ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

FACULTY OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

COMPUTATION OF SOIL COMPRESSIBILITY USING TANGENT MODULUS APPROACH

A Thesis Submitted to the School of Graduate Studies, Addis Ababa University in Partial Fulfillment of the Requirement for the Degree of Master of Science in Civil Engineering

Dessalegn Sissay

Advisor: Dr. Ing. Samuel Tadesse

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DESSALEGN SISSAY

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Approved by Board of Examiners

Advisor	Signature	Date
External Examiner	Signature	Date
Internal Examiner	Signature	Date
Chairman	Signature	Date

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“I will thank You forever because of what You have done.” Ps. 52:9.

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Notations

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

Abstract

Keywords: tangent modulus, resistance, modulus number and strain

There are several mechanisms causing settlement of a structure. Different structures have varying potential to withstand the settlement that takes place Thus, its determination has to be given due consideration by engineers as it affects these structures from functioning properly if not limited to a permissible value.

Soils consist of solid particles and void space. Gas or water or combination of both occupies the void space. When loaded the water and air gets squeezed out resulting in settlement. Determination of settlement and the rate at which the expulsion takes place is determined from the theory of consolidation.

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. It is based on the resistance concept, which is very well developed in other field of engineering, except that its application for settlement analysis in soil mechanics is new.

1. Introduction

1.1 General

Construction scheme is advancing day to day. This has made possible construction of high rising buildings, heavy structures and highways. Due to this heavy loads are induced on to the soil underneath. Besides the use of undesirable construction site is increasing. The above factors results 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 sols 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. These and other assumptions limit the validity of the approach. Thus, another method of settlement evaluation which amends at least the above mentioned drawbacks have to be adopted.

This paper introduces another method as compared to classical method for the determination of settlement, which is known as the tangent modulus approach.

1.2 Background

The probable settlement any structure undergoes on different types of soils is predicted by compressibility study. Thus, based on the compressibility characteristics the method which relies on the stress- strain analysis of soil for determining this property is dealt with.

1.3 Objective of the Present Work

The objective of the research work is to:

i) Introduce a recent and more practical way of determining settlement and compare the applicability with the existing and more popular classical approach.

ii) To verify the applicability of the approach to the red clay soils of Addis Ababa

In order to come up with this, undisturbed samples are collected from the northern part of Addis Ababa and consolidation tests are performed.

In the thesis work chapter 2 describes some of the causes of settlement. In chapter 3 the basic theory of consolidation is presented. The classical approach of settlement analysis is described in chapter 4. Chapter 5 introduces the new approach, which is the tangent modulus of approach. Laboratory test results and discussions are presented in chapter 6. Chapter 7 is dedicated to conclusion and recommendation.

2. Settlement Analysis

2.1 General

Due to loading Compressive strains develop in the soil layer due to loading. This 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. 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 the fact that its elastic properties are constant for the given loading condition. If the load on the soil is increased above the causing its elastically compacted state appreciable residual deformation will appear in it [17].

There are several mechanisms which produce ground movement and further there are many types of structures each with a varying potential to withstand or to be distressed by movements.

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 presents the most difficult tasks [17].

Settlement estimations often receive less attention than they deserve (or even non at all), as a result excessive settlements cause far more problems than do bearing capacity failures [16].

In determining the allowable bearing capacity of clay soils for example, it is initially assumed to be equal to the ultimate bearing capacity divided by a suitable factor of safety. Settlement calculations should then be carried out. If the calculated settlements are too high then the value of allowable bearing pressure must be reduced to keep them within acceptable limits [16]. This shows that settlement analysis is one of the two most important types of analysis made by the soil engineer.

Soil deformation under loading is due to rolling, slipping and sliding and to some extent by crushing at particles contact point and elastic distortion. Some of the different causes of settlement are discussed below.

2.2 Causes of settlement

Soil conditions are susceptible to variation considerably from previous to during and after excavation for constructions. One of the causes of change of soil conditions is settlement. Soil is a non homogeneous porous material consisting of three phase: solids and fluid (normally water and air) Soil settlement is caused due to one or combination effect of the following. Soils are complex multiphase system particles whose deformation depends on both the total variations of volume and deformability.

2.2.1 Compaction

Compaction is a process where by the soil particles are forced into a closer state of packing with a corresponding reduction in volume and the explosion of air but with no change in the volume of water. This is usually effected by mechanical means such as rolling, tamping or vibrating the soil.

2.2.2 Consolidation

In saturated cohesive soils the effect of increasing the load is to squeeze out some of the pore water; this process is called consolidation. A gradual reduction in volume takes place until internal pore pressure equilibrium is reached; a reduction in loading may cause swelling, providing that the soil can remain saturated.

Due to increase of loading the pore water in the soil squeezes out. Time delayed consolidation is the reduction in volume associated with a reduction in water content. It occurs in all types of soils. Consolidation occurs quickly in coarse-grained soils such as sands and gravel. In such cases it is usually not distinguished from elastic deformation. In fine-grained soils such as clays and organic materials consolidation can be significant and take considerable time to complete. Thus the most susceptible soils are normally consolidated clays and silts, and certain types of saturated fill as in such soil type reduction in volume is gradual. To start the process of consolidation there should be a change in the loading condition in the soil. According to Terzaghi (1934) it is a decrease in water content of a saturated soil with out its replacement by air.

2.2.3 Elastic Distortion

When loaded, all solid materials undergo distortion. Soils being of a particulate nature, distort partly due to compaction or consolidation and partly also as a result of elastic distortion. The distortion caused by static loads is usually small, and it occurs essentially at the same time loads are applied to the soil. The settlement caused by this process is consequently termed immediate settlement. Undrained, saturated mass of soil subject to loading sows also elastic distortion.

2.2.4 Moisture Migration

Some types of clay soils show a marked increase or decrease in volume as the moisture content is respectively increased or decreased. These type of soils contain colloidal clay mineral such as montmorillonite that experience heave and shrinkage with changes in the water content [23]

2.2.5 Effect of ground water lowering

As water is pumped out from an excavation, the water table in the surrounding ground may be lowered. Settlement can result from this reduction in hydrostatic conditions due to two processes. The first is, in some clay a decrease in moisture content will result in a decrease in volume. The soil above the reduced ground water level may therefore shrink. The second is a reduction in hydrostatic pore pressure results in an increase of the effective overburden stress on the layer below. Accordingly, the soil (especially in soft clays or peat) beneath the reduced ground water level may be consolidated by the increase in effective stress [5].

2.2.6 Lose of Lateral support

A common form of foundation movement, often leading to serious, even catastrophic structural failure is associated with the excavation of deep hole alongside the foundation. The bearing capacity of the soil directly beneath a footing is dependent on the lateral support provided by the soil along side. If this lateral support is removed, as may occur in unprotected excavation, the likely outcome is a shear slip in the soil beneath the footing taking the footing in to excavation. Similarly settlement might occur as a result of movement of natural earth slopes or cuttings, due to sliding or flowing.

2.3 Components of settlement

Soils subjected to stress undergo strain within the soil skeleton. The total settlement is the sum of immediate settlement, primary consolidation settlement and secondary consolidation settlement.

Immediate settlement is predominant in all coarse grained and dry or partially saturated fine grained soils. It is due to the elastic deformation of soil with out any change in the soil water content.

In saturated and nearly saturated inorganic silts and clays primary consolidation settlement predominates. Primary consolidation settlement is time dependent. This is because it takes place due to pore water dissipation. It is a common behavior of soil with low coefficient of permeability. The length of time taken depends on how fast the excess pore pressure that develops due to the applied load dissipates. The coefficient of permeability and the distance the pore fluid being expelled from the voids travel play a significant role on the rate of pore water dissipation {15].

Secondary compression settlement is a form of soil creep that is largely controlled by rate at which the skeleton of compressible soils particularly clays, silts and peats 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, the primary and secondary consolidation settlements may occur simultaneously.

The aim of this paper is to present a recent and a more precise way of determining settlement and compare its applicability with the existing and more popular classical method.

Settlements can be evaluated based on different insitu and laboratory tests. The insitu tests include the standard penetration tests, the cone penetration tests, the dilatometers test and the pressure meter test

Oedometer and triaxial apparatus are used when settlement is to be determined from laboratory test results.

3 Theory of Consolidation

3.1 General

The intention of this paper is to present a different approach for the determination of settlement for different types of soils. Therefore it is worth to discuss first about consolidation of soils. Due to loading, the highly stressed zone will fail if the developed stresses are too large. Failure is defined as considerable alteration or state change in soil structure (or remolding) accompanied by substantial deformation and enlargement of the stressed zone until deformation eventually halts. The resulting total deformation is the deformation under stress up to failure plus the larger deformation occurring after failure. The soil strength after failure is termed the residual strength.

When fluid, usually water, is present in the soil void spaces the particle rolls and slips will be resisted by the pore fluid. The duration of pore fluid resistance will depend on the effective coefficient of permeability [15].

Soils consist of solid particles and void space, which is occupied by gas, water or combination of both. Due to this non-homogeneous nature stress conditions are more complicated. Stress is transmitted through the soil skeleton as a multitude of small forces acting at the point of contact between soil particles. This compressive stress distorts the soil grain. The distortion caused is immediate and is recoverable. Compressive stresses in saturated clay soils, besides the distortion, they cause dissipation of water that is loosely bounded to clay minerals. This process takes long time. Thus the process is a time dependent property of clay soil. The distortion is recoverable in that once the stress is removed the clay minerals re adsorb the water and swell. The displacement caused due to application of stress is that caused from grains moving closer together. It results in the reduction of volume of voids. This type is non-recoverable [16].

The rate at which the soil particles move closer due to the application of stress depends on the ground water condition. If the soil is dry or partially saturated the air in the void gets easily compressed. Thus, 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 dissipated, 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 where it drains quickly.

In saturated soils the soil skeleton cannot immediately take up the applied compressive stress. For a surcharge of infinite limit the applied stress is initially fully taken up by the pore water in the voids. Non-is carried by the soil grain. At time, t = 0 the effective stress $\Delta\sigma' = 0$

$$\Delta\sigma = \Delta u \qquad \text{...3.1}$$

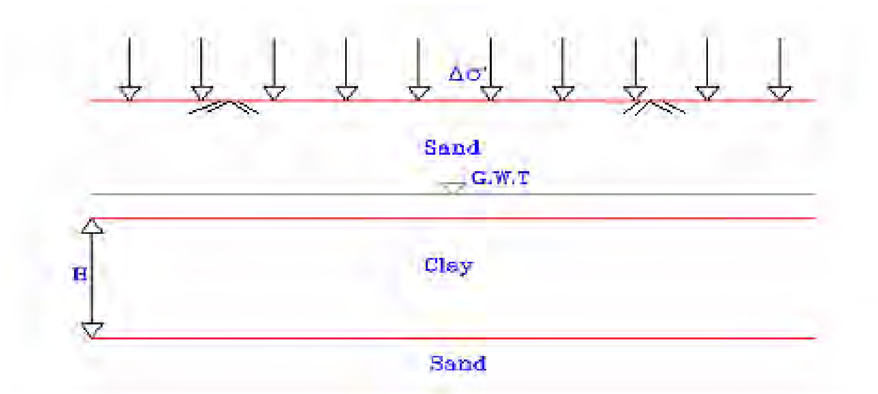


Figure 3.1 A clay layer confined between two permeable layers

After a certain time t > 0, the water in the voids squeezes out towards the highly permeable sand layer. Therefore, there is a decrease in pore pressure. The soil skeleton takes up this decrease. Thus

$$\Delta\sigma = \Delta\sigma ' + \Delta u \quad \text{.. } 3.2$$

This implies at t > 0, $\Delta\sigma > 0$ and $\Delta u < \Delta\sigma$.

The equation representing the consolidation process shall be described for the tangent modulus approach and for the classical consolidation theory separately in the forgoing discussion. When determining consolidation settlement the theory of consolidation is applied to soil. There are different factors affecting the consolidation of soil. Some of the factors affecting consolidation of soil are listed below.

3.2 Consolidation of soil

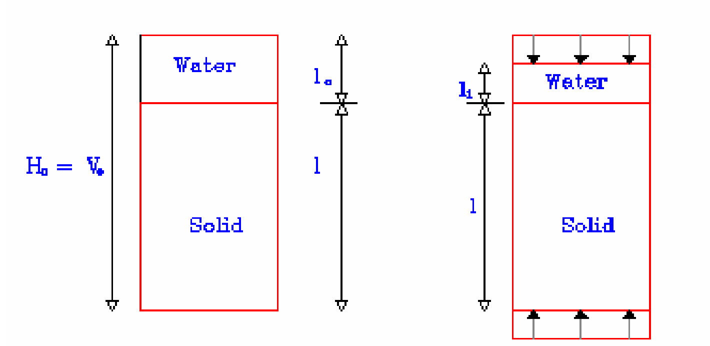
In the natural process of deposition, fine-grained soils undergo process of consolidation by the layer of soil deposited above. After a period of time a state of equilibrium is reached and compression ceases.

Due to application of external pressure on saturated clay excess pore water pressure develops. This induced excess pore water pressure initially takes up the applied external pressure.

The process of consolidation consists of gradual transfer of stress from pore water pressure to the soil skeleton. As the pore water pressure decreases the effective stress increases. In the process of consolidation the extent to which the transfer of stress has progressed is known as the degree of consolidation.

In the study of consolidation a soil may be considered as composed of selected fabric of mineral particles which are incompressible.

Considering the soil to be saturated the decrease in volume of soil is equal to volume of water squeezed out. This is in turn equal to the change in void ratio. Consider the model shown below:



a) Before loading b) after loading

Figure 3.2 Compressibility of soil:

In Figure 3.2 the characteristics of saturated soil before and after loading is shown. It can be observed that the volume of solid remains the same before and after loading.

The total vertical deformation of a soil layer under stress is a function of the following.

- Soil type
- Boundary conditions and magnitude of stress increase
- Sub soil boundary condition

When considering volume change of the soil layer attention has to be given to the following. They are

- The magnitude of the volume change
- The time required for the volume change that has taken place.

The rate of deformation is decided by:

- The volume of water that is squeezed out
- The boundary condition.

Compressibility of a soil will determine how much compression shall take place in the soil up on loading. Up on compression the soil particles rearrange themselves into a stable, and denser configuration as the pore water is forced out. The amount of rearrangement and the ensuring compression is a function of the rigidity of soil skeleton that is directly related to the soil type and its structure.

The compressibility of a soil is subject to influence by its geologic origin and stress history. Further for a particular soil deposit, compressibility is related to the magnitude of effective stress.

3.2.1 Effect of soil type

Granular materials exhibit compressibility behavior which is quite distinct from that of clay. 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 with in a short duration. The behavior of sand solely answers a question why a structure on sand soil experiences very little settlement after it has been constructed. Most of the settlements have already taken place by the time the structure is complete. Therefore, in the case of granular soil for estimating settlement it is relied on the field test than on laboratory tests.

For saturated fine-grained soils the major factor in the escape of pore water from the soil is the time required. When compared to granular soils where expulsion of pore water takes place unimpeded with a small time lag,: much longer time is needed in fine-grained soils for pore water to escape. This causes a considerable time lag between the load application and the resulting deformation.

The basic difference in the compression behavior of a granular soil and that of a fine-grained soil can therefore be expressed as granular soil compresses almost immediately up on loading. But the compression is relatively small where as fine-grained soil exhibits time dependent compression and the resulting compression is rather large. The magnitude of time lag is basically influenced by the permeability of the soil.

3.2.2 Permeability of the soil

If the pore water is at rest, the distribution of pore water pressure must be hydrostatic. Conversely any localized change in pore water pressure from the hydrostatic value will cause water to flow through the voids between the soil particles.

The rate of volume change is related directly to the permeability of soil. The amount of deformation per unit increase in stress depends on property of the soil called permeability. The permeability of a soil plays a significant role on the time taken for the process of consolidation.

The drainage of water in clay takes longer time when compared with that of free draining saturated sand. This is because the permeability of clay is tens of thousands to million 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.

3.2.3 Role of stress history

Soils tend to retain the effects of stress changes that have taken place in their geological history in the form of their structure. A soil which is subjected to a certain effective stress for the first time in its history will obviously be more compressible than when it has been subjected to a large effective stress in its earlier history but is now relieved of that effective stress due to some reason. When a soil is stressed to a level greater than the maximum stress to which it was subjected to in the past, some kind of break down in the soil structure occurs resulting in a much higher compressibility.

3.2.3.1 Normally Consolidated soils

A soil is normally consolidated when the existing effective stress σ' is the maximum that the soil has ever experienced in its history. In other words it is a soil which is subjected to an insitu effective vertical overburden pressure. These types of soils are sensitive to the effects of disturbance. The disturbance affects the behavior of the soil.

3.2.3.2 Over consolidated clays

A soil is over consolidated or pre-consolidated if the existing effective stress is less than the pre-consolidation stress ($\sigma' < \sigma_c'$). Pre-consolidation of a soil stratum may occur due to several reasons. For example a clay stratum that originally consolidated under a large pressure, but latter has been relieved of some load following some erosion of the overburden. Hence a change in total stress caused the effective stress to change and converted a normally consolidated soil to an over consolidated one. Change in pore water pressure due to desiccation of the upper layers as a result of surface drying, change in elevation of water table, desiccation due to plant life and, removal of construction are some of the factors responsible for pre-consolidation

3.2.4 Effect of effective stress

Due to application of stress to a saturated soil, the water in the pore starts to flow out. The flow continues as long as there is hydraulic gradient. As this transient flow continuous there will be reduction in volume causing an increase in effective stress. This volume reduction is related directly to the effective stress.

Free water and/or gas bubbles in the voids of mineral soils cannot transfer shear stresses. Hence the soil skeleton alone transfers shear stresses. Thus the effective normal stresses govern the internal resistance of granular soils irrespective of the shear stress.

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 of course less than the decrease in initial void ratio. This means compressibility of soil decreases as effective stress increases.

4. Classical Approach of Settlement Analysis

4.1 General

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 bed rock. The stretching in the horizontal direction is restrained by horizontal shearing resistance that develops at the top and bottom. [20].

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.

4.2 Settlement computation

From the conslidometer, 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 some what curved with a flat shape, followed by a linear relationship with a steeper slope.

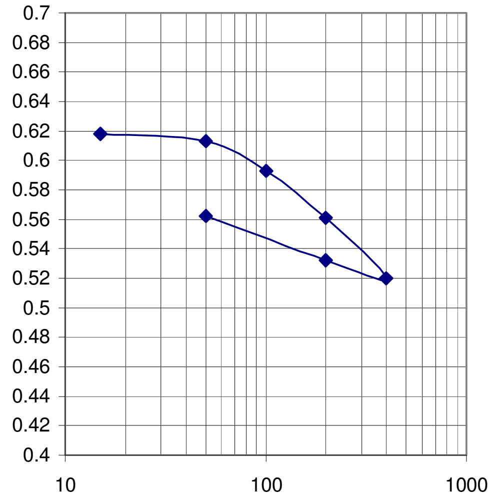


Figure 4.1 Loading –unloading curve for void ratio – stresses

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 σ' 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:

$$\frac{\Delta V}{V_o} = \frac{\Delta H}{H_o} = \frac{\Delta e}{1+e_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 4.1$$

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

$$\Delta H = \frac{\Delta e}{1+e_o} H_o \quad \ldots\ldots\ldots\ldots 4.2$$

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

$$C_C = \frac{\Delta e}{Log(\frac{\sigma'}{\sigma_o'})} \quad \ldots\ldots\ldots\ldots 4.3$$

Combining equations (4.2) and (4.3)

$$\Delta H = \frac{\Delta e}{1+e_o} HoLog(\frac{\sigma'}{\sigma_o'}) \quad \ldots\ldots\ldots\ldots 4.4$$

4.3Time 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 $\partial e/\partial \sigma'$ 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 3.1, 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 4.2.

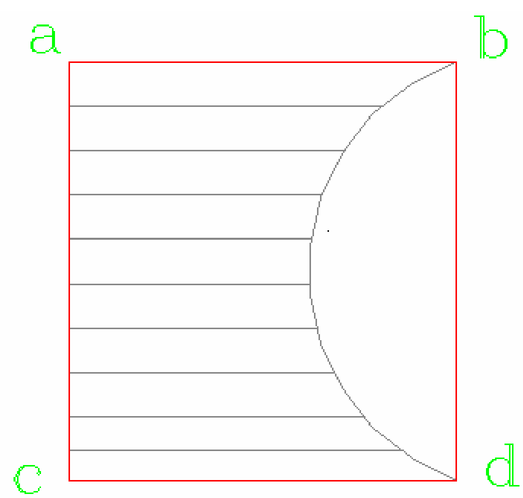


Figure 4.2 Distribution of excess pore water pressure and effective stress in clay layer confined between two permeable layers.

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 out side 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 4.3 shows 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

$$q_i = VA = KiA \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. 4.5$$

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

Out flow

$$q_o = -k(\frac{\partial h}{\partial z})dxdy - k\frac{\partial}{\partial z}(\frac{\partial h}{\partial z})dzdxdy \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..4.6$$

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

$$q_o - q_i = -k\frac{\partial^2 h}{\partial z^2}dxdydz \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots4.7$$

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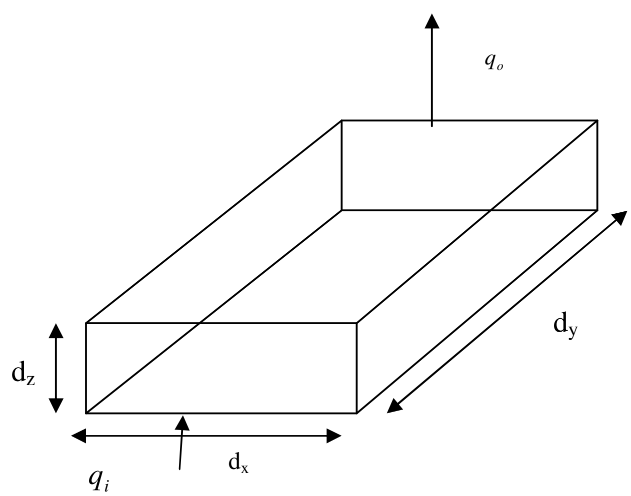


Figure 4.3 Flow or water through prismatic portion of soil element

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

$$\frac{\partial V}{\partial t} = \frac{\partial V_v}{\partial t} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..\ldots\ldots\ldots\ldots..4.8$$

But assuming that soil solids are incompressible $\frac{\partial V}{\partial t}$ =0

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

Substituting for $\frac{\partial V_s}{\partial t}$ and V_s s in equation.4.6, yields,

$$\frac{\partial V}{\partial t} = \frac{dxdydz}{(1+e_o)} \frac{\partial e}{\partial t} \quad \text{.........} \quad 4.9$$

Combining equations 4.4 & 4.6

$$\frac{-k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{1}{(1+e_o)} \frac{\partial e}{\partial t} \quad \text{.........} 4.10$$

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

$$\frac{\partial e}{\partial \sigma'} = a_v \text{.........} 4.11$$

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

Hence.

$$\partial e = -a_v \partial u \text{.........} 4.12$$

Combining equations.4.7 and .4.9

$$\frac{-k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{a_v}{(1+e_o)} \frac{\partial u}{\partial t} \quad \text{.........} 4.13$$

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.

$$\frac{\partial u}{\partial t} = \frac{k}{(\gamma_w m_v)} (\frac{\partial^2 u}{\partial z^2}) = cv \frac{\partial^2 u}{\partial z^2} \text{.........} 4.14$$

Equation 4.11 is the basic differential equation of Terzaghi's consolidation theory, which can be solved with boundary conditions.

4.4 Determination of the pre-consolidation pressure

The value of σ_p' 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 σ_p' 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 σp'.

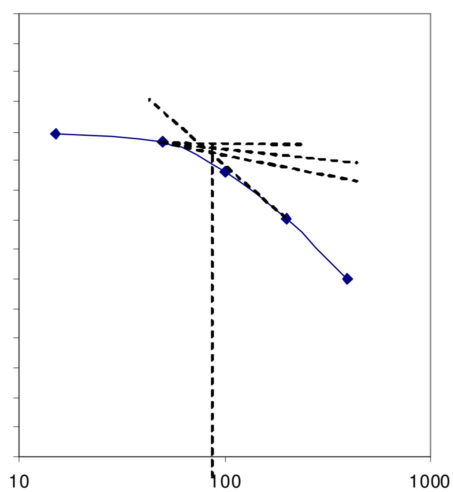


Figure4.4 Casagrande's method of determining pre-consolidation stress.

5. The Tangent Modulus approach

5.1 General

The tangent modulus approach of settlement determination is suggested initially by Janbu [11], and Stamatopoulos and Kotzias [1]. 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.

5.2 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.

$$\text{Resistance} = \frac{\text{Increamental causes}\left[\text{given}\right]}{\text{Increamental response}\left[\text{measured}\right]}$$

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.

$$\text{Electric resisance, R} = \frac{\text{Potential change}}{\text{Current change}}$$

$$\text{Elastic resisance, E} = \frac{\text{Stress change}}{\text{Strain change}}$$

$$\text{Dynamic resisance, m} = \frac{\text{Force change}}{\text{Acceleration change}}$$

$$\text{Hydraulic resisance, } k^{-1} = \frac{\text{Gradient change}}{\text{Velocity change}}$$

$$\text{Heat resisance, C} = \frac{\text{Heat change}}{\text{Temperature change}}$$

According to the out come of research work at the Technical University of Norway[12], 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.

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

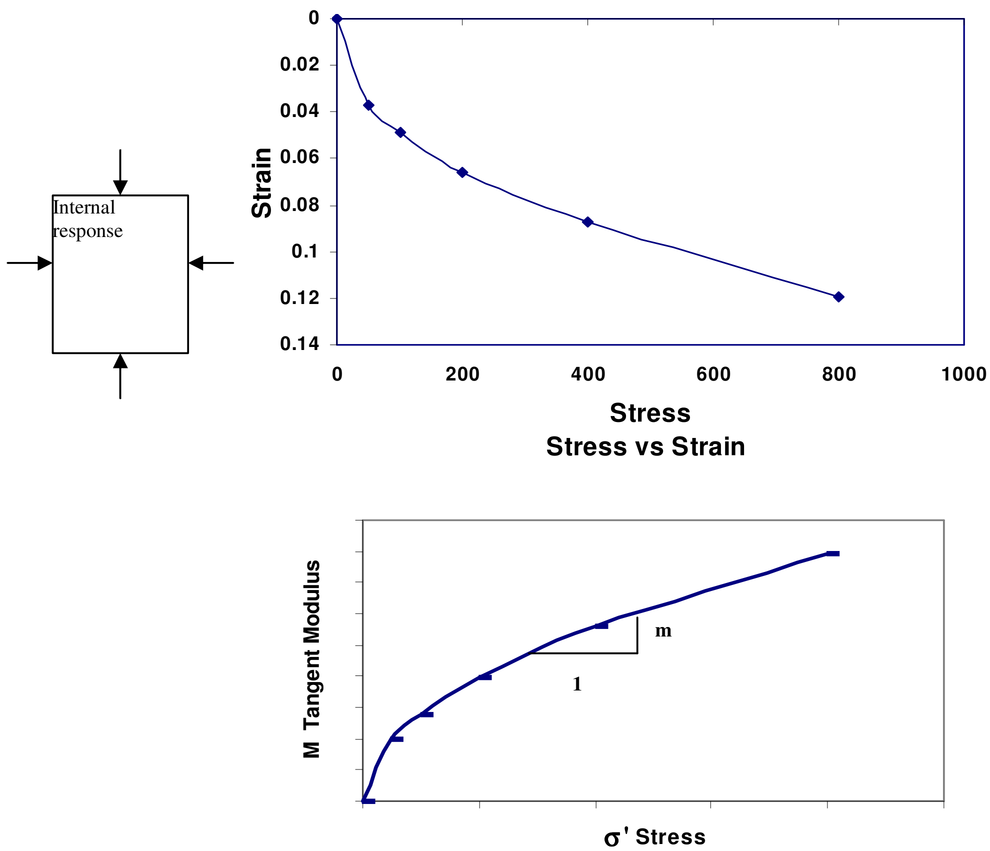


Figure 5.1 Stress-strain curve and tangent modulus–stress curve

$$M = \frac{d\sigma}{d\varepsilon} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.1$$

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.

$$\text{Resistance} = \frac{\text{Stress change}}{\text{Strain change}} \quad \ldots\ldots\ldots\ldots 4.2$$

5.3 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. [13] 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 apparatus 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) [13].

The boundary conditions of both tests can be explained by means of Figure 5.2

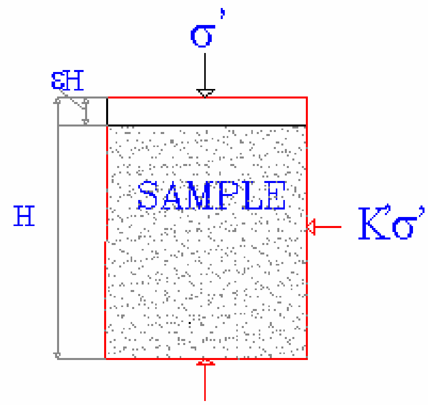


Figure 5.2 Soil Sample under Laboratory test-

To easily understand and to interpret the results reasonably it is worth observing how both are related with the field condition.

The oedometer tests correspond to no lateral yield, or plain strain hence

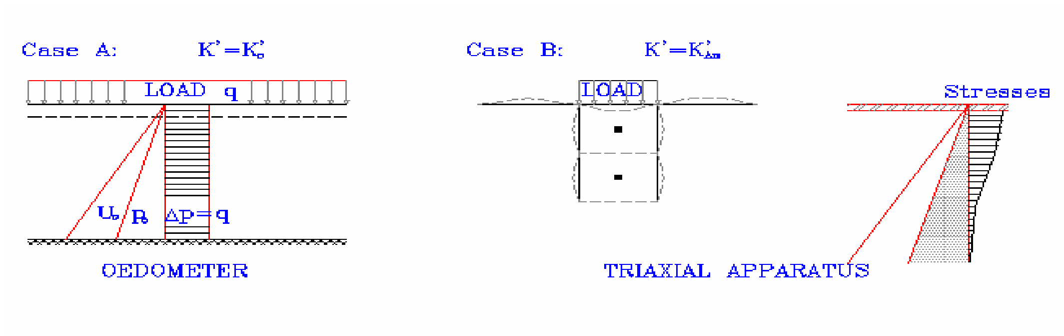
$$K' = K'_o \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.3$$

Where K'_o is effective earth pressure coefficient at rest. Such tests correspond to the field conditions shown in case A of Figure 5.3

In the triaxial tests the lateral pressure conditions can be chosen so as to simulate a larger variety of field conditions. In this case $K' < K'_o$, thus a common procedure used at the Technical University of Norway[12], can be adopted and reads as follows.

$$K' = K'_{An} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.4$$

Where the K'_{An} is nominal active earth pressure coefficient corresponding to the actual design factor of safety with respect to shear failure under the applied load.



a) oedometer b) Triaxial apparatus

Figure 5.3 Simulation of Field Condition

Unlike that of *Case A*, *Case B* of Figure 5.3 can not be exactly simulated by triaxial tests because the principal stress directions under the load are not exactly horizontal and vertical. But it is believed that the only practical and manageable way of approaching the problem is to average up and equivalent the three dimensional stress conditions in terms of principal stress to be used in a triaxial apparatus. The other alternative would be to treat the effect of each principal stress components separately, but since the principle of superposition do not apply for non linear stress strain such an approach would in general be enormously complicated and hence of restricted practical value [13].

From large number of oedometer and triaxial tests on different types of soils in Norway by N. Janbu[11] and A.C Stanmatopoulos and P.C Kotzias[1] 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 [1]

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 break 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.

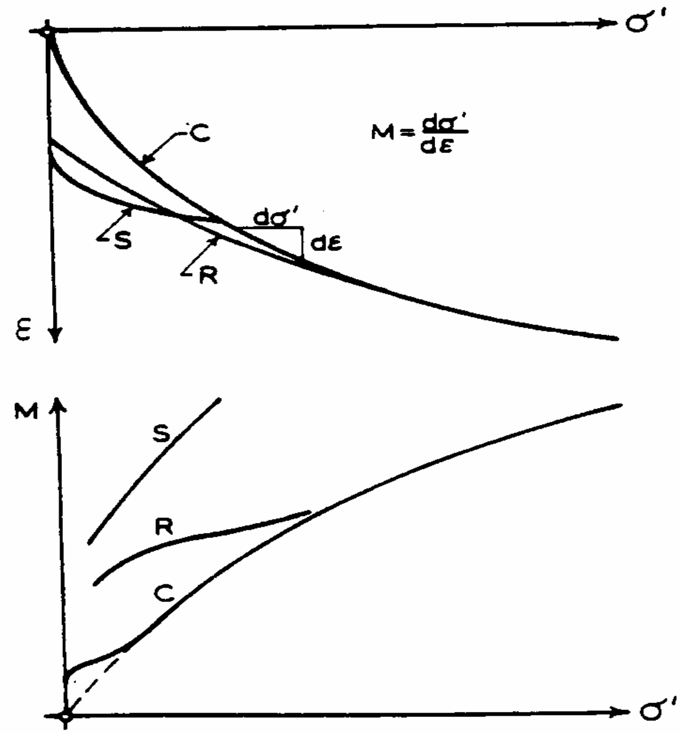
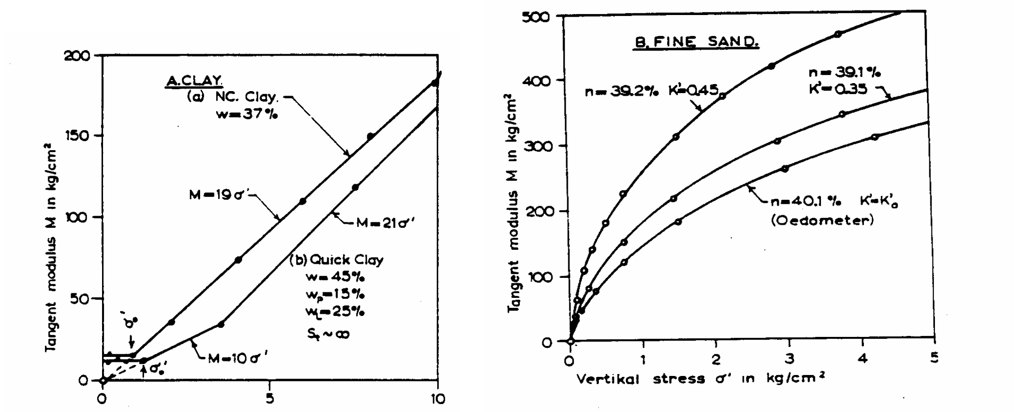


Figure 5.4 Stress strain curve and tangent modulus for virgin compression (C) swelling (S) and recompression[11]

Figure 5.4 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.

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 5.5 Examples of tangent modulus variations with stress[11]

Figure 5.5 show some typical results from oedometer tests on undisturbed Norwegian clay as compiled and explained by professor N.Janbu[11].

Figure 5.5-(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

$$M= m\sigma' \quad \ldots\ldots\ldots\ldots \quad 5.5$$

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 5.5-(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 $3\sigma_o$ 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 [14].

Figure 5.5-(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 [14].

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 drops abruptly where σ'equals to σ_o'. This drop probably corresponds to a collapse of the metastable clay skeleton in the quick clays. When $\sigma'>\sigma_o'$ the modules again increases, but now somewhat faster than σ'.

For over consolidated quick clays, OCQ, the modules remains practically constant up to the pre consolidation load σ_c'.

If the difference σ_c'-σ_o' is low (say 20-100KN/m^2) 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.

Based on the soil type and stress history the relationship in Equation 5.6 can be represented as shown in Figure 5.6.

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 change with σ' in a different fashion for this different types of soils. Thus, M can be descried as a function of the effective stress.

$$M = f(\sigma') \qquad \ldots\ldots 5..6$$

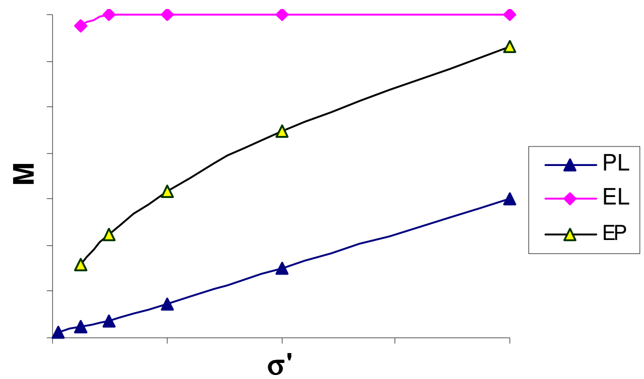


Figure 5.6 Principal types of M=f(σ')

The correlation of the tangent modulus is shown in equation 5.7. 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 5.6. Janbu[12] suggested a general formula to adequately cover all types of soil.

$$M = m\sigma_a \left(\frac{\sigma'}{\sigma_a} \right)^{1-a} \quad \text{.................................} \quad 5.7$$

The reference pressure σ_a=100Kpa is introduced solely for the purpose of obtaining a dimensionally compatible expression.

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 4.6 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 5.7.

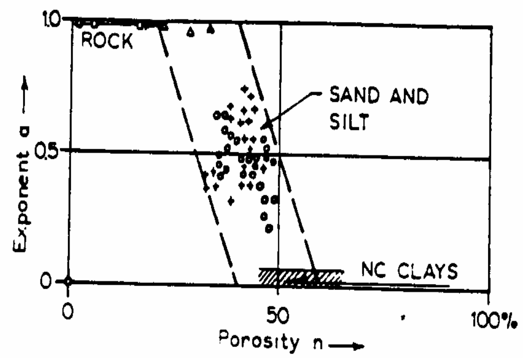


Figure 5.7 Variation of a with porosity[12]

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 5.6 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.

$$M = m\sigma_a \quad \text{......} \quad 5.8$$

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

Type EP is defined by a=0.5 corresponding to

$$M = m\sigma_a'\left\{\frac{\sigma'}{\sigma_a}\right\}^{1/2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.9$$

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

$$M = m\sigma' \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.10$$

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.

5.4 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 ε seen to be

$$d\delta = \varepsilon dz \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.11$$

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

$$\delta = \int_0^H \varepsilon dz \qquad 5.12$$

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 clay.

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 $d\varepsilon = \frac{d\sigma'}{M}$.

Therefore, as the effective stress increases from the overburden pressure σ_{ov}' until its final value $\sigma_{ov}'+\Delta\sigma'$ the resulting strain is obtained by integrating Equation 5.12.

$$\varepsilon = \int_{\sigma_o'}^{\sigma_o'+\Delta\sigma} \frac{d\sigma'}{dM} \qquad 5.13$$

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 $m\sigma_a[\sigma'/\sigma_a]^{1-a}$ ..

Introducing the value of M in to Equation 5.13 the following generalized strain formula is obtained [12].

$$\varepsilon = \frac{1}{ma\left[((\sigma_o'+\Delta\sigma)/\sigma_a)^a - (\sigma_o'/\sigma_a)^a\right]} \qquad 5.14.$$

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 wher e a=0.5, $\varepsilon=\frac{2}{m}\left\{\sqrt{\frac{\sigma_o'+\Delta\sigma}{\sigma_a}} - \sqrt{\frac{\sigma_o'}{\sigma_a}}\right)$

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

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[12] 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 It is moreover a simple matter to prove that:

$$m = \frac{1+e_o}{c_c}\ln 10 \quad \ldots\ldots 5.15$$

5.5 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 [13].

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 [13].

Thus the plot of degree of consolidation U 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 [12].

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 ε- distribution that determines the time rate of consolidation.

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

$$\varepsilon = f(t, z) \qquad \text{5.16}$$

The first step in obtaining a practical solution for Equation 5.16 must be to derive a differential equation on the basis of strain, instead of pore pressure Janbu[12] quite independent have obtained such equation, and probably at about the same time. Short out line shall be presented below.

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

$$\frac{\partial \varepsilon}{\partial t} = \frac{\partial v}{\partial t} \qquad \text{5.17}$$

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

$$v = \frac{c_v \partial\varepsilon}{(\partial z - v_o)} \quad \text{...5.18}$$

Where coefficient of consolidation c_v is given by the equation

$$c_v = \frac{m}{\gamma} k \quad \text{... .5.19}$$

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.

$$v_o = k\frac{(i_o + 1)}{\gamma_w} dq / dz \quad \text{..5.20}$$

From Equation 5.18 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

$$v_o = c_v d\varepsilon_1 / dz \quad \text{...5.21}$$

where ε_1 is strain for t = ∞.,
Inserting equation 5.18 into equation 5.17 the differential equation governing the consolidation process itself is obtained.

$$\partial e / \partial v = \partial / \partial z (c_v \partial\varepsilon / \partial z) - v_o / \partial z \quad \text{...................................5.22}$$

The degree of consolidation in terms of strain reads as

$$U = \frac{\int \varepsilon \, d\varepsilon}{\int \varepsilon_p \, d\varepsilon} \quad \text{..5.23}$$

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 as it is observed from the laboratory tests conducted.

6 Laboratory test results

6.1 General

To verify whether the relation developed for the normally consolidated soils holds true for the red clay soils of Addis Ababa, oedometer tests were carried out collecting samples from one of the area where red clay soils dominant. Before discussing about the test, it is important to describe about the origin and mineralogical content of on the red clay soils of Ethiopia.

Ethiopian red clay soils are mainly residual and derived from the weathering of volcanic rocks. Residual soil is one, which was formed on its present location through weathering of the parent or bedrock. These soils are widespread in tropical areas where they are termed laterites. Olivine basalt, basalt and trachyte are the parent rock for black and red clays The red color of Ethiopia soils indicate the present of iron.

Red clay soils are developed where rainfall is plenty and there is good drainage. The principal clay minerals in red clay soil are kaolenite and hallaysite. Even though the fore mentioned minerals are predominant montimorillonite is also frequently present in significant amount.
Montmorillonite formed initially in all Ethiopian soils developed over basalt. In many Ethiopian soils transformation of montimorllonite to kaolinite has taken place and is continuing at present. Kaolinite and halloysite are predominant clay minerals of red soils.
This indicates that alteration of montimorillonite is more complete in these soils.

The abundance of montmorillonite even in the red clay soils indicates that Ethiopian soils have not reached the degree of maturity that other tropical soils of Africa have attained.
The Ethiopian soils have similar pH as that of the African soils. The black clays are always basic and the red clays are always acidic.
Due to greater amount of kaolinite and halloysite as compared to montimorillonite red clay soils are less expansive than the black clay soils.

Basically when conducting the test on the red 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 undisturbed 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 execs 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 relation ship 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.

6.2 Interpretation of the test results

This test method uses conventional consolidation theory based on Terzagh's consolidation equation to compute the coefficient of consolidation. The test result has to be interpreted finally after completion of the test. 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.

6.2.1 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 provide a straight line portion which again is important for determining the compression index which is one of the two important consolidation parameter. 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 insitu 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).

$$OCR = \frac{\sigma'_p}{\sigma'_{ov}} \text{ ....(.6.1)}$$

- At the end after a stress increased loading beyond the curvature becomes some what straight line.

The plot of the laboratory test results for the red clay soil of Addis Ababa is plotted on the semi log and arithmetic scale and is shown in Figure 6.1 to Figure 6.22.

6.2.2 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 clay layer using both the tangent modulus approach and the classical approach. The modulus number m, Cc 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.

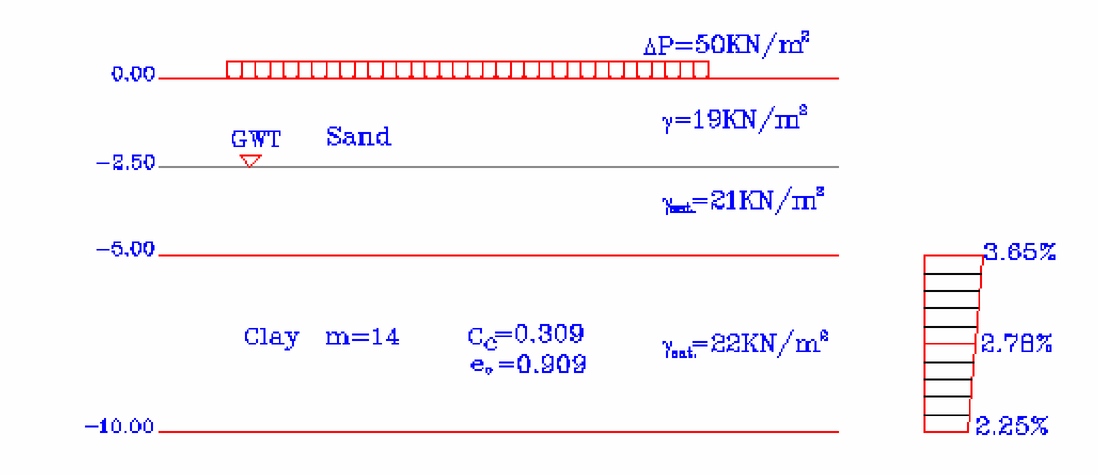


Figure E 6.1 Soil Profile

Additional stress =50KN/m^2

Initial effective stress in the clay layer

$P_{1\ (\text{At the top})}$ =19x2.5+(21-10)x2.5

=75KN/m^2

$P_{1\ (\text{At the middle})}$=75+(22-10)x2.5

=105KN/m^2

$P_{1\ (\text{At the bottom})}$=105+(22-10)x2.5

= 135KN/m^2

Average effective stress =1/6x (75+4x105+135)

=105KN/m^2

After application of additional stress

$P_2= \Delta P+P_1$

$P_{2(\text{ At the top})}$=50+75

=125KN/m^2

$P_{2(\text{ At the middle})}$=50+105

=155KN/m^2

$P_{2(\text{ At the bottom})}$=50+135

=185KN/m^2

1. Classical Approachp

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

Inserting the values of P_2 and P_1 at the middle of the clay layer the resulting settlement is ΔH=13.68cm.

2.0 Tangent Modulus Approach

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

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

$\varepsilon_{(\text{at the top})}$ =0.0365 =3.65%

$\varepsilon_{(\text{at the middle})}$ =0.0278 = 2.78%

$\varepsilon_{(\text{at the bottom})}$ =0.0225 =2.25%

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

ΔH=0.0225x500+0.5x (0.0365-0.0225) x500cm

=14.75cm

6.3 Similarities and Differences

The similarities of the two procedures are that both graphs show some what 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 void ratio 'e' or 'C_c' as an intermediate step in the calculation of settlement. The third difference is that the scales on which the graphs are plotted.

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_c', using Cassagrande approach is highly dependent on the human element and is empirical.
- The laboratory correction for field disturbance is not rational.
- The determination of the initial void ratio. e_o is both tedious and subject to many variables. For instance, it requires knowledge of the specific gravity of the soil and end of test water content. It also presumes that the water content al the end of the test is full saturation water content. It is common knowledge that it is very sensitive to both water content and specific gravity. Besides it is generally not the sample value at the start of the consolidation test since some swell has occurred from loss of in situ overburden pressure.
- 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, Janbu(1965 1998) amply 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.