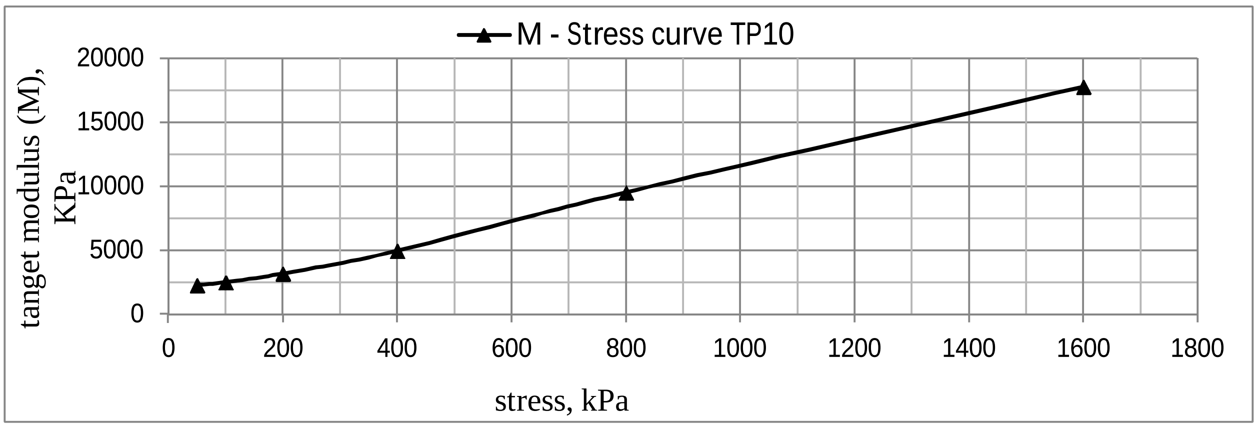


c, Void ratio – Stress curve

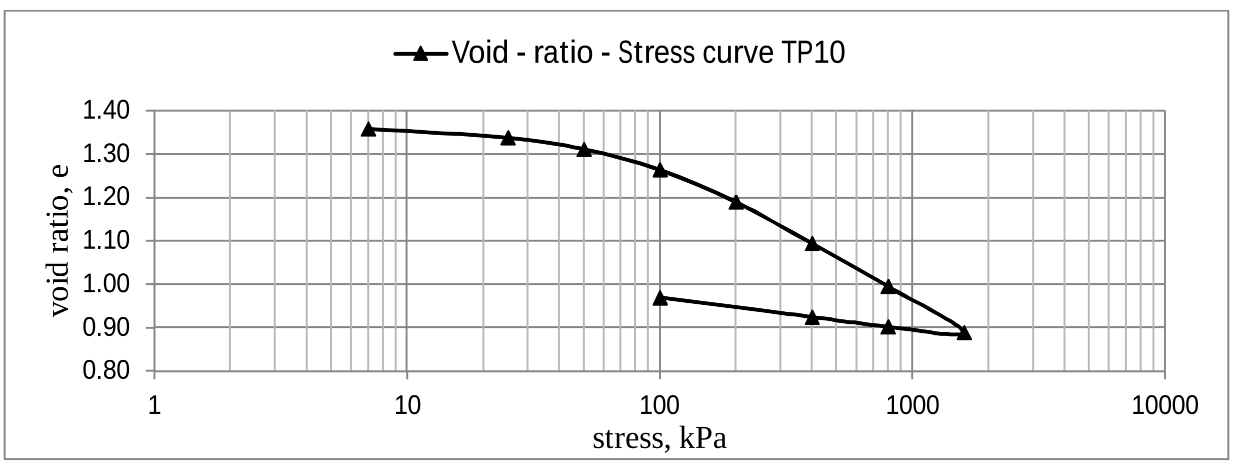
Figure A –9 Oedometer test results of TP9



a, Stress – Strain curve



b, Tangent modulus – stress curve



c, Void ratio – Stress curve

Figure A –10 Oedometer test results of TP10

APPINDEX B

Table B-1 Typical Ranges of “m” and “a” of Addis Ababa Red Clay Soil [17]
Red Clay Soil

Firmness	Tangent modulus number, m		Stress Exponent, a
	NC	OC	
Soft – Firm (medium)	5 - 10	20 - 40	0
Stiff - very stiff	10 - 20	20 - 40	0

Expansive Soil

Firmness	Tangent modulus number, m		Stress Exponent, a
	NC	OC	
Soft – Firm (medium)	1 - 5	1 - 10	0
Stiff - very stiff	1 - 5	1 - 10	0

Table B-2 Typical Conservative Modulus Number of Scandinavian Countries [10]

Soil type	Soil state	Modulus number, m	Stress exponent, a
Till	Dense to very dense	300 - 1000	1
Gravel	Dense to very dense	40 - 400	1
Sand	Very dense	250 - 400	0.5
	dense	150 - 250	0.5
	loose	100 - 150	0.5
Silt	Very dense	80 - 200	0.5
	dense	60 - 80	0.5
	loose	40 - 60	0.5
Clays			
Silty clay	Stiff - hard	20 - 60	0
	Stiff - firm	10 - 20	0

Clay silt	soft	5 - 10	0
Soft marine clays and organic clay	-	5 - 20	0
Peat	-	1 - 5	0

Table B-3 Consistencies in terms of Consistency Index [14]

Consistency	Consistency Index (%)	Characteristics of soil
Very soft	0 - 25	Fist can be pressed in to soil
Soft	25 - 50	Thumb can be pressed in to soil
Medium (firm)	50 - 75	Thumb can be pressed with pressure
stiff	75 - 100	Thumb can be pressed with great difficulty
Very Stiff (Hard)	>100	The soil can be readily indented with thumb nail
Hard	>100	The soil can be indented with difficulty by thumb nail.

Table B-4 Degree of Colloidal Activity [20]

Activity	Degree of activity
<0.75	Inactive clay
0.75-1.25	Normal clay
>1.25	Active clay