A STUDY OF SETTLEMENT USING TANGENT MODULUS APPROACH:

IN THE CASE OF BISHOFTU TOWN

By

Amanuel Adugnaw

A Thesis Submitted to

The department of Civil Engineering,

School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirement for the Degree of Master's in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies

Adama Science and Technology University

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Approval of Board of Examiners

We, the undersigned, members of the Board of Examiners of the final open defense by Amanuel Adugnaw Maru have read and evaluated his thesis entitled "A Study of Settlement Using Tangent Modulus Approach: In the Case of Bishoftu Town" and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirement of the Degree of Masters in Civil Engineering (specialization in Geotechnical Engineering).

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Declaration

I hereby declare that this MSc Thesis is my original work and has not been presented for a degree in any other university, and all sources of material used for this thesis have been duly acknowledged.

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This MSc Thesis has been submitted for examination with my approval as thesis advisor.

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LIST OF ACRONYMS AND ABBREVIATIONS

H	Soil Layer thickness
OCR	Over Consolidation Ratio
C_c	Coefficient of Compression Index
C_v	Coefficient of Consolidation
e_o	Initial Void Ratio
e	Void Ratio
k	Permeability
k’	Earth pressure coefficient
m	Modulus number
m_v	Coefficient of Volume Compressibility
M	Tangent Modulus
q	Discharge
u	Pore Water Pressure
U	Degree of Consolidation
ε	Strain
γ_w	Unite Weight of Water
σ	Total Stress
σ'	Effective Vertical Stress
σ_p'	Pre-consolidation Stress

ABSTRACT

The estimation of settlement often receives less attention than they deserve, with the result that excessive settlements cause far more problem of structural failure. Settlement analysis was therefore one of the most important types of analysis made by soil engineer for designing safe and economical structures. The main aim of this research work focuses on developing and promoting the concept of tangent modulus in the field of soil compressibility in case of Bishoftu town. The classical approach, which is widely used in our country and other parts of the world, has limitations. These limitations have contributed for the development of the tangent modulus approach. The tangent modulus approach provides better way of determining settlement for various types of soil ranging from hard to very soft soil. In the method the soil property, which is determined from laboratory soil deformation tests, are used. In the approach a compression modulus also called the constrained modulus, the tangent to a linear plot of stress versus strain curves determined from laboratory tests play a significant role. It is emphasized on this property of the soil, as it is a measure of resistance of the soil against deformation due to change in loading condition the compression modulus, measures of the resistance of a media or an isolated part of it against a forced change of equilibrium condition. From the computation of settlement results, it has been observed that the effective stress is 31.41KN/m^2 and the total stress is 131.23 KN/m^2. Also, it has been observed that the compressibility value of soil in classical approach is 8.46cm and in tangent modulus approach is 10.05cm. The Therefore, the simplicity and rationally inherent in the tangent modulus concept made it the preferred method of analysis of the compressibility of soil. It has been shown herein that the tangent modulus concept which has a good scientific explanation is applicable for Bishoftu soil to compute compressibility

***Keywords**: Tangent modulus, Settlement, Oedometer test, Stress-strain curve*

CHAPTER ONE

INTRODUCTION

1.1 Background of the study

Bishoftu town is rapidly expanding to rural area surrounding the town day to day. This has made possible construction of group housing buildings around the town. Due to these loads are induced on to the soil underneath. Besides the use of undesirable construction site is increasing. The above factors result in compressive strains causing settlement of a structure. The resulting settlement has to be limited to a minimum to allow the structure function properly and maintain the required aesthetic value. Further cost of maintenance has to be limited to a minimum.

For estimating the settlement that occurs in cohesive soils the classical approach is widely used in most parts of the world including Ethiopia. This approach assumes the compressibility of the soil remains constant. Further it was believed that the initial pore water pressure describes the consolidation characteristics of thick clay. Thus, another method of settlement evaluation which amends at least the above-mentioned drawback has to be adopted.

This study develops a better understanding of the application of tangent modulus approach for the computation of settlement for soil in the case of Bishoftu town. The material data accessed via two test pits and the oedometer test used for determination of stress-strain curve for different linear increment of loading. And finally, the result recorded compared simultaneously with classical approach.

1.2 Statement of the problem

Rapidly expanding of rural area surrounding the city will increase construction of group housing buildings. These loaded structures generally induce high stresses on the soil underneath which undergoes compressive strain resulting in the settlement of structures.

Bishoftu is one of fast developing city in Ethiopia in which there are so many rural areas have been included in urban areas in recent years due to its suitable topography and urbanization. The soil in this area was more susceptible to settlement problem due to those loads induced on to the soil underneath.

Excessive or unequal settlement may prevent the structure from functioning properly, which results in unnecessary cost of maintenance and even in extreme case become dangerously unstable and finally it may collapse. Settlement determination has to be given due considerations by engineers as it affects engineered structures from functioning properly and has to be also managed before causing serious damage. Different countries used different approaches to estimate settlement of structures. Here in our country the classical approach is used in order to estimate settlements. The other method, which is used in other countries, is the tangent modulus approach, which has many advantageous.

Thus, it is needed to compare these two approaches (i.e., classical method and tangent modulus approach) and verify which one is more effective and applicable in soil of Bishoftu town.

1.3 Objective of the Study

1.3.1 General objective

The main objective of this research work focuses on developing and promoting the concept of tangent modulus in the field of soil compressibility in case of Bishoftu town.

1.3.2 Specific objective

- To promote a recent and more practical way of determining settlement
- To compare the applicability with the existing and more popular classical approach
- To verify the applicability of the approach to the soils of Bishoftu town

1.4 Significance of the Study

After successive completion of the study of tangent modulus approach for settlement computation and careful interpretation of the results, the output of the work may:

- Develop a better understanding of the applicability of the approach in the study area.
- Verify the previous investigation done and enhance the confidence of the designers (esp. Geotechnical Engineers) to apply this preferable alternative in study area.
- Contribute as reference for future work.

1.5 Limitation of the Study

This study developed a better understanding of the application of tangent modulus approach for the computation of settlement in the case of Bishoftu town. The material data accessed via two test pits depth of 3m with two samples at each pit for trial and the oedometer test used for determination of stress-strain curve of load to ultimate pressure of study area. This area and depth are narrowed for computation of long and those including basement buildings because of economy capacity. To get for long and those including basement buildings settlement computation result there should be deep excavation have taken for soil sample that need enough economic capability. And finally, the result recorded will be compared simultaneously with classical approach.

1.6 Organization of the study

This document is organized in five chapters. The first chapter deals with the background of the study, statement of the problem, objective, significance, and limitation of the study. The second chapter briefly reviews the major literature works conducted related to the topic. Both tangential modulus approach and classical approach are presented. The third chapter deals with the methods of analysis, laboratory procedures and consolidation record taken from consolidation test. In the fourth chapter, the test result obtained from primary data source, has to be interpreted. The interpretation of the results helps in the determination of soil parameter, which govern design. In the fifth Chapter, conclusions are drawn and recommendations are put forth. In addition, recommended future research works are identified.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

The increase of stress in soil layers due to load imposed by various structures at the foundation level will always be accompanied by some strain. If the load is greater than the strength of the soil both elastic and residual deformation are absorbed. For soils the load deformation relationship is usually complex, varying widely with different soils, particularly in the plastic range of cohesive soils where time plays a major role. Yet it is such time-related soil deformation (i.e., consolidation) that accounts for most of the settlement in cohesive soils and that the foundation engineer must cope with.

Cohesionless soils (sand, gravel) will generally compress during a relatively short time. In fact, quite frequently most of anticipated settlement of a foundation resting on a cohesionless soil has taken place during the construction phase of the structure. Furthermore; the compression of such soil by induced vibration effect could be accomplished with much greater ease than of cohesive soils (N.Cornia., 1991).

Unlike cohesionless soils, a saturated cohesive soil mass (e.g., fine silts, clays) with low permeability will compress quite slowly since the expulsion of the pore water from such soil occurs at a significantly slower rate than from cohesionless ones (ARORA, 2000). Hence the deformation in such soil takes time after the construction of the structure.

Even though surface loads results in the vertical and horizontal deformations, one notes that it is the vertical compression or consolidation is the largest and it is frequently the most important component.

When cohesive soils are subjected to stress due to foundation loading the pore water pressure will immediately increase, however, because of the law permeability of the soil, there will be a time lag between the application of load and the extrusion of water and thus, settlement, this phenomenon, which is called consolidation.

2.2 Theory of consolidation

As the purpose of this paper is to present a different approach for the determination of settlement for different types of soil it is worth to discuss first about consolidation of soil.

When a soil layer is subjected to a compressive stress, such as during the construction of a structure, it will exhibit a certain amount of compression. This compression is due to rearrangement of the soil solids or extrusion of the pore air and/or water. According to Terzaghi (1943), “a decrease of water content of a saturated soil without replacement of the water by air is called processes of consolidation" (Terzaghi K.Peck, 1943).

The rate at which the air or water dissipated so that the soil particles closer towards each other due to the application of stress depends on the degree of saturation of the soil mass. If the soil is dry or partially saturated the air in the void gets easily compressed. That is the air is easily expelled out resulting in almost immediate settlement. In a saturated soil it is not easy for the soil particles to get closer, as the water in the voids is relatively incompressible. For reduction in voids to take place the water has to get dissipated. The rate of out flow of water depends on the permeability of the soil, and the length through which the pore water travels to reach when it drains quickly.

To understand the basic concept of consolidation, consider a clay layer of thickness H located below the ground water level in between two permeable sand layers as shown in Fig 2.1. If a surcharge of intensity Δσ applied over a large area the pore water pressure in the clay will increase. For a surcharge of infinite extent, the immediate increase of the pore water pressure, Δu, at all depths of the clay layer will be equal to the increase of the total stress, Δσ.

Thus, at t=o

$$\Delta\sigma = \Delta u$$

Since the total stress is the sum of the effective stress and the pore water pressure, at all depths of the clay layer the increase of the effective stress due to the surcharge at t=o will be equal to zero (Δσ=0). In other wards at t = o the entire stress is taken up by the pore water pressure and none by the soil skeleton. Here one has to note that for a load applied over a limited area, it may not be true that the increase of the pore water pressure is equal to the increase of a vertical stress at any depth at time t=0.

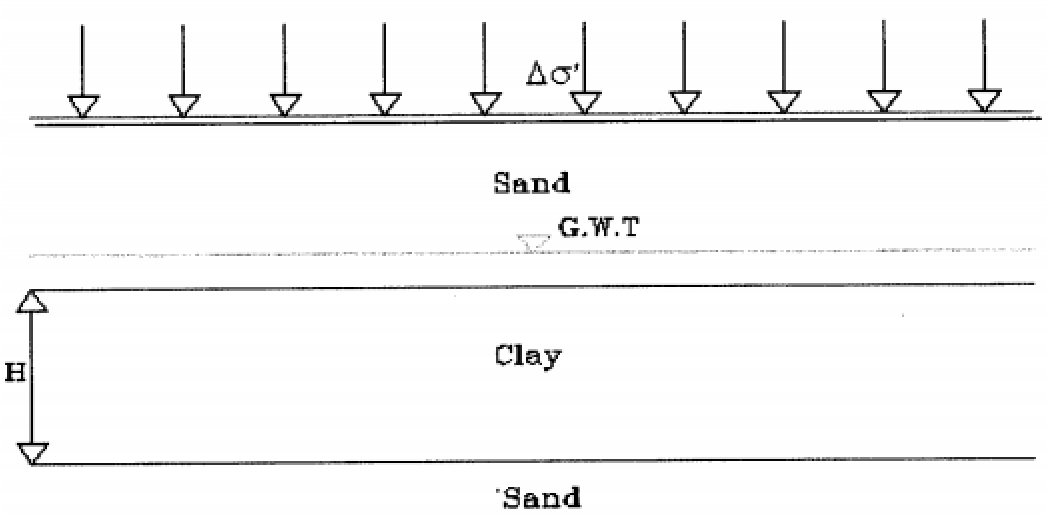


Figure 2. 1 A clay layer confined between two permeable layers

After application of the surcharge (i.e., at time t>0) the water in the void space of the clay layer will be squeezed out and will flow towards both permeable sand layers, thereby reducing the excess pore water pressure. This will increase the effective stress, by an equal amount, since:

$$\Delta\sigma' + \Delta u = \Delta\sigma \text{-- (2.1)}$$

Thus, at time t > 0

$$\Delta\sigma' > 0$$

And

$$\Delta u < \Delta\sigma$$

Theoretically, at time t = ∞ the excess pore water pressure at all depths of the clay layer will be dissipated by gradual drainage. Thus, at time t= ∞.

$$\Delta\sigma' = \Delta\sigma$$

$$\Delta u = 0$$

It is this gradual increase in effective stress which results in settlement called process of consolidation.

These equations representing the consolidation process shall be described for the concept of resistant / tangent modulus approach/ and for the classical consolidation theory.

2.3 Factors Affecting Consolidation

2.3.1 Compressibility of the soil

Granular materials exhibit compressibility behavior quite distinct from clay soils. The rate of compression decreases as the load increases. The major part of compression occurs almost instantaneously. As the permeability of granular soil is very high, the pore water dissipates within a short duration. Most of the settlements have already taken place by the time the structure is complete.

For saturated fine - grained soils the major factor in the escape of pore water from the soil is the time required for dissipation. The extrusion of pore water takes time i.e., much time is required in fine grained soil for pore water to escape. Hence, in fine - grained soil the compression is larger than in granular soils (ARORA, 2000).

2.3.2 Permeability of the soil

Water from a soil is flowing through the voids between the soil particles if the at rest hydrostatic pore pressure changes in any way. This flow of water from the soil particles will produce a volume change and this is related directly to the permeability of the soil. The effect of consolidation settlement occurs in clays where the value of permeability is low so that it prevents the initial excess pore water pressure to dissipate immediately. The permeability of a soil, therefore, plays a significant role in the time taken for the process of consolidation. (Smith, 1978)

The drainage of water in clay takes longer time when compared with that of free draining saturated sand. This is because that the permeability of clay is tens of thousands to millions of times less than that of sands and the movement of water occurs very much slowly resulting in considerable time for water to squeeze out completely (Das., 1997).

2.3.3 Role of stress history

Stresses are induced in the soil mass by past history such as surcharge load from soil later eroded away by natural causes, lowering of ground water table and desiccation by drying from the surface (U.S. Army Crops of Engineers, 1900). A soil which is subjected to a certain effective stress for the first time in its history (normally consolidated) will obviously be more compressible than when it has been subjected to a large effective stress in its earlier history (over consolidated). When a soil is stressed to a level greater than the maximum stress to which it was even subjected to in the past, some kind of break down in the soil structure occurs resulting in a much higher compressibility.

2.3.4 Effect of effective stress

Pore water starts to flow out from saturated soil due to the application of stress as long as there is a hydraulic gradient. When this flow continues, there is a reduction in pore water pressure and an increase in effective stress.

Free water and/ or gas bubbles in the voids of minerals soil cannot transfer shear stress. Hence the soil skeleton alone transfers shear stress. Thus, the effective normal stress governs the internal resistance of granular soils irrespective of the shear stress (Das., 1997).

As a soil is subjected to a stress, the soil undergoes a decrease in void ratio. If the same stress is applied again the soil undergoes a decrease in void ratio which is less than the decrease in initial void ratio. This means compressibility of soil decrease as effective stress increase provided that there is no brakeage of the particles.

2.4 Evaluation of settlement

The increase of stress in soil layers due to the load imposed by various structures at the foundation level will always be accompanied by some strain, which causes deformation of the soil layer. If the load is greater than the structural strength of the soil, both elastic and residual deformations are observed. Though elasticity is a property inherent in all material bodies, soils can be regarded as elastic bodies only under definite conditions. With a repeated loading and unloading, however, the soil will generally acquire an elastically compacted state which is characterized by that its elastic property are constant for the given loading condition. If the load on the soil is increased above that causing elastically compacted state appreciable residual deformation will appear in it (R.Whitlow, 1990).

When thinking of a new construction, it is important to realize that due to change of soil condition settlement might be as equally critical as bearing capacity. It is the prediction of these changes that present the most difficult task (Craig, 1974).

The estimation of settlement often receives less attention than they deserve (or even none at all), with the result that excessive settlements cause far more problem than do bearing capacity failures (M.Carter and M.V.Symons, 1989).

The deformation of soils under loading is basically due to rolling, slipping and sliding and to some extent by crushing of particles at contact point and elastic deformation.

Some of the different causes of settlement are: compaction, consolidation, elastic distortions, moisture migration, loss of lateral support and ground water lowering. It is one or the combination of these causes which brings about settlement (A.C.Statopoulos and P.C Kotzius, 1978).

2.4.1 Component of settlement

The response of stress applied to a soil brings about a total settlement, which is the sum of immediate or elastic settlement, primary consolidation settlement and secondary compression settlement.

Immediate settlement is the change in shape or distortion of the soil caused by the applied stress. This type of settlement is usually occurring in all coarse grained and dry or partially saturated fine grain soils. Immediate settlement is due to the elastic deformation of soil without any change in the soil water content.

Primary consolidation settlement occurs in cohesive or compressible soil during dissipation of excess pore fluid pressure, and it is controlled by the gradual expulsion of fluid from voids in soil leading to the associated compression of the soil skeleton. Excess pore pressure is pressure that exceeds the hydrostatic fluid pressure. The hydrostatic fluid pressure is the product of unit weight of water and the difference in elevation between the given point and elevation of free water.

Primary consolidation settlement is normally insignificant in cohesionless soil where settlement occurs rapidly because these soils have relatively large permeability. Hence this type of settlement takes substantial time in cohesive soils because they have relatively low permeability. Time of consolidation increase with thickness of the soil layer and inversely related to the coefficient of permeability of the soil (U.S. Army Crops of Engineers, 1900).

Secondary compression settlement is a form of soil creep which is largely controlled by the rate at which the skeleton of compressible soils, particularly clays, silts, and peat, can yield and compress. Secondary compression is often conveniently identified to follow primary consolidation when excess pore fluid pressure can no longer be measured; however, both processes may occur simultaneously (U.S. Army Crops of Engineers, 1900).

2.4.2 Consolidation settlement

2.4.2.1 Description

Vertical pressure σ_{st} from foundation loads transmitted to a saturated compressible soil mass is initially carried by fluid or water in the pores because of water is relatively incompressible compared with that of the soil structure. The pore water pressure, u, induced in the soil by the foundation loads is initially equal to the vertical pressure σ_{st} and it is defined as excess pore water pressure because this pressure exceeds that caused by the weight of water in the pores. Primary consolidation begins when water starts to drain from the pores. The excess pressure and its gradient decrease with time as water drain from the soil causing the load to be gradually carried by the soil skeleton. This load transfer is accompanied by decrease in volume of the soil mass equal to the volume of water drained from the soil. Primary consolidation is complete when all excess pore water pressure has dissipated so that u =0. Primary consolidation settlement is usually determined from the results of one - dimensional (1-D) consolidometer test (R.Whitlow, 1990).

Normally consolidated soil

A normally consolidated soil is a soil which is subjected to an in situ effective vertical overburden stress σ'_{ov} equal to or greater than the pre-consolidated stress σ_{pv}. Virgin consolidation settlement for applied stress exceeding σ_{pv} can be significant in soft and compressible soil with a skeleton of low elastic modulus such as plastic CH and CL clays, silts, and organic MH and ML soils (U.S. Army Crops of Engineers, 1900).

Over consolidated soils

An over consolidated soil is a soil which is subjected to an in situ effective overburden stress σ'_{ov} less than the pre-consolidated stress σ_{pv}. Consolidation settlement will be limited to recompression from stress applied to the soil up to σ_{pv} (U.S. Army Crops of Engineers, 1900).

2.4.2.2 One dimensional consolidation

The theory of consolidation is developed based on one dimensional consolidation. One dimensional consolidation occurs under fills and embankment that are wide when compared with the thickness of the underlying compressible ground. When a large area is loaded uniformly, every element at every depth is confined by adjacent element that is subjected to the same state of stress. There is no horizontal deformation of the soil except near the boundaries of the loaded area. If the layer is overlain and confined by a desiccated crust or granular layer thick enough to minimized heave of the clay layer around the foundation, the condition also approximated as a state of one-dimensional consolidation. If the foundation is located deep enough, so that the surrounding overburden prevents lateral deformation of the clay located directly below the foundation, a similar boundary condition exists in one-dimensional condition is when the compressible ground is overlain by a stiff layer such as dense granular soil or bed rock. The stretching in the horizontal direction is restrained by horizontal shearing resistance that develops at the top and bottom (Terzaghi K.Peck, 1943).

If the thickness of a compressible layer is large compared to the loaded area, the condition of one-dimensional consolidation does not exist. This is due to the fact that some settlement is caused by lateral displacement of the soil. But even if there is lateral displacement, it is very much smaller when compared with that of the vertical displacement.

Thus, to determine settlement due to compression of clay stratum under confined conditions, that approximate one dimensional compression can be derived from compression of laterally confined specimen. This is carried out in consolidometer.

Hence, for the vast majority of practical settlement problems, it is sufficient to consider that both seepage and strain takes place in one direction only, the usually being vertical which is used throughout this research.

2.5 The tangent modulus approaches

The tangent modulus approach of settlement determination is suggested initially by Janbu (Janbu, 1963) and Stanmatopoulos and Kotzius (A.C.Statopoulos and P.C Kotzius, 1978). It is based on the stress strain relation of the soil obtained from laboratory test results. The tangent to the arithmetic plot of stress-strain curve is called compression modulus or the constrained modulus. This provides unified procedure of practical settlement calculation for different types of soils. This unified procedure of settlement calculation making use of the elementary concepts and principles of classical mechanics amended the drawbacks of the classical method. Further the concept is suitable for settlement determination regardless of soil type and boundary conditions.

2.5.1 The concept of Resistance

The classical resistance concept is a unifying concept being widely applied in all fields of engineering, where action reaction systems require analysis. All media possess resistance against a forced change of existing equilibrium conditions. The resistance of a medium or of an isolated part of it can therefore be determined by measuring the incremental response to a given incremental action.

$$\text{Resistance} = \frac{\text{Increamental causes [given]}}{\text{Increamental response [measured]}} \text{------------------------------------} (2.1)$$

The concept relates the action on a body of medium to the response of the medium to the action. It is rationally defined in familiar engineering and mathematical language. The following examples show the usage of the resistance concept in other fields of engineering.

$$\text{Electric resistant (R)} = \frac{Potential\ change}{Current\ change} \text{------------------------------------} (2.2)$$

$$\text{Elastic resistant (E)} = \frac{Stress\ change}{Strain\ change} \text{------------------------------------} (2.3)$$

$$\text{Dynamic resist. mass (m)} = \frac{Force\ change}{Accelleration\ change} \text{------------------------------------} (2.4)$$

$$\text{Hydraulic resistant } (K^{-1}) = \frac{\textit{Gradient change}}{\textit{Velocity change}} \text{-----} \quad (2.5)$$

$$\text{Heat resistant } (c) = \frac{\textit{Heat change}}{\textit{Temperature change}} \text{-----} \quad (2.6)$$

According to the outcome of research work at the Technical University of Norway, the common term 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 (M.Carter and M.V.Symons, 1989).

Modulus of a soil is defined as change of stress to change of strain. It can be described as shown in the figure below.

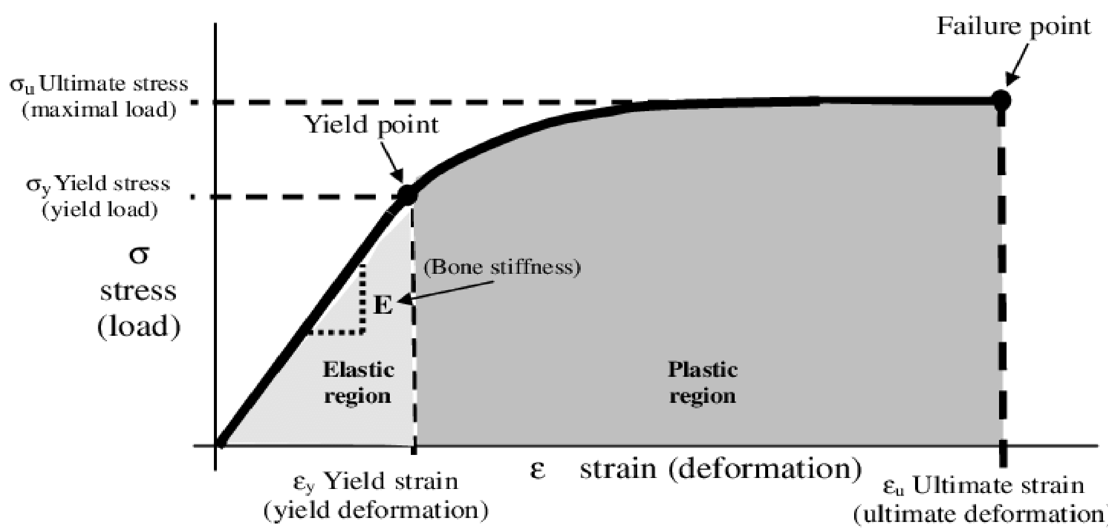


Figure 2. 2 Stress-strain curve

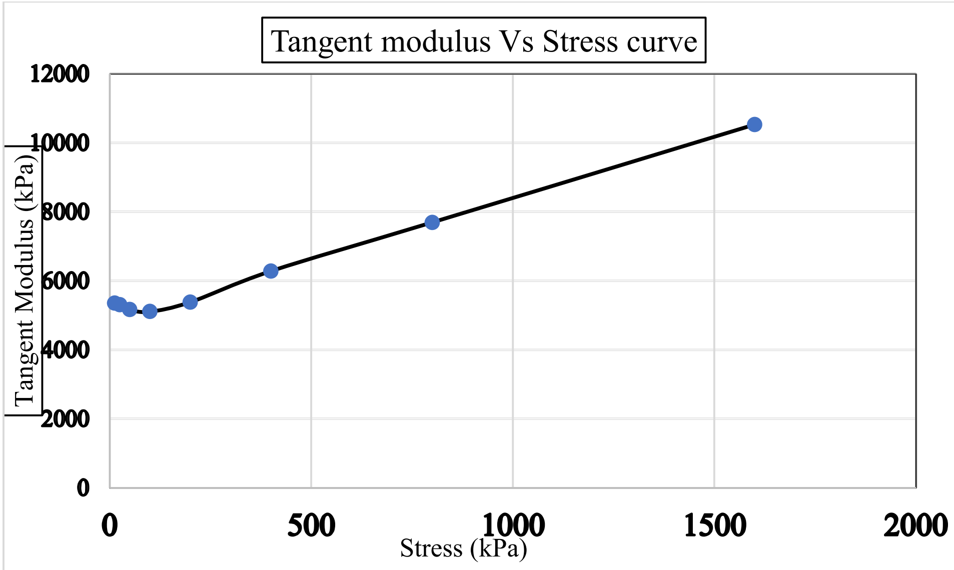


Figure 2. 3 Tangent modulus–stress curve

$$M = \frac{d\sigma}{d\varepsilon} \text{--- (2.6)}$$

The deformation tests on soils are carried out as three-dimensional, thus it is important that the state of stress has to be described more specifically in order to have a meaningful definition of deformation modulus.

The plot of stress versus strain curves and modulus versus stress curves have separate branches for the complete loading cycle (loading - unloading - reloading). Thus, the corresponding modulus has to be distinguished as M for compression modulus, M_s for swelling modules and M_{rc} for recompression modulus. From its wide application, the meaning of the tangent modulus M can be said to be a volumetric resistance against deformation.

$$\text{Resistance} = \frac{\text{Stress change}}{\text{Strain change}} \text{--- (2.7)}$$

2.5.2 Characteristics of soil resistance

The soil resistance against deformation is generally volumetric in nature because of three-dimensional stress conditions during testing. The values of M will therefore depend on both the state of relative stress as well as on the absolute magnitude of stress in the primary direction where the stress-strain measurements are made. Thus, careful effort should be made in order to simulate the stress condition in the field as much as possible (R.Whitlow, 1990). Though the actual Stress State on ground is complex, it can be explained in two different simple cases.

- ☞ Isotopic case: there is equal stress application in all direction. Applicable to triaxial test before shearing
- ☞ One-dimensional case: application of stress is only in the vertical direction. The horizontal strains are zero. Common cases are oedometer test, and in the ground below wide foundation embankments and excavation.

The deformation tests, usually carried out in the laboratories are either oedometer test or triaxial test. Thus, it is evident that these two conditions are well compatible with the cases that are simplified to simulate the field condition.

By using either of these laboratory apparatuses the soil behavior is determined by subjecting a 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)

From large number of oedometer and triaxial tests on different types of soils in Norway by N. Janbu (Janbu, 1963) and A.C Stanmatopoulos and P.C Kotzias (A.C.Statopoulos and P.C Kotzius, 1978) in Greece, they were able to deduce that the tangent modulus derived from the vertical stress strain curve is a suitable direct measure of soil compressibility. The plots of stress strain and tangent modulus stress on a linear scale have a U- shape. This shape holds for soils of variable type and origin: clay, silt, sand, organic, sedimentary, residual and so on. It appears therefore to constitute a general mode of soil behavior to which reference can be made when evaluating the test results (A.C.Statopoulos and P.C Kotzius, 1978).

This plot of tangent modulus on linear scale has the following advantages:

- ☞ It reveals the minimum value of tangent modulus and the stress where it occurs (the pre-consolidation pressure)
- ☞ It shows the range of pressure where the soil is strain softening (decreasing value of M) with grater sharpness than it would show on a stress-strain plot.
- ☞ It illustrates one of the causes of the variation of C_v with σ'.
- ☞ It is applicable to a wide range of soil type.

The tangent module M representing the behavior of soil is dependent on the nature of the soil. Its variation with stress is also remarkable. For a low stress level on the loading branch the tangent modules M against deformation is large. While the stress increases this high resistance eventually decreases appreciably owing to partial collapse of the grain skeleton. This breaks down of the resistance occurs around the pre-consolidation stress level σ_c'. This leads to a very practical and reasonable way of determining the pre-consolidation pressure from a linear scale plot.

What is said so far in general shows that the tangent modulus varies with stress, and the variation can be expressed with sufficient accuracy by means of a formula containing two-dimensional parameters. The magnitude of these parameters has been determined for various soils of highly different porosity.

From the study of a large number of stress-strain curve for different types of soils ranging in porosity from (nearly zero) rock to about (90%) it is believed that for engineering purposes one can adequately cover the variation in compressibility by means of single definition, namely tangent modulus.

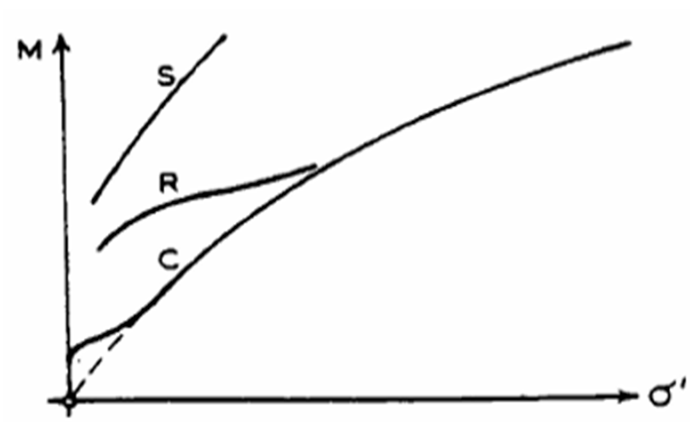


Figure 2. 4 Stress strain curves for virgin compression (C) swelling (S) and recompression (R)

Figure 2.4 is a typical stress-strain curve for an oedometer test on a cylindrical soil specimen with different value of M (C) for virgin compression (S), for swelling and (R) for recompression.

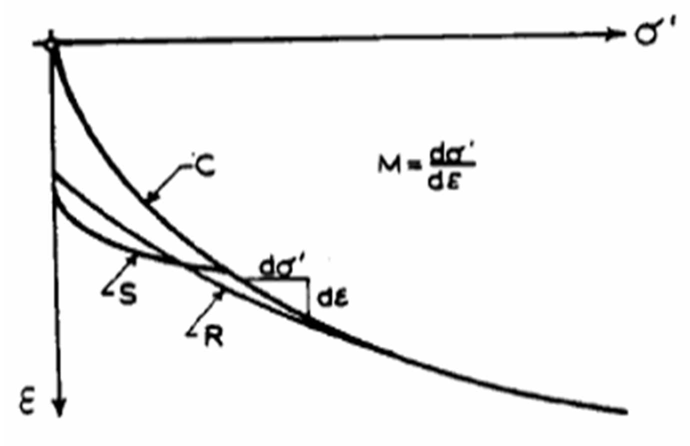
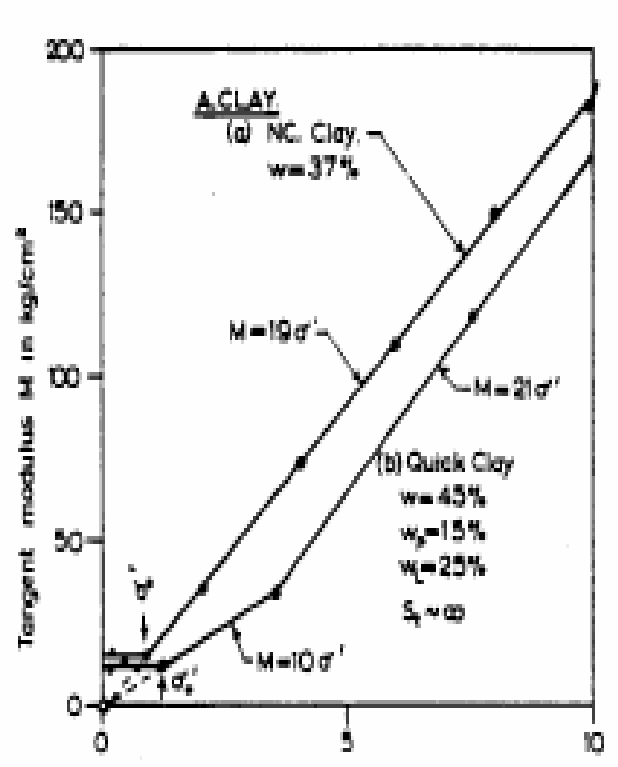


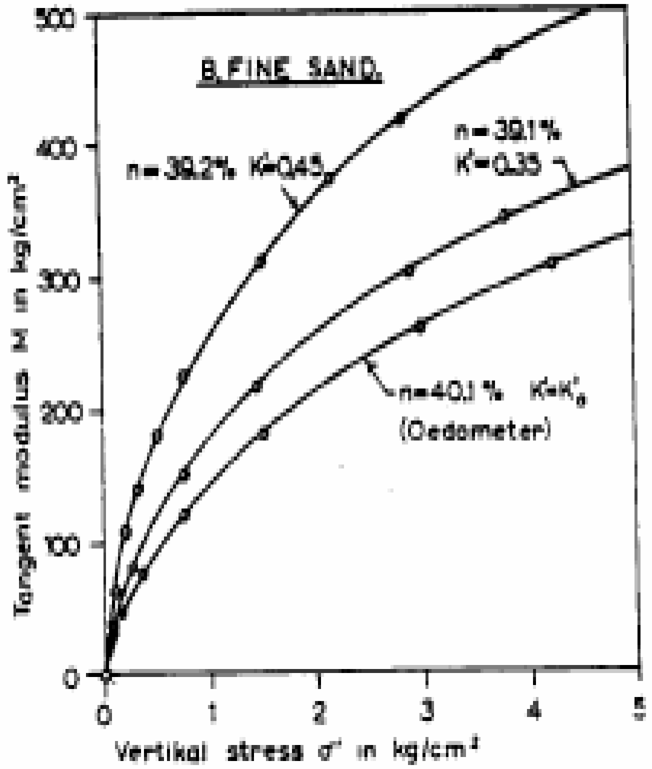
Figure 2. 5 Stress -tangent modulus curves for virgin compression (C) swelling (S) and recompression(R)

Figure 2.5 is a typical Stress -tangent modulus curve for an oedometer test on a cylindrical soil specimen with different value of M (C) for virgin compression (S), for swelling and (R) for recompression.

In general, it shows the dependency of the tangent modulus on effective stress and stress history. This dependency is shown in detail in the forgoing discussion.



a)



b)

Figure 2. 6 Examples of tangent modulus variations with stress

Figure 2.6 show some typical results from oedometer tests on undisturbed Norwegian clay as compiled and explained by professor Janbu (Janbu, 1963).

Figure 2.6-(a) shows how the tangent modulus for normally consolidated clay varies with effective vertical stress. It is seen that over this normal consolidation range M is a simple linear function of effective stress in excess of the pre-consolidation stress. Hence

$$M = m\sigma' \qquad (2.8)$$

For stress below the pre-consolidation stress σ_c', the value of M is almost independent of the stress change $\Delta\sigma$. For stress above σ_c' as mentioned above the modulus increases.

In Figure 2.6-(a) curve 'b' shows a typical variation with stress of the tangent modulus for undisturbed Norwegian quick clay. For vertical stress slightly above the pre-consolidation load the tangent modulus increases moderately with effective stress to begin with (collapse of grain structure). For effective stresses in excess of say σo the tangent modulus increase can be even steeper than for with low sensitivity but with the same water content. Thus, it is believed that for such stress, the originally quick clay is no longer quick (Teferra, 1981).

Figure 2.6-(b) shows the tangent modulus values obtained from an oedometer test. The indicated variation of M with σ' is typical for a large number of such tests on sand of different porosity and grain size distribution characteristics. However, the looser the sand the more will the M-σ' diagram approaches a straight-line (Teferra, 1981).

For over consolidated clays and rock the tangent modulus M remains constant regardless of the applied stress. The same applies for the undrained (Initial) compression modulus for clay (corresponding to initial settlement condition).

For normally consolidated quick clay, NCQ, the modules drop abruptly where σ' equals to σ_o'. This drop probably corresponds to a collapse of the metastable clay skeleton in the quick clays. When σ'>σ_o' the modules again increase, but now somewhat faster than σ'. For over consolidated quick clays, OCQ, the modules remain practically constant up to the pre consolidation load σ_c'.

If the difference σ_c'-σ_o' is low (say 20-100KN/m2) then one may get a sharp and substantial drop of M near σ_c'.

If the deference σ_c'-σ_o' is large, the transition from the over consolidated to the normally consolidated part may be smooth, without drop.

For sand silt the modules varies with the effective stress raised to a power of 0.3 to 0.7 with an average near 0.5 for practical purpose.

From the intensive research, which is based on a vast number of stress-strain curves for a variety of soils, it is seen that the tangent modulus changes with σ' in a different fashion for this different type of soils. Thus, M can be descried as a function of the effective stress.

$$M = f(\sigma') \qquad (2.9)$$

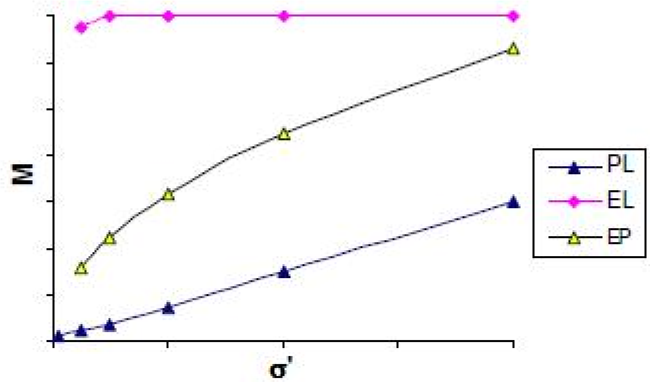


Figure 2. 7 Principal types of M=f(σ')

The correlation of the tangent modulus is shown in equation 2.10. It makes possible handling of each soil type separately and enable determination of strain ε (total) by integration.

For the purpose of being able to formulate the function f (σ') systematically it is of interest to note that all the variations encountered so far can in general be described within the area defined by the two straight lines EL and PL in Figure 2.7. Janbu suggested a general formula to adequately cover all types of soil (Janbu N. , 1967).

$$M = m\sigma_a\left(\frac{\sigma'}{\sigma_a}\right)^{1-a} \quad \text{.......... (2.10)}$$

Where; M= Tangent modulus

m= modulus number

a= stress exponent

σ′=effective stress

$\sigma_{a=}$ reference pressure, a constant which is equal to 100kPa

The reference pressure σa=100Kpa is introduced solely for the purpose of obtaining a dimensionally compatible expression (atmospheric pressure).

The governing parameter 'm' and 'a' have been obtained through experimental investigations over a period of 10 years. The general trend of variations of these variables is demonstrated in Figure 2.8 with three different sets of M~σ'. The soil types considered range from sound rock with low porosity to the softest clay with porosity near 90% Systematic analysis by oedometer and triaxial tests of the compression modulus of several types of natural deposits have shown that in general the compression modulus depends primarily on the porosity of the soil and on the intensity of the effective stress. Therefore, based on the porosity it is believed sufficient to classify compression behavior of soils in three main categories. From the tests conducted porosity of soil is related with exponent 'a' as shown in Figure 2.8.

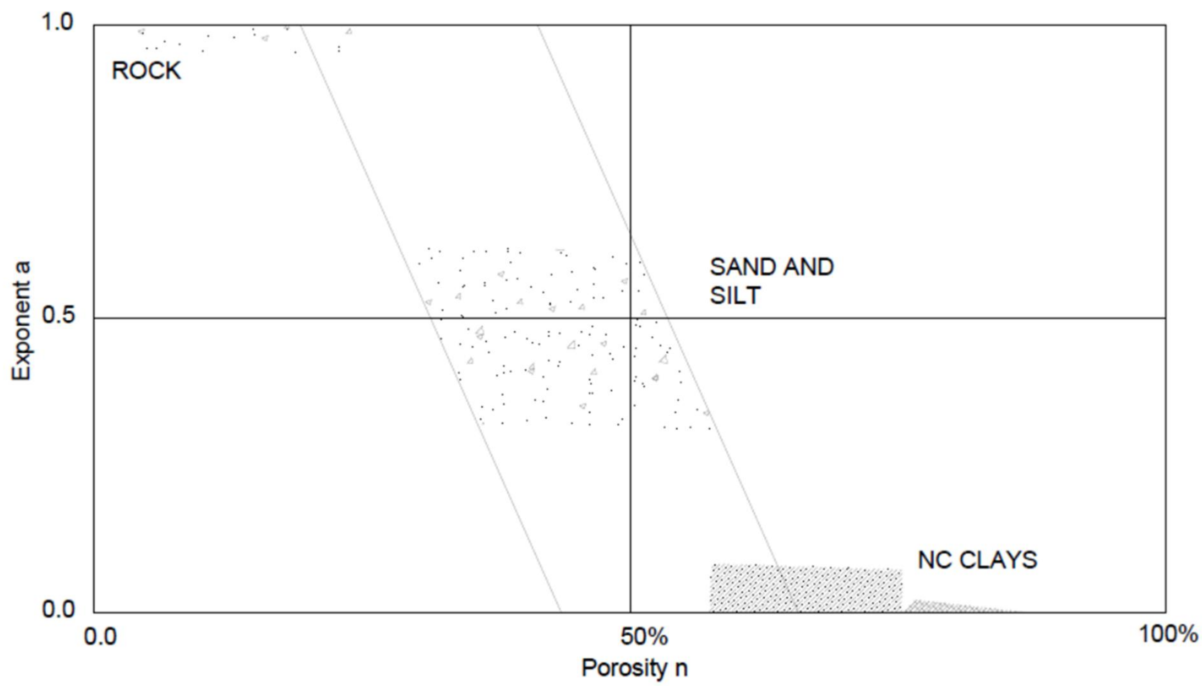


Figure 2. 8 Variation of exponent a with porosity

From the experiments it was found that for rock and hard dense moraines the stress exponent (a=1) corresponding to a modules M= constant, i.e. independent of stress variation. It is also found that for such soils the modules number 'm' is generally high, in the order of magnitude of from 10^3 to 10^7.

For high porosity soils (normally consolidated clays) the stress exponent a ~0, corresponding to a linear variation of modules versus stress (M= mσ'). The modules number is generally low, say from 5 to 50.

For soil of intermediate porosity (sand – coarse silt) the stress exponent 'a' varies around 0.5 as an average, corresponding to tangent modules which is about proportional to square root of the stress. The corresponding modules number m is generally located between the low values assigned on the one hand up to those of rock- moraines on the other, say m= 50 to 1000. The graphs are meant only for illustration of the general tendencies for variation of ‘a’ and ‘m’ with porosity but not sufficiently detailed for practical application.

As shown in Figure 2.8 the various soil types are assembled into the following three main categories with respect to modules variation.

Type EL= “elastic “(a=1)

Type EP = elastic – plastic (a=0.5)

Type PL = plastic (a=0)

The above grouping is based on specific values of exponent ‘a’.

Type EL defined by a=1 corresponding to constant M.

$$M = m\sigma_a \quad (2.11)$$

Typical examples of soil belonging to type EL are rock hard moraines over consolidated clays undrained modules of clay and for purely practical reasons peat.

Type EP is defined by a=0.5 corresponding to

$$M = m\sigma_a'\left(\frac{\sigma'}{\sigma_a}\right)^{1/2} \quad (2.12)$$

Sand (and partly also silt) may be considered typical example of soil belonging to type EP.

Type PL is defined by a=0 corresponding to

$$M = m\sigma' \quad (2.13)$$

The normally consolidated clay is the most typical example for type PL.

A more detailed theoretical analysis of the individual components of the deformation modulus has in general disclosed that the magnitude of the modulus is primarily governed by the shear stress. This finding should therefore mean that the deformation modules is mainly a resistance against shear deformation.

2.5.3 Tangent Modulus as applied to settlement Analysis

It has been possible to establish a common definition for the compression modules independent of soil type. Therefore, the various steps of the settlement analysis can be derived from the same basic principle. The entire calculation procedure will in principle be the same for all types of soil irrespective of the modulus number '*m*' and the exponent '*a*'.

The definition of the compression modules utilizes familiar classical concepts, such as incremental stress and strain. Obviously, settlement is produced due to change of the state of stress. It is important to determine the vertical stress profile. This profile consists of the effective overburden σ_o' and stress distribution of net stress with depth. Consider an infinitesimal element having thickness *dz* at an arbitrary depth z below a foundation level. It undergoes deformation $d\delta$ due to an additional stress $d\sigma'$ and the effective over burden prior to load application σ_{ov}' as the strain is generally dependent on both these stresses. The infinitesimal verified compression $d\delta$ of the layer *dz* is by definitions of e seen to be

$$d\delta = \varepsilon dz \quad \text{……………………………………………… (2.14)}$$

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

$$d = \int_0 \varepsilon dz \quad \text{……………………………………………… (2.15)}$$

According to this equation the settlement is equal to the area of the ε−diagram. This elementary observation is of great practical importance for several reasons. First, it may reduce to a minimum the number of ε-values that need to be calculated. Secondly, by plotting ε-z, the integration is reduced to a simple area determination and finally the ε−distribution with depth helps calculating the time rate of settlement of soil.

Thus, to determine settlement of any soil deposit, strain distribution with respect to depth has to be obtained. The principle of calculation of vertical strain is based on the definition of deformation modulus.

$$d\varepsilon = \frac{d\sigma'}{M} \quad \text{……………………………………………… (2.16)}$$

Therefore, as the effective stress increases from the overburden pressure σ_ov' until its final value σ_{ov}'+Δσ' the resulting strain is obtained by integrating Equation 2.16.

$$\varepsilon = \int_{\sigma o}^{\sigma o' + \Delta\sigma} \frac{d\sigma'}{dM} \quad \text{.. (2.17)}$$

Expressing M as a function of σ' is important to obtain explicit strain formula.

The generalized experimental value of the compression modulus M is given by $m\sigma_a[\sigma'/\sigma_a]^{1-a}$

Janbu (1967) suggested a general formula to adequately cover all types of soil.

$$\varepsilon = \frac{1}{ma[\left(\frac{\sigma_{ov}' + \Delta\sigma}{p_a}\right)^a - \left(\frac{\sigma_{ov}}{p_a}\right)^a]} \quad \text{.................................... (2.18)}$$

which is directly applicable for the entire range of *a* from 0 to 1, except a=0, which represents a special boundary case.

For soil types corresponding to type EL where a=1, $\varepsilon = \Delta\sigma/M$.

For soil types corresponding to type EP where a=0.5, $\varepsilon = \frac{2}{m}\{\sqrt{\frac{\sigma_o' + \Delta\sigma}{\sigma_a}} - \sqrt{\frac{\sigma_o'}{\sigma_a}}\}$

For soil types corresponding to type PL where a=0, $\varepsilon = \frac{1}{m} \ln \left(\frac{\sigma_o' + \Delta\sigma}{\sigma_a}\right)$

The stress strain curve for normally consolidated clays becomes a straight line on a semi-logarithmic plot. These empirical findings appear now to have found its clear mechanical explanation. According to Janbu (1967) the implication is that "The tangent modulus is a linear function of effective stress above the present effective overburden".

This means there must be a close relationship between modulus number and compression index c_c. That is:

$$m = \frac{1+e_o}{c_c} \ln 10 \quad \text{...................................... (2.19)}$$

Whereas; m= modulus number

c_c= compression index

e_o=initial void of soil

2.5.4 Time rate of settlement

The determination of the total settlement that the foundation of a structure undergoes is one part of the solution to the problem of soil compression under structural load. The second and equally important is the time rate of consolidation.

For coarse grained soils such as sand and gravel the time dependency is usually of little practical interest because the pores are sufficiently large to allow drainage almost simultaneously with stress change. In this case what is important is the total settlement under the prevailing stress change.

For rocks and heavily over consolidated clays, the deformation behavior resembles somewhat elastic material. Time rate of settlement is rarely considered in practice.

For non-saturated soils, and for organic soils, the time rate of settlement may very well be of considerable importance, but it is not included in this study.

The time dependency of settlement is of considerable importance for normally consolidated fully saturated clays and this is described in this paper.

When defining the degree of consolidation in the classical theory, it was important to assume the change of strain is proportional to the change of stress, which is true only for 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 structure after completion (R.Whitlow, 1990).

Thus, the equation has to be modified to include the actual soil properties and at the same time has to give values in good consent with already completed structures.

However, decades of 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 (R.Whitlow, 1990).

Thus, the plot of degree of consolidation versus time factor T_v 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 a constant M in which is the case for over consolidated clays (M.Carter and M.V.Symons, 1989).

The stress distribution in which the classical theory of consolidation is based are needed only as an intermediate step-in order to calculate the strain.

Hence, the distribution of strain with depth has no resemblance what so ever with the stress distribution. Consequently, the shape of the stress distribution diagram is of no direct value for the time rate evaluation.

It is the area of the strain diagram that per definition gives the settlement; it is quite obvious that is the e- 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 layer.

$$\varepsilon = f(t, z) \qquad (2.20)$$

The first step in obtaining a practical solution for Equation 2.20 must be to derive a differential equation on the basis of strain, instead of pore pressure Janbu quite independent have obtained such equation, and probably at about the same time (Janbu N. , 1967). Short out line shall be presented below.

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

$$\frac{\partial \varepsilon}{\partial t} = \frac{\partial v}{\partial t} \qquad (2.21)$$

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

$$v = \frac{c_v \partial \varepsilon}{\partial z - v_o} \qquad (2.22)$$

Where coefficient of consolidation c_v is given by the equation

$$c_v = \frac{m}{\gamma} k \qquad (2.23)$$

Unlike the classical consolidation theory, no simplifying assumptions has been necessary neither regarding the modulus M nor the permeability k.

v_o represents a nominal velocity expressed as follows.

$$v_o = k\frac{(i_o+1)}{\gamma_w}dq/dz \text{.....................................(2.24)}$$

From Equation 2.24 it is seen that since v=0 for time t = ∞ the nominal velocity v_o is also given by the final strain distribution corresponding to a nominal stationary condition at the end of the primary consolidation. Since

$$v_o = c_v\, d\varepsilon_l/dz \text{ .... (2.25)}$$

where ε_1 is strain for t = ∞.,

Inserting equation 2.24 into equation 2.25 the differential equation governing the consolidation process itself is obtained.

$$\partial e/\partial v = \partial/\partial z(c_v\partial\varepsilon/\partial z) - v_o/\partial z \text{ .... (2.26)}$$

The degree of consolidation in terms of strain reads as

$$U = \frac{\int \varepsilon d\varepsilon}{\int \varepsilon_p d\varepsilon} \text{ .... (2.27)}$$

where ε is strain at a specified time t and ε_p is strain at the end of primary consolidation.

The main defect of the classical theory of consolidation is due to the fact that the meaning of stress has easily been mistaken for strain and vice versa because of the assumption that m_v is constant. But the fact is that it varies with depth.

2.6 Classical approach of settlement analysis

To help one understand clearly the advantage of the use of actual soil parameters (in-built properties of soils) it is worth to discuss the classical theory of consolidation.

Karl Terzaghi develops the theory in 1943 introducing a method for determining consolidation of normally consolidated clays under a stress.

The theory of consolidation is developed based on one-dimensional consolidation. One-dimensional consolidation occurs under fills and embankment that are wide when compared with the thickness of the underlying compressible ground. When a large area is loaded uniformly, every element at every depth is confined by adjacent elements that are subjected to the same state of stress. There is no horizontal deformation of the soil except near the boundaries of the loaded area. If the layer is overlain and confined by a desiccated crust or granular layer thick enough to minimize heave of the clay layer around the foundation. the conditions also approximated as a state of one-dimensional consolidation. If the foundation is located deep enough, so that the surrounding overburden prevents lateral deformation of the clay located directly below the foundation, a similar boundary condition exists. The one-dimensional compression condition is also favored when the compressible ground is overlain by a stiff layer such as dense granular soil or bedrock. The stretching in the horizontal direction is restrained by horizontal shearing resistance that develops at the top and bottom (ARORA, 2000).

If the thickness of a compressible layer is large as compared to the loaded area, the condition of one-dimensional consolidation does not exist. This is due to the fact that some settlement is caused by lateral displacement of the soil from the loaded area. But even if there is lateral displacement, it is very much smaller when compared with that of the vertical displacement. Thus, to determine settlement due to compression of clay stratum under confined or conditions that approximate one-dimensional compression can be derived from compression of laterally confined specimen. This is usually carried out in consolidometer. To come up with such theory he made an analogy between clay stratum subjected to loading and spring piston model. The model consisted of a spring model to simulate the soil skeleton- the network of soil grain, and the water in the vessel represents the water filling the voids in the soil. The perforations in the piston are analogous to the voids that impair permeability to the soils. In the model the area of the piston on which the load is to be placed is almost equal to the area of the vessel, thus the compression is one-dimensional.

From this model he deduced some points about the process of consolidation. Prior to the application of the total stress, the pore water pressure is the same as the hydrostatic pressure. Excess hydrostatic pressure is set upon loading and these generate the transient flow condition. Dissipation occurs first at location close to the drainage face and progress gradually to locations far from the drainage face. Consolidation or the volume decrease results with stress transfer from water to soil grains. The share of stress carried by the spring gradually increases with time. With this as a starting point the strain of a saturated clay layer subjected to a stress increase is analyzed.

2.6.1 Settlement computation

From the consolidometer, the compressibility of the soil for one-dimensional consolidation condition is determined. Settlement due to one-dimensional compression results only from decrease in the volume of voids. The data obtained from the test are used to determine the relationship between the effective stresses and void ratio or strains.

The data are computed so as to plot the time-deformation curve, which helps for the determination of the end of primary consolidation. The other is a plot of the void ratio versus the logarithmic of the stress. The graph of the time versus the effective stress is divided in to three stages.

- Initial compression- which is mainly due to pre-loading
- Primary consolidation during which the expulsion of pore water, excess pore water pressure is gradually transformed into effective stress
- Secondary compression-after complete dissipation of excess pore water pressure- some deformation of the sample is caused by plastic readjustment.

In the void ratio-pressure plot is drawn on a semi log graph paper. In this curve the complete loading –unloading can be described as shown in the figure below. It is somewhat curved with a flat shape, followed by a linear relationship with a steeper slope.

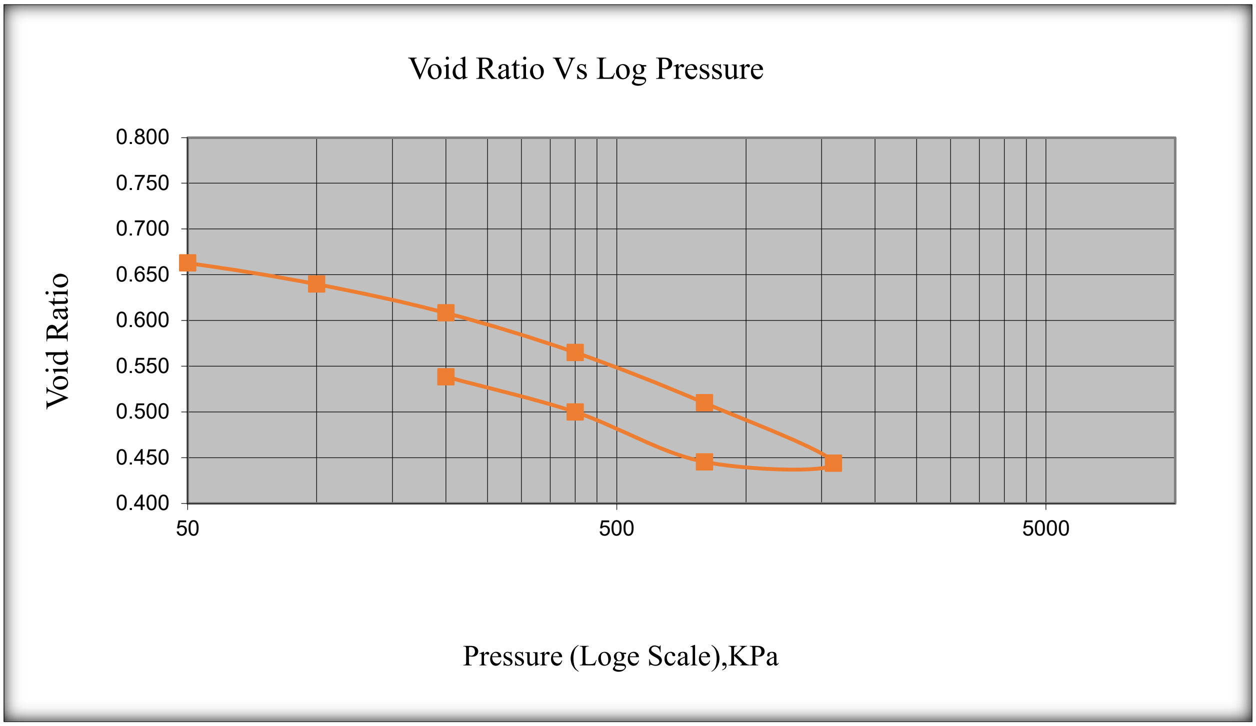


Figure 2. 9 Loading –unloading curve for void ratio – stresses

A soil in the field at some depth has been subjected to a certain maximum effective overburden pressure in its geologic history. During soil sampling the existing overburden pressure is released, resulting in expansion. When this sample is subjected to a consolidation test, a small amount of compression (change in void ratio) will occur if the total pressure applied is less than the maximum past effective overburden pressure. If the total pressure applied on the sample is greater than the maximum past effective overburden pressure, the change in void ratio is much larger and the e-log s' is practically linear.

It is this linear portion of the plot, which is used for determining the settlement of a normally consolidated clay layer.

The change in the soil layer in the consolidometer can be generalized based on the change in the thickness (ΔH) or the change in the void ratio. Thus, the volumetric strain is related as:

$$\frac{\Delta V}{V_o} = \frac{\Delta H}{H_o} = \frac{\Delta e}{1+e_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.28)$$

The change in thickness of a layer initially Ho thick is:

$$\Delta H = \frac{\Delta e}{1+e_o} H_o \text{ .... } (2.29)$$

The coefficient of consolidation, as obtained from the linear portion of the virgin curve is

$$C_c = \frac{\Delta e}{Log(\frac{\sigma'}{\sigma_o'})} \text{ .... } (2.30)$$

Combining equations (2.29) and (2.30)

$$\Delta H = \frac{C_c}{1+e_o} H_o Log(\frac{\sigma'}{\sigma_o'}) \text{ .... } (2.31)$$

2.6.2 Time rate of consolidation

In this section the governing equation in one-dimensional consolidation theory and the distribution of excess pore water pressure, distributed with in the soil when load is applied, shall be discussed. As mentioned earlier one-dimensional consolidation is where deformation takes place in the direction of loading. The natural loading and unloading of a soil stratum during deposition and erosion of overlying material takes place under condition of one-dimensional consolidation.

Karl Terzaghi presented the theory of one-dimensional consolidation in 1925 for evaluating primary consolidation of saturated clays. The consolidometer is used to simulate in ground a soil under a wide foundation and an embankment.

When coming up with such theory he made various assumptions. These are:

- ☞ The soil is homogeneous
- ☞ The soil is fully saturated
- ☞ The soil grain and water are both incompressible
- ☞ Darcy’s law is valid
- ☞ Compression and flow are one-dimensional
- ☞ The change in volume corresponds to the change in void ratio and ∂e/∂σ’ remains constant

The state of consolidation for a homogeneous soil depends on the soil permeability, the thickness and the length of the drainage path.

In the derivation of the formulae the following two points are used:

- ☞ The change in volume of the soil (ΔV) is equal to the change in volume of pore water expelled (ΔV_w), which is equal to the change in the volume of the voids (ΔV_v). Since the area of the soil is constant (the soil is laterally confined), the change in volume is directly proportional to the change of length.
- ☞ At, any depth the change in vertical effective stress is equal to the change in excess pore water pressure at that depth. That is, $\Delta\sigma' = \Delta u$.

As water is incompressible, the change in volume determined from the principles of continuity. Referring to Figure 2.1, the clay layer is located between two highly permeable sand layers. When the clay is subjected to an increase of vertical pressure $\Delta\sigma$ which is distributed uniformly over a semi-infinite area the pore water pressure in the layer will increase by Δu. This is represented by diagram *abcd* in Figure 2.10

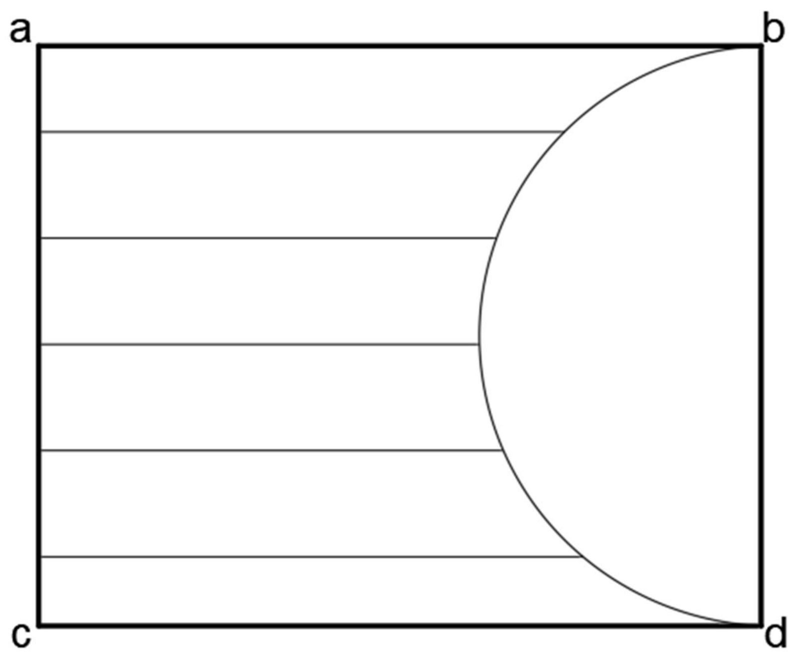


Figure 2. 10 Distribution of excess pore water pressure and effective stress in clay layer confined between two permeable layers.

In one-dimensional consolidation water will be squeezed out in the vertical direction towards the sand layer. After time t has elapsed, drainage into the sand layer above and below will have caused the excess pore water pressure to be reduced to the profile shown by the area outside the shaded portion of *abcd.*

Considering an elemental layer within the clay stratum of thickness dz, in which at time t, the excess pore water pressure is u.

Figure 2.11 shows that a prismatic portion of an elemental layer having dimension dx, dy and dz. The drainage across the sample is one - dimensional in the z direction, with a hydraulic gradient of $-\partial h/\partial z$.

For the soil element:

Inflow

$$q_i = VA = KiA \quad \dots \quad (2.31)$$

$$= -k\frac{\partial h}{\partial z}dxdy$$

Out flow

$$q_o = -k\frac{\partial h}{\partial z}dxdy - k\frac{\partial}{\partial z}\left(\frac{\partial h}{\partial z}\right)dxdydz \quad \dots \quad (2.32)$$

The rate of change in volume of water expelled, which is equal to the rate of change of volume of the soil, must equal the change in flow. That is

$$q_o - q_i = -k\frac{\partial^2 h}{\partial z^2}dxdydz \quad \dots \quad (2.33)$$

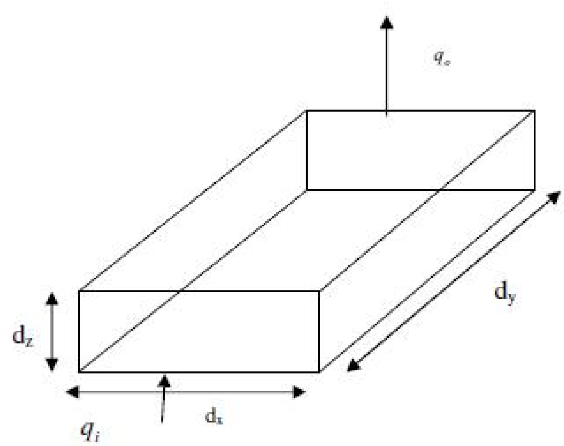


Figure 2. 11 Flow of water through prismatic portion of soil element

During consolidation the rate of change of volume is equal to the rate of change of volume of voids. Therefore,

$$\frac{\partial V}{\partial t} = \frac{\partial V_v}{\partial t} \qquad (2.34)$$

But assuming that soil solids are incompressible $\partial V/\partial t = 0$

And

$$V_s = \frac{V}{(1+e_o)} = \frac{dxdydz}{(1+e_o)}$$

Substituting for $\partial V_s/\partial t$ and V_s in equation.2.34, yields,

$$\frac{\partial V}{\partial t} = \frac{dxdydz}{(1+e_o)}\frac{\partial e}{\partial t} \qquad (2.35)$$

Combining equations 2.33 & 2.35

$$\frac{-k}{\gamma_w}\frac{\partial^2 u}{\partial z^2} = \frac{1}{(1+e_o)}\frac{\partial e}{\partial t} \qquad (2.36)$$

The change in void ratio, ∂e, is due to the increase of effective stress assuming that these are linearly related. Thus

$$\frac{\partial e}{\partial \sigma'} = a_v \qquad (2.37)$$

Again, the increase of effective stress is due to the decrease of excess pore water pressure, ∂u

Hence

$$\partial e = -a_v \partial u \qquad (2.38)$$

Combining equations.2.36 and 2.38

$$\frac{-k}{\gamma_w}\frac{\partial^2 u}{\partial z^2} = \frac{a_v}{(1+e_o)}\frac{\partial u}{\partial t} \qquad (2.39)$$

But $\partial h = \frac{\partial u}{\gamma_w}$ and $m_v = -a_v \partial u$

And $\frac{\partial e}{\partial \sigma'}$ is constant and σ remains constant

$$\frac{\partial u}{\partial t} = \frac{k}{\gamma_w m_v}\left(\frac{\partial^2 u}{\partial z^2}\right) = c_v\left(\frac{\partial^2 u}{\partial z^2}\right) \qquad (2.40)$$

Equation 2.40 is the basic differential equation of Terzaghi's consolidation theory, which can be solved with boundary conditions.

2.6.3 Determination of the pre-consolidation pressure

The value of σp' represents the highest level of stress to which the soil has historically been subjected prior to the current application of load. The pre-consolidation pressure can be useful guide to limit settlement in over-consolidated clays. Generally speaking, it is of great value to geotechnical engineer to know the peak stress as it helps to determine whether the soil stratum in the field is either in the state of normally consolidated or is an over-consolidated over the stress range relevant to that soil stratum. The implication is that its determination is never undermined. It is determined from the same e vs log σp' using the most popular graphical procedure suggested by A. Casagrande. The procedure is summarized as follows; the point of maximum curvature is determined from the consolidation curve by visual inspection. Then two lines are drawn passing through this point of maximum curvature. One is a tangent to the curve and the other is a line parallel to the stress axis and passing through the point of tangency. A line bisecting the angle obtained by the intersection of the two lines is drawn. The straight-line part of the curve is extended back to meet the bisector line obtained. The projection of the point of intersection of these lines gives the approximate value for the pre-consolidation stress σ_p'.

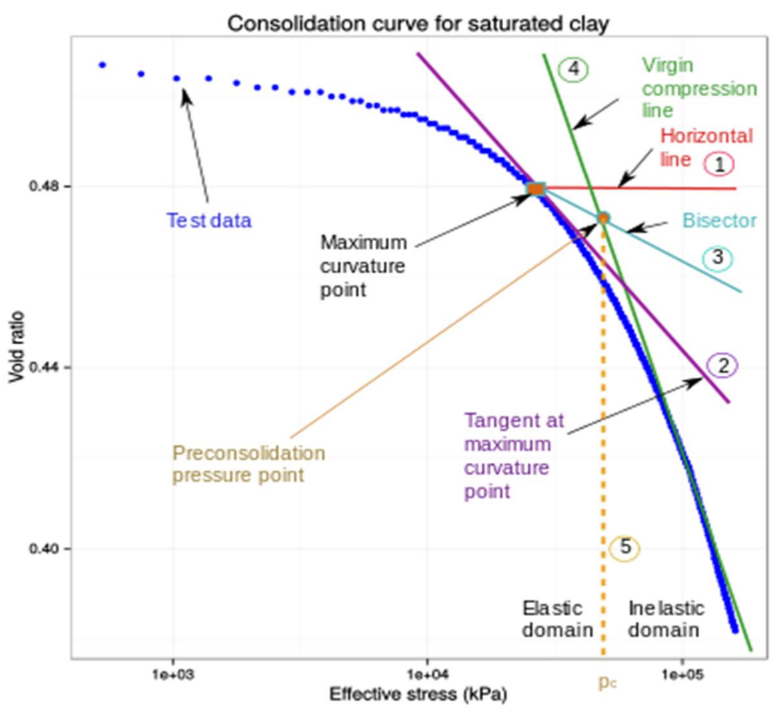


Figure 2. 12 Casagrande's method of determining pre-consolidation stress.

2.7 Previous related work

M.Carter and M.V.Symons, (1989) studied on Site Investigations and Foundations Explained. They suggested that the common term so called tangent modulus 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.

Dessalegn Sissay (2003) conducted on the computation of soil compressibility using tangent modulus approach. The author concluded that the tangent modulus approach provides better way of determining settlement for various types of soil ranging from hard to very soft soil. In the method the soil property, which is determined from laboratory soil deformation tests, are used.

Tesfay Nerea (2004) carried out the determination of parameters for the tangent modulus approach. He presented that in this approach a compression modulus, the tangent to stress *Vs* strain curves obtained from laboratory tests play a significant role. It is a measure of the resistance of soil against deformation due to application of loading. The compression modulus describes the resistance of the media or an isolated part of it against a forced change of equilibrium conditions.

G.H. Yang and J.H Wang (2011) presented the application of undisturbed soil tangent modulus method for computing nonlinear settlement of soil foundation. They suggested that based on the in-site plate loading curve, the undisturbed soil tangent modulus of soil can be computed based on its stress level; then the foundation settlement can be computed by using undisturbed soil tangent modulus in the layer wise summation method. This method takes the nonlinearity and original state of soil into consideration, and can be used to different kinds of loading curves and in the large scale. It can also overcome the shortcomings of the original tangent modulus method which supposes the loading curve to be a hyperbola. The validity of the method is also proved by the in-site loading test and engineering practice.

Hua Huang *et. al* (2018) studied on the calculation of tangent modulus of soils under different stress paths. This study focused on the tangent modulus of soil under different stress paths and provided theoretical basis for the deformation calculation of soil.

2.8 Summary of literature review

The following table shows the summary of literature review.

Name	Description
Professor N.Janbu,N. (1967)	According to Janbu (1967) the implication is that “The tangent modulus is a linear function of effective stress above the present effective overburden”.
M.Carter and M.V.Symons, (1989)	They suggested that the common term so called tangent modulus can as well be applied to soil mechanics. Their 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.
G.H. Yang and J.H Wang (2011)	They suggested that based on the in-site plate loading curve, the undisturbed soil tangent modulus of soil can be computed based on its stress level; then the foundation settlement can be computed by using undisturbed soil tangent modulus in the layer wise summation method.
Hua Huang *et. al* (2018)	This study focused on the tangent modulus of soil under different stress paths and provided theoretical basis for the deformation calculation of soil.

Table 2. 1 Summary of literature review

CHAPTER THREE

METHODOLOGY

3.1 Area of study

The site where primary data samples were collected is located in Bishoftu kebele 01 and kebele 07. Bishoftu is a city found in Oromia, Ethiopia. It is situated a few dozens of miles (29.8 mil or 47.9km) southeast of Addis Ababa. The city has been developing greatly since the second half of the 20th century, and its population number has been increasing greatly since 1990s. The area is famous with its lakes, including lake Bishoftu, lake Hora, lake Kuriftu, and others.

3.1.1 Altitude, latitude and longitude

The latitude of Bishoftu is $8^0$44’4.7400” N and longitude of $39^0$0’30.7188” E. of the largest and most populated cities in Oromia National Regional State. It is located in the East Shewa Zone of Oromia at 8o33`35``N - 8o36`46`` N latitude and 39o11`57`` E – 39o 21`15`` E longitude. The altitude of the city is 1,920 m above mean sea level.

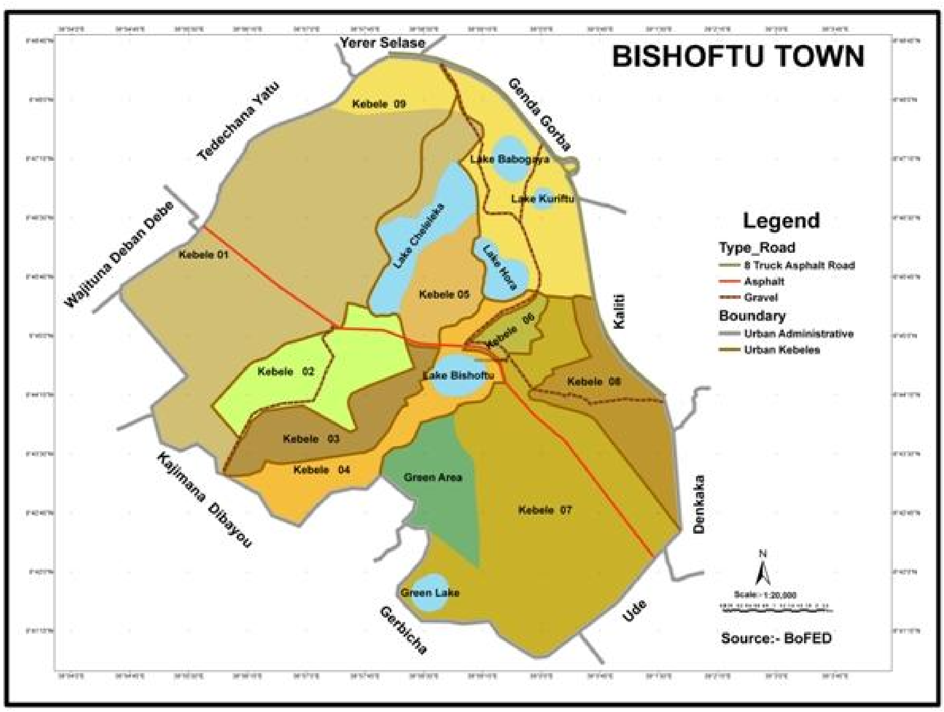


Figure 3. 1 Bishoftu city map (source: Google map)

3.1.2 Precipitation and Temperature Conditions

The climate here is mild, and generally warm and temperate. In winter, there is much less rain fall in Bishoftu than summer. The average annual temperature is 18.7^0C and the average rainfall is 892mm.

3.2 Methods of analysis

In order to meet the desire objective of the research a comprehensive literature review in the area of study had conducted to the better understanding of the both classical and tangent modulus approach.

Then, a series of one-dimensional consolidation tests were performed on disturbed samples collected from different parts of Bishoftu town. Prior to applying normal loads each sample was allowed to saturate. After the saturation process was completed the samples were subjected to series of normal loads, while changes in thickness were recorded against time. The vertical normal load increments were applied at regular time intervals for 24 hrs. The loads were doubled with each increment up to the required maximum loads (25, 50, 100, 200, 400, 800 kpa etc.). From the change in thickness at the end of each normal load and from the change in thickness recorded against time during a load stage the compressibility and rate of consolidation of soils sample were calculated. From the data obtained stress Vs strain graph were plotted. This plot was used to compute the resistance of the soil to deformation, and hence plots of tangent modulus Vs stress for all samples were made. It was these plots used for the analysis and computation of this research work.

3.2.1 Material Tested

To verify whether the relation developed for the normally consolidated soils holds true for the soils of Bishoftu, oedometer tests were carried out collecting samples from the areas.

Samples were collected from different sites that comprise the common soil types in Bishoftu. In order to meet the objective of the project, samples (disturbed) of different properties were taken. The trial test pits were usually dug to a depth of 3m by giving enough space to digging and sampling. Two samples were taken from each test pit in order to represent the field condition.

For the vast majority of practical settlement problems, both seepage and strain were considered so as to take place in one direction only, this is only being vertical. Hence these samples soils were tested in one dimensional consolidation apparatus called oedometer test where there were no lateral strains. Cylindrical specimen of soil, enclosed in metal ring was subjected to a series of increasing static loads, while changing in thickness were recorded against time. Through this test, the compression characteristics of these soils were measured.

From the change in thickness at the end of each load stage the compressibility of the sample soils, and from the changes in thickness recorded against time during a load stage the rate of consolidation, were calculated.

It was these raw data that read from the oedometer which were used for the verification of the applicability and determination of constant for the tangent modulus approach to our local sample soils.

3.2.2 Laboratory procedures and test

This test is performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. From the measured data, the consolidation curve (pressure-void ratio relationship) can be plotted. This data is useful in determining the compression index, the recompression index and the pre-consolidation pressure (or maximum past pressure) of the soil. In addition, the data obtained can also be used to determine the coefficient of consolidation of the soil.

Purpose:
The consolidation properties determined from the consolidation test are used to estimate the magnitude and the rate of both primary and secondary consolidation settlement of earthen material. The data from the consolidation test are used to estimate the magnitude and rate of both differential and total settlement of a structure or earth fill. Estimates of this type are of key importance in the design of engineered structures and the evaluation of their performance.

Standard Reference:
This test is based on the ASTM D 2435 - Standard Test Method for One-Dimensional Consolidation Properties of Soils.

Apparatus Used:

Consolidation device (including ring, porous stones and load plate), Dial gauge (0.002mm = 1.0 on dial), Sample trimming device, Glass plate, Metal straight edge, Clock, Moisture can, Filter paper.

Procedures

1.1 First, they taken an oven dried 3kg soil and mix it with water at optimum water content. This wet soil was compacted in standard proctor mold. Each layer was compacted at 25 of blows.

2.1 After then the compacted sample was soaked in water for 1 day to be saturated.

3.1 They measured the weight of empty consolidation ring.

4.1 Then they measured the height (h) of the ring and its inside diameter (d).

5.1 They extruded the soil sample from the sample which was soaked for 1 day after compaction by using metal ring. Here proper caution was taken to fit the soil sample exactly to the metal ring without any void space.

6.1 Then they measured the weight of the specimen plus metal ring.

7.1 They centered the porous stones that have been soaking, on the top and bottom surfaces of the test specimen. Filter papers were placed between porous stones and soil specimen.

8.1 Being careful to prevent movement of the ring and porous stones, they placed the load plate centrally on the upper porous stone and adjust the loading device

9.1 Add water to the consolidometer to submerge the soil and porous stone to keep it saturated.

10.1 They adjusted the dial gauge to a zero reading.

11.1 Then they applied loads on the load plate. First, they start by 3kg load then after the load is doubled after each consecutive load.

12.1 They recorded the consolidation dial readings at the elapsed times given on the data sheet.

In all the above listed procedures, several measurements and records were taken and this are listed below.

Type of specimen	Remolded
Specimen Diameter(D), mm	62.00
Specimen Thickness (H_o), mm	20.00
Area(A), cm2	30.18
Volume(V), cm3	60.35
Mass of Ring, gm	42.16
Mass of Ring +Specimen, gm	139.02
Mass of specimen, gm	96.86
Specific Gravity (Gs)	2.52
Initial moisture (wo), %	27.15
Bulk Density (ρ_o), gm/cc	1.60
Dry Density (ρ_d), gm/cc	1.26
Initial Void Ratio: e_o =(Gs/Dd)-1	1.00
Hight of Solid: H_s=H_o/(1+e_o), mm	10.02
Initial Saturation: S_o=W_o Gs/ e_o, %	68.67

Table 3. 1 Records taken from consolidation test

CHAPTER FOUR

RESULT AND DISCUSSION

4.1 Laboratory test results

The test result, which is obtained from primary data source, has to be interpreted. The interpretation of the result helps in the determination of soil parameters, which governs design. The data obtained are analyzed basically by using a graphical procedure. Readings are recorded from the compression as loading proceeds of course at the specified loading interval. The results obtained from the test are interpreted by using the classical approach and the tangent modulus approach.

Basically, when conducting the test on the fat clay soils the conventional procedure is adapted. That is the test is performed with constant load increment duration of 24 hour. It is performed on disturbed saturated samples.

In the test method a soil specimen is restrained laterally and loaded axially with total stress increments. Such condition prevents lateral deformation from which the one-dimensional consolidation parameters are derived. Such stress increment is maintained until excess pore water pressure is completely dissipated.

During the consolidation process measurements are made of change in the specimen height and these data are used to determine the relationship between effective stress and void ratio or strain and the rate at which consolidation can occur by evaluating the coefficient of consolidation. From the relationships derived from the test data the following two parameters are determined.

- ☞ The compressibility of the soil – which is a measure of the amount by which the soil will compress when loaded and allowed to consolidate.
- ☞ The time related parameters – which indicates the rate of compression and hence the time –period over which consolidation settlement will take place.

The following table shows the time vs deflection and the corresponding time for coefficient of consolidation calculation by using the Logarithm of time method.

	Dial gauge reading in mm							
Time,min	12Kpa	25Kpa	50Kpa	100Kpa	200Kpa	400Kpa	800Kpa	1600Kpa
0	0.000	0.132	0.343	0.822	1.418	1.890	2.432	2.883
0.25	0.040	0.162	0.460	0.997	1.525	2.131	2.615	3.112
0.5	0.042	0.165	0.470	1.020	1.534	2.156	2.642	3.132
1	0.046	0.168	0.488	1.052	1.548	2.188	2.678	3.158
2	0.050	0.172	0.520	1.088	1.562	2.222	2.722	3.178
4	0.054	0.180	0.549	1.130	1.588	2.225	2.757	3.197
8	0.058	0.192	0.583	1.163	1.619	2.232	2.781	3.221
15	0.065	0.200	0.614	1.198	1.655	2.310	2.800	3.249
30	0.077	0.222	0.650	1.236	1.698	2.335	2.820	3.289
60	0.083	0.244	0.691	1.270	1.743	2.360	2.830	3.314
120	0.096	0.275	0.730	1.312	1.781	2.383	2.841	3.330
240	0.111	0.310	0.760	1.360	1.812	2.400	2.850	3.378
480	0.121	0.328	0.799	1.382	1.869	2.419	2.880	3.408
1440	0.132	0.343	0.822	1.418	1.890	2.432	2.883	3.444

Table 4. 1 Time vs deflection

The data obtained are analyzed basically by using a graphical procedure. Readings are recorded from the compression as loading proceeds of course at the specified loading interval. The results obtained from the test are interpreted by using the classical approach and the tangent modulus approach.

4.1.1 Application of the classical approach

Pressure	D-100	Do	D-50	T-50	ΔH	Δe=ΔH/Hs	e=eo-Δe	H/2=(Ho-D-50)/2	Cv=0.197*(H/2)2/T-50
12	0.127	0	0.080	60	0.127	0.0126	0.9851	9.9599	0.3257
25	0.336	0.2	0.246	70	0.336	0.0335	0.9643	9.8771	0.2746
50	0.811	0.4	0.621	15	0.811	0.0810	0.9168	9.6894	1.2330
100	1.400	0.9	1.171	13	1.400	0.1398	0.8579	9.4145	1.3431
200	1.880	1.5	1.691	25	1.880	0.1877	0.8100	9.1546	0.6604
400	2.426	2.1	2.250	9	2.426	0.2423	0.7555	8.8751	1.7241
800	2.882	2.6	2.717	6	2.882	0.2878	0.7099	8.6416	2.4519
1600	3.426	3.1	3.246	15	3.426	0.3422	0.6556	8.3770	0.9216
800	3.418	3.4	3.431		3.418	0.3414	0.6563		
400	3.320	3.4	3.369		3.320	0.3316	0.6661		
200	3.201	3.3	3.261		3.201	0.3197	0.6780		

Table 4. 2 Calculation of the coefficient of consolidation for different load

4.1.2 Application of the Tangent Modulus to Local Soils

The vertical normal load increments were applied at regular time intervals for 24 hrs. The loads were doubled with each increment up to the required maximum loads (12, 25, 50, 100, 200, 400, 800, and1600Kpa) From the change in thickness at the end of each normal load and from the change in thickness recorded against time during a load stage the compressibility and rate of consolidation of soil sample were calculated.

The following table shows the Stress vs strain and the final reading of deformation of local soil sample.

P'(kPa)	log p'(kPa)	Final reading deformation (mm)	H_0 (mm)	ε (strain)
12	1.0791812	0.132	20	0.0066
25	1.39794	0.343	20	0.01715
50	1.69897	0.822	20	0.0411
100	2	1.418	20	0.0709
200	2.30103	1.89	20	0.0945
400	2.60206	2.432	20	0.1216
800	2.90309	2.883	20	0.14415
1600	3.20412	3.444	20	0.1722
800	2.90309	3.418	20	0.1709
400	2.60206	3.32	20	0.166
200	2.30103	2.142	20	0.1071

Table 4. 3 Calculation of the coefficient of consolidation for different load

The following figure shows the Stress vs strain and the Tangent modulus vs stress of local soil sample.

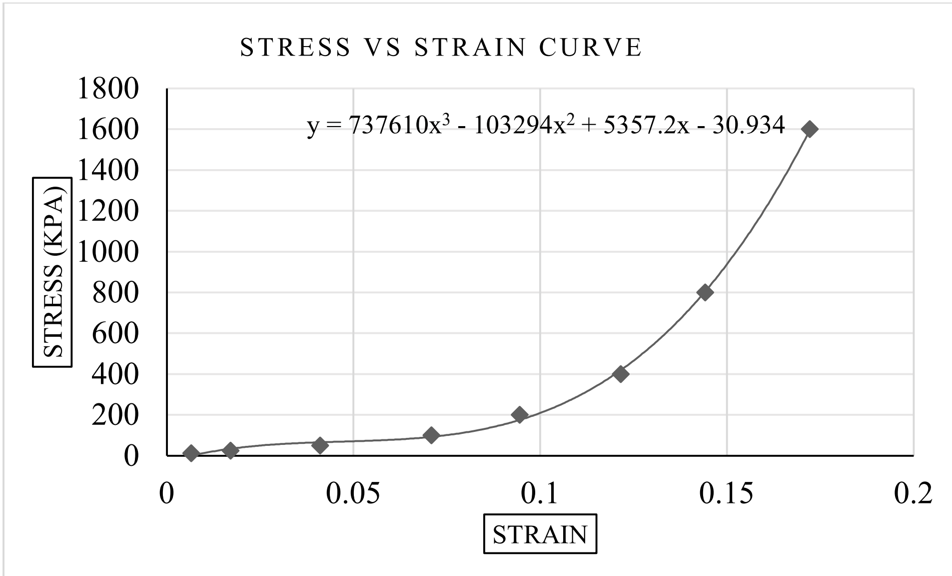


Figure 4. 1 Stress vs strain curve

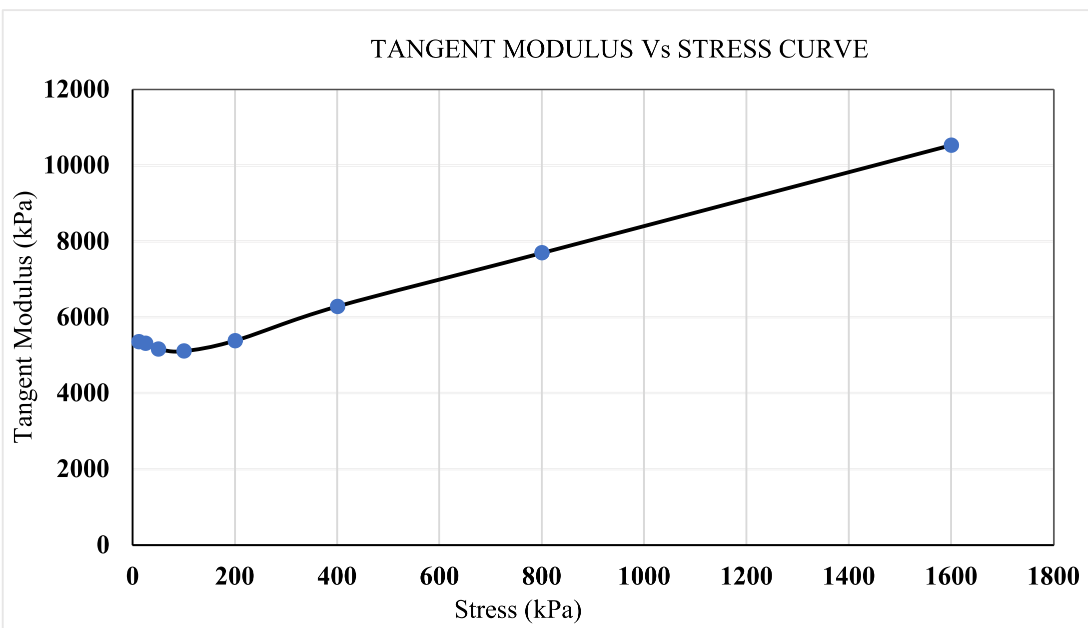


Figure 4. 2 Tangent Modulus Vs Stress curve

From tangent modulus versus stress curve estimation of pre consolidation pressure is much easier and more accurate than the classical approach value. The value is $\sigma_p' \approx 97$kPa.

4.2 Interpretation of the test results

The data obtained are analyzed basically by using a graphical procedure. Readings are recorded from the compression as loading proceeds of course at the specified loading interval. The results obtained from the test are interpreted by using the classical approach and the tangent modulus approach.

4.2.1 Interpretation using classical Approach

The results from one-dimensional consolidation test analyzed according to the procedure produced by Terzaghi.

The data obtained from dial reading while conducting the test is settlement of the specimen under the applied loading. These dial readings are converted to either void ratio or strain through computation. The applied load is divided by the sample area to obtain stress. Then the stress versus strain or stress versus void ratio plots are prepared plotting is done on the semi-log paper. As already mentioned previously the plot on the semi log paper helps in determining the pre-consolidation stress, which is later made use of in settlement computation. Further the plot provides a straight-line portion which again is important for determining the compression index which is one of the two important consolidation parameters. The other is the coefficient of consolidation.

The semi-log plot of either strain versus log stress or void ratio versus log stress has the following characteristics

- The initial branch of the curve has a relatively flat slope
- At some pressure the plot curvature sharply increases. This point at which major structural changes is assumed to take place is considered as the maximum past effective stress encountered by that particular soil. When the curvature sharply increases (breaks) close to the in situ effective overburden pressure σ'_{ov}, it is concluded that the soil is normally consolidated. If the break occurs at a pressure greater than σ'_{ov}, it is said to that the soil is pre-consolidated. The relative amount of the pre-consolidation pressure is usually reported as the over consolidation ratio (OCR).

$$OCR = \frac{\sigma_p'}{\sigma_{ov}'} \qquad (4.1)$$

☞ At the end after a stress increased loading beyond the curvature becomes somewhat straight line.

4.2.2 Interpretation based on the tangent modulus approach

In this approach the consolidation test result is plotted on the arithmetic scale as stress versus strain. The modulus number is determined from the curve. Depending on the nature and type of the soil the curve has different shape. On this plot the pre consolidation pressure is depicted from the graph by simple visual inspection. For the loading less than this pressure the stress versus strain curves decreases and for the loading beyond the same pressure the graph increases upwards. Thus, the point at which the graph shows minimum point is close to the pre-consolidation pressure to which that particular soil is exposed to in its past history. The procedure for the determination of the settlement of the soil using the method is described in chapter three. The following example describes computation of settlement for the soil layer using both the tangent modulus approach and the classical approach. The modulus number m, C_c and e_o are obtained from the laboratory test results. In the absence of practically measured value, it is not possible to compare the numerical values obtained using the two approaches.

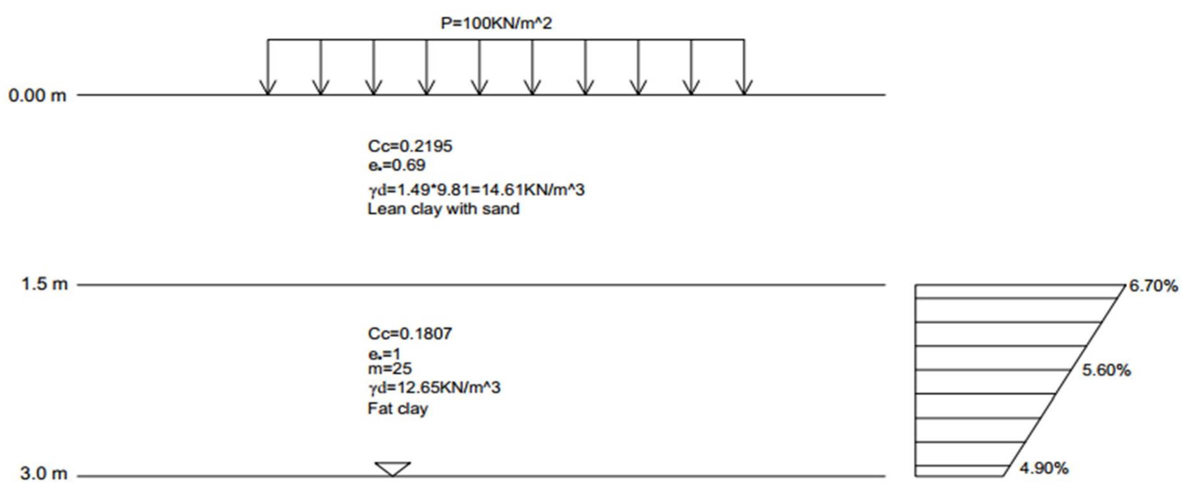


Figure 4. 3 Soil Profile of BH1

Additional stress =100KN/m2

Initial effective stress in the fat clay layer

P1 (At the top) =14.61x1.5 =21.92KN/m2

P1 (At the middle) =14.61x1.5 + 12.65x1.5 x0.5 = 31.41KN/m2

P1 (At the bottom) =31.41+12.65x1.5x0.5 = 40.9KN/m2

Average effective stress =1/6x (21.92+4x31.41+40.9) = 31.41KN/m2

After application of additional stress

P2= ΔP+P1

P2(At the top) = 100+21.92 = 121.92KN/m2

P2(At the middle) = 100+31.41 = 131.14KN/m2

P2(At the bottom) = 100+40.9 = 140.9KN/m2

I. Classical Approach

$$\Delta H = \frac{C_c}{1+e_o}\log\left(\frac{P_2}{P_1}\right) * H$$

Inserting the values of P2 and P1 at the middle of the fat clay layer the resulting settlement is ΔH=8.46cm.

II. Tangent Modulus Approach

The resulting strain is computed at the top, middle and the bottom of the fat clay layer by inserting the corresponding values:

$$\varepsilon = \frac{1}{m}\ln\left(\frac{P_2}{P_1}\right)$$

$\varepsilon_{(\text{at the top})}$ =0.067 =6.70%

$\varepsilon_{(\text{at the middle})}$ =0.056 = 5.60%

$\varepsilon_{(\text{at the bottom})}$ =0.049 =4.90%

Thus, settlement is the area of the strain curve shown in the figure,

ΔH = 0.049x150+0.5x (0.067-0.049) x150cm

= 10.05cm

4.3 Comparison of classical approach and tangent modulus approach

The similarities of the two procedures are that both graphs show somewhat the same characteristics at the point where the pre-consolidation stress occurs. This point is seen on both curves as a point where there is a significant change on the nature of the graph. The second similarity is that the same data obtained from the test is used in plotting the curves.

The differences are first, the tangent modulus approach is very suitable to determine settlements for different types of soil while the classical approach provides procedure for determining settlement only for normally consolidated soils. Second it is not required to calculate 'C_c' as an intermediate step in the calculation of pre consolidation pressure.

From what has been said and seen on the graphs the advantages of the tangent modulus approach can be appreciated by first considering the limitations in the classical approach.

The limitations of the classical approach can be summarized as follows:
In a more precise and concise way the limitation of the classical approach, which of course initiated the need for the tangent modulus, can be stated as follows

- ☞ The logarithmic scale in the e-log σ' distorts and hence hides away the details of the soil behaviors. Virtually all the information at the start of the test is erased.
- ☞ The determination of the pre-consolidation pressure σ_p', using Casagrande approach is highly dependent on the human element and is empirical.
- ☞ The approach is limited to cohesive (clayey) soils only

On the contrary, the tangent modulus concept offers the following advantageous.

- ☞ The concept is fundamentally sound and is based on principles that have been successfully applied in other field of engineering and applied science.
- ☞ It is not sensitive to material type and hence presence in itself. a unifying framework for characterizing engineering material behaviors and it's demonstrated the versatility of the concept in handling the full range of materials from soft soils to hard rock (even concrete).
- ☞ 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.

Therefore, the simplicity and rationally inherent in the resistance concept made it the preferred method of analysis of the compressibility of soil.

4.4 Discussion

The settlement computation value for the soil layer using the tangent modulus approach gave us greater value than the classical approach, this used for limit to a minimum to allow the structure function properly and maintain the required aesthetic value. Further cost of maintenance has to be limited to a minimum.

As observed from the results the tangent modulus approach clearly showed the over consolidated and normally consolidated ranges, and the pre-consolidated pressure, σ_{pc} these are basic characteristic of the soil which required a lot of efforts to obtain them in the classical method. The pre-consolidated pressure, σ_{pc} is specifically the most characteristics of soils by which their behavior is greatly dependent.

The deformation time graph shows that the deformation after two-hour reading is almost constant. Hence, primary consolidation for most of the loads took two hours. The reading of deformation after two hours, therefore, means computing deformations in secondary consolidation range than in primary consolidation range.

The ranges of over consolidated and normally consolidated are equally important to determine the soil behavior. In normally consolidated range the values almost correspond to the values found in the other part of the world. But in the over consolidated range the response of local fat clay soils is different from the soil 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.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

In this research, an intensive literature review and study has been made to assess the applicability of the tangent modulus concept for our local soils to verify its effectiveness over classical approach. In order to meet the objective of the research, samples of different properties of disturbed soils were taken. The trial test pits were usually dug to a depth of 3m by giving enough space to digging and sampling. Two samples were taken from each test pit in order to represent the field condition.

Based on the results the following conclusions were reached

- It has been shown herein that the tangent modulus concept which has a good scientific explanation is applicable for Bishoftu soil to compute compressibility.
- The tangent modulus approach can be used to compute the strain distribution throughout the soil strata for different modulus number in both ranges of consolidated in one compressive equation.
- The linear relationship between tangent modulus and stress in all ranges of consolidated enables us to conclude that the time rate of settlement is independent of pore water pressure distribution but it is on strain distribution.
- Therefore, the simplicity and rationally inherent in the tangent modulus concept made it the preferred method of analysis of the compressibility of soil.

5.2 Recommendation

The present work has attempted to assess the applicability of the tangent modulus concept over classical approach in the case of Bishoftu town. 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.

A detail comparison of this method with the classical method, for different soil strata in different area, is left for future works.

This study mainly focuses on building structure. The future research may investigate on road embankment settlement performance by using tangent modulus approach.

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APPENDIX A: Test results of stress Vs strain, tangent modulus Vs stress and void ratio Vs pressure plot.

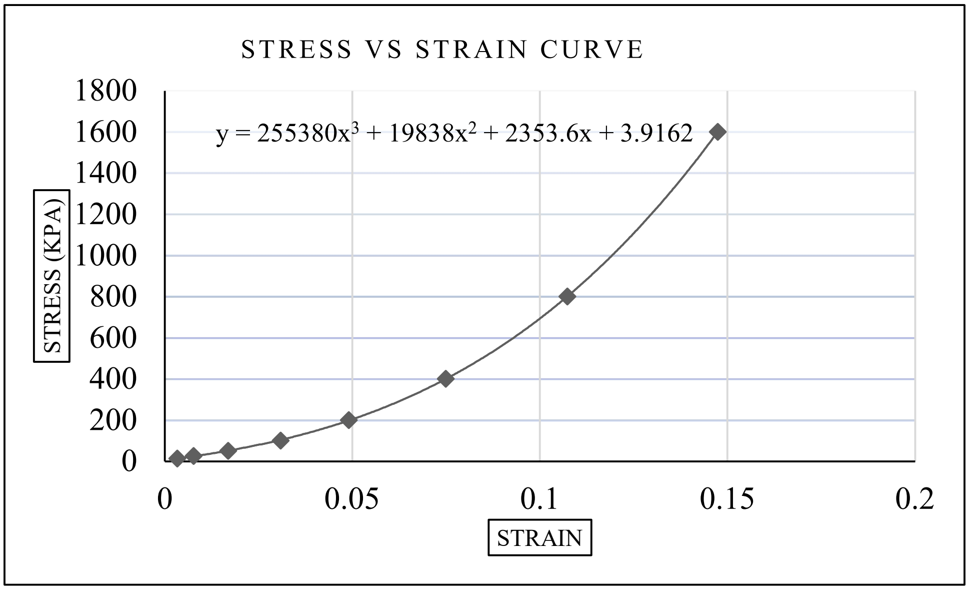


Figure A. 1 Stress – Strain Curve (Depth at 1.5m for BH-1)

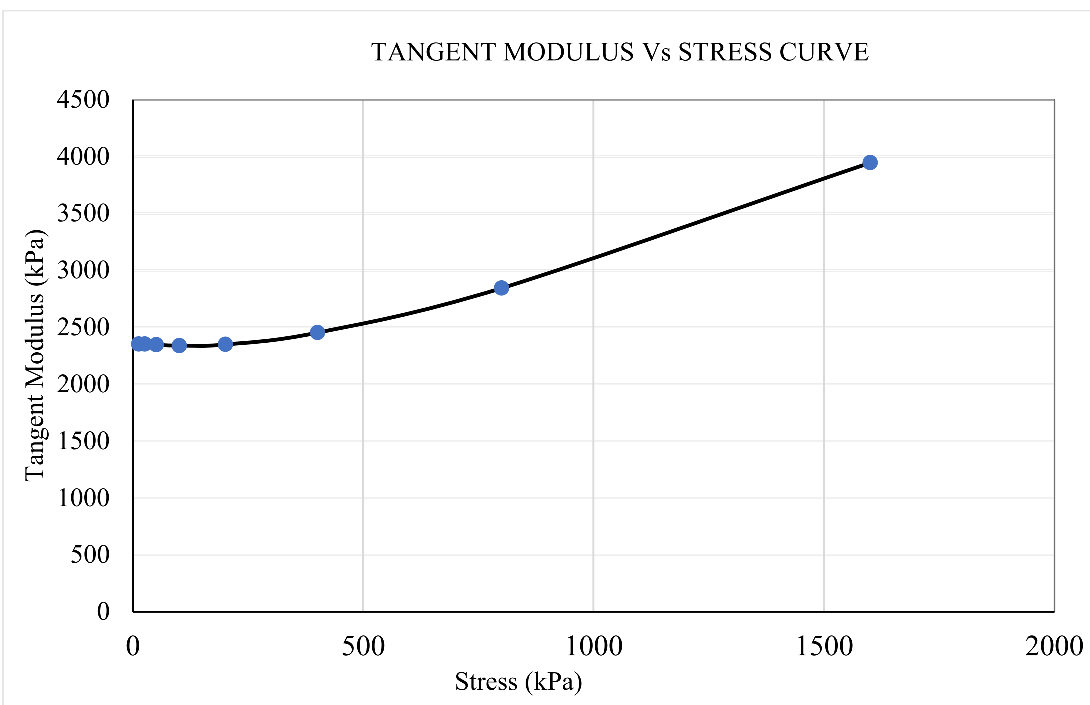


Figure A. 2 Tangent Modulus Vs Stress (Depth at 1.5m for BH-1)

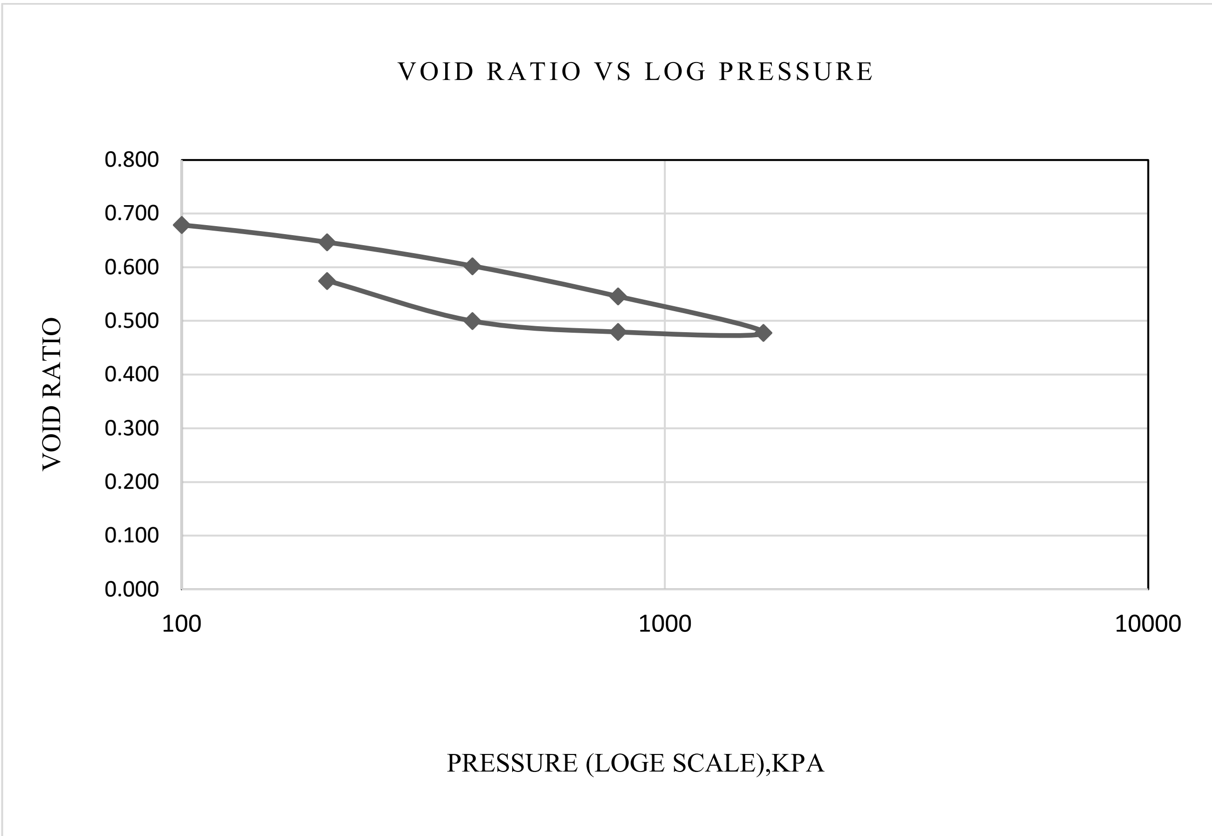


Figure A. 3 Pressure Vs Void Ratio (Depth at 1.5m for BH-1)

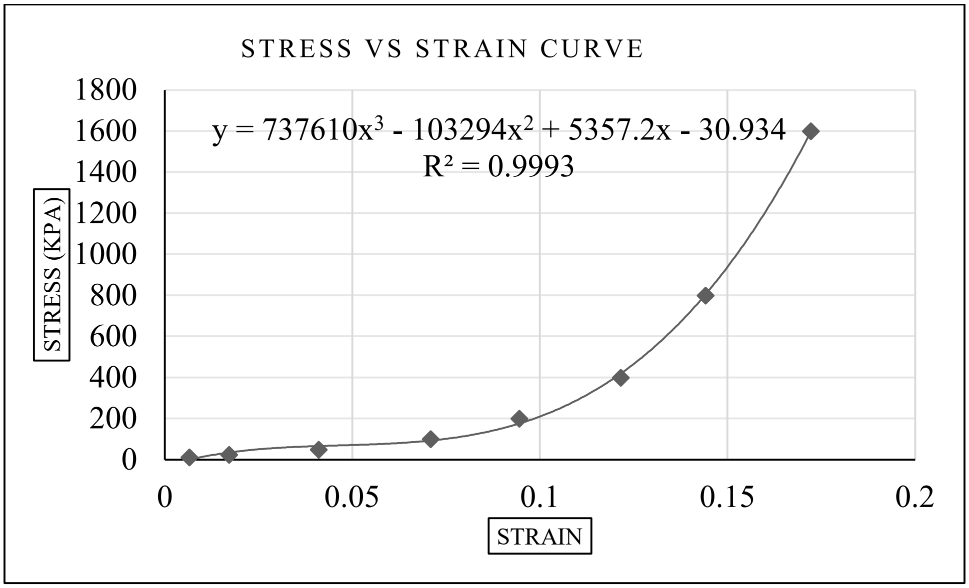


Figure A. 4 Stress – Strain Curve (Depth at 3m for BH-1)

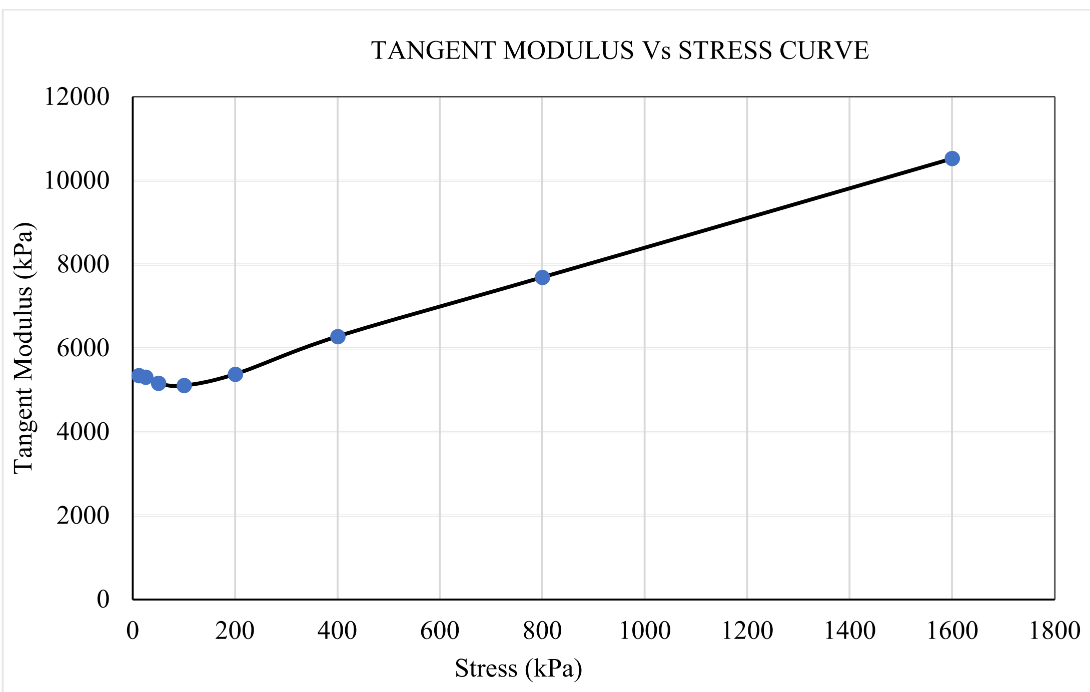


Figure A. 5 Tangent Modulus Vs Stress (Depth at 3m for BH-1)

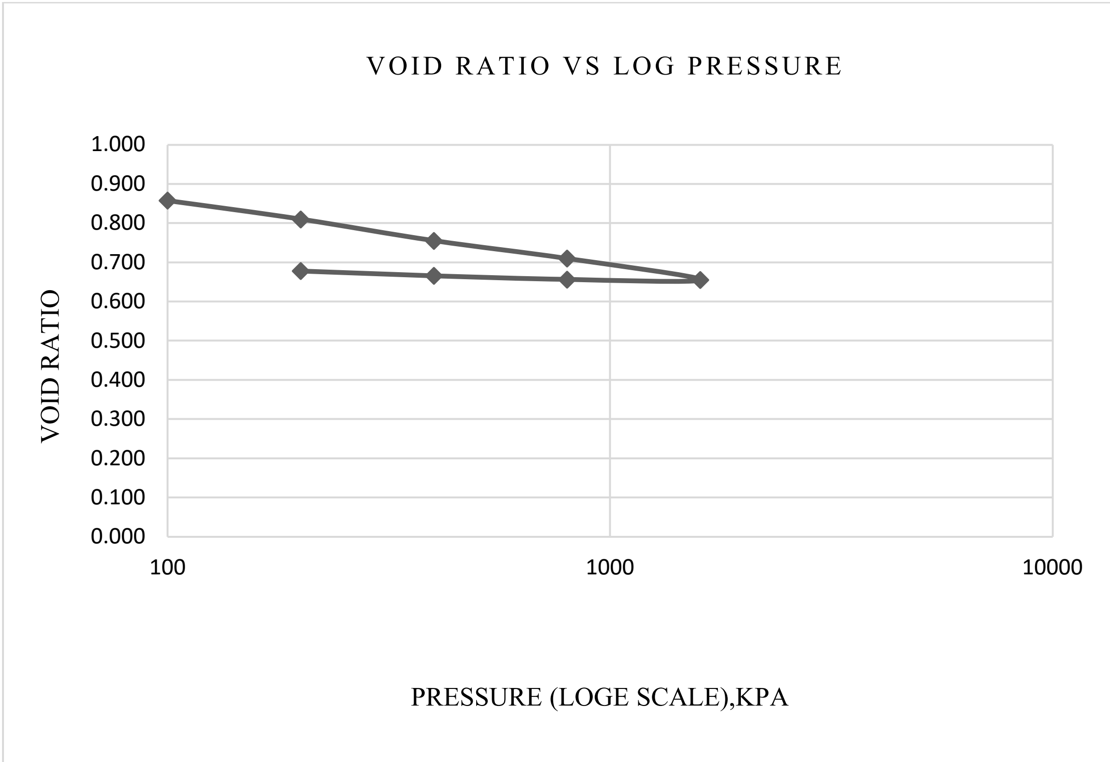


Figure A. 6 Pressure Vs Void Ratio (Depth at 3m for BH-1)

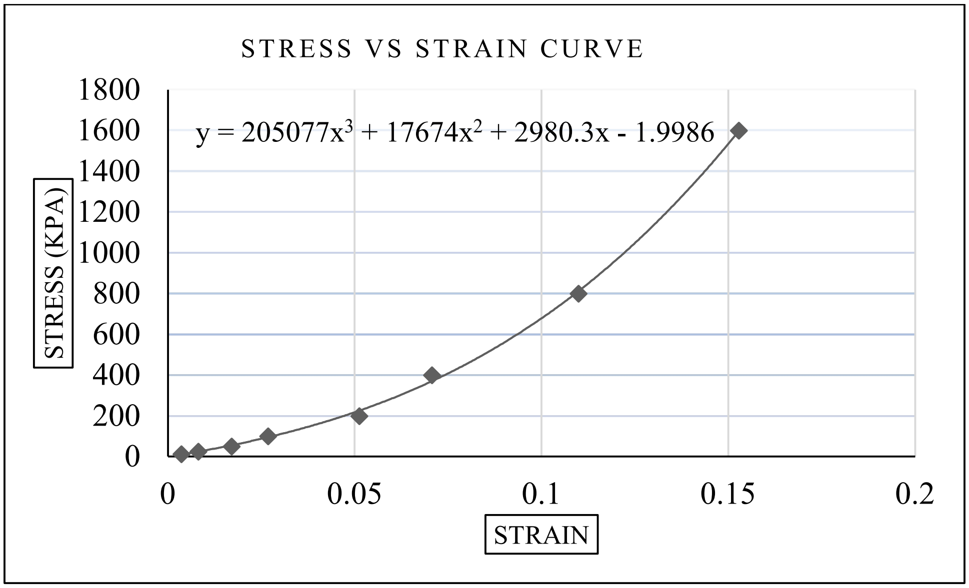


Figure A. 7 Stress – Strain Curve (Depth at 1.5m for BH-2)

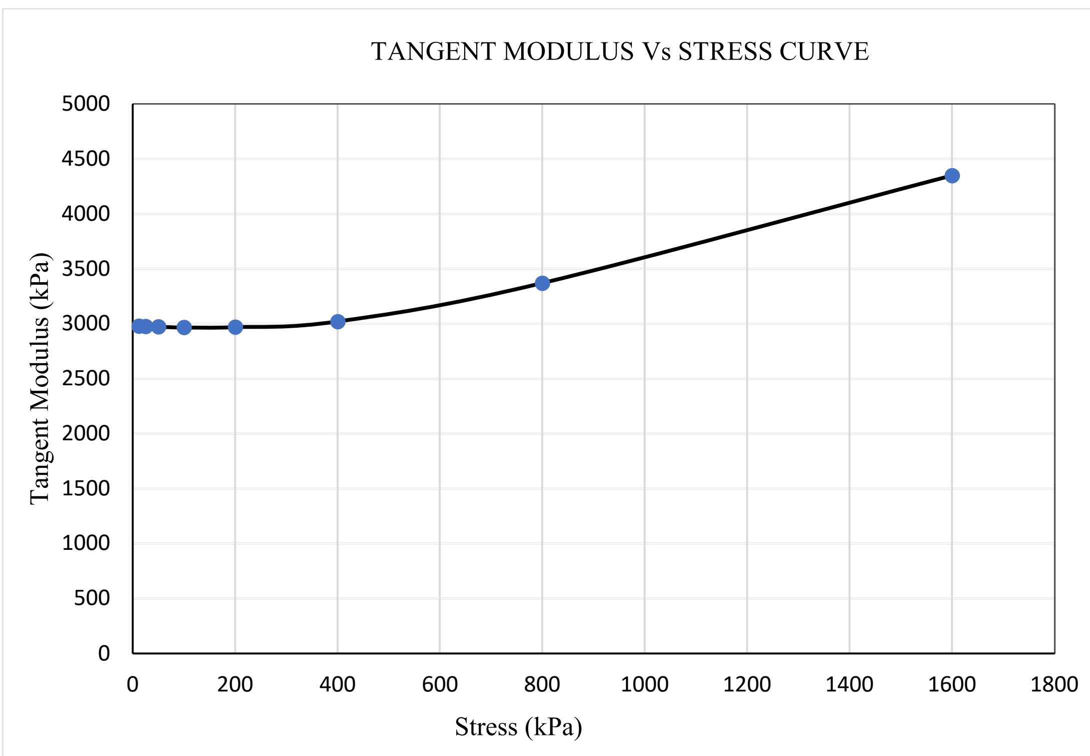


Figure A. 8 Tangent Modulus Vs Stress (Depth at 1.5m for BH-2)

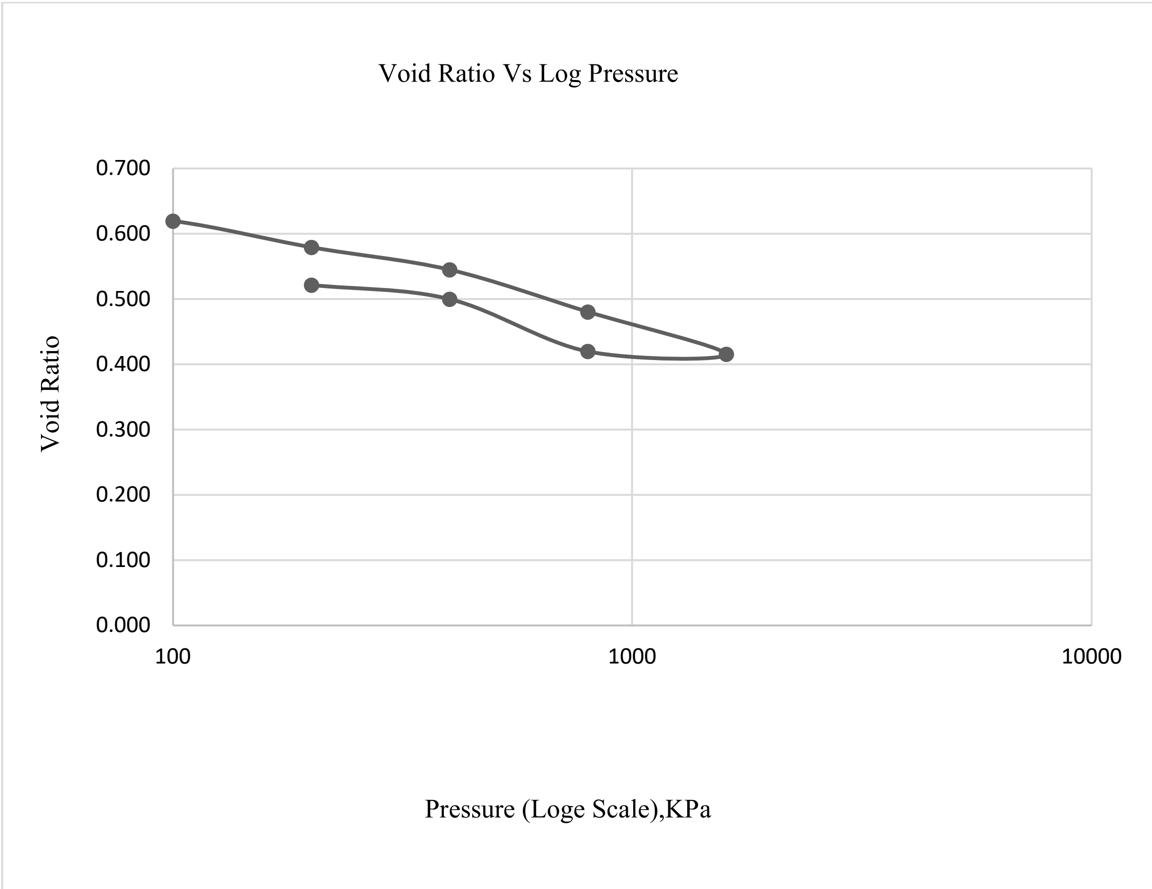


Figure A. 9 Pressure Vs Void Ratio (Depth at 1.5m for BH-2)

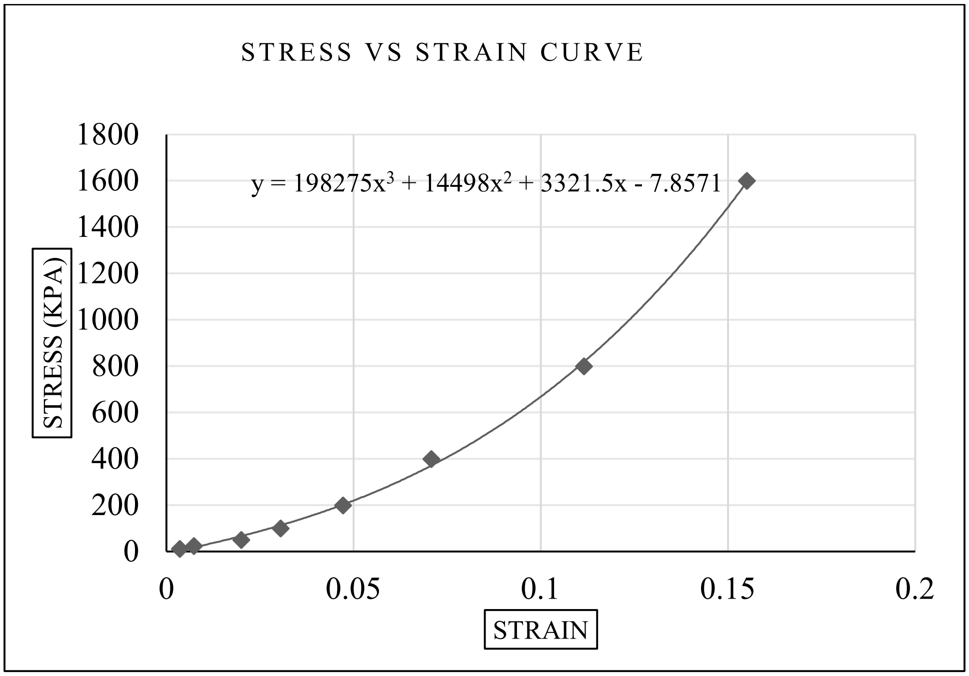


Figure A. 10 Stress – Strain Curve (Depth at 3m for BH-2)

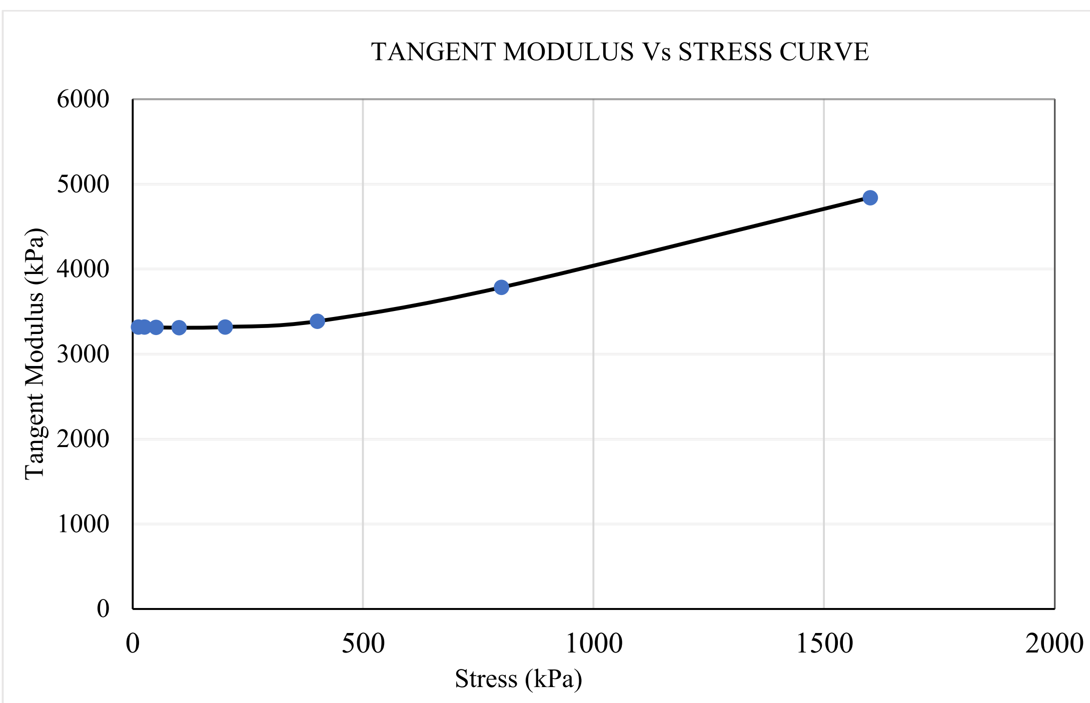


Figure A. 11 Tangent Modulus Vs Stress (Depth at 3m for BH-2)

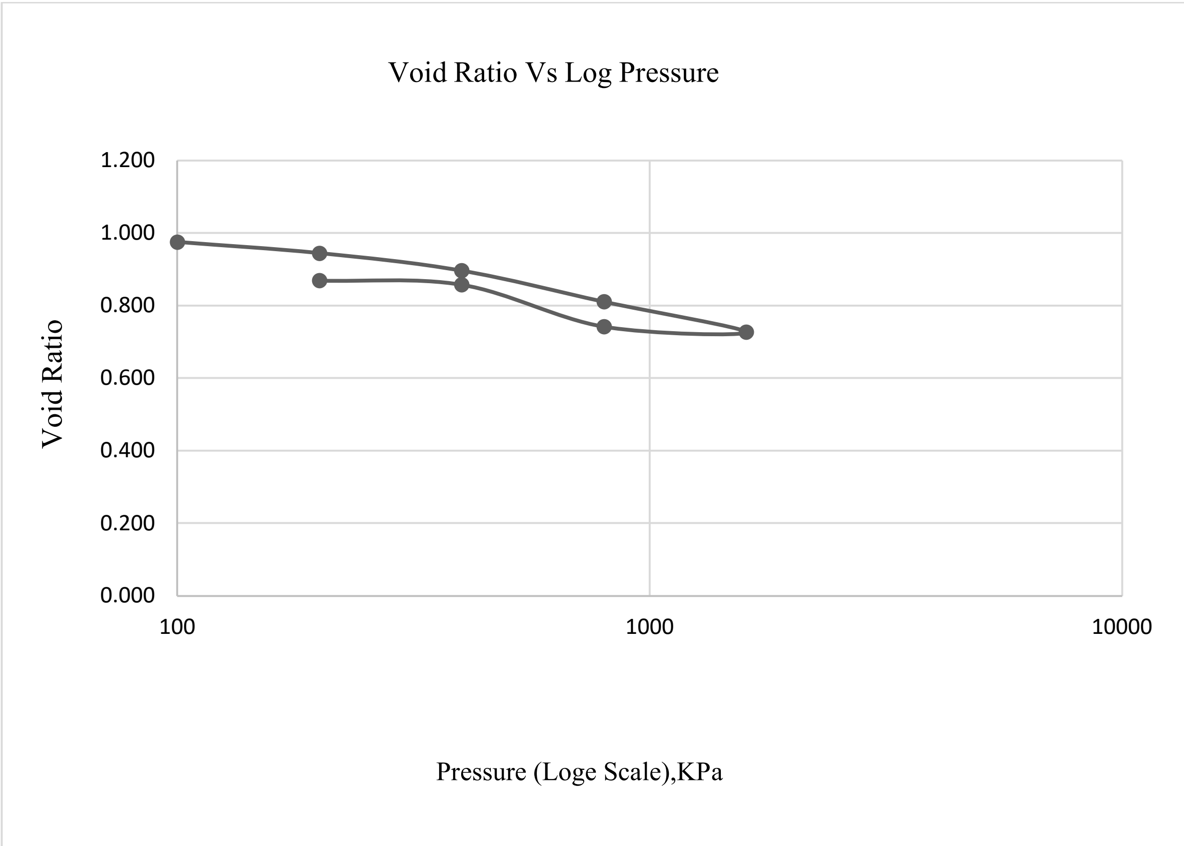
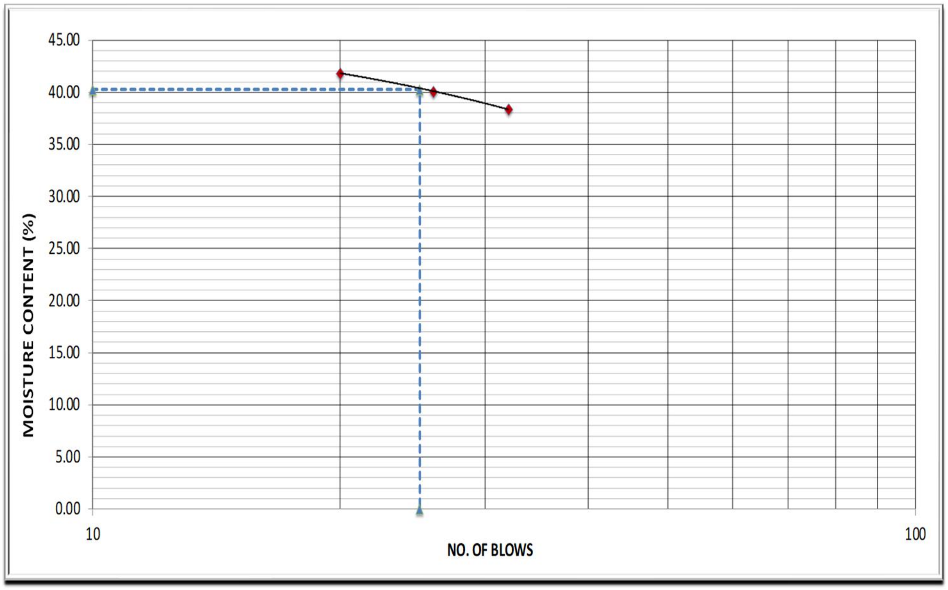


Figure A. 12 Pressure Vs Void Ratio (Depth at 3m for BH-2)

LIQUID AND PLASTIC LIMITS TEST REPORT

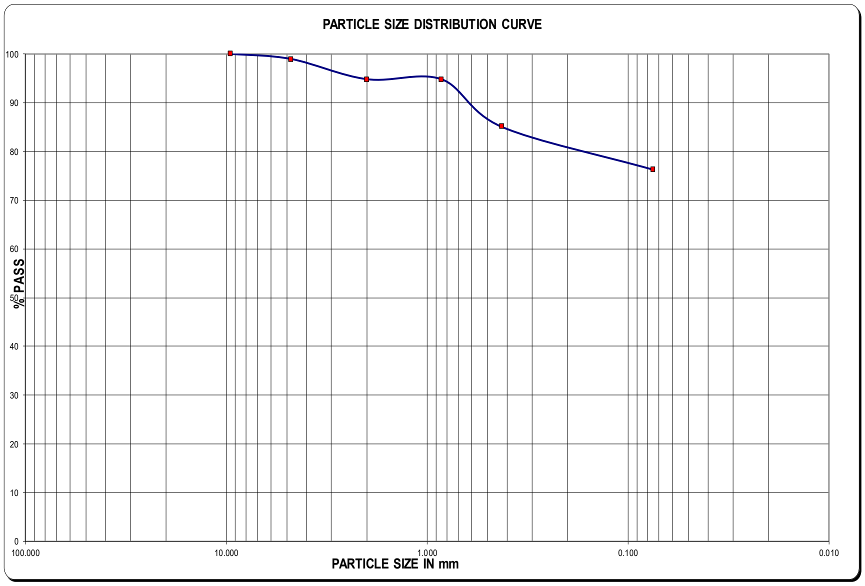


	LIQUID LIMIT			PLASTIC LIMIT	
No. Blows	32	26	20		
Wt. wet soil (g.)	17.10	17.02	17.09	1.61	1.73
Wt. dry soil (g.)	12.36	12.15	12.05	1.34	1.43
Moisture content (%)	38.35	40.08	41.83	20.15	20.98
				AV. PL (%)	20.6

Liquid Limit LL(%)	Plastic Limit PL(%)	Plasticity Index PI	WET SEIVE ANALYSIS, % PASS			Free swell (%)
			2mm	0.425mm	0.075mm	
40	21	20	95	85	76	30

Figure A. 13 Liquid and plastic limits test report (Depth at 1.5m for BH-1)

PARTICLE SIZE DISTRIBUTION REPORT



SIEVE SIZE (MM.)	% PASSED
9.500	100
4.750	99
2.000	95
0.850	95
0.425	85
0.075	76

Soil Description
Lean CLAY with Sand

Atterberg Limit

LL	40
PI	20

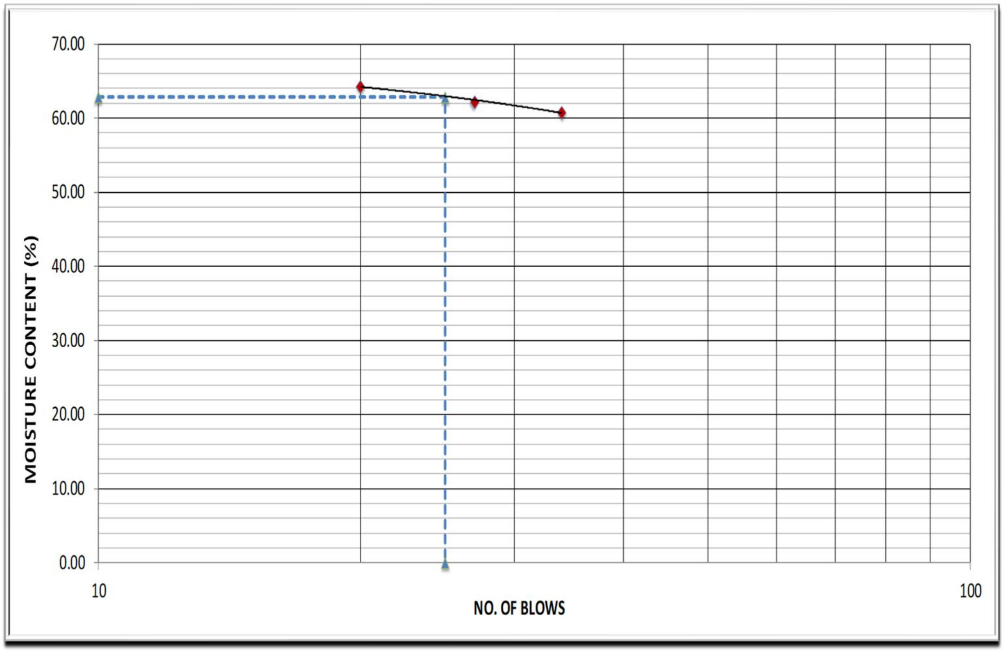
Classification

USCS	**CL**

% COBBLE	% GRAVEL	% SAND	% FINE
-	1	23	76

Figure A. 14 Particle size distribution report (Depth at 1.5m for BH-1)

LIQUID AND PLASTIC LIMITS TEST REPORT

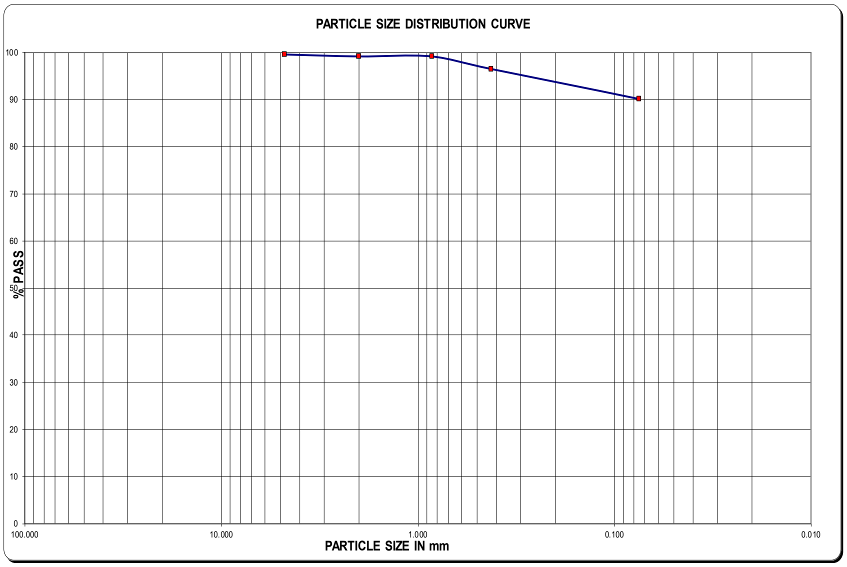


	LIQUID LIMIT			PLASTIC LIMIT	
No. Blows	34	27	20		
Wt. wet soil (g.)	18.30	18.17	18.42	2.29	1.79
Wt. dry soil (g.)	11.38	11.20	11.21	1.75	1.36
Moisture content (%)	60.81	62.23	64.32	30.86	31.62
				AV. PL (%)	31.2

Liquid Limit LL(%)	Plastic Limit PL(%)	Plasticity Index PI	WET SEIVE ANALYSIS, % PASS 2mm	0.425mm	0.075mm	Free swell (%)
63	31	32	99	97	90	30

Figure A. 15 Liquid and plastic limits test report (Depth at 3m for BH-1)

PARTICLE SIZE DISTRIBUTION REPORT



SIEVE SIZE (MM.)	% PASSED
4.750	100
2.000	99
0.850	99
0.425	97
0.075	90

Soil Description	
Fat CLAY	
Atterberg Limit	
LL	63
PI	32
Classification	
USCS	**CH**

% COBBLE	% GRAVEL	% SAND	% FINE
-	0	9	90

Figure A. 16 Particle size distribution report (Depth at 3m for BH-1)

CONSOLIDATION TEST RESULTS OF BH-1

Pressure,KPa	Void Ratio,%	Cv,mm2/minutes (0.197*H2/t50)-Coefficient of consolidation
12	0.725	0.44
25	0.718	0.19
50	0.703	0.28
100	0.679	0.94
200	0.647	1.21
400	0.602	1.72
800	0.546	2.02
1600	0.478	1.06
800	0.480	
400	0.500	
200	0.575	

Copression Index(Cc)	**0.2247**
Swelling index(Cs)	**0.0049**

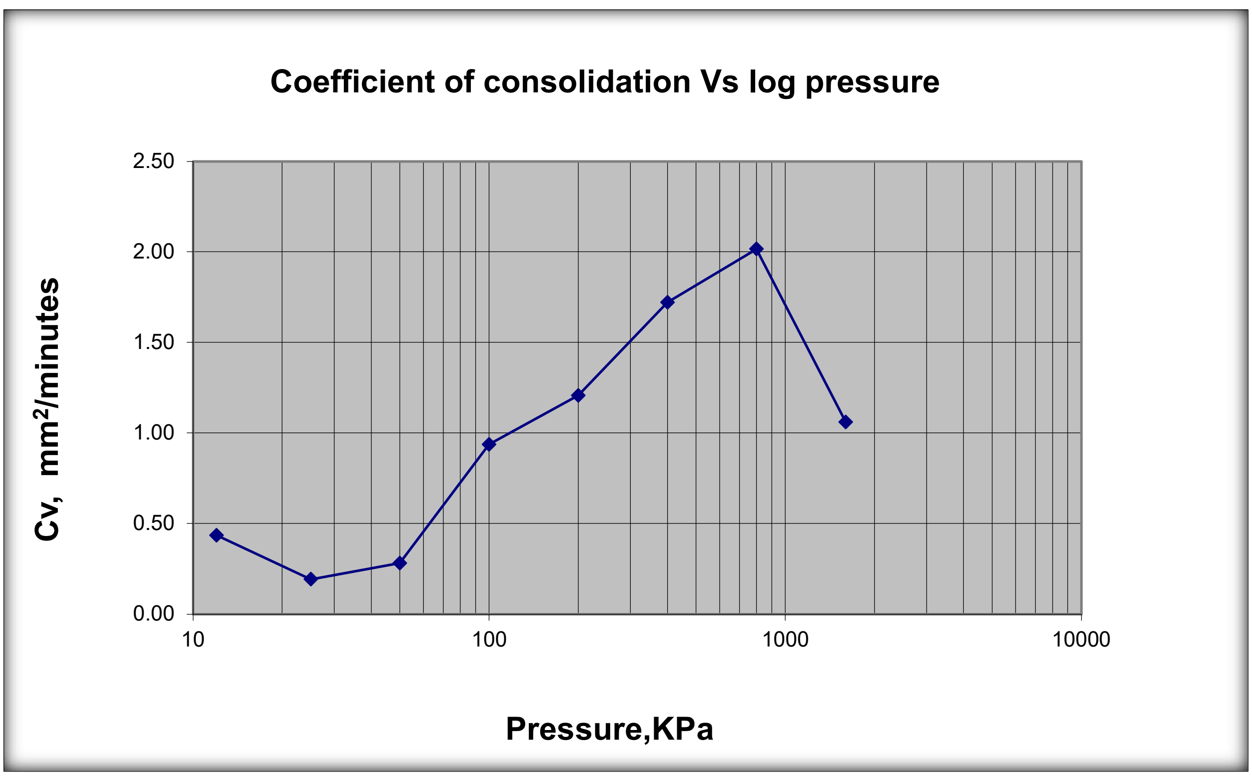


Figure A. 17 Consolidation test report (Depth at 1.5m for BH-1)

Pressure,KPa	Void Ratio,%	Cv,mm2/minutes (0.197*H2/t50)- Coefficient of consolidation
12	0.985	0.33
25	0.964	0.27
50	0.917	1.23
100	0.858	1.34
200	0.810	0.66
400	0.755	1.72
800	0.710	2.45
1600	0.656	0.92
800	0.656	
400	0.666	
200	0.678	

Copression Index(Cc) =	**0.1807**
Swelling index(Cs) =	**0.0027**

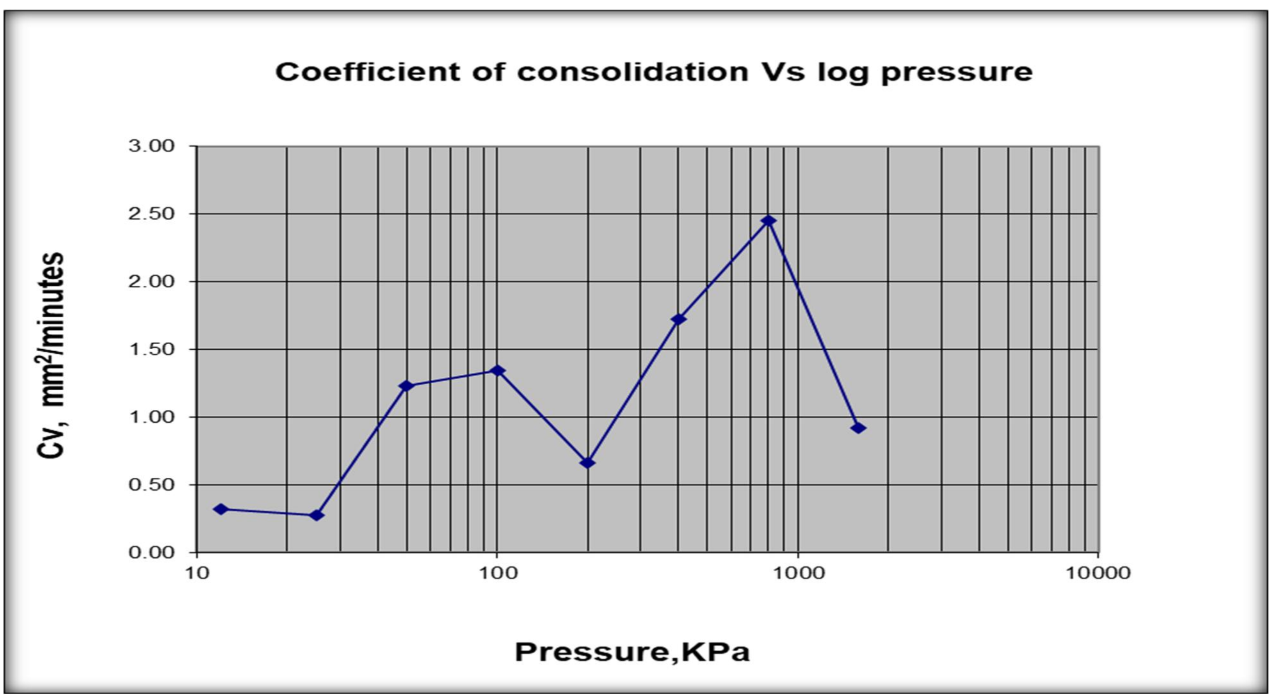
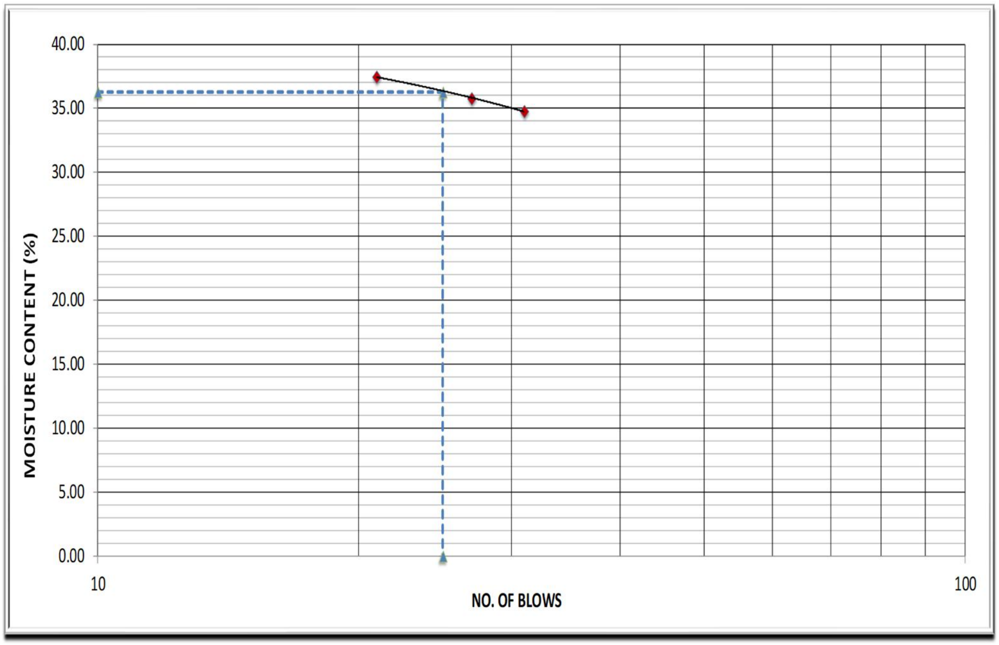


Figure A. 18 Consolidation test report (Depth at 3.0m for BH-1)

LIQUID AND PLASTIC LIMITS TEST REPORT

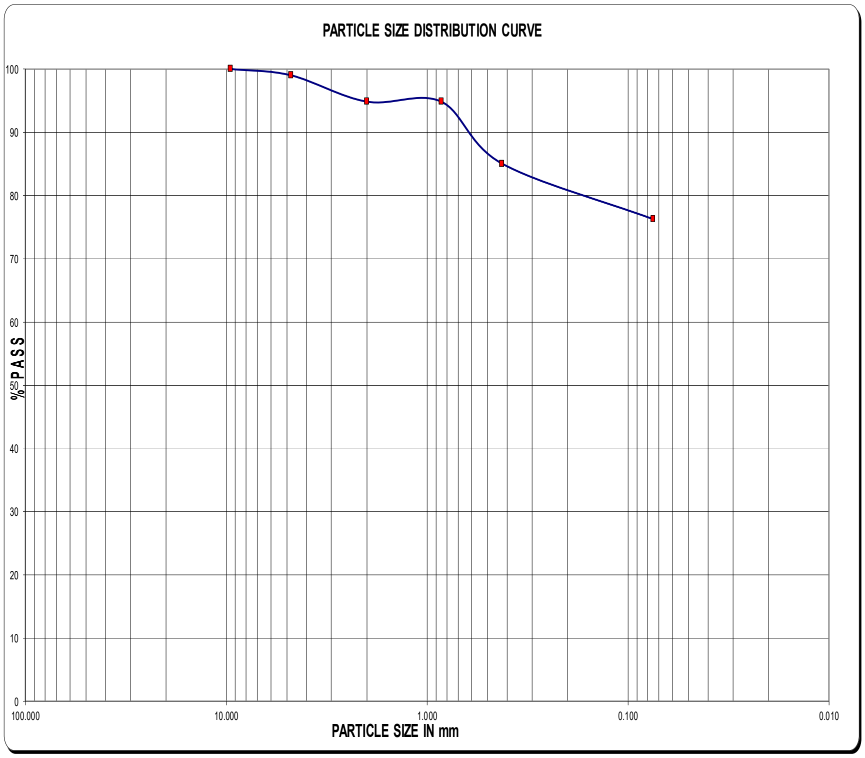


	LIQUID LIMIT			PLASTIC LIMIT	
No. Blows	31	27	21		
Wt. wet soil (g.)	17.99	17.98	18.07	4.61	4.73
Wt. dry soil (g.)	13.35	13.25	13.15	3.34	4.43
Moisture content (%)	34.76	35.70	37.41	38.02	6.77
				AV. PL (%)	22.4

Liquid Limit LL(%)	Plastic Limit PL(%)	Plasticity Index PI	WET SEIVE ANALYSIS, % PASS			Free swell (%)
			2mm	0.425mm	0.075mm	
36	22	14	95	85	76	31

Figure A. 19 Liquid and plastic limits test report (Depth at 1.5m for BH-2)

PARTICLE SIZE DISTRIBUTION REPORT

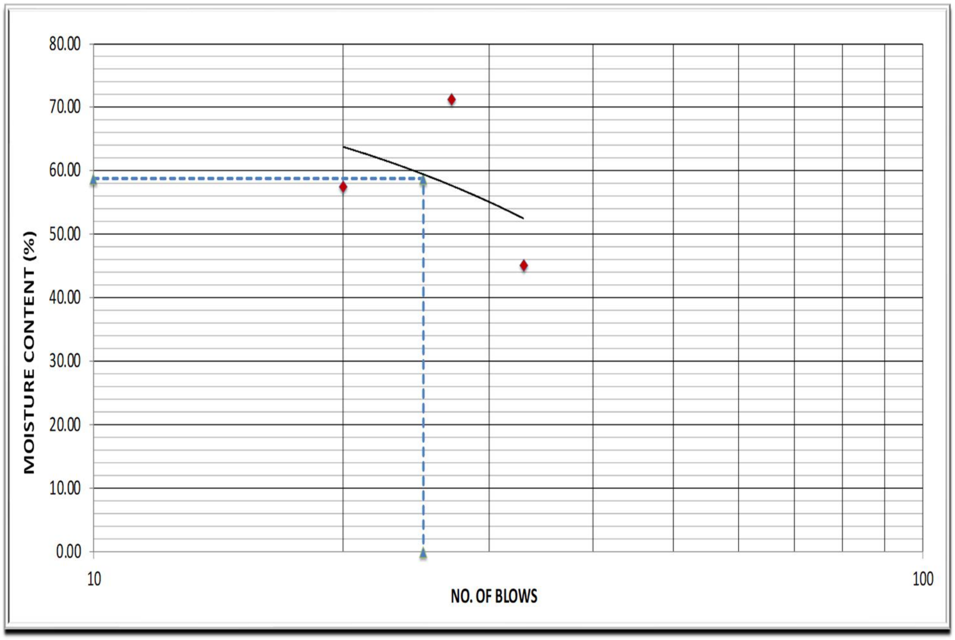


SIEVE SIZE (MM.)	% PASSED
9.500	100
4.750	99
2.000	95
0.850	95
0.425	85
0.075	76

Soil Description	
Lean CLAY with Sand	
Atterberg Limit	
LL	36
PI	14
Classification	
USCS	**CL**

Figure A. 20 Particle size distribution report (Depth at 1.5m for BH-2)

LIQUID AND PLASTIC LIMITS TEST REPORT

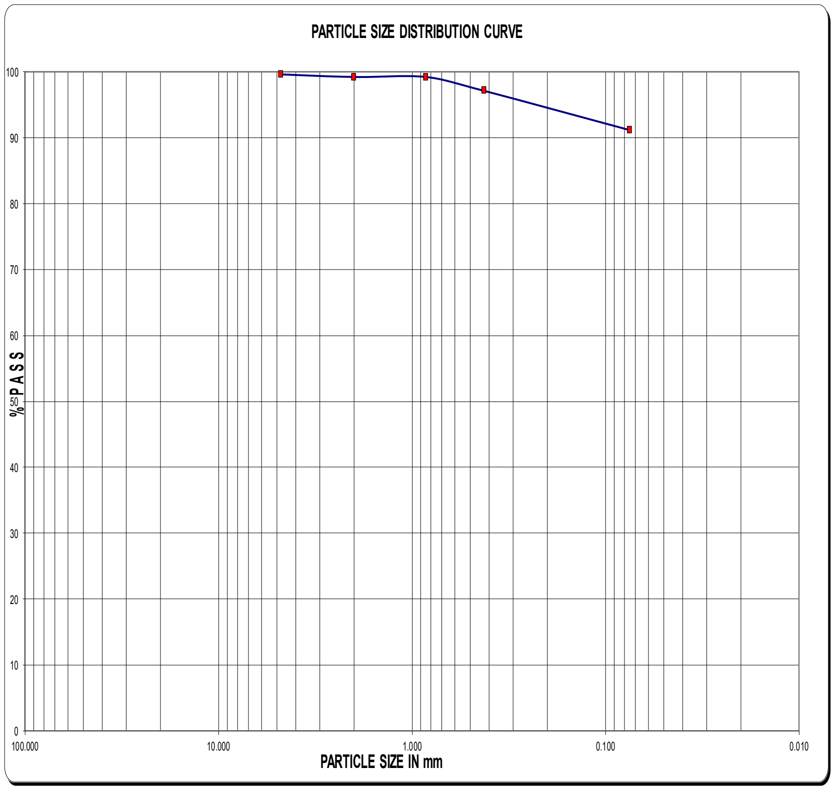


	LIQUID LIMIT			PLASTIC LIMIT	
No. Blows	33	27	20		
Wt. wet soil (g.)	19.40	19.18	19.22	2.20	1.80
Wt. dry soil (g.)	13.37	11.20	12.21	1.70	1.26
Moisture content (%)	45.10	71.25	57.41	29.41	42.86
				AV. PL (%)	36.1

Liquid Limit LL(%)	Plastic Limit PL(%)	Plasticity Index PI	WET SEIVE ANALYSIS, % PASS			Free swell (%)
			2mm	0.425mm	0.075mm	
59	36	23	99	97	91	32

Figure A. 21 Liquid and plastic limits test report (Depth at 3.0m for BH-2)

PARTICLE SIZE DISTRIBUTION REPORT



SIEVE SIZE (MM.)	% PASSED
4.750	100
2.000	99
0.850	99
0.425	97
0.075	91

Soil Description	
Fat CLAY	
Atterberg Limit	
LL	59
PI	23
Classification	
USCS	**CH**

Figure A. 22 Particle size distribution report (Depth at 3.0m for BH-2)

CONSOLIDATION TEST RESULTS OF BH-2

Pressure,KPa	Void Ratio,%	Cv,mm2/minutes (0.197*H2/t50)- Coefficient of consolidation
12	0.656	0.44
25	0.648	0.19
50	0.633	0.28
100	0.619	0.94
200	0.579	1.21
400	0.545	1.74
800	0.480	2.04
1600	0.415	1.07
800	0.420	
400	0.500	
200	0.521	

Copression Index(Cc)	**0.2161**
Swelling index(Cs)	**0.0152**

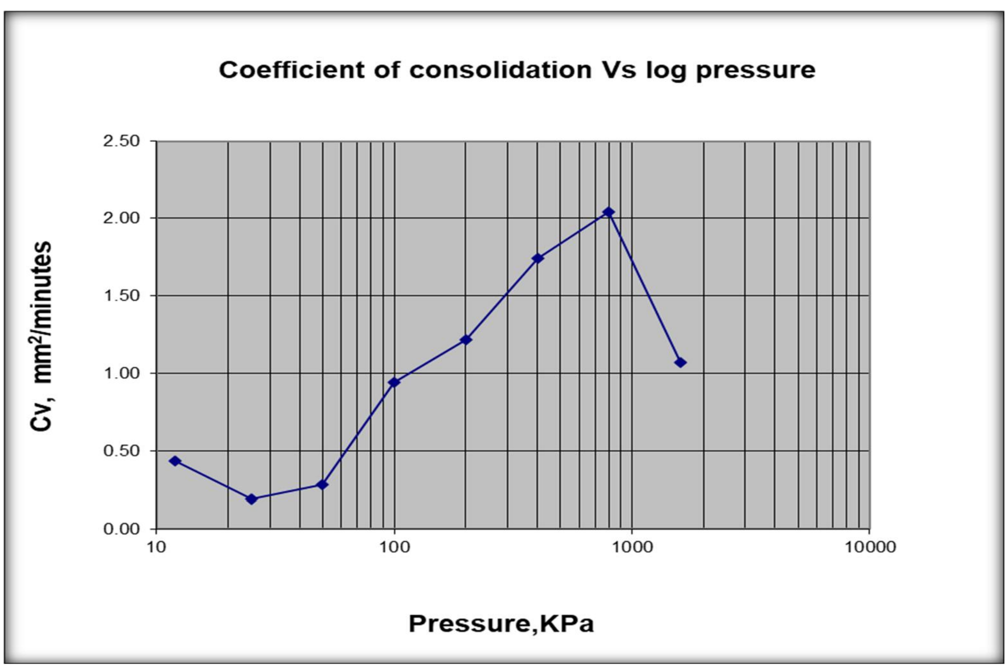


Figure A. 23 Consolidation test report (Depth at 1.5m for BH-2)

Pressure,KPa	Void Ratio,%	Cv,mm2/minutes (0.197*H2/t50)- Coefficient of consolidation
12	1.030	0.33
25	1.023	0.28
50	1.000	1.28
100	0.976	1.44
200	0.945	0.73
400	0.897	1.94
800	0.811	2.74
1600	0.727	0.99
800	0.742	
400	0.858	
200	0.869	

Copression Index(Cc) =	**0.2783**
Swelling index(Cs) =	**0.0501**

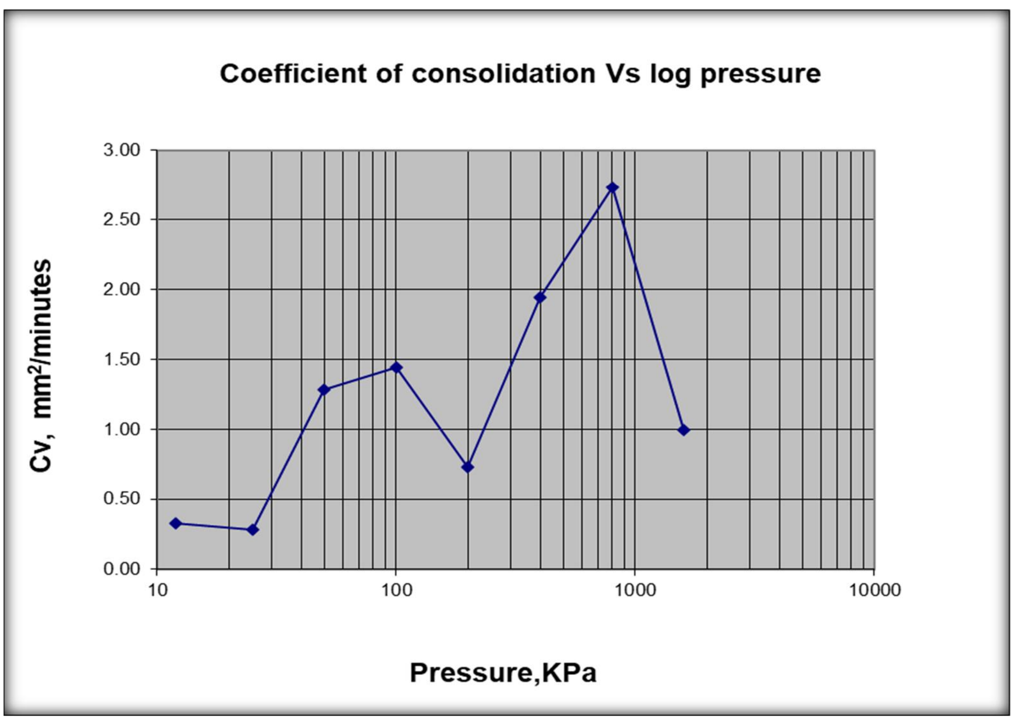


Figure A. 24 Consolidation test report (Depth at 3.0m for BH-2)