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Pore water pressure behaviour and evolution in clays and its influence in the consolidation process

Thesis submitted for examination for the degree of Master´s Programme in Building Technology

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Abstract

Consolidation is linked to changes in effective stress, which result from changes in pore water pressure. Upon application of an external or internal load, there is an increase in pore water pressure throughout the sample known as excess pore water pressure. Accordingly, flow takes place due to the hydraulic gradient generated by the initial excess pore pressure distribution. At any stage of the consolidation process, the pore water pressures will vary within the soil layer. Terzaghi's one-dimensional consolidation theory is commonly adopted to describe the dissipation of excess pore water pressure within a consolidating soil over time.

The tests performed in this study are a variant of the standard incrementally loaded oedometer test. With these tests, it is intended to observe precisely how pore water pressures develop in a soil sample, during the loading process. One of the objectives is to determine exactly what is happening during load steps: pore water pressure values, times required, and how its dissipation develops. In the same way, this allows to know better how settlements generate, as well as any other kind of effects. Another objective is to study the potentials of this testing method, and to point out possible disadvantages of the current testing equipment and method, and to suggest future modifications. The chosen material is a soft clay from Ossinlampi (Otaniemi), taken from the vicinity of Aalto-University's laboratory of soil mechanics and foundation engineering in Espoo, and a total of twelve tests have been carried out.

The results show that the development of pore water pressures present a behavior somewhat different to that generally assumed. These differences are manifested mainly in the very early phase after the application of the load on the soil sample. The transfer of the total pressure to the water particles does not occur instantaneously but requires a period of time to be completed which is generally around 10 seconds. In addition, not all of this load is transferred to the sample, but values close to 90 % of the load.

The test is relatively new, and does not consist of a defined procedure, so it sometimes requires more time and preparation. The sample and cell preparations are the most problematic phases, which may cause some problems. However, it has many applications in this field of geotechnics.

Keywords Pore water pressure, oedometer test, consolidation, Pore pressure increment

Preface

This thesis was made in Aalto University´s School of Engineering, at the Department of Civil Engineering. A research work on the behavior of clay has been carried out in the laboratory of foundation engineering and soil mechanics. This work continues a series of studies related to Samuli Laaksonen's Master Thesis "Improvement of research methods and testing equipment for deformation properties of clay", 2014. The work was supervised by Professor Leena Korkiala-Tanttu, and doctoral student Monica Löfman was the advisor. My greatest thanks go to Professor Leena Korkiala-Tanttu for the supervisor task and to Monica Löfman, whose advice and guidance have made this work possible. In addition to the already aforementioned people, a large part of the work has been done by Matti Ristimäki, whose help on the performance of experiments has been vital. Thanks also to the staff of the base building and the soil mechanics laboratory for help and advice with the tests. In addition, I would like to thank my family, who gave me the opportunity to study in Helsinki, my friends' support during the work, and specially my friend Alberto.

Espoo 28.06.2017

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Markings

A	[m^2]	Cross section area
C_C	[-]	Compression index
C_S	[-]	Swelling index
Gs	[-]	Relative density of particles
H	[m]	Height or thickness
IP	[-]	Plasticity index
Q	[m^3/s]	Flow
T	[°C]	Temperature
T_v	[-]	Time factor
U_z	[-]	Consolidation degree
$U_{z,t}$	[kPa]	Excess pore water pressure at *z* point, at *t* time
U_0	[kPa]	Initial excess pore pressure
V	[m^3]	Total volume
V_g	[m^3]	Gas volume
V_s	[m^3]	Solid phase volume
V_v	[m^3]	Volume of voids
V_w	[m^3]	Water volume
V_z	[m/s]	Vertical velocity of the flow entering the element
$V_{(z+dz)}$	[m/s]	Vertical velocity of the flow leaving the element
W	[kN]	Total weight
W_g	[kN]	Gas weight (Considered zero)
W_s	[kN]	Weight of solid particles
W_{sat}	[kN]	Weight of water when the sample issaturated
W_w	[kN]	Water weight
a_v	[1/kPa]	Compressibility coefficient
c_v	[m^2/year]	Coefficient of consolidation
d	[m]	Drainage path
d_{50}	[m]	Drainage path at 50% of time
d_{90}	[m]	Drainage path at 90% of time
dz	[m]	Differential element of soil
e	[-]	Void ratio
g	[m/s^2]	Gravity acceleration
h	[m]	Hydraulic load
h_p	[m]	Piezometric height
h_e	[m]	Piezometric height over water table
h_h	[m]	Piezometric height under water table
i	[-]	Hydraulic gradient
k	[m/s]	Coefficient of permeability
n	[%]	Porosity
t	[s]	Time
t_{50}	[s]	Time at U = 50%
t_{90}	[s]	Time at U = 90%

u	[kPa]	Pore water pressure
u_e	[kPa]	Pore water pressure over water table
u_h	[kPa]	Pore water pressure under water table
v	[m/s]	Flow velocity (Considered zero)
w	[%]	Water content
w_L	[%]	Liquid limit
w_P	[%]	Plastic limit
w_S	[%]	Retraction limit
z	[m]	Load or position with respect the reference plane
γ	[kN/ m^3]	Unit weight or apparent unit weight
γ'	[kN/ m^3]	Submerged unit weight
γ_d	[kN/ m^3]	Dry unit weight
γ_{sat}	[kN/ m^3]	Saturated unit weight
γ_w	[kN/ m^3]	Water unit weight
ε_1	[%]	Deformation in principal axis 1
ε_2	[%]	Deformation in principal axis 2
ε_3	[%]	Deformation in principal axis 3
ε_x	[%]	Deformation in *x* direction
ε_y	[%]	Deformation in *y* direction
ε_z	[%]	Deformation in *z* direction
σ	[kPa]	Total pressure
σ'	[kPa]	Effective pressure
σ_0	[kPa]	Total initial vertical stress
σ'_0	[kPa]	Effective initial stress
σ_1	[kPa]	Stress in principal axis 1
σ_2	[kPa]	Stress in principal axis 2
σ_3	[kPa]	Stress in principal axis 3
σ'_p	[kPa]	Preconsolidation pressure
μ	[kg/(m.s)]	Viscosity coefficient
ρ_w	[kg/ m^3]	Water density

1. Introduction

All materials, when subjected to changes in stress conditions, undergo deformations, which may or may not be time dependent. The relationships between stresses, deformations and time vary according to the material to be analyzed.

1.1. Introduction to the problem

The deformations of the soil due to the application of an external load are the result of a decrease in the total volume of the soil mass. Specifically, it is a reduction of the volume of voids, since the volume of solids is constant; therefore, such deformations are product of a decrease of the relation of voids of the soil as shown in Figure 1.

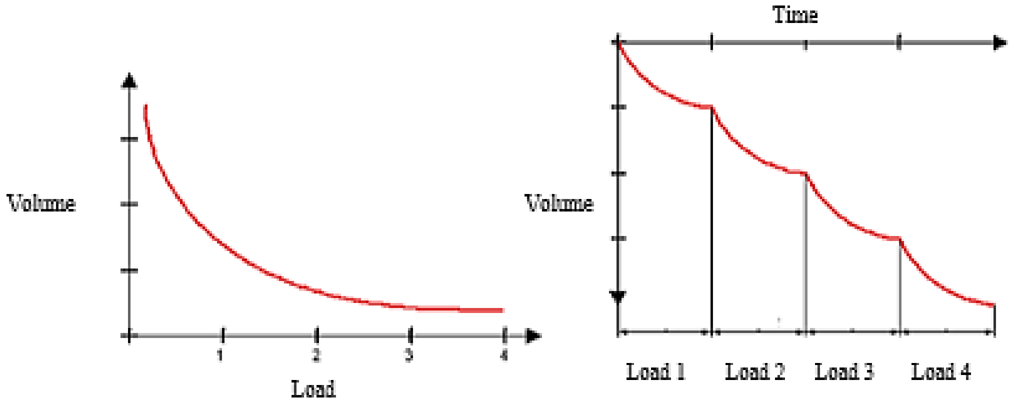

Figure 1. Variation of volume during consolidation (Poliotti & Sierra, 2001).

If these voids are filled with water (saturated soil), considering the fluid incompressible, this decrease in the void ratio is only possible if the volume of liquid decreases, therefore a flow of liquid to some permeable layer is taking place. The expulsion of the water, and its corresponding change of volume imply a change in the tensional stress of the soil. Excess pore water pressure or excess pore pressure, since soil will be considered always saturated, is transferred to soil particles, which means effective pressure (Poliotti & Sierra, 2001).

In cohesive soils, like clays to be tested, permeability is very low, so the water flow is very slow, and the dissipation of excess pore water pressure is likewise very slow. Consequently, the soil can continue deforming for years, even decades, because effective pressure affects the soil, not total pressure (Badillo & Rodríguez, 1967).

Therefore, the development of pore pressures will mark the behavior of the soil. Knowing how these pressures behave during the different phases of a load step will help to better understand their evolution over time.

1.2. Scope and objectives

This work continues the studies initiated by Samuli Laaksonen in his Master's Thesis "Improvement of research methods and testing equipment for deformation properties of clay", 2014. The present work concentrates in the use of a new pore water oedometer test method based on the Standard Oedometer Test. This method allows measuring the pore pressures of the soil samples during the application of an external load. The objective is to test several soil samples and to analyze the results related to pore pressure. With these tests it is possible to know what happens to pore pressure when a soil sample is subjected to pressure increases, and therefore, to analyze these results with the theoretical bases of geotechnics.

At the same time, by obtaining all the necessary information, it is possible to learn to know the test equipment and method better. Determining how it behaves, the best way to use it, precautions that must be taken into account, as well as possible complementary uses it may have. It is important to become familiar with the equipment in order to be able to detect where and how mistakes can be made, how to avoid them, and thus, make a more efficient use of the equipment. With all this information, the objective is to develop a list of possible suggestions for future uses, as well as a possible working method.

1.3. Thesis outline

The development of the thesis is divided into a series of chapters, beginning with the theoretical framework until the analysis of the results obtained.

First, in Chapter 2, the necessary information for the development and understanding of laboratory tests has been compiled. This literature review ranges from the most basic and general one, like index properties, until reaching the most specific concepts related to the work, such as consolidation curves.

Once the necessary information and theory has been presented, the following section, Chapter 3, focuses on explaining in detail how experiments are performed. It begins explaining the Standard Oedometer Test, and then describes in detail how pore water pressure oedometer tests were performed. It explains what is related to the laboratory equipment, how to prepare and how to test the specimens.

The following section (Chapter 4) shows the results obtained in the tests, as well as their analysis. These results are analyzed taking into account different parameters such as time or pore pressure itself.

The section corresponding to Chapter 5 is a compilation of the conclusions and expertise gained in the use of the laboratory equipment. This section provides a list of possible measures or recommendations to improve the use of equipment in future uses, as well as the performance of the tests and their analysis.

1.4. Limitations

During the execution of the thesis, it has been necessary to modify some aspects of the content due to a series of limiting factors that have arisen. The initial idea was to test various types of clay (stiff and soft clays) and thus, to compare their results. The first tests that were carried out were with stiff clay (Kimola Clay). However, the results obtained were not as desired, since the readings were not recorded well. The material was somewhat irregular, since it contained small stones that altered the internal structure of the material. At the same time, the problems in the preparation of the specimens due to their difficult handling, made it necessary to discard the option of stiff clays. Finally, it was decided to use only one kind of material, the soft clays (Otaniemi Clay).

As for the content of the research, at first, one of the objectives of the thesis was to carry out this type of tests in order to deeply analyze the phenomenon of secondary consolidation or creep. However, because of the lack of time and complexity involved in this aspect of consolidation, this creep analysis was left out. In addition, another objective was to test specimens with different thickness but it was also left due to the lack of time.

Another aspect that was contemplated was the study or analysis of the effect of the temperature in the processes of consolidation of clays. Since the test equipment is highly accurate, it would be possible to monitor the effects of temperature on soil samples. But the lack of time was again a limitation. Actually, temperature measurement would be quite easy to include into the data, but temperature effects were not the main objective of this study, so other aspects have prevailed.

2. Background

Before starting with the development of the thesis, it is necessary to make a review of the most important concepts on geotechnics, as well as all the theoretical bases that have been used for the elaboration of this work. It starts from the most general knowledge until reaching those more specific concepts that are vital importance for a good understanding of pore pressure behaviour and consolidation process.

2.1. *Index properties*

Soil is constituted of a set of mineral particles (with organic matter to a lesser extent, usually) in contact with each other. Among these particles are voids (pores) which may be totally or partially occupied by water, or where appropriate by air. It can be understood, therefore, that the soil is a system constituted by three phases: a solid one (the particles), a liquid one (the water) and a gaseous one (the air), as it is schematized in the Figure 2:

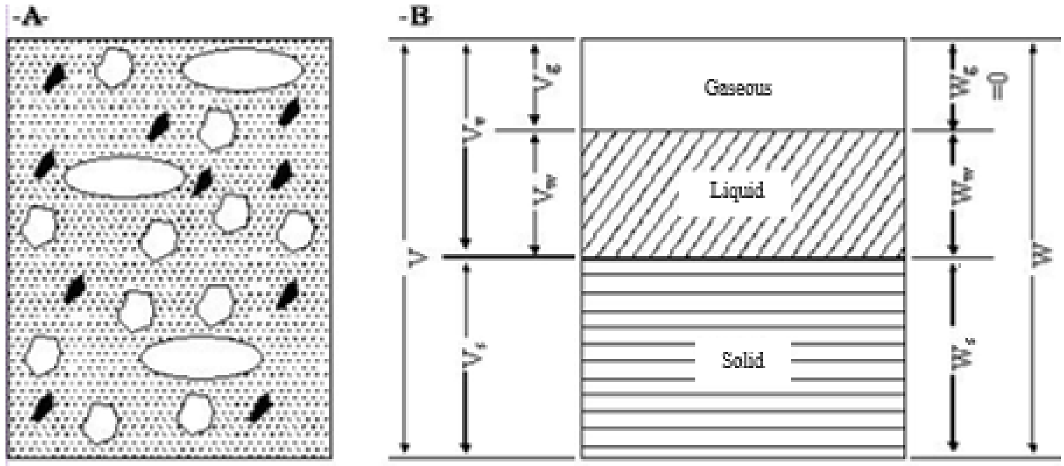

Figure 2. Relations between soil phases: A-Soil in natural state; B-Soil divided in phases (Torrijo, 2008)

Basic properties help to identify and classify soils. They are called index properties. Two main groups can be distinguished:

- Intrinsic soil properties: these are the ones, which do not depend on the soil structure. It means that they keep the same value despite of how the soil has been manipulated at the laboratory:
  - Grain size distribution
  - Atterberg Limits
  - Relative density of particles (Gs)

- State soil properties: the ones dependent on the way the samples have been obtained, or their manipulation at the laboratory:
  - Porosity (n)
  - Void ratio (e)
  - Water content (w)
  - Degree of saturation (S_r)

- Unit weight (γ)

The basic descriptors of a soil are related to its easiest to determine characteristics: the size of the particles that form the soil, and the susceptibility of the soil to vary their consistency with the variation of the material's moisture (plasticity).

2.1.1. Water content

Let us consider W_s to be the weight of the solid particles of a certain mass of a soil (Figure 2), and W_w the weight of the water that mass of soil contains in its pores. The soil water content w is defined as the following quotient expressed as a percentage (Franch, 2012):

$$\mathrm{w} = {}^{W_\mathrm{w}}/_{\mathrm{W_s}} \tag{2.1}$$

2.1.2. Unit weight and related parameters

The unit weight of a soil (γ), or apparent density, is defined as the ratio of its weight, both the solid fraction and the water contained ($W_\mathrm{s} + W_\mathrm{w}$), and its volume ($V$):

$$\gamma = {}^{W_\mathrm{s} + W_\mathrm{w}}/_{\mathrm{V}} \tag{2.2}$$

The dry unit weight of a soil is referred to as the ratio of the weight of the particles to the total volume of the soil:

$$\gamma_\mathrm{d} = {}^{W_\mathrm{s}}/_{\mathrm{V}} \tag{2.3}$$

The relation between the weight of the solid phase and its volume gives soil particles unit weight:

$$\gamma_\mathrm{s} = {}^{W_\mathrm{s}}/_{\mathrm{V_s}} \tag{2.4}$$

The total mass of the particles and the water that occupies the totality of the pores, divided by the total volume is called saturated unit weight of a soil:

$$\gamma_{sat} = {}^{(W_s + W_w\ sat)}/_{V} \tag{2.5}$$

Submerged unit weight (γ') of a soil is understood to be its saturated density minus the density of water (γ_w):

$$\gamma' = \gamma_{sat} - \gamma_w \tag{2.6}$$

2.1.3. Degree of saturation

A soil can have a variable water content between zero and the one in which the pores are completely filled with water. In this second case it is said that the soil is saturated. The degree of saturation of the soil (S_{r}) is the ratio of natural water content to saturation humidity, and it is expressed as a percentage:

$$S_{\mathrm{r}} = \mathrm{W_w} \,/\, W_{\mathrm{sat}} \tag{2.7}$$

The voids ratio (e) is the ratio of the volume of the pores to the volume occupied by the solid particles:

$$e = V_{\mathrm{v}} \,/\, V_{\mathrm{s}} \tag{2.8}$$

2.1.4. Plasticity

Plasticity is the property that some soils have to modify their consistency (or, in other words, their resistance to be cut) as a function of water content. Plasticity is an exclusive property of fine soils (Franch, 2012). There are four states in which a plastic soil can be found depending on its consistency, which varies according to the humidity: solid, semi-solid, plastic and liquid.

Conceptually, the liquid limit (w_L) corresponds to the water content above which the shear strength of a soil is near to zero (typical of a liquid). The plastic limit (w_P) corresponds to the water content threshold of a soil above which the material shows a deformation of plastic type in relation to the applied stresses (Casagrande, 1932).

The larger the range of moisture between the different limits of plasticity is, the bigger amount of water can be assumed by a soil without changing its state of consistency or plasticity.
The difference between the moisture value corresponding to the liquid limit and the plastic limit is defined as "plasticity index" (I_P). Thus, the greater the plasticity index is, the more plastic a soil will be (Casagrande, 1932).

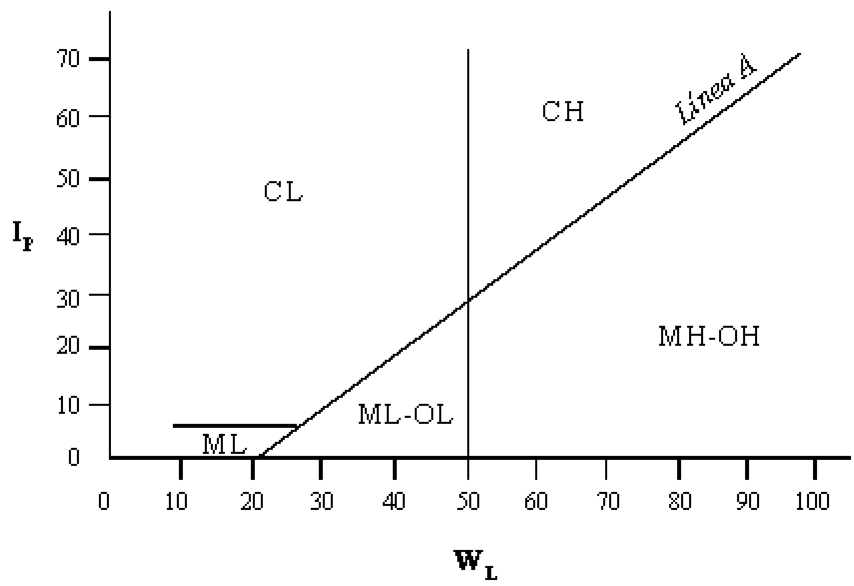

Figure 3. Casagrande graph (Casagrande. 1932)

The different kind of soils defined on graph in Figure 3 are (ASTM-D4318):

- Low plasticity inorganic clay (CL)
- High plasticity inorganic clay (CH)
- Inorganic mud with low plasticity (ML)
- Soil with colloidal organic matter and low plasticity (OL)
- High plasticity organic mud (MH)
- Soil with colloidal organic (OH)

2.2. Darcy´s Law

Darcy's law rules the water flow through porous media. It states that such flow is directly proportional to the hydraulic gradient, resulting in the following equation (Darcy, 1856):

$$Q = \frac{\partial V}{\partial t} = k \times i \times A \qquad (2.9)$$

Where
- Q: Flow [m^3/s]
- $\frac{\partial V}{\partial t}$: Volume change at a time differential [m^3/s]
- k: Permeability coefficient [m/s]
- i: Hydraulic gradient [Dimensionless]
- A: Cross section area [m^2]

For each point in the sample, for example, b, the total charge h_b is defined as (Bernoulli, 1738):

$$h_b = z_b + \frac{u_b}{\gamma_w} + \frac{v^2}{2 \times g} \qquad (2.10)$$

Where
- z_b: Load with respect to a plane [L]
- $\frac{u_b}{\gamma_w}$: Load because of pore water pressure
- $\frac{v^2}{2 \times g}$: Load because of flow velocity

In geotechnics, it is usual considering flow velocity zero, since velocities reached by flows throw the ground is very low.

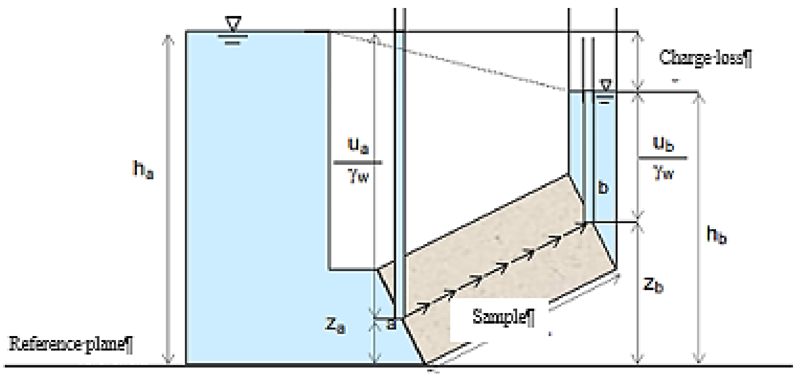

Figure 4. Darcy's experiment (Darcy, 1856))

By comparing the values of total load at b and a, it is observed that there is a difference between them. That value is the loss of load or hydraulic load:

$$\Delta h_b = h_a - h_b \tag{2.11}$$

The relationship between the load loss and the filtration line path provides the hydraulic gradient i, in the flow direction:

$$i = \frac{\Delta h}{L} \tag{2.12}$$

2.3. Consolidation

The application of loads on the terrain produces deformations that give rise to changes in volume. If the soil is saturated, volume changes involve expulsion of water from the pores. As the permeability of the soil is finite, it takes time for such ejection of water to occur. In granular soils, the permeability is so great that, for the usual speed of the loads, the flow is almost instantaneous; therefore, the application of the load is with drainage. In clay soils, however, the permeability is so small that the flow may last long, and it is usual to assume that the load is applied without drainage. Then, this charge without water flow causes an increase in pore water pressure, which then dissipates over time. This process is called consolidation (Jiménez & Alpanés, 1975)

2.3.1. Terzaghi´s Principle

Because of the previous process, the soil undergoes deformations that are evolving temporarily as the excess pore water pressure is dissipated, increasing the effective stress of the soil.

Stresses at a point of the soil can be calculated from the total stresses in principal axis ($\sigma_1, \sigma_2, \sigma_3$) acting at that point and physically measurable. If the soil is saturated, at each point the water will have a pressure (u) (pore water pressure), measurable and positive or negative. Total stresses in principal axis are composed of two parts; u, which acts on water and solid particles in all directions, and by the difference:

$$\sigma'_1 = \sigma_1 - u \tag{2.13}$$

$$\sigma'_2 = \sigma_2 - u \tag{2.14}$$

$$\sigma'_3 = \sigma_3 - u \tag{2.15}$$

This latter part acts only in the solid phase of the soil and is called effective stress. All measurable changes in a soil due to stresses (deformability and resistance) are due exclusively to a
effective stress change (Terzaghi, 1925).

2.3.2. One-dimensional consolidation

The most used deformation model and the one to be followed in this thesis is the one-dimensional consolidation. The one-dimensional consolidation occurs in situations where terrain layers subjected to a surface load of theoretically infinite dimensions.

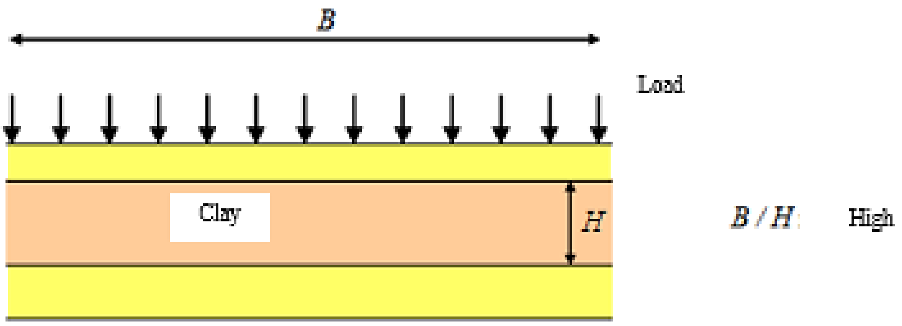

Figure 5. Piece of soil under external load

In the situation described in Figure 5, lateral deformation is zero ($\varepsilon_x + \varepsilon_y = 0$), the only possible deformation being vertical (ε_z). Therefore, the volumetric deformation directly results in the value of the vertical deformation:

$$\varepsilon_{vol} = \varepsilon_1 + \varepsilon_2 + \varepsilon_3 = \varepsilon_x + \varepsilon_y + \varepsilon_z = \varepsilon_z \tag{2.16}$$

The vertical deformation that occurs with an increase of vertical stress ($\Delta\sigma$) can be due to different phenomena:

- Compression of the solid matter (volume change of solid phase), which can be produced by deformation of the particle or crushing in the particle contacts.
- Variation in the volume of voids, which needs a rearrangement of the particulate matter. This variation can be caused by any of the following causes: compression of water and/or air, dissolution of air in the water, or expulsion of water and/or air.

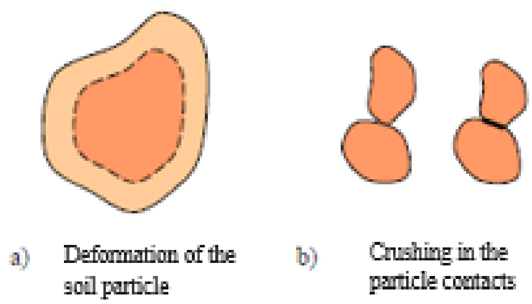

Figure 6. elastic compression of solid matter (Ortuño, 2009)

Regarding these phenomena, the following must be taken into account:

- Volume changes due to compression of solid matter are small for the usual external load ranges.
- In saturated soils, the variation of voids due to elastic compression of air, dissolution of air in water and air expulsion, is practically nonexistent since there is no air inside the voids.
- Water can be considered practically incompressible.

Consider a homogeneous, saturated soil deposit of infinite lateral length and subjected to a uniform load (q) applied throughout the surface area. The soil lays on an impermeable base as indicated in Figure 7, where:

- h_p: is the piezometric height
- z: is the position with respect to a reference plane
- h_h: Piezometric height under water table
- h_e: Piezometric height over water table
- H: is the thickness of the stratum

The excess water pressure dissipation, at any point, will only take place throughout a vertical flow of the water. Thus, the soil will undergo vertical deformation as a result of this water flow.

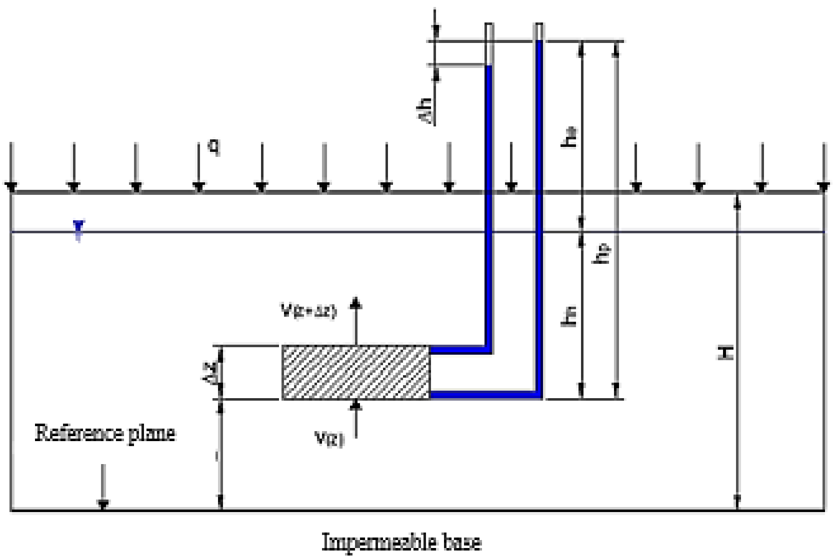

Figure 7. Model of the deposit (Poliotti & Sierra, 2001)

The following hypotheses are established (Terzhagi & Flörich, 1936):

- The soil is homogeneous.
- The soil is saturated and will remain so throughout the consolidation process. In the case of unsaturated soils, the results of this theory are unreliable.
- Particles in soil and water are incompressible.

- The compression is one-dimensional in the vertical direction and there is no movement of particles in the horizontal direction. This is true in the laboratory, but approximate in situ.
- Water drainage occurs only vertically.
- Darcy's law and all its hypotheses are valid.

Considering the flow in a differential element located at *z* of the reference plane (Taylor, 1948), where:

- V_z is the vertical velocity of the flow entering the element
- $V_{(z+dz)}$ is the vertical velocity of the flow leaving the element

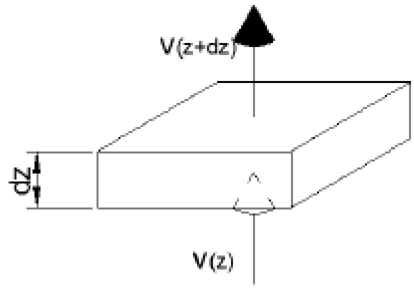

Figure 8. Differential element of the soil (Poliotti & Sierra, 2001)

Since *dz* is very small, it may be assumed that the terms of the second or higher order are insignificant and then it turns out that:

$$v_{(z+dz)} = v_z + \frac{\partial v_z}{\partial z} dz \rightarrow [v_z + \frac{\partial v_z}{\partial z} dz] \times A - v_z A = -\frac{\partial V}{\partial t} \quad (2.17)$$

If soil particles and pore water are assumed incompressible, then the volume change velocity of the element ∂V / ∂t is equal to the velocity of change of volume of voids $\partial V_v / \partial t$:

$$V \frac{\partial v_z}{\partial z} = -\frac{\partial V_v}{\partial t} \quad (2.18)$$

So if void ratio $e = {V_v}/{V_s}$ and $V_v = e \times V_s$, the problem leads to void ratio e in time, $\frac{\partial e}{\partial t}$

:

$$V \frac{\partial v_z}{\partial z} = -V_s \frac{\partial e}{\partial t} \quad (2.19)$$

$$\frac{\partial v_z}{\partial z} = -\frac{1}{1+e} \times \frac{\partial e}{\partial t} \quad (2.20)$$

From Darcy´s law *($v = ki$; $i = h/z$)* it is possible to obtain for a vertical flow through the element:

$$v_z = -k_z \frac{\partial h}{\partial z} \quad (2.21)$$

By replacing (2.21) in (2.20), it is obtained:

$$k_z \frac{\partial^2 h}{\partial z^2} = -\frac{1}{1+e} \times \frac{\partial e}{\partial t} \tag{2.22}$$

Assuming that neither the water table nor the position of the element vary during the consolidation process *(z + h_h = permanent)*, and the only thing that varies is the height of the water corresponding to the excess of pore water pressure h_e, we obtain:

$$\frac{\partial^2 h}{\partial z^2} = \frac{\partial^2 h_e}{\partial z^2} \tag{2.23}$$

$$u = \rho_w \times g \times h \tag{2.24}$$

By substitution in (2.23) we obtain:

$$k_z \frac{\partial^2 h}{\partial z^2} = \frac{1}{\rho_w g} \times \frac{\partial^2 u_e}{\partial z^2} \tag{2.25}$$

By substitution (2.22) we obtain:

$$\frac{\partial e}{\partial t} = \frac{k_z(1+e)}{\rho_w g} \times \frac{\partial^2 u_e}{\partial z^2} \tag{2.26}$$

Equation 2.26 is obtained with two unknowns. To pose the problem completely, an additional equation is needed which relates the excess pore pressure and the void ratio. This is obtained by considering the behavior of the soil under vertical stress-deformation. Terzaghi took this behavior as linear for a particular load increase $\partial\sigma'$. Since the change in deformation is proportional to the change in voids ratio, this also implies the existence of a linear relation $e - \sigma'$. The slope of line is called a_v, and it is defined as compressibility coefficient:

$$a_v = -\frac{\partial e}{\partial \sigma'} \tag{2.27}$$

Where:

$$\sigma = \sigma' + u \tag{2.28}$$

$$u = u_h + u_e \tag{2.29}$$

Therefore:

$$\sigma = \sigma' + u_h + u_e \tag{2.30}$$

Reaching the expression:

$$\frac{\partial \sigma'}{\partial t} = \frac{\partial u_e}{\partial t} \tag{2.31}$$

This expression demonstrates what has already seen in Terzaghi's analogy, as the excess pressure decreases, an increase in the effective pressure occurs, that is, the pressure is transferred from the pore water to the soil particles.

In addition:

$$\frac{\partial e}{\partial t} = \frac{\partial e}{\partial \sigma\prime} \frac{\partial \sigma\prime}{\partial t} \tag{2.32}$$

$$\frac{\partial e}{\partial t} = a_v \frac{\partial u_e}{\partial t} \tag{2.33}$$

$$\frac{\partial u_e}{\partial t} = \frac{k_z(1+e)}{\rho_w g} \times \frac{\partial^2 u_e}{\partial z^2} \tag{2.34}$$

This last equation can be also expressed in the *one-dimensional consolidation equation* as:

$$\frac{\partial u_e}{\partial t} = c_v \frac{\partial^2 u_e}{\partial z^2} \tag{2.35}$$

Where c_v is the consolidation coefficient:

$$c_v = \frac{k_v}{m_v * \gamma_w} \tag{2.36}$$

In order to express the *one-dimensional consolidation equation* as a dimensionless one, it is necessary to define the following two parameters:

$$T_v = \frac{c_v}{d^2} \times t \tag{2.37}$$

Which is the time factor, and the consolidation degree U_z:

$$U_z = 1 - \frac{U_{z,t}}{U_0} \times t \tag{2.38}$$

With all this, the solution of the equation will be:

$$U_z = U_z\left(\frac{z}{H}; T_v\right) \tag{2.39}$$

2.3.3. Oedometer test

The one-dimensional soil consolidation process is simulated in the laboratory by the compression of a sample in an apparatus called oedometer. Figure 9 shows the diagram of an oedometer cell with all its components. Inside the cell, a soil sample is placed inside a rigid ring that prevents lateral deformation and flow during compression. At the base and at the top of the sample two porous stones allow drainage during consolidation.

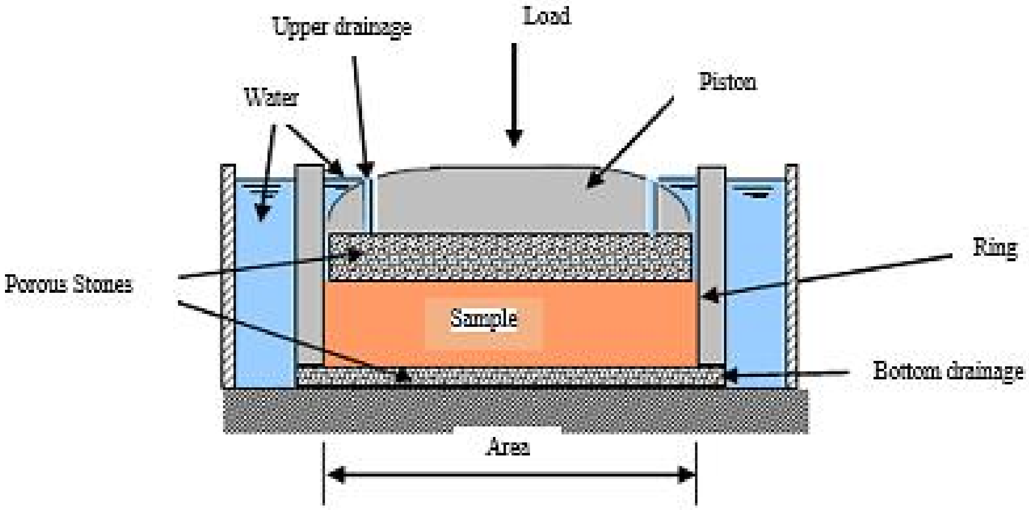

Figure 9. Sample disposition (Izquierdo & Garrido, 2014)

Once the sample is inside cell, the test consists of applying a sequence of vertical loads. The soil is getting more and more rigid as it squeezes. Therefore, in order to ensure that deformation in each step has the same order of magnitude, load increments must be bigger each time. It is normal to double the existing load. A usual sequence is: 5, 10, 20, 40, 80, 150, 300, 600, 1000, 1500 kPa and unloading.

After the application of each load step, the sample is allowed to consolidate until the excess of pore water pressure produced inside it is reduced to zero. Theoretically, this time is infinite, however, in practice it is established that at 24 hours the consolidation can be considered finished (CEN ISO/TS 17892-5). The loading step loads are carried on until reaching the maximum pressure that is desired, after which it is proceeded to unload, also by successive steps. During the unloading, the soil increases in volume (swelling).

During each load step, settlement reader registers the value of the settlement produced for different times. These values allow obtaining the deformation of the soil sample at different times, and therefore, they are a measure of the evolution of deformations over time.

2.3.4. Evolution of the settlement and the level of stresses over time

Figure 10 shows an oedometric sample of clay soil, which is subjected to a total initial vertical stress state equal to the effective one, $\sigma_0 = \sigma'_0$. In this situation, it the soil is consolidated at σ_0:

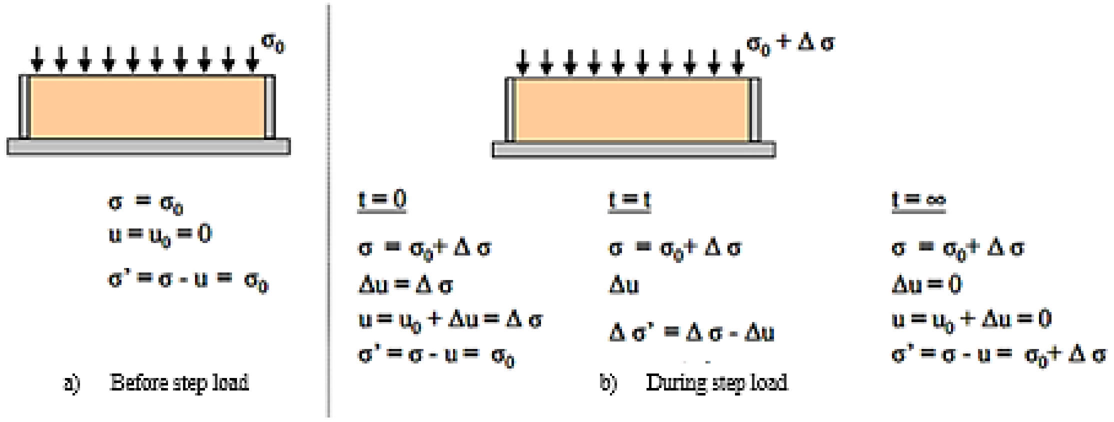

Figure 10. Pressure development (Ortuño, 2009)

Starting from the described situation, a stress increase $\Delta\sigma$ is applied and, allowing drainage of the sample, the evolution of the settlements is analyzed until the soil becomes consolidated at a total compressive stress $\sigma = \sigma_0 + \Delta\sigma$.

Each time a new load step is applied the total vertical pressure increase $\Delta\sigma$ applied is integrally and instantaneously transmitted to the pore water Δu. According to Terzaghi's postulate, the effective stress do not vary at this point. Due to the increase in pore pressure, the water will want to "escape" the sample, but can only do so through the upper and lower porous stones. They are the only "draining or permeable boundaries". Consequently, after the loading, a flow of water starts, rising in the upper half of the specimen and descending in the lower half (Terzaghi, 1925).

Prior to the application of the loading step, the distribution of pore water pressure (u_0) on the soil test piece is hydrostatic. The application of a loading step $\Delta\sigma$ immediately results in an pore water pressure increase of equal magnitude $\Delta\sigma = \Delta u$. At permeable boundaries (porous stones), excess pore pressure dissipates instantaneously. In contrast, in the center of the test tube it is more difficult to "escape" (it has longer drainage distance), and it will take longer to consolidate.

At any time (t) after loading, the excess remaining pore water pressure will vary from one point to another as a function of its distance from the draining boundaries. Figure 11 below shows the schematic form of the succession of pore pressure laws for the different times in the application of isochrones (Taylor, 1962).

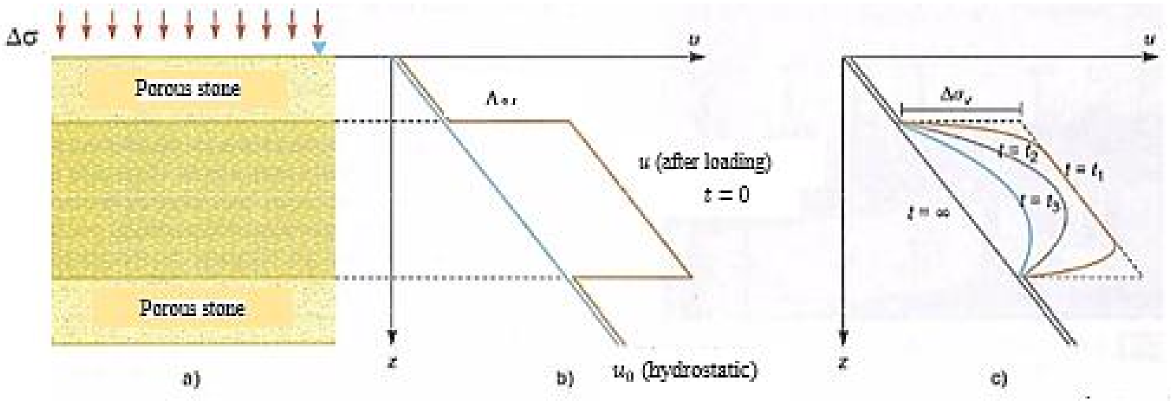

Figure 11. Pore pressure development (Ortuño, 2009)

The excess pore pressure distribution in a stratum at each point of its depth z, at time t, it is called the isochronous distribution $u_s(z, t)$. Generally, these distributions are curves that approach sufficiently to a parabola, reason why they are called parabolic isochrones (Taylor, 1925).

There is a time t_c (critical time), for which the parabola corresponding to that isochronous will have a vertical tangent in the center of the stratum, and it will be that central point the only one in which the pore excess pressure has not yet dissipated.

It is possible to make an analysis based on three phases:

1) If $0 < t < t_c$ there will be a thickness or height for the stratum or simple, where the excess pore pressures will have started dissipating, while the rest of the stratum $(H - L)$ won´t be suffering any changes in pressures.
2) If $t = t_c$ there is only one point $(z = H)$ where excess pore has not dissipated.

If $t_c \geq t > t_f$, excess pore pressures have begun dissipating at any point of the stratum. The maximum pore pressure value is at the centre of the stratum, when drainage is permitted on both sides.

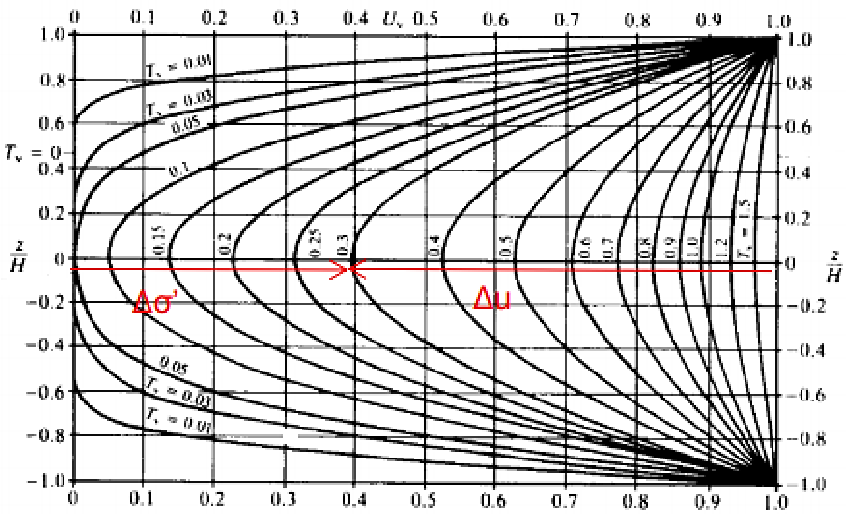

Figure 12. Abacus for degree of consolidation (Taylor, 1925)

2.3.5. Oedometric curve

The representation of all these values is carried out by the so-called oedometric curve. It is possible to establish pairs of values (*s, σ '*), where *s* is the settlement at a particular time corresponding to the end of each step, and σ' is the effective pressure of the sample at that time. These pairs of values are usually transformed into values *(e, σ')*, where *e* is the value of the void ratio of the sample at the end of each step. The effective pressures are located on abscissa, and in ordinates, the void ratio reached at the end of the consolidation period corresponding to a given pressure.

The most usual representation of the oedometric curve is to use a logarithmic scale for the pressures, obtaining a curve like that of Figure 13. The different parts of the oedometric curve receive the following names: compression curve, unloading or swelling curve and reloading curve.

In this case, the different curve are approximately straight. The compression curve is represented by an equation of the type:

$$e_1 - e = C_C \times log_{10} \frac{\sigma'}{\sigma'_1} \tag{2.40}$$

C_c is a constant called compression index; e_1 and σ'_1 are the values of void ratios and effective pressure of a point on this line.

Each unloading or swelling curve can be represented by a similar equation, but in this case the constant C_s is called the swelling index:

$$e_1 - e = C_S \times log_{10} \frac{\sigma'}{\sigma'_1} \tag{2.41}$$

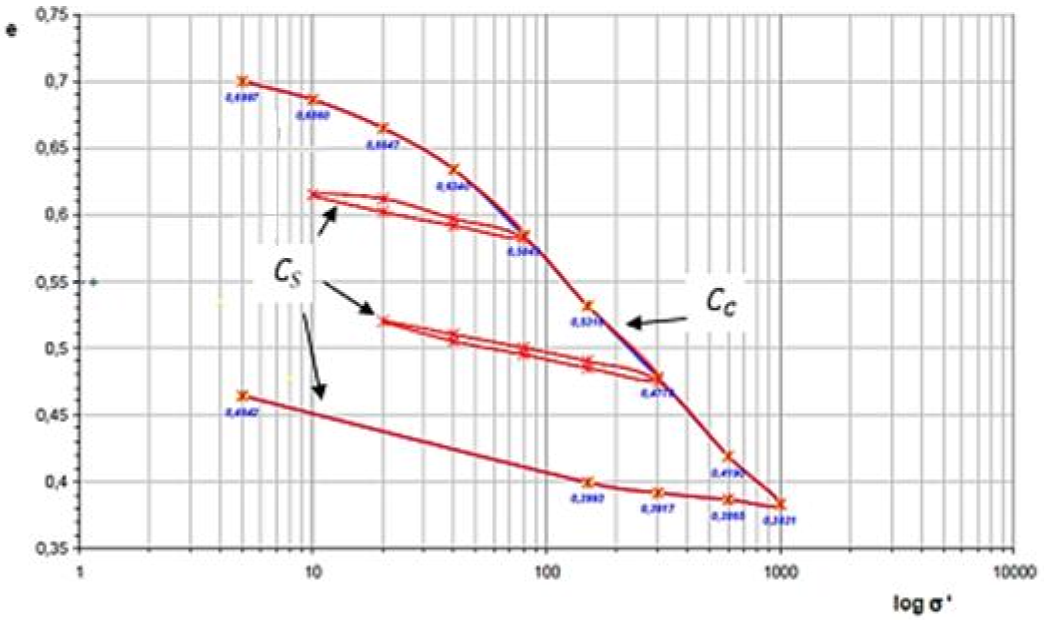

Figure 13. Oedometric curve (Izquierdo & Garrido, 2014)

In the compression curve it is observed that deformations are smaller for a same increase of pressure, as this increases, so the soil is stiffened. Therefore, the deformation modulus is not constant for the different load levels.

It can also be observed that in both unloading and reloading, the soil is stiffer, so deformations are smaller compared to those along compression curve. In addition, in the unloading, the soil just recovers a percentage of the settlements generated during the load, reason why there are plastic deformations.

2.3.6. State of consolidation

Soil is normally consolidated when it has never been subjected to effective pressures higher than it has at the present time. In this case the oedometric curve begins according to a branch of compression. Otherwise it is said that the soil is overconsolidated, and the oedometric curve begins then according to a recharge branch until it reaches the preconsolidation pressure. The overconsolidation ratio is the relationship between the preconsolidation pressure and the in-situ effective pressure.

$$OCR = {}^{\sigma'_p}\!/_{\sigma'_0} \qquad (2.42)$$

OCR can be determined from an oedometer test conducted for an undisturbed sample. However, there are no completely undisturbed samples. Sampling procedures produce some disturbance. The maximum disturbance of a sample would be its total remolding. The disturbance of the sample means:

- Reduction of the void ratio for a given vertical pressure
- The history of soil stress and its preconsolidation pressure is obscured
- The slope of the noval branch, i.e. its compression index, decreases

As for the branch of unloading, it does not undergo appreciable change with the kneading.
However, what is interesting to obtain from an oedometric test is the compression curve in the field, which is the one that provides the values of the coefficients C_C, C_S and the preconsolidation pressure in its case. For this, it is necessary to eliminate the effects of the perturbation, from the oedometric curve obtained in the laboratory test, to obtain the curve corresponding to the compression in the field by means of a series of corrections. However, these kind of corrections are not very used in Finland.

There are different methods of determining σ'_p from laboratory oedometer data. Casagrande (1936) developed the most commonly used method, used in this thesis.. Figure 14 shows this method as described in Holtz and Kovacs (1981). The following steps describe this construction.

1. Choose by eye the point of minimum radius (or maximum curvature) on the consolidation curve (point A).
2. Draw a horizontal line from point A
3. Draw a line tangent to the curve at point A.

4. Bisect the angle made by steps 2 and 3.
5. Extend the straight-line portion of the compression curve up to where it meets the bisector line obtained in step 4. The point of intersection of these two lines is the preconsolidation stress (point B).

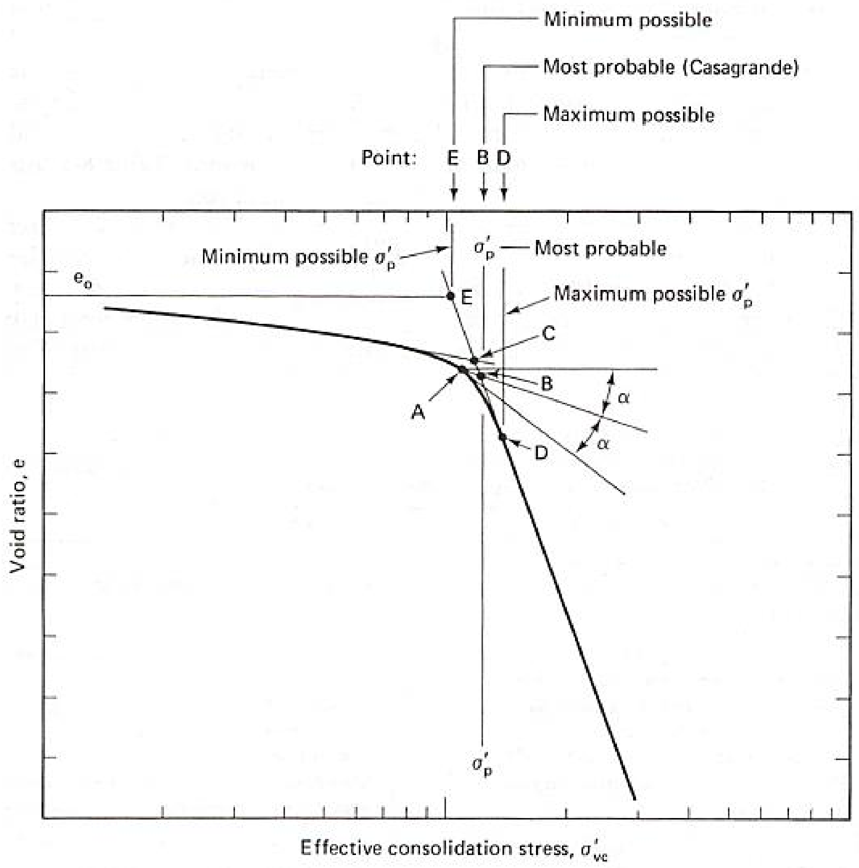

Figure 14. Casagrande construction for determining preconsolidation stress (Casagrande, 1936)

Since this whole construction is very laborious and it can only be done by hand, the method has been simplified. The preconsolidation stress will be considered where the extended straight-line portion of the compression curve meets the extension of the straight-line portion of the recompression curve (Point C). It gives a very close approximation and saves time and effort (Koskinen, Karstunen & Lojander, 2003).

2.3.7. Consolidation curve

When representing the deformations of a soil, for each load step, three types of consolidation appear:

1. Immediate deformation: volume change in the soil produced by the increase of the total pressure. Caused by elastic deformation of soil without change in moisture content. Effects such as dissolution of air bubbles (incomplete saturation) fissure closure or rearrangement of particles are the main causes.

2. Primary consolidation: caused by volume change in saturated cohesive soils due to exclusion of water in voids. It means the excessive pore pressure dissipates and as a result, the effective pressure rise.
3. Secondary consolidation (creep): volume change in soil that happens after the excess pore pressure has dissipated, which means, effective stress remains uniform. It is due to multiple factors such as displacement and reorientation of particles, decomposition of soil organic matter or plastic adjustment of soil fabrics.

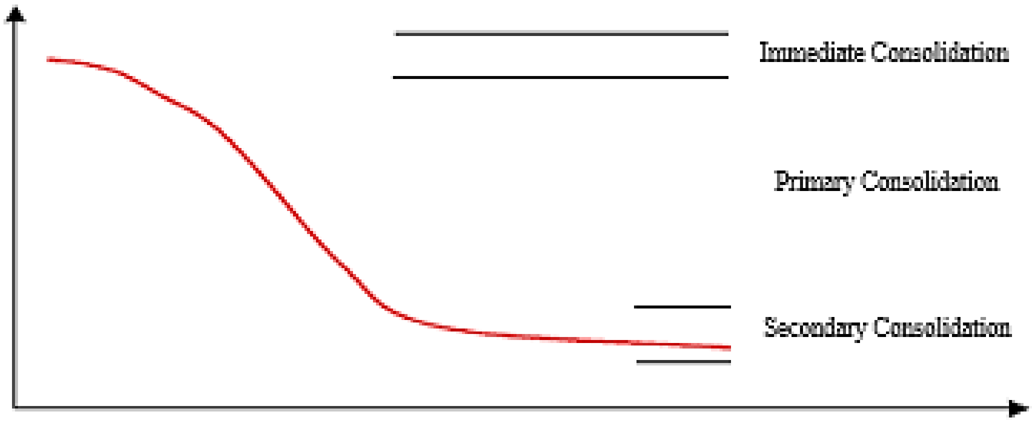

Figure 15. Consolidation curve (Izquierdo & Garrido, 2014)

In oedometer tests, as many consolidation curves (time-settlement) are obtained as step loads are applied.

The purpose of these curves is to determine the velocity a soil consolidates at. As time elapses, the settlement increases, as well as the degree of consolidation of the sample. The consolidation velocity is quantified by the consolidation coefficient c_v, which is also expressed as follows:

$$c_v = \frac{T_v \times d^2}{t} \tag{2.43}$$

The two most common methods for obtaining c_v are the Taylor method and the Casagrande method.

<u>Taylor Method</u>

In this method, c_v is obtained for a degree of consolidation $U = 90\%$. The consolidation curves are plotted representing the deformation of the sample, in millimeters, against the square root of time, in minutes. The procedure to obtain this c_v is:

1. Identify the straight section of the laboratory curve and extend it until it coincides with the ordinate axis (Point R_0).
2. From the extension of the line at the bottom, choose any point and measure the distance to the axis of ordinates (distance x)
3. Prolong the x distance along the abscissa axis, a length of $0.15x$.

4. Join points R_0 and the previous one. The point of intersection with the curve has the consolidation settlement for a degree of consolidation of 90%.

$$c_v = \frac{0.848 \times d_{90}^2}{t_{90}} \tag{2.44}$$

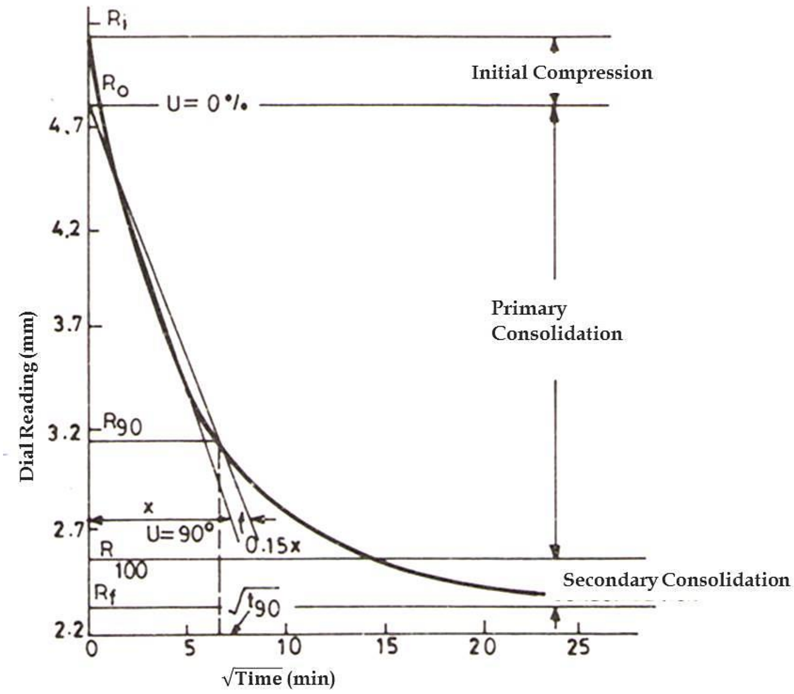

Figure 16. Taylor Method (Taylor, 1962)

<u>Casagrande Method</u>

In this method, c_v is obtained for a degree of consolidation $U = 50\%$. The curves are plotted representing the deformation of the sample as a function of the time logarithm. The procedure is:

1. Take any two points on the curve having an abscissa ratio of 4:1.
2. Determine the length in ordinates between the chosen points (z). Apply this distance from the point of least ordinate, parallel to the axis of the abscissa upwards. If a horizontal line is drawn from that distance, the ordinate at the origin obtained, R_0, defines the beginning of the primary consolidation.
3. Draw a tangent to the laboratory curve where the slope is greater.
4. Prolong the final part of the curve, obtaining the point whose ordinate is L_{100}, which represents the end of the primary consolidation.
5. Obtain L_{50} as the midpoint of the L_{100}-L_0 section.
6. Draw a horizontal by that point until it cuts to the laboratory curve. The resulting point has as its coordinates the seat corresponding to U = 50%.

$$c_v = \frac{0.196 \times d_{50}^2}{t_{50}} \tag{2.45}$$

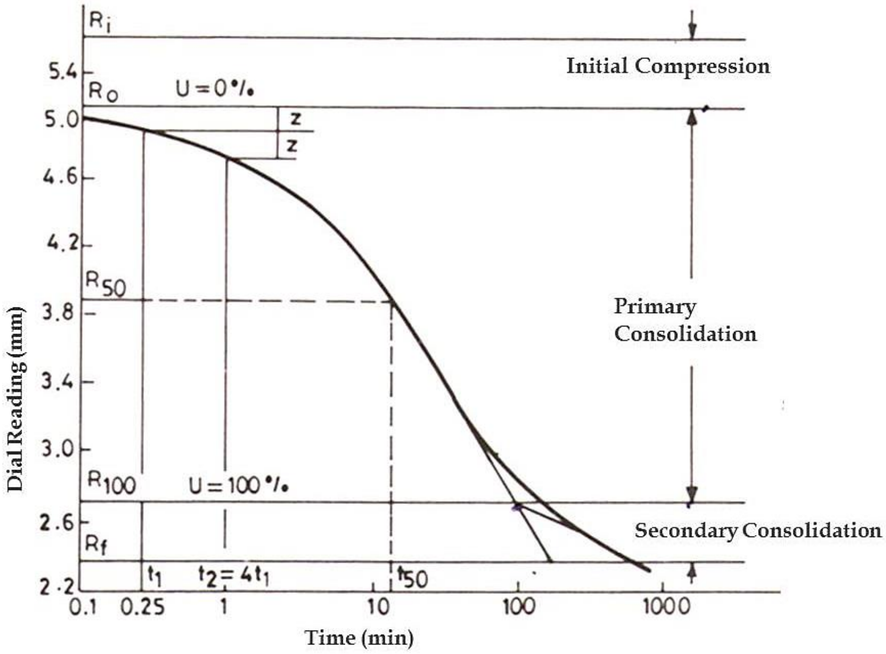

Figure 17. Casagrande Method (Casagrande, 1932)

3. Test procedure

This thesis is mainly based on a modification of the conventional oedometer test, which is called Pore Water Pressure Oedometer Test. In order to better understand what it is about and how it works, first it is necessary to describe the oedometer test and then, explain the main singularities or differences among them.

3.1. Description of the site

Samples were taken in spring and autumn in 2016 from Otaniemi (Figure 18). Unscrupulous samples are taken with the Norwegian Piston Circuit (NGI 54). According to the CEN standard, the Norwegian Piston Circuitry is able to take samples in the sampling class A and thus samples of laboratory grade class 1 (SFS-EN ISO 22475-1, 2006). Taking uninterrupted samples was started about a depth of 1.5 meters from the ground.

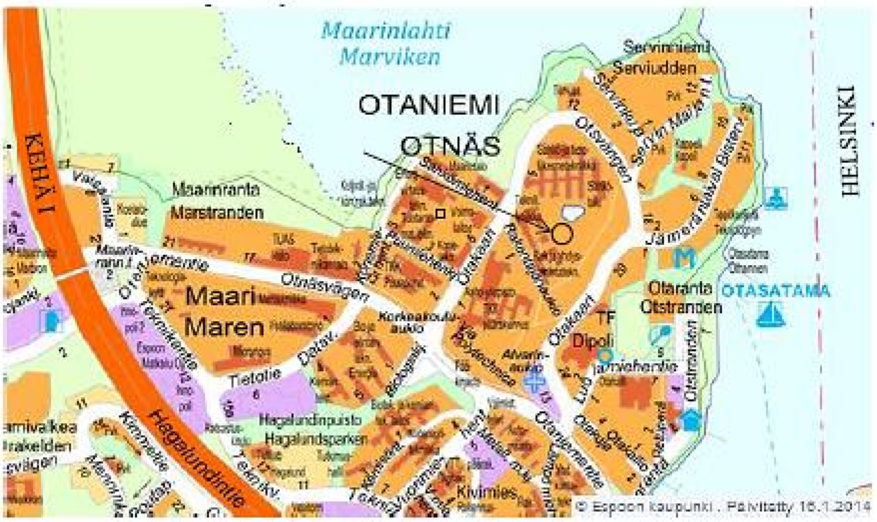

Figure 18. City of Espoo, 2014 (Laaksonen, 2014)

All of the sampling pipes used during the work are defined at least from the lower end of the tube, the following classification characteristics: color, water content, bulk density, specific weight, humus density, and granularity according to GEO and Eurocode classification. In the second sampling, bag samples were taken from the ground at about 20 cm increments. From all bag samples, water content and, as far as possible, hardness and granularity were determined (Laaksonen, 2014).

3.1.1. Properties

The water content of the samples was determined in accordance with CEN Technical Specification CEN ISO / TS 17892-1, 2007. Water content was determined from the trimmings gotten after carved the sample into the ring. Additionally, the sample was weighted before the test and after the experiment the sample was dried and weighed to obtain the test sample water content. The samples processed in the work are mainly from a depth of 1.8-2.2 meters. In the majority of the area, the average water content is about 73%.

Unit weight values were determined by pyknometric test in accordance with CEN technical specification CEN ISO / TS 17892-3, 2007. The unit weight at the studies depth varies between 15.16 – 15.43 kN / m^3.

3.1.2. Classification test results (HUT-Clay)

Water content and unit weight are properties known from the tested samples. However, these tests do not allow obtaining some other properties also necessary. Therefore, this information is taken from classification test results carried out in Samuli Laaksonen's Master's Thesis "Improvement of research methods and testing equipment for deformation properties of clay" (2014), where tested clay is from the same spot.

Table 1. HUT-CLAY Classification properties

HUT-CLAY Classification properties															
Samples from tubes (undisturbed)															
Point	Tube	Depth from ground surface	Water content	Undrained shear strength	Remoulded shear strength	Sensitivity	Liquid limit	Plastic limit	Bulk density (moist)	Unit weight (moist)	Specific weight (particle density)	Organic content	Clay content	Soil type (GEO)	Soil type (ISO)
number	number	z, m	w, %	c_u, kPa	c_{ur}, kPa	S_t	%	%	ρ_m, g/cm^3	γ_m, kN/m^3	ρ_s, g/cm^3	%	%	name	name
2013-R1	1	1,95-2,00	64,0	12,8	1,5	8,7	61,5		1,60	15,73	2,78	-0,4	66,2	liSa	Cl
2013-R2	2	2,05-2,10	75,4	9,6	1,1	8,9	67,9		1,51	14,77	2,80	-0,5	76,2	liSa	Cl
2013-R3	3	3,05-3,10	45,2	7,5	0,7	11,2	36,1		1,77	17,37	2,76	-0,5	37,1	laSa	siCl
2013-R4	63	2,95-3,00	48,1	7,5	1,2	6,3	41,8	25,0	1,77	17,33	2,78	-0,6	41,6	laSa	Cl
2013-R5	100	1,95-2,00	73,2	12,7	1,5	8,6	69,6	29,0	1,54	15,10	2,76	-0,1	79,8	liSa	Cl
2013-R6	49	2,12-2,17	71,9	13,3	1,5	9,0	68,3		1,58	15,46	2,75	0,1	81,5	liSa	Cl
2013-S2	T2	2,19-2,24	73,3	12,4	1,2	10,5	63,7		1,57	15,39	2,80	0,0	78,1	liSa	Cl
2013-S3	T3	2,15-2,20	72,8	9,0	1,2	7,6	63,4		1,61	15,78	2,81	0,0	78,1	liSa	Cl
2013-S4	T4	2,99-3,04	56,9	8,9	0,5	19,3	42,7		1,68	16,51	2,79	-0,4	52,1	liSa	Cl
2013-S5	T5	2,09-2,14	73,0	9,0	1,5	6,1	63,5		1,54	15,11	2,80	-0,1	78,4	liSa	Cl
Samples from bags (disturbed)															
2013-S3	bag	0,00-0,20	37,5												
2013-S3	bag	0,20-0,40	25,3												
2013-S3	bag	0,40-0,60	38,4												
2013-S3	bag	0,60-0,80	57,4								2,59	6,3	27,9	saSi	siCl
2013-S3	bag	0,80-0,95	78,0		5,9		96,8				2,65	4,6	37,4	laSa	siCl
2013-S3	bag	1,10-1,20	38,2												
2013-S4	bag	1,00-1,20	64,3								2,65	3,1	35,1	laSa	Cl
2013-S4	bag	1,20-1,40	44,3								2,69	1,2	26,5	saSi	saCl

3.1.3. Summary of the tests

To obtain the necessary data to carry out this thesis, a total of 12 laboratory tests have been performed. In turn, these tests have been divided into two groups. The first group has been tested following the guidelines of the standard incrementally loaded oedometer test. For the second group of tests, a modification has been implemented with respect to the previous one. The load increment ratio (LIR) has been modified with respect to the original, which was LIR = 2.

One of the fundamental objectives is to determine the behavior and the development of the pore pressure of the soil samples, but trying to follow a methodology based on the standard oedometer test (LIR = 2) (ASTM-D4318):. In this way, criteria can be unified, and thus the comparison of results is easier. For this reason, 8 out of the 12 samples tested correspond to the first group of tests.

However, it is also an objective to determine how the pore pressure behaves against other types of stress. Therefore, a part of the tests have been modified to be able to observe how the soil samples behave under different conditions to those stipulated in the incrementally loaded oedometer test. Thus, the remaining 4 samples have been used for the second set of tests. Two out of these four samples will follow a new load increment ratio (LIR =1), and the other two will maintain the original one. This way, it easier to compare the results. This pursues to analyze if the load distribution affects the final state of the sample, since, at the conclusion of the tests, all samples will be equally loaded. At the same time, applying load steps of the same value consecutively allows an easier comparison as the sample evolves during the test, both in the settlements and in the pore pressure distribution.

Table 2. Tests and classification properties

KIMOLA-CLAY CLASSIFICATION PROPERTIES						
Classification data from Oedometer tests						
Test number	Point	Tube	Depth	Water content Sample	Bulk density	Unit weight (moist)
N°	Id	Id	z, m	w, %	ρ_m, g/cm^3	γ_m, kN/m^3
6527u	3	68	4,09-4,06	44,4	1,73	17,01
6528u	3	68	4,03-4,00	46,5	1,73	16,97
OSSINLAMPI-CLAY CLASSIFICATION PROPERTIES						
Classification data from Oedometer tests						
Test number	Point	Tube	Depth	Water content Sample	Wet density	Wet unit weight
N°	Id	Id	z, m	w, %	ρ_m, g/cm^3	γ_m, kN/m^3
6535u	V4	4	2,12-2,09	78,9	1,55	15,16
6536u	V4	4	2,09-2,06	73,2	1,56	15,32
6544u	V4	4	2,06-2,03	75,1	1,56	15,27
6545u	V4	4	2,03-2,00	74,0	1,56	15,30
6549u	V4	4	2,00-1,97	70,4	1,56	15,29
6550u	V4	4	1,97-1,94	70,1	1,56	15,33
6557u	V4	4	1,94-1,91	70,9	1,57	15,41
6558u	V4	4	1,91-1,88	71,7	1,56	15,35
6559u	V4	4	1,88-1,85	71,6	1,56	15,35
6560u	V4	4	1,85-1,82	75,6	1,57	15,43

3.2. Standard incrementally loaded Oedometer test

The conventional oedometric test, based on Terzaghi´s consolidation theory, consists of applying successively increasing step loads to which the test sample is subjected until reaching a maximum. For each step load, at the end of the consolidation, the time-settlement curve is obtained from which it is possible to obtain the parameters of the theoretical one-dimensional consolidation model C_v (consolidation coefficient), E_m (oedometric module) and k (permeability).

It consists of placing a cylindrical undisturbed sample of soil (small thickness compared its extension), confined in a rigid ring to prevent lateral deformations, between two porous stones that allow water to flow (vertical flow).

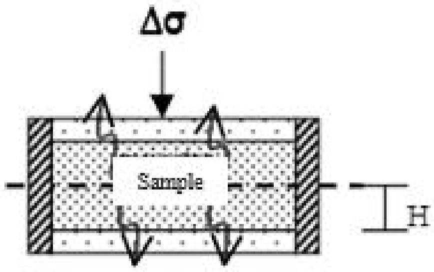

Figure 19. Oedometer cell scheme

The set is placed in a cell that is kept flooded to ensure soil saturation conditions. It is subjected to a series of step loads and unloading (increases of effective stress), measuring in each one of them the vertical displacements (settlements) produced along the time.

The oedometer cell has the following components:

- Metal base with a lateral closing body forming a container with the base.
- Rigid metallic ring
- Porous stones that allow drainage
- Loading piston, which should allow the sample to drain freely.

An analogue or digital deformation gauge must also be available, with precision of thousandth of a millimeter. Finally, a rigid bench where the cell is placed for the application of the different step loads.

3.2.1. Test procedure (Standard Oedometer test)

First phase: preparing operations

- Determination of the mass of the soil sample with its natural moisture (soil + initial water), as it differs from the previously sampled ring and ring mass, previously weighted. Sample residues that have been removed in their preparation are brought to the furnace to determine their water content.
- Assembly of the oedometer cell:
    - The bottom porous stone is placed inside the base of the consolidometer mold and a filter paper is placed on it. It is very important that both the porous stone and the filter paper are pre-saturated. The objective is to eliminate possible air bubbles that may have occluded.
    - Then the ring containing the soil sample to be tested is introduced, a filter paper and the upper porous stone are placed on the sample.

  - The fixing ring of the upper porous stone is fixed with the corresponding screws. Again, to prevent porous stones from drawing sample moisture, they must be free of occluded air before mounting the unit. It is important to properly center the porous stones to prevent binding against the ring during the test. It is advisable to fasten the screws bit by bit, to avoid movement of the sample.
- Assembly of the bed: the cell is placed on the bed and, in order to preserve the saturation of the sample during the test, it is filled with water. By adjusting the counterweight on the back of the bench, the balance is checked, thus all the transmission elements are in contact, without any load being applied.
- The strain gauge is placed in the center of the load piston. Finally, the initial reading, corresponding to zero deformations, is recorded.

<u>Second phase: application of loads and discharges</u>

Loading process: consists of the application of different levels of pressure in the sample. Each load must be maintained until the recorded vertical deformations are stabilized. In most cases with 24 hours is enough, although it depends on the nature of the soil.

It is recommended to record the greatest number of measurements in the first instants of each load step and to space the record as the consolidation phenomenon slows down. Loads are normally doubled after each load step.

<u>Third phase: Unloading process</u>

Unloading starts immediately after the last loading step. At least two steps of the series of loads chosen must be selected and follow the same procedure as in the previous process in terms of the time that each load is maintained and the deformation record. The deformations will be negative since the soil swells during unloading.

<u>Fourth phase: disassembly of the cell</u>

Once the loading and unloading process is complete, and the load plates are removed from the bed, the oedometric cell is removed from the bed and disassembled.

3.3. Pore Water Pressure Oedometer Test

Once the common oedometer test is described, the pore water pressure oedometer test can be explained taking into account all features presented before.

3.3.1. Equipment

Samuli Laaksonen in his Master´s Thesis in 2014 “Improvement of research methods and testing equipment for deformation properties of clay” designed the equipment used, based on the standard oedometric cell, to which certain changes have been implemented.

It is a metallic cell of 120 millimetres in diameter, in which two compartments are distinguished: a central one, with a diameter of 50 millimetres, in which the sample will be placed, and another one formed by the remaining space, which will be full of water.

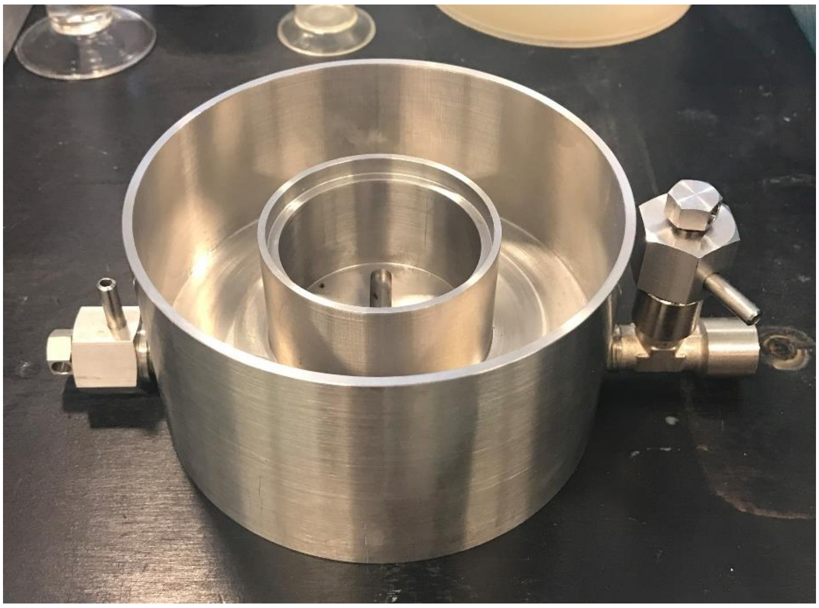

Figure 20. Modified oedometric cell

This test maintains the condition of lateral deformation prevented ($\varepsilon_x = \varepsilon_y = 0$) with modifications according to the objectives pursued in that test. However, the main difference of the cell implements to the test is that it only has a draining path, unlike the conventional oedometer test that drains to upwards and downwards. Thus, when pressure is applied on the sample, water can only exit through the upper surface of the specimen.

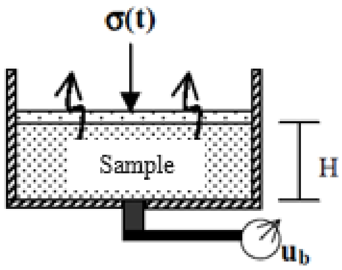

Figure 21. Modified oedometric cell scheme

Consequently, the cell consists of two valves in its lower part, which is used to measure the variations of pore pressures throughout the test. One of these valves serves to evacuate the water when placing the sample. The second valve was not necessary to perform these tests.

3.3.2. Test procedure

The procedure followed is a modification of the conventional oedometer test. Since the test and its fundamental objective present some differences, there are also some variations in the way it is performed.

In relation to the preparation of the sample, the process is the same as in the oedometer test. The size and the metallic ring used in this test will be the same, so the way the samples are prepared is the same. The sample is removed from the tube and the ring is pressed into the sample.

In these tests, the samples will have a height of 20 mm, and the rings an area of 20 cm^2. The ring is then weighted with the sample and the excess is taken to the oven to determine material properties such as the water content. On this occasion, before introducing the ring with the sample into the cell, a few previous steps are required to prepare the cell. Since the objective of the test is to monitor pore water pressures, it is very important to ensure that the sample is fully saturated, which implies that there is no air bubble left in either the cell or the water to be used in the test.

The importance of this factor was noticed in previous tests carried out by Samuli Laaksonen on his Master's Thesis "Improvement of research methods and testing equipment for deformation properties of clay", 2014. In this work, different samples were tested with different treatment concerning to air content to determine whether it affects or not the results. Figure 22 shows test results obtained by Samuli Laaksonen, when porous stones were not vacuumed to remove the air content. It shows that pore pressure peak values appear after 15 minutes since load has been applied.

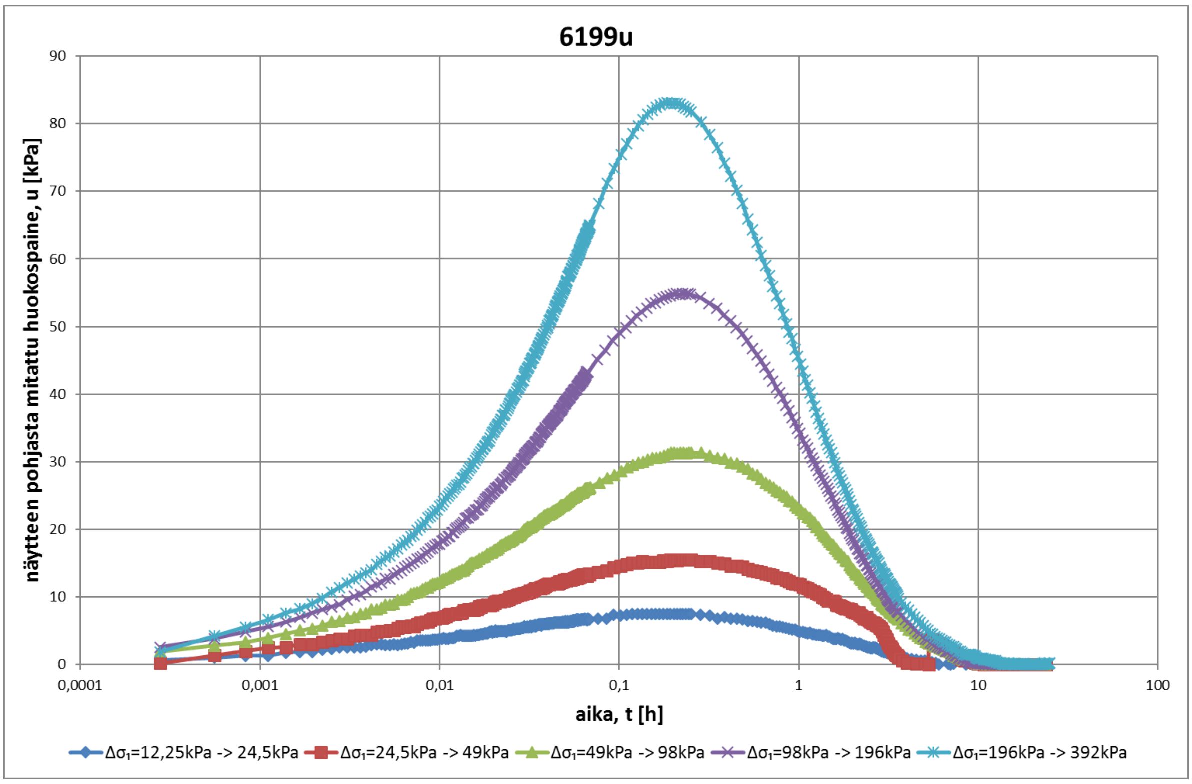

Figure 22. Pore pressure behaviour in standard conditions (Laaksonen, 2014)

In the same way, the following tests were performed with different conditions. Finally, in the last one, the whole cell and porous stones were inside the vacuum. In this test, the time lag was only about 10 seconds to reach the peak values.

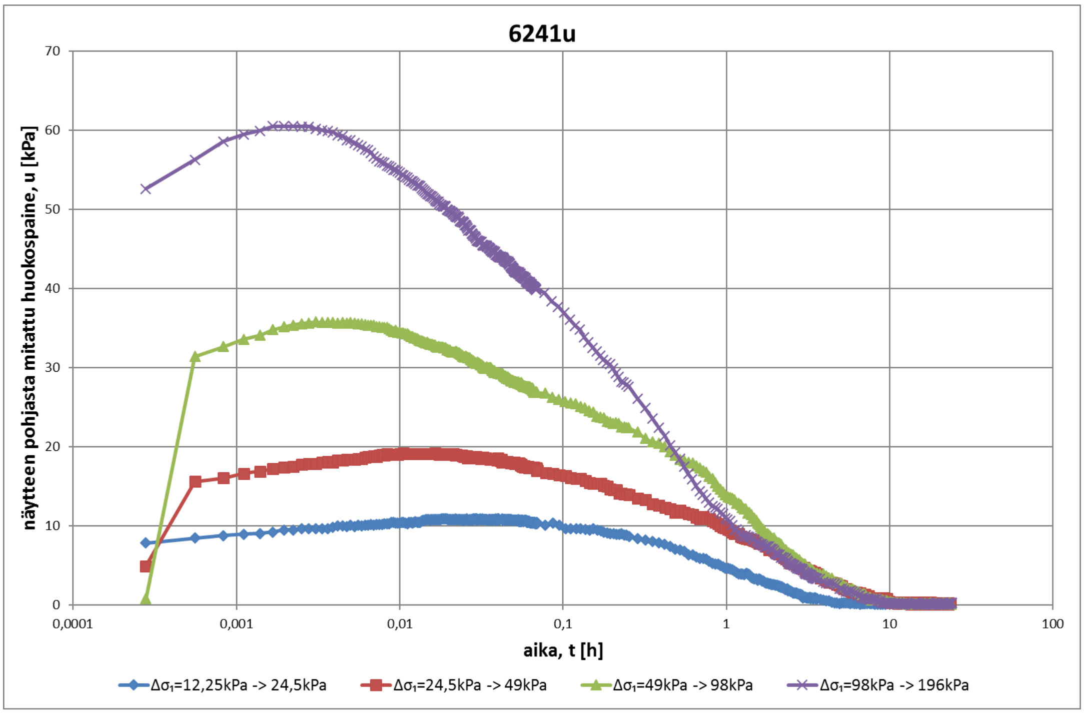

Figure 23. Pore pressure behaviour with no air content (Laaksonen, 2014)

Therefore, first of all, it must be ensured that the holes and valves of the cell correctly allow the passage of water. To do this, water is circulated through all these ducts and it is certified that they are not obstructed. Once this is done, the cell is flooded and placed in a vacuum machine. This is done to eliminate all the air that may be in the water. In the same way, a container with water is also introduced into the machine. This water will be used to fill the cell during the test and after the sample has been put into the cell, before test starts. In this way, the water will always be free of air and thus will not affect the results. The bottom porous stone and filter paper will be already placed inside the cell. It is also necessary to introduce into the vacuum machine bowls with water containing the upper filter paper and porous stone, so that they also get rid of the air they may contain.

Once enough time has passed for the water to be free of air, it is time to proceed to the assembly of the cell. Since the water must be in the vacuum machine time enough to remove all air bubbles, the soil sample must be prepared when the water is ready. In this way, the sample is not exposed for too long time.

Since the bottom porous stone and filter paper is already inside the cell, the first step is placing the ring with the sample in the sample compartment, so that it is fitted, but not inside the hole. A filter paper and another porous stone are placed on the top of sample, and with the help of the piston, it is necessary to gently press the sample, to avoid altering the sample as little as possible. In this way, the sample leaves the ring and goes into the sample compartment. At the same time as the sample is pushed down, the valve is open so the water can be poured out.

Figure 24. Sample placement

Once inside, the cell is ready. It is important to check that there are no remains of the sample in the ring during the lowering of the sample. If so, the results can be distorted.

With all this, it would only be necessary to calibrate the bench and adjust the different measuring devices: both the manual reader and the digital one. Normally the digital readings are recorded automatically in a spreadsheet. On this occasion, a specific software used in triaxial testing has been chosen for the reading of pore water pressures. This measurement software records changes in height of the sample and pore pressure, and measurement frequency is 1 second.

When everything is ready, the loading process can begin. The procedure is similar to the conventional oedometer test. Since the intention is to better understand the behaviour of water pressures, the load increments are carried out when the pressures have stabilized. Likewise, in these tests, the whole process of increasing the load will not be carried out either. The sequence is shown in Table 3.

Table 3. Load step sequence

Load step	Weight (g)	Load increment $\Delta\sigma$ (kPa)	Current pressure $\Delta\sigma$ (kPa)
1	62.5	3.123	3.123
2	62.5	3.123	6.246
3	125	6.246	12.492
4	250	12.492	24.984
5	500	24.984	49.968
6	1000	49.968	99.936

The reason is that it is not necessary to load the sample that much, since the settlements are not the main objective. The pore pressure measurements are the objective, so that these steps are considered sufficient. In addition, the capacity of the pore pressure sensor is limited. The new load sequence implemented in the second set of samples is the one shown in Table 4.

Table 4: New load sequence

Load step	Weight (g)	Load increment $\Delta\sigma$(kPa)	Current pressure (kPa)
1	62.5	3.123	3.123
2	62.5	3.123	6.246
3	125	6.246	12.492
4	250	12.492	24.984
5	250	12.492	37.44
6	250	12.492	49.932
7	1000	49.968	99.936

In this way, the loads to which the sample is subjected vary, but the total load or vertical stress at the end of the test is the same. Therefore, it is possible to compare the results at the end of the tests, since all samples are equally loaded.

When the loading process is completed, the sample is unloaded. The unloading process is not exactly the same as in the oedometer test. In this case, the different weights are removed at intervals of 1 min and the readings are taken. Finally, the last step is left 15 min to finally disassemble the bench and the cell. This is because the aim is not to study the swelling behaviour. Therefore, a faster, but controlled, unloading is performed.

It is important to say that this second group or set of tests will not be analysed in the same degree of detail than the eight first testes. The importance of these first eight tests is greater, and the second set of tests will be analysed and described taking into account what has been previously determined.

4. Data analysis

The first phenomenon to be analyzed in this test is the development of the load steps, once it is transferred to the sample. Within this process, the first thing to be analyzed is how these loads are transferred to the sample, and how long it takes the porepressure to reach its maximum value. It is important to compare these values both for the same step loads at different depths and for the different step loads for the same sample.

Since it is assumed that, the whole vertical pressure transferred to the sample is absorbed by the water particles ($\frac{u}{\Delta\sigma} = 1$), it is necessary to analyse the peak values reached in the tests. Furthermore, it is also assumed that this phenomenon takes place instantly, so it is al also interesting to monitor the times.

For the development of this section, only the last four load steps have been used. The reason is that, during the realization of the tests, these were the steps with more prolongation in time, so that it is more comfortable to operate with them. In the same way, since they are greater loads, the behaviors and results are seen more clearly. The first two steps are perfectly valid, but on this occasion, it has been preferred not to take them. In addition, in this way, the results reflected in tables and graphs are more ordered and clear.

4.1. Pore pressure rise and time required

The application of a load, or what is the same, the increase of the total pressure in a saturated soil sample has a particular behavior. The application of this load, according to the conventional consolidation models, results in an immediate increase in the water pressure of the soil pores. However, in the tests performed it can be seen from Figure 25 that this phenomenon does not follow, exactly, this behavior.

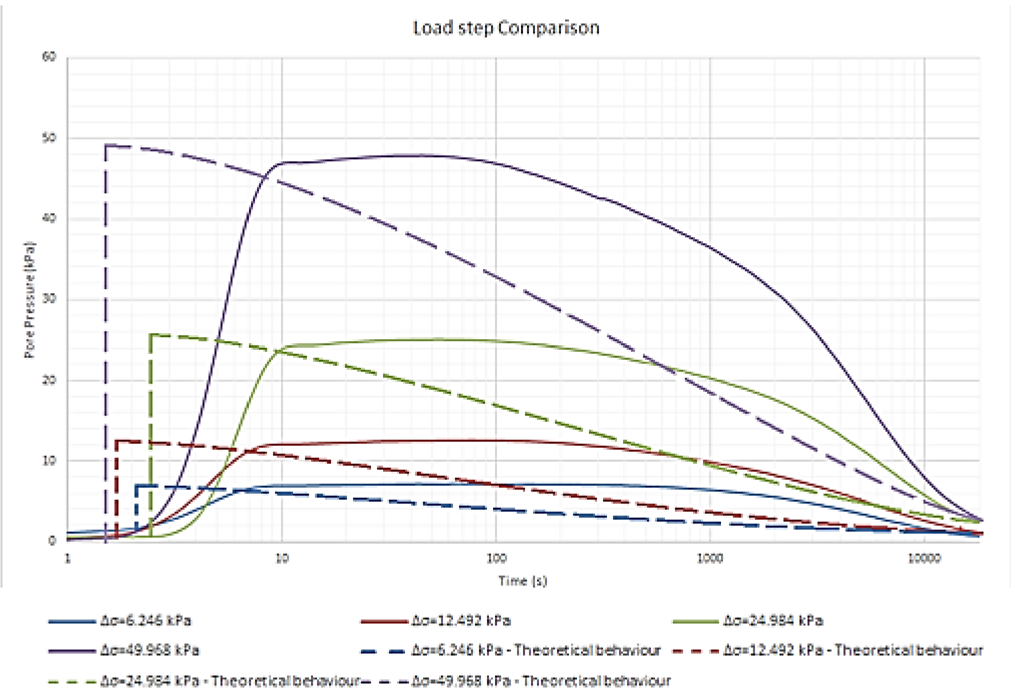

Figure 25: Step load comparison

The first points of the graph correspond to the state at rest. The sample has been applied a step load, but it has already dissipated the excess pore pressure. Then, a branch of great slope follows, which represents the sudden rise of pore water pressures, because of the application of the load corresponding to that step. In theory, this branch should be vertical. When the loading is assumed to be instantaneous, therefore, the graph should rise vertically until it reaches the value corresponding to the total pressure level that has been applied at that instant. However, this branch is not vertical, but has a lower slope, although it is very sharp. This implies that the terrain requires some time to mobilize all the pressure that is transferred to it. In addition, the pore pressure is measured from the bottom of the cell, while the pressure is applied on the top. This might be also a reason for this behavior, since there might be some delay in the transmission of the stresses.

Another very important aspect that can be seen in Figure 25 is that the maximum value of the pore pressure is not reached directly in that first branch. In the first branch or period, a high percentage of the total load applied is reached, but not all. The remaining value of the pressure is developed in a very different way. This is done throughout a branch of slope much smaller than the previous one, which implies much more time.

Thus, the process of ascent or transmission of the total pressure to the water particles can be divided into two clearly differentiated stages. First, there is a gross increase in pore pressure, reaching much of the total pressure transmitted to the sample. After reaching this value, this pore water pressure continues to increase, albeit much more slowly and gradually, until it finally reaches its peak value. From that moment, the pore pressure will begin to dissipate.

It would be reasonable to consider that, at lower pressure increase, less time would be required to reach that value. In the same way, more time would be required in the opposite case. However, it can be seen that the time elapsed from applying the load until the maximum pressure value is reached is lower when the step load is bigger. In other words, the lower the step is, the more time the sample requires to reach the maximum pore pressure value. Figure 26 and Figure 27 show the development of all the step loads carried out on the first set of tests until the maximum pore pressure value is reached:

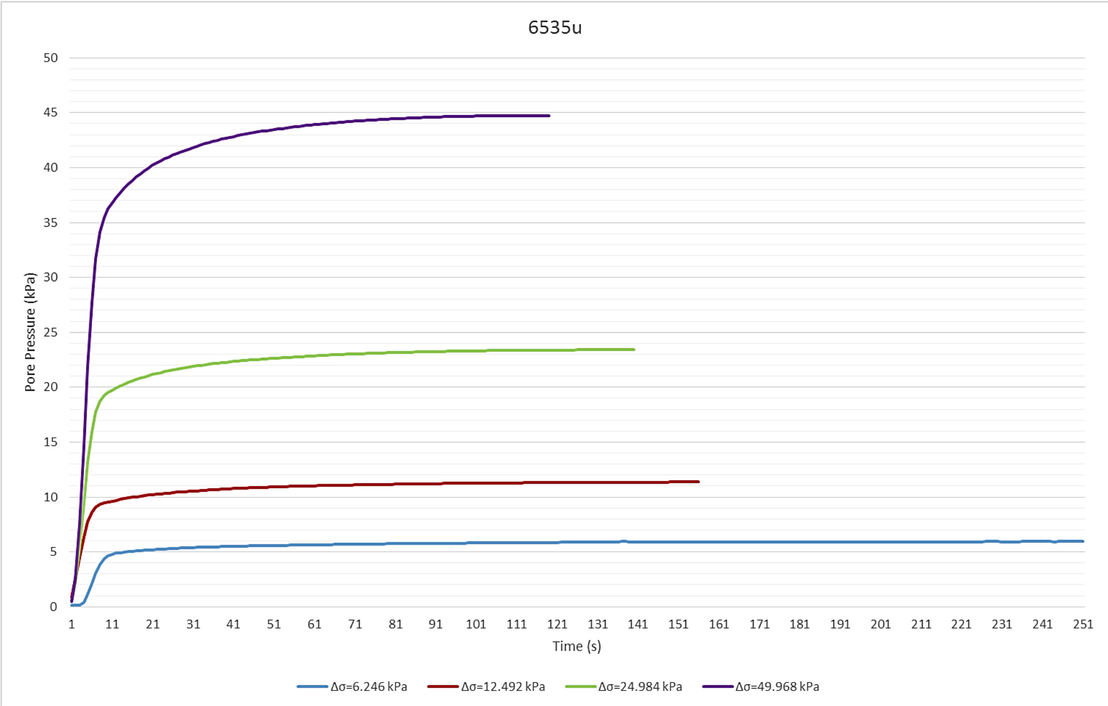

Figure 26. Test 6535u

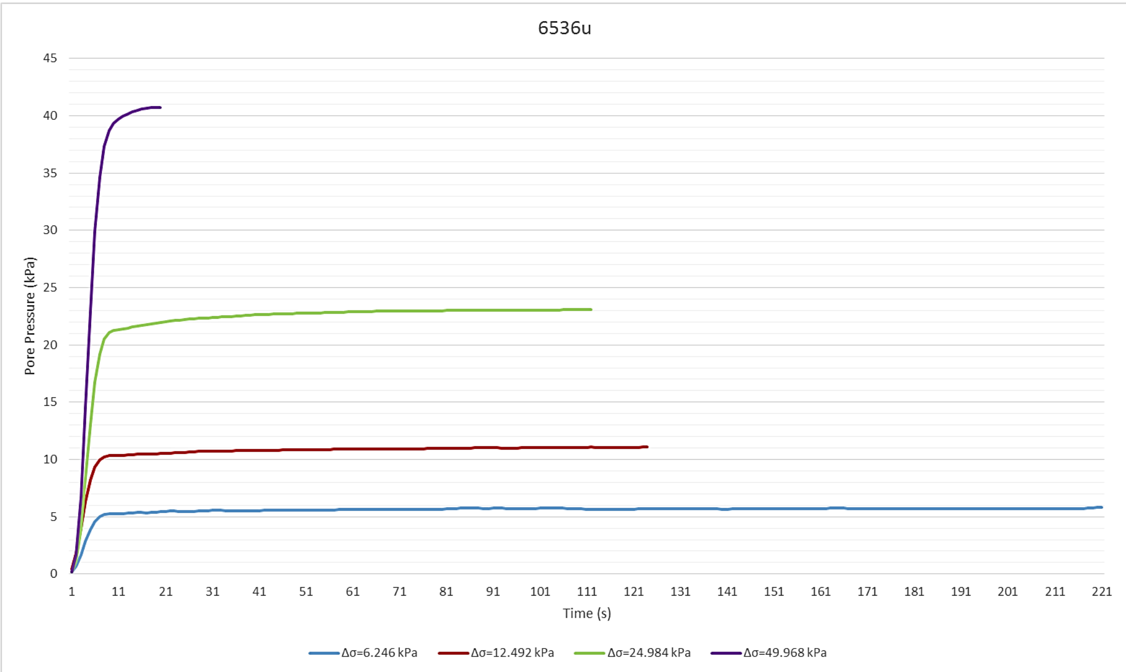

Figure 27. Test 6536u

In these graphs can be seen clearly how in the steps of higher load, the time required to reach the maximum pressure is lower than in the previous steps, although there are some exceptions. As it can be seen, if the analysis is done comparing samples or depth, the amount of time needed to dissipate the excess pore pressure is continuously reduced. In the same way, if the focus is on the step load, the behavior is the same: the excess pore pressure dissipates before as the depth decreases.

Table 5 resumes the amount of time required for each sample to reach the peak value at each load step, and also show how it evolves:

Table 5: Time required to reach peak values

	6535u 2.12-2.09 m	6536u 2.09-2.06 m	6544u 2.06-2.03 m	6545u 2.03-2.00 m	6549u 2.00-1.97 m	6550u 1.97-1.94 m
12.492 kPa	251	221	43	115	109	55
24.984 kPa	156	124	78	66	84	95
49.968 kPa	140	112	54	11	79	28
99.936 kPa	119	20	45	13	68	13

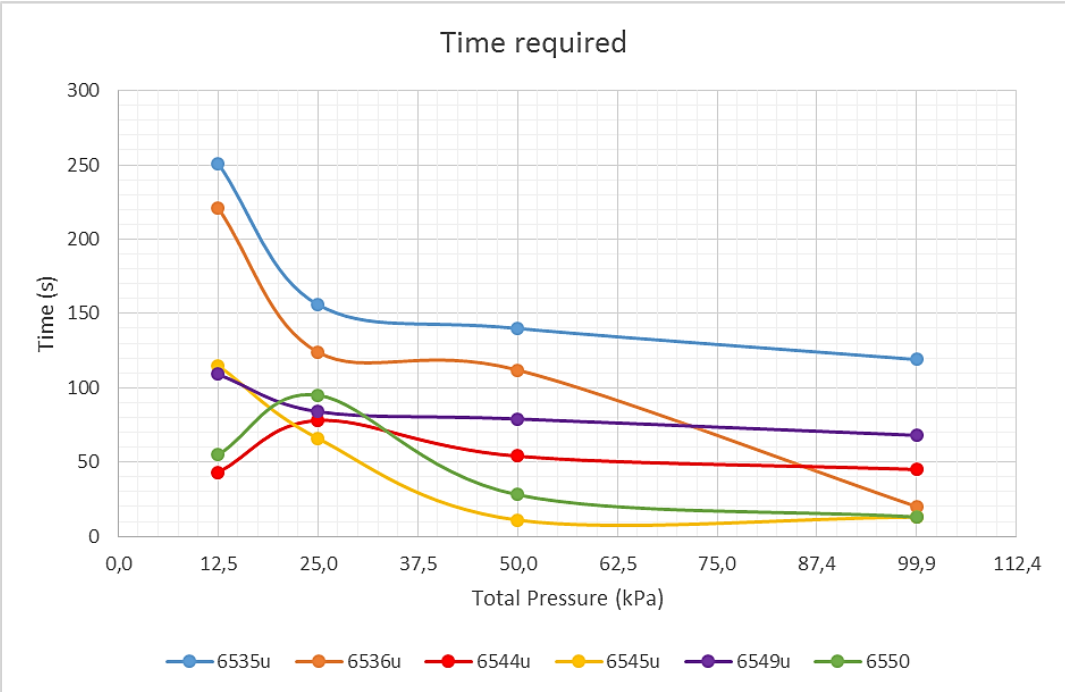

Figure 28. Time required to reach peak values

It is important to note that at 12,5 kPa load increment, the current load is around 25 kPa, and preconsolidation pressure is around 25-30 kPa (see 4.2.2. Preconsolidation pressure) The behavior is quite different in these first steps, it means that, at overconsolidated state, pore pressure increments behave differently.

However, once the process of sudden pore water pressure increase ends, the second branch of pore pressure rise is formed by a set of very similar values. The differences between these values are so small that there is hardly any difference between the beginning of this branch and the final value of it. This may be due to small disturbances that may occur in the measuring device due to blows or vibrations. These kind of disturbances may lead to small rises or decreases in pore pressure that make longer its dissipation. It also possible that there are some irregularities in the sample structure, so the water does not leave sample in a regular way. This leads to think that something of the said above may be happening, and that, therefore, it is not representative of the load step.

In fact, within this slow and prolonged ascent of the pore pressure, slight ascents and descents of this pressure occur, when it should be constantly increasing, if it is in the phase of pressure rise. Figure 29 shows this phenomenon:

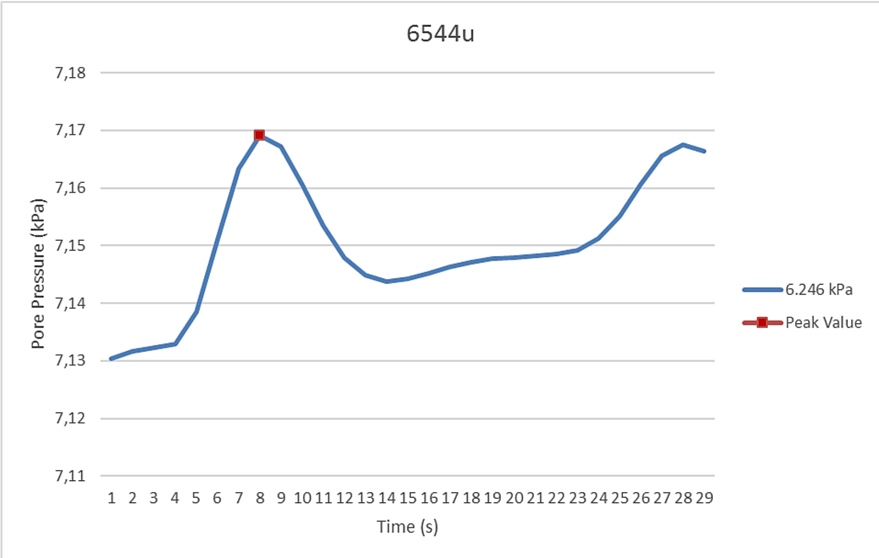

Figure 29: Peak value at test 6544u

Once the maximum pressure value has been reached, it should begin to slowly dissipate. However, what happens is that the values of the pressure go oscillating upwards and downwards without following any pattern. As stated above, this may be due to the expulsion of some air bubbles in the soil sample. It could also be due to some type of contact or vibration on the table or the oedometer cell, which is reflected in the data reader. In any case, these differences are very small, without becoming significant.

However, this suggests that the peak value reached by the pressure in the sample could also be the result of some type of disturbance as already mentioned above. Especially because most of peak values reached during the tests need quite much time.

Therefore, it can be concluded that the data provided by the pressure reader immediately after the application of the load are more reliable. In the very early stage, the pore pressure increments are much bigger, so any disturbance that may happen is not so crucial. In the late stage, where pore pressures are much smaller, those disturbances may change the results significantly. This implies that the interesting data is the one read immediately after the strong rise of pore pressures. As soon as the path of the curve acquires an almost horizontal slope, we will consider that the pressure has reached its peak value.

To select which data are useful, or which are interesting for pressure analysis, it is necessary to apply some kind of criteria. The first is based on the relationship between the increase in pore pressure and elapsed time. So that according to the value of this relation, the data will be accepted or rejected.

Since once the rectilinear section of the pressure rise has started, the differences between the values are very small, it has been decided to take as threshold an increase in the pore pressure of 0.02 kPa. From this value, the variations in the pore pressure are very small and with little variety. It is important to note that, once this threshold is reached, all other data will be rejected. As it has been said before, it is possible that in subsequent readings higher pressure increases are recorded, and therefore they should be taken into account. However, as before, this may be due to disturbances in the sample or work equipment, so they will not be taken into account. By applying this criteria, the step loads would have this shape:

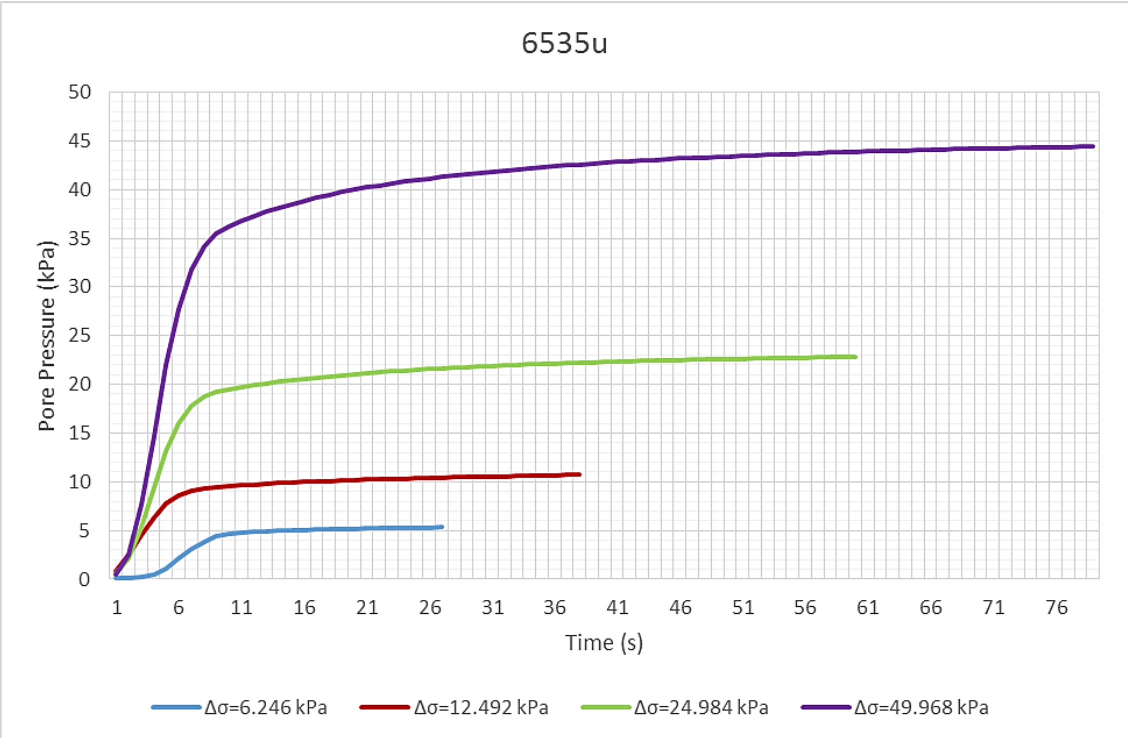

Figure 30: Corrected Step load test 6535u

The shape of the load step is very similar to those shown above: it still has two distinct branches. However, on this occasion, the slow ascent branch is much smaller, and its importance can be evaluated. Despite having a lower slope, it can be seen how the pressure increases significantly especially at the biggest steps.

In addition, by applying this new criterion, the situation previously described is different. Now the biggest step loads are the ones that take more time to reach their maximum values, contrary to what was happening before.

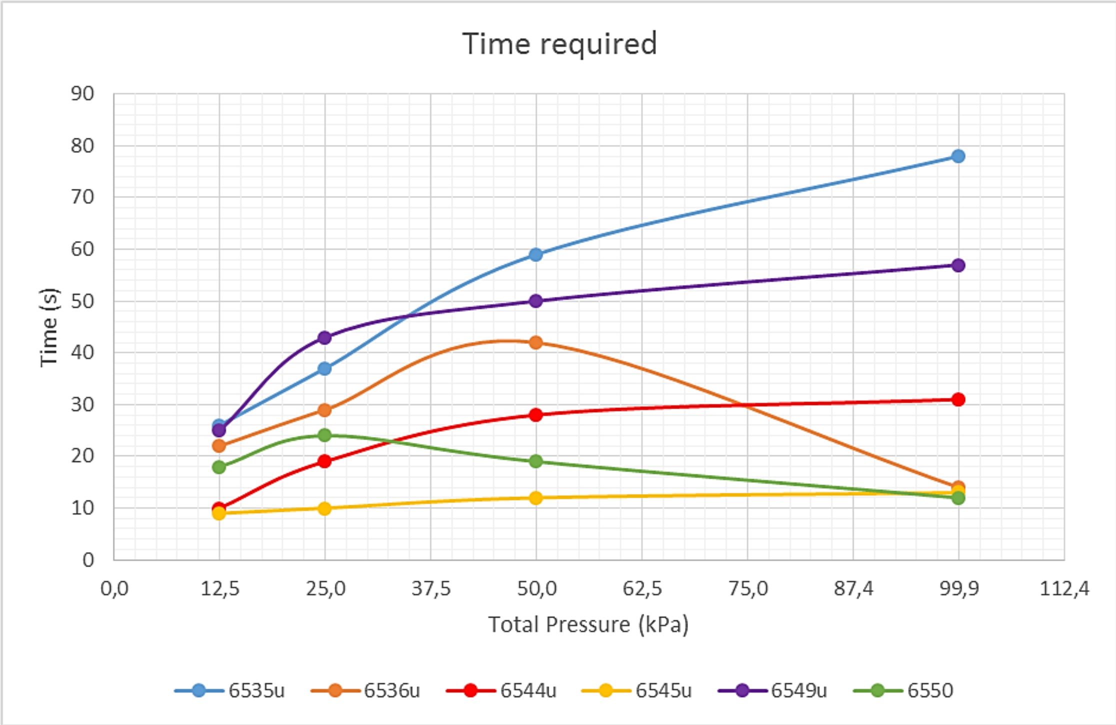

Figure 31: Time required to reach peak values in corrected load steps

The values of the pore pressure reached at the end of the step, once the correction has been applied with this criterion, are not very different from those previously achieved. In spite of having eliminated a considerable amount of readings corresponding to the branch of slow and gradual rise of the pressure, its value is not affected in a relevant way. This indicates that the increases in pressure after their strong ascent are so small that, despite their high number, it is not a significant proportion of the final value of the pore pressure.

However, even though they are very small values, it is such time span that it really should be a relevant proportion of the value of this pressure. This is because, as mentioned above, throughout this time in which it seems that the pressure continues to increase slightly, what happens is that small increases and decreases of pressure are alternating, although in general pressure keeps increasing slightly. This may be due to possible disturbances that the cell may suffer during this period. That is why it is interesting to remove some of this data.

Table 6 shows the differences in the pressure value when applying this reduction (Figure 30, Figure 31) of the readings:

Table 6: Comparison between corrected and original values (kPa)

	6535u		6536u		6544u	
	Corrected	Normal	Corrected	Normal	Corrected	Normal
6.246 kPa	5,331	5,957	5,469	5,829	6,923	7,169
12.492 kPa	10,713	11,406	10,700	11,069	12,269	12,589
24.984 kPa	22,839	23,409	22,658	23,079	24,863	25,037
49.968 kPa	44,405	44,734	40,157	40,719	47,769	47,846

	6545u		6549u		6550u	
	Corrected	Normal	Corrected	Normal	Corrected	Normal
6.246 kPa	5,945	6,651	5,959	6,416	7,178	7,410
12.492 kPa	11,457	11,852	11,115	11,404	11,227	11,284
24.984 kPa	22,777	22,777	22,326	22,627	22,688	22,761
49.968 kPa	41,582	41,767	42,203	42,297	42,058	42,058

When comparing these values with the original ones, the corrected values of the pore water pressure are, in all cases, higher than 90% of the maximum-recorded value. Thus, in order to simplify the reduction of the data and thus to unify all of the above in a single more general criterion, it is possible to adopt as representative value of the test, that the maximum pressure reached equals 90% of the maximum value registered by the measure. In this way, it is possible to obtain a value of the pore water pressure very close to the maximum value truly reached, and at the same time, a significant reduction of the data or values located in that branch of slow ascent is carried out. The result is a more defined step load that fits the phenomenon better.

Thus, after these corrections in the data collection for the elaboration of the load steps, it is determined that at a higher load, more time is required to reach the peak value of the pore pressure. The reasons why the heaviest steps require more time to reach their peak values lies in several factors.

The first reason, and most obvious, is that they must achieve a higher value of pore pressure. When the water pressure rise becomes slow and gradual, the higher the applied load, the longer the soil sample needs to reach that peak pressure value. The lower the load steps are, once the increase in pore pressure weakens, the more easily arrives at the peak value of the step.
This second part of the rise in water pressure, represented by this horizontal branch in the previous graphs, is a phenomenon that presents an equal behavior in all cases. It could be said that the time required is directly proportional to the magnitude of the load applied to the soil sample. The higher the load is, the longer it will take to reach the maximum pressure value.

However, the beginning of the pressure rise, which corresponds to the great slope in the graphs, has a very similar behavior, regardless of the load step applied. The development of the sharp rise of pore water pressures at the beginning of the load step is the determining factor in this process. Figure 32 shows the sections corresponding to this phase of the load step.

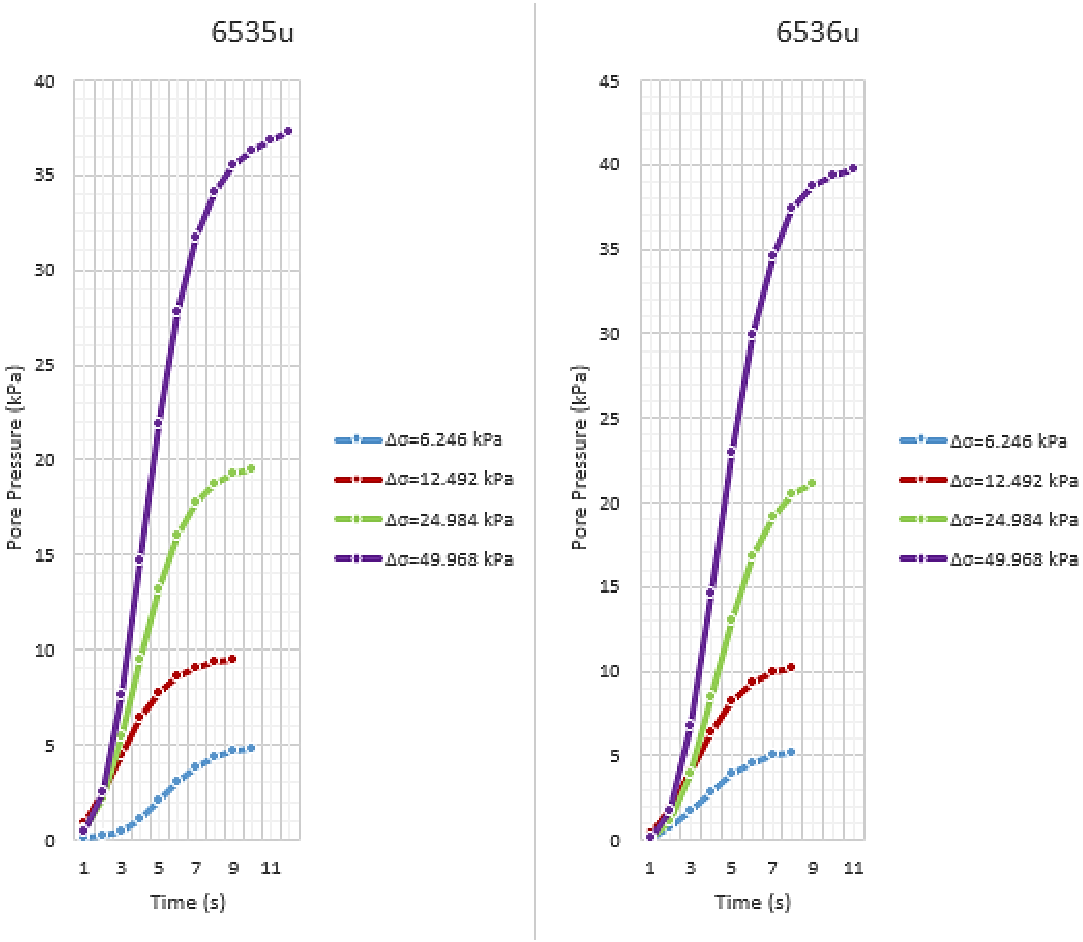

Figure 32. Load step early phase

The time elapsed since the load is applied until the pressure increments change their behavior is very similar, regardless of the value of the load. Thus, it takes about the same time to stabilize a sample loaded with the first step, than with the last one.

Since the required time is very similar, the higher the load applied, the greater the pressure increase per unit time. Table 7 shows the times of each of the steps, as well as Figure 33.

Table 7. Early phase time required

	6535u	6536u	6544u	6545u	6549u	6550u
	Time (s)					
6.246 kPa	10	8	9	9	9	8
12.492 kPa	9	8	9	9	10	8
24.984 kPa	10	9	10	11	11	10
49.968 kPa	12	11	10	10	16	10

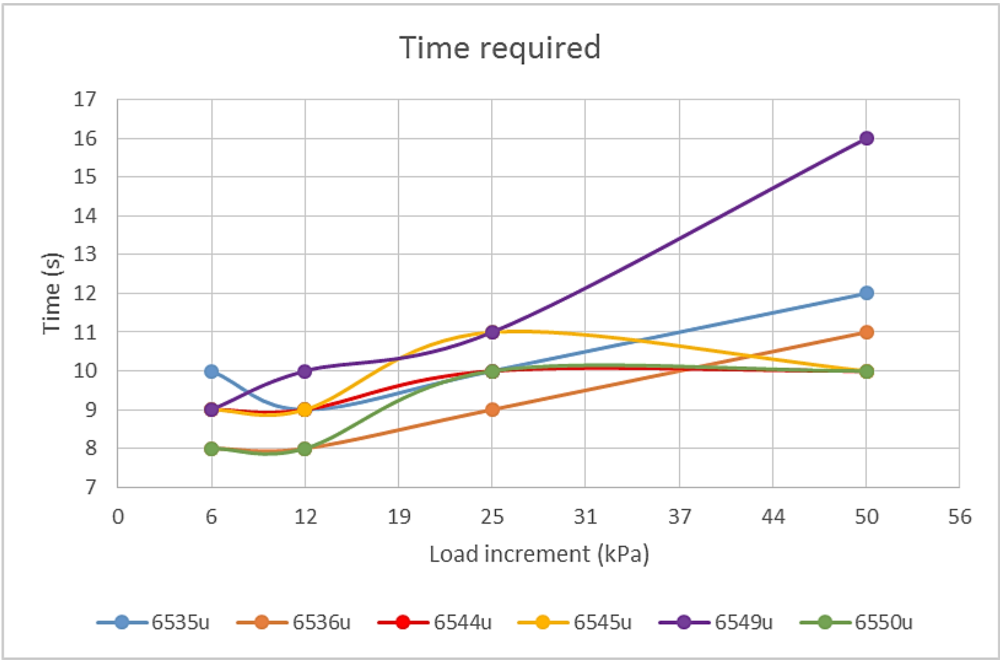

Figure 33. Time required in load step early phase

All the times have similar values, between 8-12 s. This shows that pressure, regardless of its value, requires a specific time to reach its maximum value until its behavior changes. Therefore, this implies that, for the times to be always so similar, it is necessary that, in the bigger load steps, the water pressure increases more drastically than in steps of less intensity. The pressure will increase at different rates, but always occupying a time within this interval. This is reflected in the slopes of this phase of the phenomenon. The steps of bigger load have a sharper slope. Consequently, this will also be reflected in the increments of pore water pressure per unit time. If the time that the steps occupy is similar, during the more loaded steps there will be larger increases of the pressure per unit of time. The final steps are those that undergo much greater ascents per unit of time, or increment rate.

Table 8.Pore pressure increments per unit time (kPa/s)

	6535u	6536u	6544u	6545u	6549u	6550u
	Pore Pressure increment (kPa)					
6.246 kPa	0,979	1,177	1,323	1,125	0,984	1,166
12.492 kPa	2,015	2,304	2,586	2,439	1,822	2,348
24.984 kPa	4,024	4,619	5,289	5,051	3,535	4,745
49.968 kPa	7,196	8,357	10,505	9,629	6,161	9,253

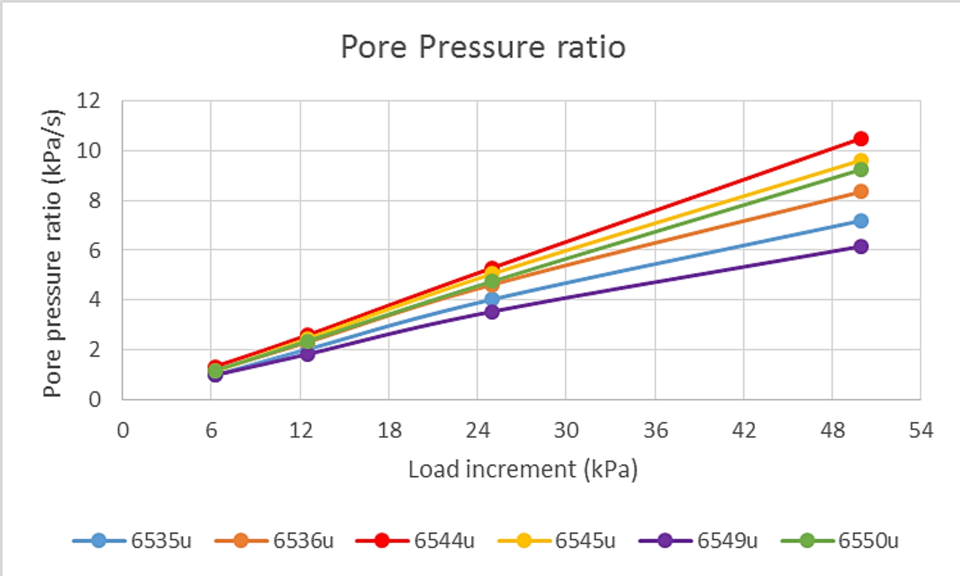

Figure 34. Pore pressure – Time ratio

The most important of the load step set is the percentage or proportion of the maximum pore pressure that the sample reaches during the first few seconds of the load. This factor conditions everything mentioned previously. The higher the value of the pressure reached at the beginning of the process, the lower the remaining value to be completed in the second phase of the pressure rise. Therefore, if the remaining pressure is lower during this stage, less time will be consumed to reach the final value. Therefore, it is very important to know what percentage of the maximum pore pressure is reached in this first phase.

4.2. Pore pressure maximum value

The next aspect to be analysed is the maximum value of pore pressure that is reached during the tests, in relation to the applied pressure value. In other words, if it is possible to transmit the totality of the load applied to the soil sample, and if not, in what proportion it is achieved.

In order to make a comparison between the value of the applied load and the value that is transmitted to the sample, first, it is important to know if the equipment used works properly and the load is transmitted correctly. For this purpose, it is necessary to carry out load tests on the different oedometers to verify if they behave properly. The same step loads have been reproduced the different soil samples will hold, and these are the results obtained.

Table 9: Efficiency load test results (kPa)

	Theoretical value	Performed value			
Oedometer number	-	6	7	8	9
Area (cm2)	20	20	20	20	20
Stress (kPa)	**3,123**	3,08	3,06	3,09	3,06
	6,246	6,152	6,120	6,173	6,125
	12,492	12,30	12,24	12,35	12,25
	24,984	24,61	24,48	24,69	24,50
	49,968	49,21	48,96	49,38	49,0

None of the tests show the theoretical value of the pressure to be reached when applying the different loads. The load cell is quite accurate, but there are small differences in the oedometers (dimensions, small deformations caused by previous testing etc), and that's why the load transfer is a bit different. In any case, the values are very close to the theoretical ones, and the load is considered to be satisfactorily transmitted.

Since the applied load is not fully transmitted, for the maximum pressure analysis or study, it is logical to make this comparison based on the performance of the laboratory equipment. The most accurate way to do this analysis would be to carry out the comparison of each test or each result in comparison to the values obtained in the oedometer in which it has been tested. However, this is a much slower and laborious process. In addition, since the differences between the results provided by the different oedometers are very similar, it can be assumed that all laboratory equipment is in an identical state.

Thus, assuming all of the above, the average of the values obtained for each oedometer would be carried out, and that provides the final value to be used for the comparison between pressure values in the test and in theory (Table 10):

Table 10: Equipment accuracy

	Theoretical value	Mean value	Efficiency (%)
Stress (kPa)	3,123	3,07	98,344
	6,246	6,14	
	12,492	12,29	
	24,984	24,57	
	49,968	49,14	

Before comparing the data, it is important to note that in some of the tests carried out, the final value of the pore water pressure is higher than the value of the load step applied. In other words, pore water pressure peaks are higher than the applied load. This can have its origin in several reasons. The first would be related to the process of application of the load itself. The weights used to simulate the load steps must be placed on the oedometer manually. Therefore, it is possible that the placement of these elements is carried out in a more abrupt way than recommended. This implies that the soil sample is subjected to some kind of additional load. These overloads are very small but can be large enough to significantly alter the results. The second reason why this may occur, which is more likely to happen, is that at the end of a load step, before proceeding to start the next one, the pore water pressure has not completely dissipated. So when the next load is applied, the pore water pressure generated is added on the current pore pressure in the sample and the final value increases. These values are in the approximate range of 0.5-1.5 kPa, which is not very high. However, since we are performing checks between very similar values, these added pressures may be of importance.

To correct this situation, it is necessary to exclusively pay attention to the step or increase of the pressure itself. That is, the peak value of the pore water pressure will be the one given by the measuring instrument, but subtracting the value of the pore water pressure before applying the load step. The calculations and values obtained when performing this correction are reflected in Table 11, Figure 35 and Figure 36.

Table 11: Peak value and accuracy at each load step

	6535u - 6536u				
	Initial value	Maximum value	Peak value	Mean value	Accuracy
6,246 kPa	0,148	5,957	5,809	6,140	0,946
	0,154	5,829	5,676	6,140	0,924
12,492 kPa	0,885	11,406	10,521	12,290	0,856
	0,404	11,069	10,665	12,290	0,868
24,984 kPa	0,553	23,409	22,855	24,570	0,930
	0,190	23,079	22,889	24,570	0,932
49,968 kPa	0,485	44,734	44,249	49,140	0,900
	0,167	40,719	40,552	49,140	0,825

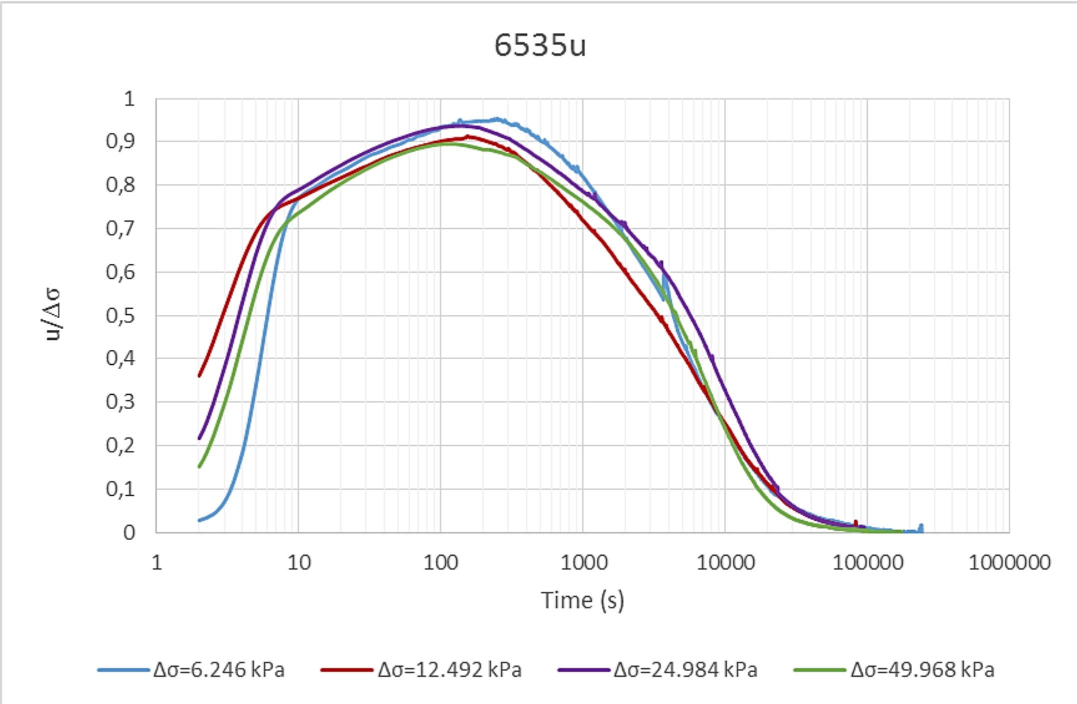

Figure 35. Relationship between pore and total pressure

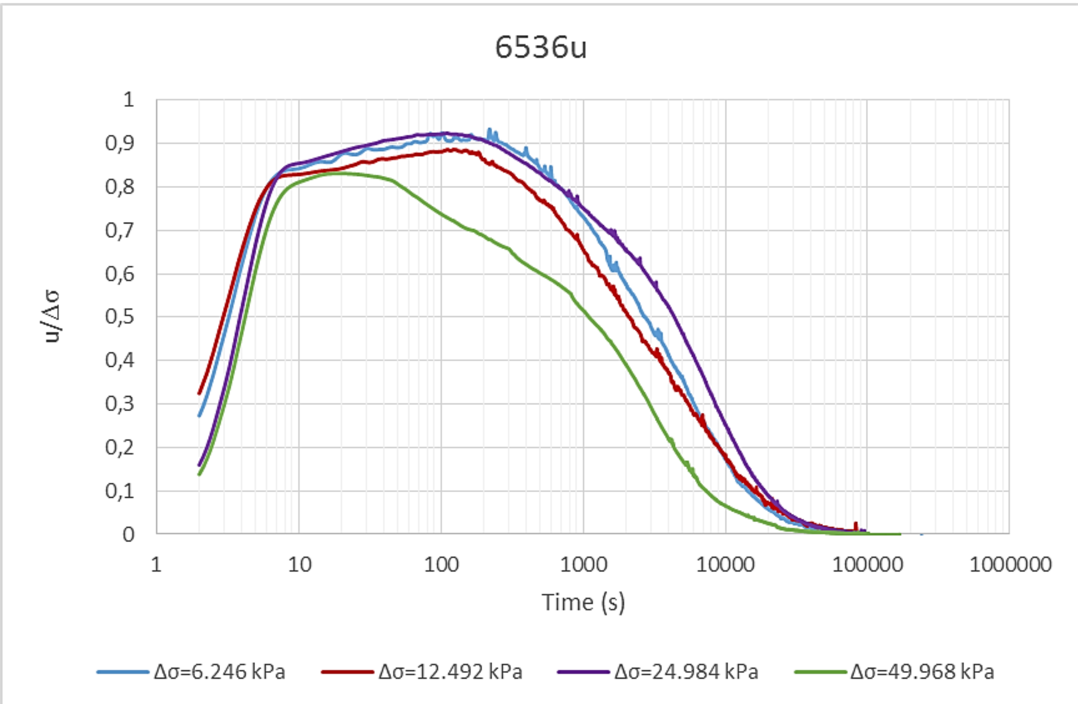

Figure 36. Relationship between pore and total pressure

As expected, none of the readings indicates that there has been an increase in pore pressure greater than the load applied at each step. This indicates that the loading procedure has been performed properly, without generating excess pressure. Since the performance of the laboratory equipment is 98%, it is expected that the load transmitted to the soil samples will be reflected in the pore water pressure with the previously corrected peak values. However, the results obtained when applying loads are not as expected.

Although it is true that the recorded values of the pore pressure are always near to the load steps, none of these steps has succeeded in achieving an ascent of the pore pressure corresponding to 100% of the applied load. Assuming the test equipment has worked correctly, and the load has been properly transmitted, some of this load has not been absorbed by the water. A percentage has not been transmitted to the sample water particles, and therefore has been transmitted directly to the solid phase of the soil.

Under normal conditions, the load must be fully absorbed by the water particles when a soil is saturated. This implies that something is failing in the saturation conditions of the sample. It would be understandable not to record a 100% pressure increase, since the theoretical model is very difficult to reproduce in reality, and there is always some kind of disturbance or error. However, in some cases, there are pressure values that are far from reaching that maximum value (around 80%).

As can be seen in Figure 35 and Figure 36, generally, as the test goes forward, the maximum values of the pressure are lower in relation to the value that they should reach theoretically. When the load is applied, the water ejects from the pores, as well as the excess water from the interior of the cavity in which the sample is stored. It is possible that the water can not be able to access the sample cavity completely, so that the sample loses part of its saturation condition. Thus, as the test progresses, the degree of saturation of the sample is less. Consequently, less of the load is absorbed by the water, and hence, it is the soil that has to take care of the remaining load.

4.3. Second set of tests

In the previous tests, it has been verified how the soil samples subjected to incrementally bigger loads behave. This time, the results are obtained from testing the samples to equal load step to compare and analyze them.

4.3.1. 12 kPa / 250 g load step comparison

As shown in Table 3, the steps of 12.49 kPa are those that will be repeated a total amount of three times. Therefore, the behavior of the soil samples during these loading steps will be analyzed.

Figure 37 shows these load steps, which have been applied samples 6558u and 6560u. As can be seen, all results show very similar characteristics. However, the load steps corresponding to each test can be perfectly distinguished. It can be seen that the curves are divided into two groups. The three curves that form the lower group of the graph correspond to the same test sample (6558u). In contrast, the group at the top forms the other test (6560u). Each of these branches represents the first, second and third step of 12.49 kPa of each test.

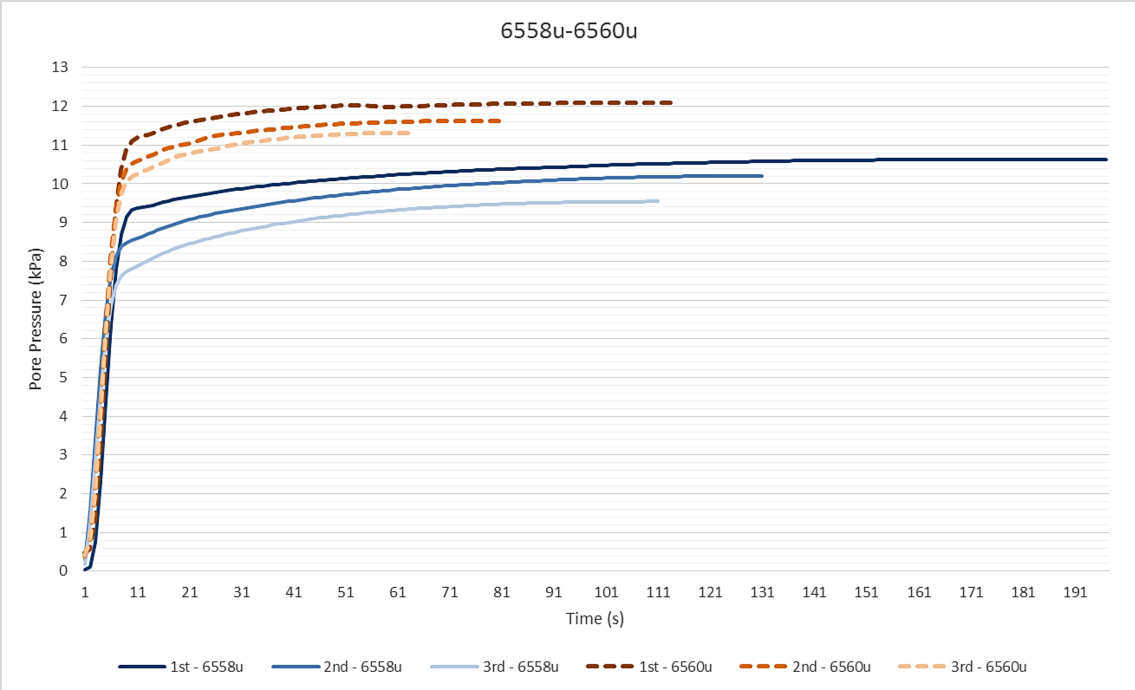

Figure 37. 12.49 kPa load step representation

Both samples show the same behavior and evolution throughout the test. It would be reasonable to assume that the following steps require the same time to reach the peak pressure value, since it is the same load step on the same sample. However, it can be seen how the value of time decreases as the loading steps are applied. The first load steps are those that require more time to reach the maximum value of the pore pressure. These variations in time are also related to the maximum pressure value. Therefore, as explained above, the lower this time, the lower the pore pressure will be. The different steps are decreasing in time and final value of the pressure successively in the two samples. Likewise, the maximum values of the pressure reached in the very early instants after the load application also decrease as the test goes forward.

The two samples have identical behavior, but differ in the time values and the pressure reached at their maximums. This is due to the different depth to which the samples were in the soil, which makes them slightly vary their properties, such as permeability, and this affects these values.

4.3.2. Final load step comparison

In this second group of tests it is also interesting to analyze the last load step, corresponding to the addition of 49 kPa. As explained above, all samples have been equally loaded at the end of the test (a total of 2 kg or 99 kPa). However, two of the samples (6558u and 6560u) have reached this load throughout a different paths. Figure 38 shows the results obtained after the application of the last load step at all these four samples:

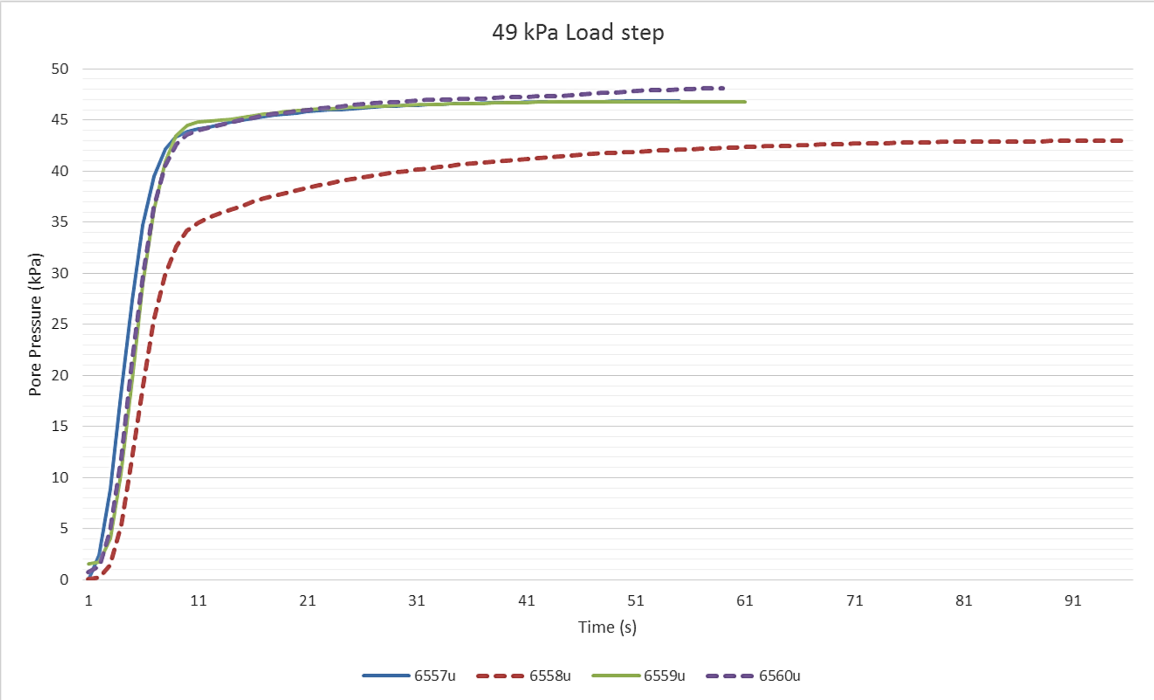

Figure 38. 49 kPa load step representation

Since the samples come from a equal state, in which they are all subjected to a total pressure of 49 kPa, there are no major differences in the development of the loading step. All samples present similar values of the maximum pressure reached, both the peak value and the value in the very early seconds after the load application. However, one of the tests shows a different behavior, although the form of the development of the pressures is similar. These differences may be due to factors such as placement of weight in the equipment in a different manner, or variations in the degree of saturation with respect to the other samples.

The conclusion that can be drawn from these tests is that, if samples reach the same stress state, the stress trajectory or sequence does not affect the development of pore pressures. This is a result that should be expected, since all samples start at the same stress level and all are subjected to the same load step.

Since the development of water pressure in the saturated soil is directly related to the settlement, it would be logical to think that the values of their values will also be similar.
Table 12 represents the settlement at the beginning and end of the last load step, as well as the total settlement produced:

Table 12. Settlement during last load step (mm)

	6557u	6558u	6559u	6560u
Initial height	17,85	18,21	18,21	18,60
Final height	16,68	16,81	16,95	17,19
Settlement	1,17	1,40	1,26	1,41

The total seat produced in the different tests is very similar, although the applied loads have been different. As before, the sequence of loads is not a relevant factor in the development of consolidation settlements. In any case, it can be seen that the samples that have experienced this variation in the application of the loads (smaller LIR) have suffered slightly higher settlements than the other two samples.

4.4. Consolidation process comparison

Once all the test results have been obtained and analysed, it is also interesting to compare them with traditional methods. The objective is to see if the obtained results are similar to what these methods say, and to verify that they represent all these phenomena well. Consequently, it will also be necessary to analyse how pressure is transmitted to the water, and also how it is dissipated. However, in order to compare these phenomenon with other models of analysis, in this section are also included time-settlement analysis and definition of preconsolidation pressure or soil permeability.

4.4.1. Casagrande and Taylor models

Taylor and Casagrande models allow to establish the theoretical limits between the different types of consolidation that a saturated soil suffers when total pressure increases. In addition, it allows to know the settlements for these limits of the consolidation.

The primary consolidation is the change in volume water removal from soil voids. Only when all the excess pressure has been transmitted to the soil particles, the primary consolidation is finished. The other changes in the volume of the soil are due to another series of phenomena that are included in the secondary consolidation.

The pore pressure readings obtained in the tests can be analysed together with the consolidation models according to Casagrande and Taylor. In this way, it is possible to verify if the development of the pressures confirms to these models or if there are variations. In addition, it is interesting to observe how the settlement has evolved during the execution of the tests. As explained above, the measuring devices not only provide the readings corresponding to the pore pressure of the sample, but also the deformation.

In order to verify this phenomenon, the Casagrande model is easier and more visual. In the consolidation curve of Casagrande, it is possible to see more clearly the different phases of consolidation. Figure 39 is a superposition of the development of pore pressures and the consolidation curve total pressure of 24.98 kPa.

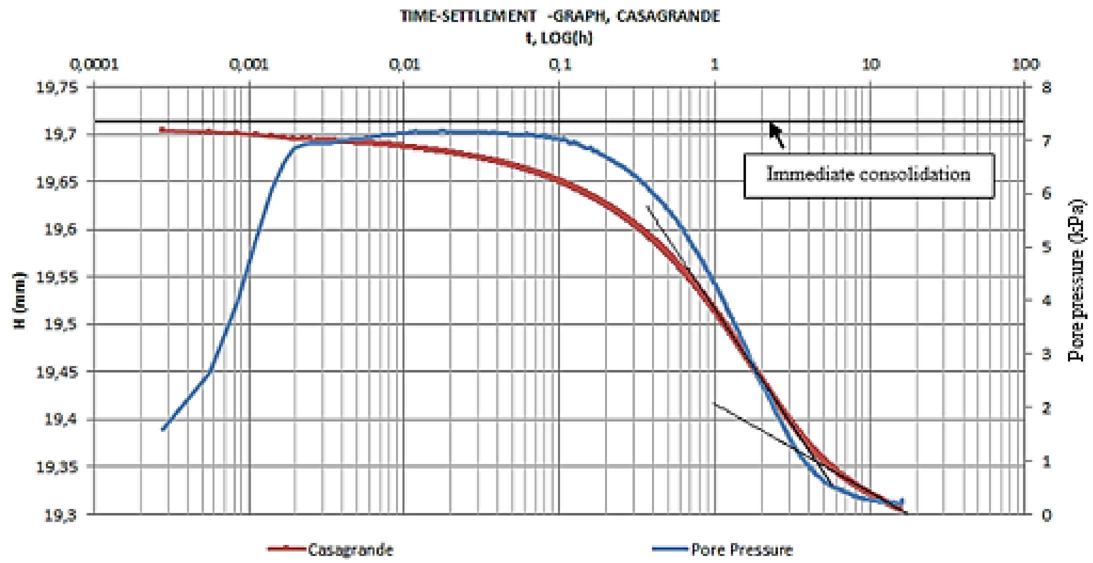

Figure 39. 24.98 kPa – Casagrande graph

The soil sample does not experience immediate consolidation, since the boundaries fall outside the data range. Therefore, the sample only experiences primary and secondary consolidation. This may be because the previously applied load steps have completed the elastic behavior of the sample. Likewise, these loading steps have been used to close the possible fissures in the sample, or to eliminate any air bubbles that might remain.

If it is considered that immediate consolidation does not exist at this stage of the test, no sample settlements should be produced until water pressure dissipation begins. However, in Figure 39 it is seen how the sample begins to settle during the process of ascent of the pore pressure. Theoretically, the sample should remain unchanged for as long as the water pressure increases, and only when it begins to decrease should the consolidation settlement be produced. This may be because, as mentioned in previous sections, there has been some problem during the placement of the sample in the cell that has affected in some way the degree of saturation. In the same way, it is possible that due to the effect of the evaporation of the water from the cell, this parameter has also been affected. If the sample is not completely saturated, part of the pressure corresponding to the load step is transmitted directly to the solid phase of the specimen. It may also be due to the fact that the Casagrande model does not fully adjust to what is happening in the tests, since it is a variant of the one-dimensional consolidation test.

In addition, secondary consolidation (creep) is supposed to start immediately after loading, even before pore water pressure reaches its maximum value. That is obvious near the upper part of the sample where drainage is arranged through the porous stone (Laaksonen, et al, 2015).

Figure 39 also shows the point at which the primary consolidation ends, and gives rise to the beginning of the secondary consolidation or creep. Theoretically, at this time the water pressure should have dissipated, which means, its value should be zero. Therefore all settlements after this point would be produced with a constant effective pressure level. However, as shown in the graph, when this time is reached, the water pressure has not yet dissipated completely. It is true that throughout the different tests that have been carried out, in none of them the water pressure becomes completely dissipated. The final values of the pore pressure when it reaches a steady state are 0.5-1 kPa. It is assumed that these values correspond to the dissipation of the pore pressure. When the moment in which the primary consolidation ends, according to the Casagrande model, the water pressure has a value close to this range. However, it is possible to see how the pressure is still dissipating. Primary consolidation, therefore, is not yet complete.

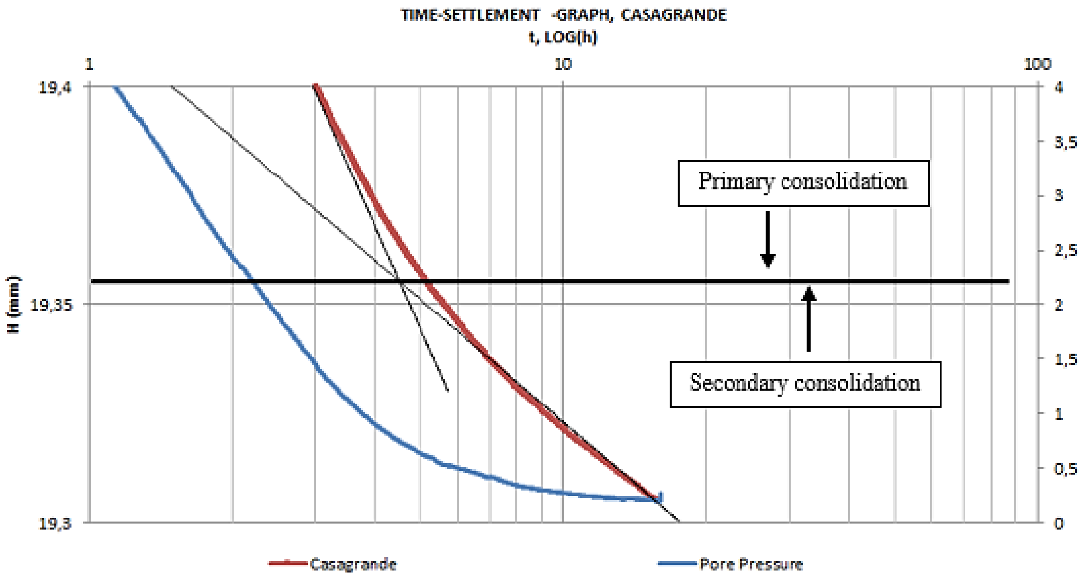

Figure 40. 24.98 kPa – Casagrande graph (Detail)

The Taylor method also allows to know the limits of consolidation, although not as clearly as the Casagrande method. As shown in Figure 39, as before, the immediate consolidation is not represented. Thus, it can be concluded, since both methods confirm it, that the immediate consolidation does not appear in this phase of the test.

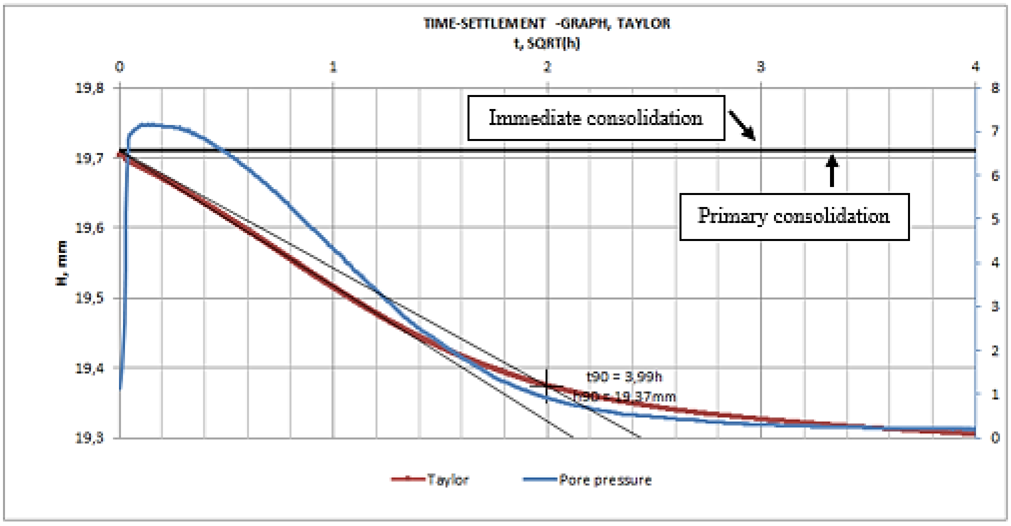

Figure 41. 12.49 kPa load step – Taylor graph (Detail)

As in the previous case, Figure 41 shows the evolution of the soil sample settlement along with the development of water pressure. The final stretch of graph is very similar to Figure 39. The Taylor model estimates the end of the primary consolidation for a t_{100} = 4,84 h. At this time, the pore pressures must be zero, or a value close to zero indicating that the pressure increase has dissipated. However, at this time t_{100}, it can be seen that the pressure if it has values close to zero, but has not yet stabilized completely. It still requires more time to reach a lower and stable value.

Thus, the Taylor and Casagrande models satisfactorily represent the development of primary consolidation, but they do not adjust precisely to the boundaries. It is true that factors named above may be involved as possible disturbances in the test equipment during the development of the tests. However, these inaccuracies occur at the different load steps of the tests. Therefore, this suggests that these models are not the most appropriate methods for establishing the limits of primary consolidation.

4.4.2. Preconsolidation pressure

Another objective pursued with the realization of these tests is related to the preconsolidation pressure. The objective is to know its value for the different samples tested, as well as its influence on the development of the tests.

The preconsolidation pressure has an important role in this kind of tests, in which the settlements of the soil samples are directly involved. The value of the preconsolidation pressure determines the behaviour of the samples and therefore, the results of the test. For values of the applied pressure lower than preconsolidation pressure, the behaviour of the soil will be governed according to the characteristics of reload or swelling branch (overconsolidated state) and thus stiffer response, and as such one could expect different pore pressure behavior compared to normally consolidated state. If it is higher, the behaviour will follow the guidelines of the load or compression branch.

As mentioned above, the development of the settlements is directly related to the evolution of pore pressures. Thus, this section seeks to determine how the pore pressures behave as a function of the preconsolidation pressure. Since the settlements are marked by the preconsolidation pressure, it is also expected that the behavior of the water pressure is also somewhat conditioned by this value.

As explained previously, the preconsolidation stress will be considered where the extended straight-line portion of the compression curve meets the extension of the straight-line portion of the reloading curve.

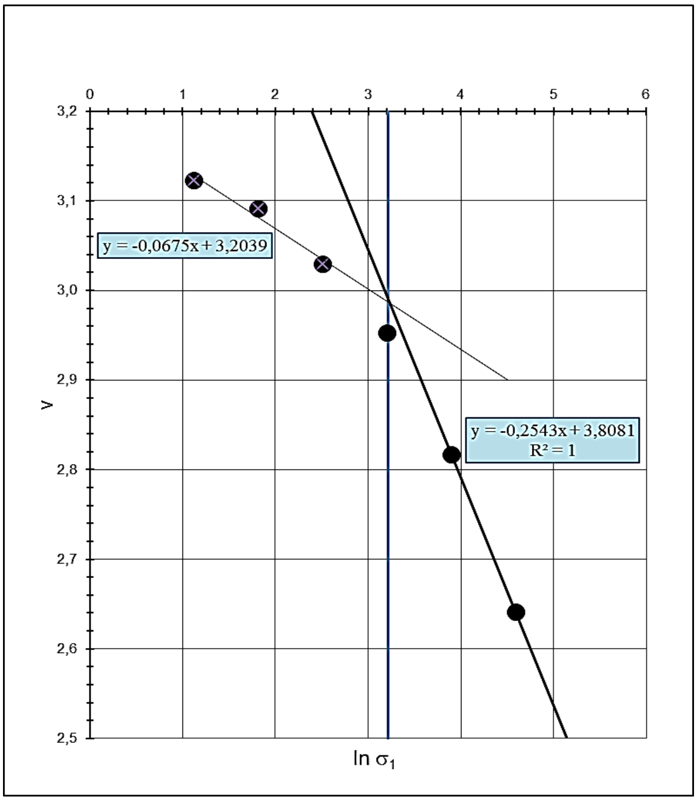

Figure 42. Preconsolidation stress simplified construction

The results show that the value of the preconsolidation pressure in the material tested has an average value of 25.6 kPa. In this way, the distribution of the load steps is very adequate to make a comparison before and after reaching the consolidation pressure. This distribution allows to have the sample loaded in different phases up to 24 kPa, which is very close to the preconsolidation pressure. At the same time, the following steps allow loading the sample to values until 98 kPa, which far exceed the preconsolidation pressure. Therefore, it is possible to test the sample in both situations and check their behavior. Table 13 summarizes the preconsolidation pressures of the specimens.

Table 13. Preconsolidation stress of the specimens

Test	6535u	6536u	6544u	6545u	6549u	6550u	6557u	6559u
Depht (m)	2.12 - 2.09	2.09 - 2.06	2.06 - 2.03	2.03 - 2.00	2.00 - 1.97	1.97 - 1.94	1.94 - 1.91	1.88 - 1.85
σ'_p (kPa)	23,5	22,5	23,85	31	25	24,5	25	30

In order to compare the soil behavior before and after the preconsolidation pressure is exceeded, the sample loading process is first analyzed. As discussed previously, the rise of pore pressure has a similar behavior in the different loading steps. The time required to reach the maximum pressure value is similar in all cases. However, it can be seen that the slope, before and after the preconsolidation pressure is exceeded, is slightly different (Figure 43). Once this value is reached, the pressure rise rate is higher, with the slope corresponding to the upper loading step. However, this does not allow a clear comparison of the behavior of the soil samples before and after the preconsolidation pressure. Since the load steps are increasing, the rate of rise of the water pressure is also higher, and the influence of the preconsolidation pressure value may not be the most relevant factor in this process.

To analyze the importance of this factor, it is more convenient to employ the second set of tests, described above. The application of equal load steps coincides with pressure values lower and bigger than the preconsolidation pressure. In this way, it is possible to observe more clearly how the material behaves when it exceeds the preconsolidation pressure, but being subjected to equal pressure increments.

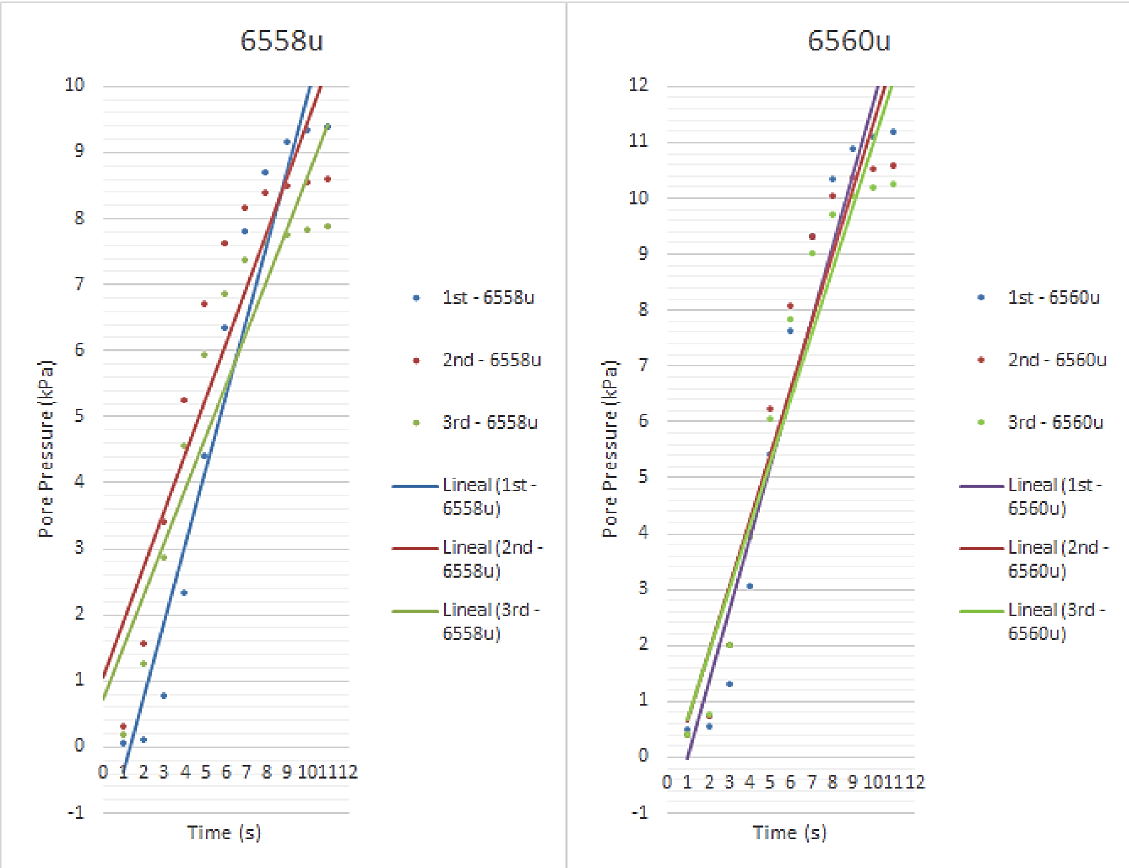

Figure 43. Load step early phase

Figure 43 shows the first instants of the load steps of 12.49 kPa (250 g) which is repeated in the second set of tests. The values of the pore pressure in the soil samples and the lines representing the linear ascending section of the pressure are represented. Unlike the first group of tests, this time the pressure behavior is almost the same. In previous tests, the rise in water pressure was clearly differentiated by steps. Although always very steep ascents, the greater the load, the greater the slope of these branches.
In these tests, all steps show a similar shape. After the first step, pressure will be 24.98 kPa, which achieves a value very similar to the preconsolidation pressure. Therefore, the later steps exceed this value. However, there is no appreciation of a different behavior that makes them clearly distinguishable. The rise in pressure follows the same behavior. Table 14 shows the values of the slopes of each line:

Table 14. Load step slopes during early phase

	6558u	6560u
1st	1,135	1,299
2nd	0,839	1,184
3rd	0,791	1,142

The slopes for all sections are very similar. The slight variations between these values do not seem to be due to the preconsolidation pressure as the main cause. There is no markedly different behavior between the steps, so it can be concluded that the preconsolidation pressure is not an important factor in the rise of pore pressures.

However, it can be observed that the values of the slopes decrease as the sample is loaded. On the contrary, this may be due to a change in soil permeability. When subjected to higher pressure, the structure of the soil is reorganized and there are fewer holes through which water can circulate. Figure 44 shows the section corresponding to the phase of dissipation of the pore pressures for the steps of 12.49 kpa (250 g). The evolution of the pore pressure in time has been represented until it reaches a value of 0.05 kPa. From this value, the pore water pressure is considered to be stabilized.

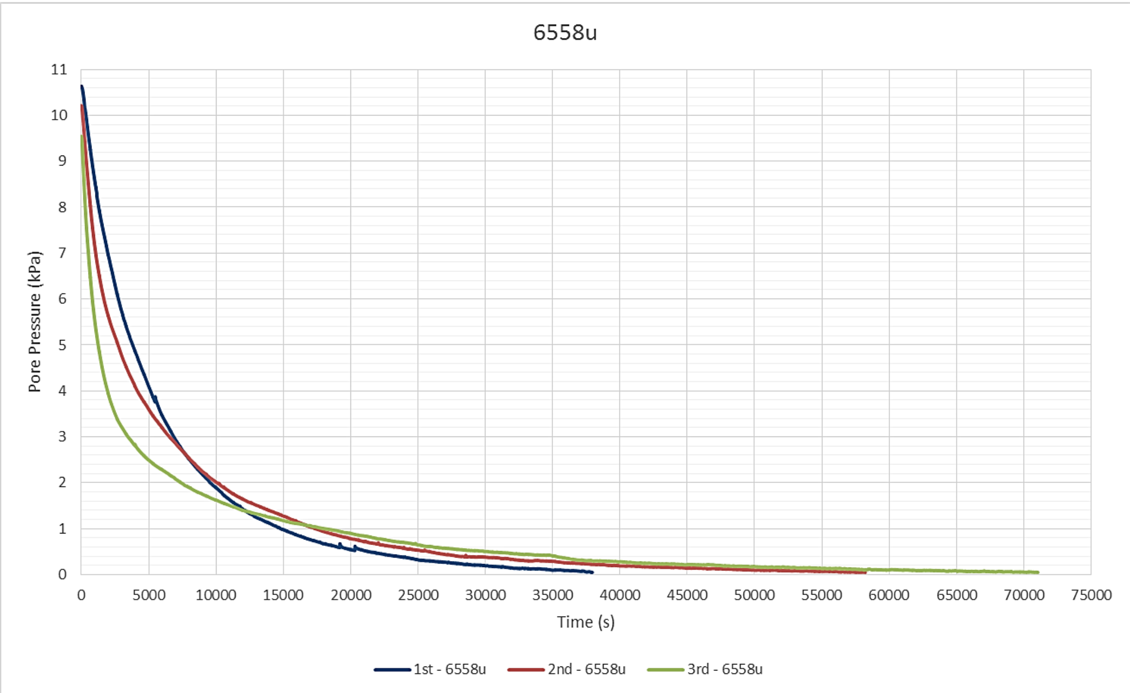

Figure 44. Test 6558: dissipation time

As can be seen, the soil sample is taking longer to dissipate the same pressure. In figure 43, although the slopes of the straight sections vary slightly, it is not a relevant factor in the development of water pressure. Although permeability varies, by producing a remarkable total pressure increase, this permeability variation is not a determining factor. However, when it comes to dissipating pressures, since they are much lower values, these small variations in soil permeability are a very important aspect. The time required to dissipate the pressure definitively is significantly greater as the permeability is lower.

5. Conclusions

The objective of the thesis was divided into two sections. First of these was to know how the pore pressure develops in clays, and what effect this behaviour has on the consolidation process. The second objective was to obtain a working method in these new tests, as well as to detect possible failures or improvements that can be implemented in future works.

5.1. Test results

The results obtained in the tests have answered many of the questions that were raised at the beginning of this work.

The first phenomenon studied has been the transmission of pressure to the sample. The results confirm that this transmission of the external load to the liquid phase of the sample does not occur instantaneously. In all tests, the rise of pore pressure has required a minimum time, always near to 10 seconds, even though it is assumed to occur instantaneously. This may be due to the presence of air in the sample, which can delay the development of the process. Similarly, the pressure is transmitted through the upper face of the sample, while the readings are recorded on the lower face, so this can also slow down the process. In future, it would be advisable to test samples of different thickness to see these differences.

The value reached by the pore pressures has also been analysed, always being lower than the value of the added load. The added pressure must be totally absorbed by the water in the soil, since it is saturated. The loads registered were around 90% of the total load applied, so that around 10% of the applied load is being absorbed by other elements. Some of this load may be absorbed by friction between the sample and the cell, so it is also advisable to carry out different tests with different coefficients of friction in the cell.

The influence of the load increment ratio (LIR) has also been studied. It has been shown not to affect the development of pore pressure, and therefore does not significantly affect consolidation either. The tests carried out with LIR = 1 have shown similar behavior in the evolution of pore pressure, as well as a final value of the settlement near 3 mm in all cases for a total load of 99 kPa.

Finally, the value of preconsolidation pressure is not a relevant factor in the processes of ascent and evolution of pore pressure. It is true that in the times required to reach the peak values, there is a somewhat different behaviour before and after reaching the preconsolidation pressure. However this behaviour does not seem to have great differences. Tests should be done in which the load increments are closer to preconsolidation stress value, in order to be able to analyse in depth what is happening in these ranges

In spite of all this, it would be interesting to test samples of different materials and thicknesses, to confirm more clearly and if these phenomena occur uniformly in the differentclays.

5.2. Test recommendations

The test equipment used to carry out the experiments is a relatively new material. There is not much earlier information regarding its use, and similar equipment in other laboratories is not known. Therefore, there is no standard procedure or working methodology. It is true that it is a test based on the Standard Oedometer Test, and therefore, its methodology of use can be used as base. However, this new equipment presents some singularities that makes it necessary to point some warnings out and follow some specific steps or recommendations, both for the assembly of the cell as for specimen testing.

5.2.1. Assembling / Testing

1) The preparation of the specimen is performed in the same way as in the oedometer test. As described in the test procedure, the water used for this test must be air free. Therefore, it is very important that when the samples are prepared, this water is already prepared so that the less time elapses from the preparation of the sample until it is placed in the cell. This is very important, because this prevents the material from suffering disturbances, and thus the results are more uniform.

Recommendation: Before preparing the sample, the first step should be to prepare the water to be used. In this way, the water will always be available and there will not be a delay between the preparation of the sample and its placement. In addition, when the water is prepared in the vacuum machine, the already flooded cell is also introduced. So everything will be ready for the test set up.

2) To insert the sample into the cell, it is necessary to place the ring, and then push with the help of the piston to insert the specimen into its cavity. At this stage two problems may arise. First, pushing the sample with the piston can generate excess pore pressures in the sample that may alter the results. In the same way, this pressure can generate settlements in the sample. The second problem may occur when the introduction of the sample into the cavity is completed. The piston is neither attached nor fixed, so it can move, allowing air to enter to the top of the sample. If this occurs, it is useless to remove the air to the water, since air can to enter.

Recommendation: A possible solution to this problem would be to know when to stop pushing the sample so as not to produce settlements or excess pressure. The depth of the cavity should be measured so it is possible to know when the specimen is already placed.

3) It is very important that the placement of the weights is as appropriate as possible. These tests try to emulate an increase in pressure as close as possible. That is why it is important that the placement is carefully done, to avoid any excess pressure besides the one wanted. At the same time, the placement should be done as quickly. The objective is to transfer the pressure as quickly as possible to study the phenomenon of the load step.

4) During the test, the cell must remain full of water at all times to ensure saturation conditions. However, water from the surface of the cell evaporates and it is necessary to be careful to fill it to avoid the lowering of the level.

Recommendation: The ideal way to proceed would be to find a way to cover the cell so it would not lose water and would not need to be refilled. If this is not possible, a timetable should be set to check and control the water level. It is important to register the instants in which water is introduced into the cell, in case any anomaly is detected in the tests.

5.2.2. Data obtaining

As explained before, the placement of the specimen inside the cell can generate excess pressures and settlements. It is important to know if these phenomena happen, and if so, it is also important to know if they are relevant and how they may affect the test. Therefore, it would be interesting to start the program that records the data before introducing the specimen into the cell. In this way, everything that happens during the placement of the specimen can be monitored.

The existence of the excess pressures generated by the placement of the sample seems difficult to avoid. Therefore, in order to try to make the test as unaltered as possible, it would be advisable to let the sample rest. If the measurement software has already been started, it is possible to check the existing pore pressure. Thus, a certain amount of time is allowed to elapse until the pressure has dissipated, and thus to begin the test.

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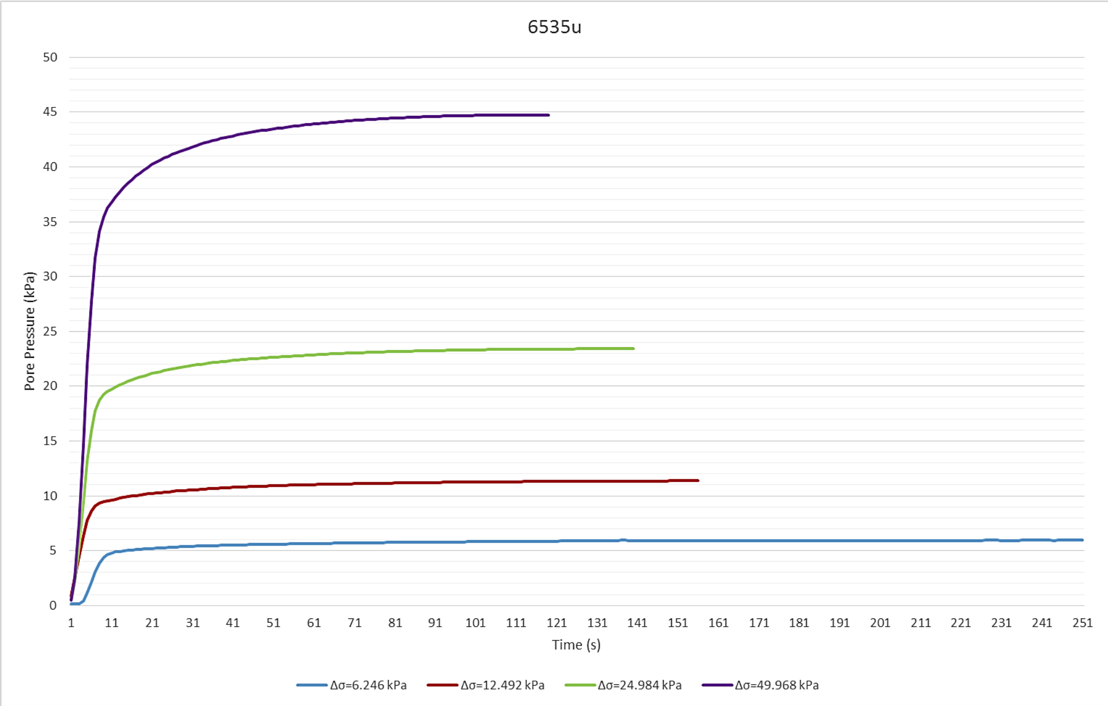
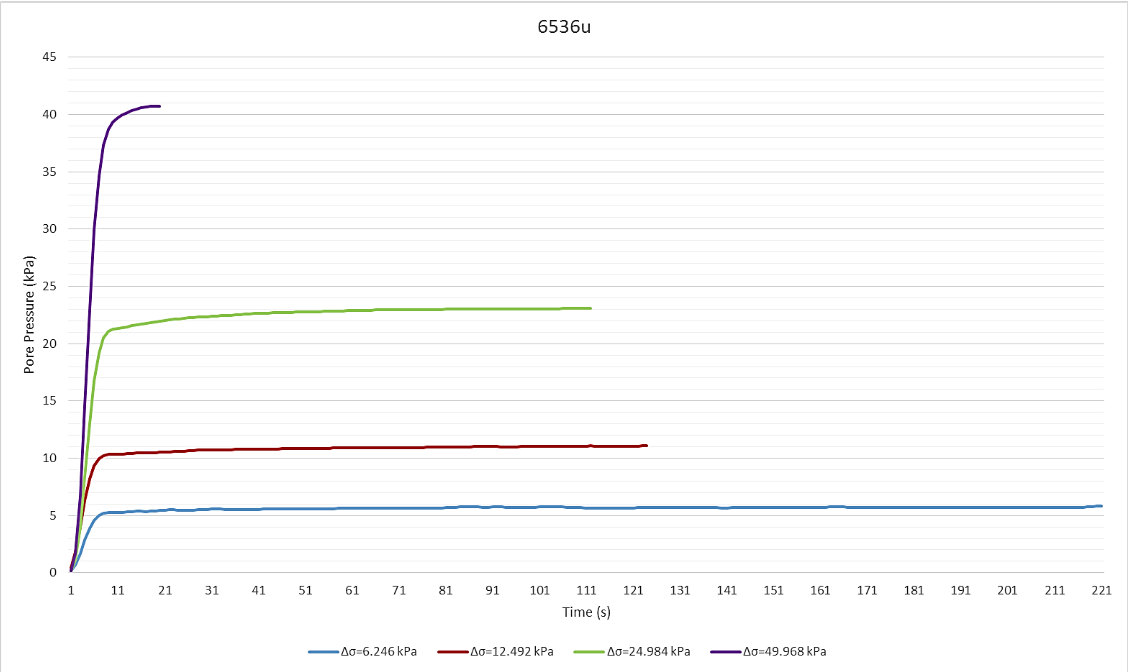
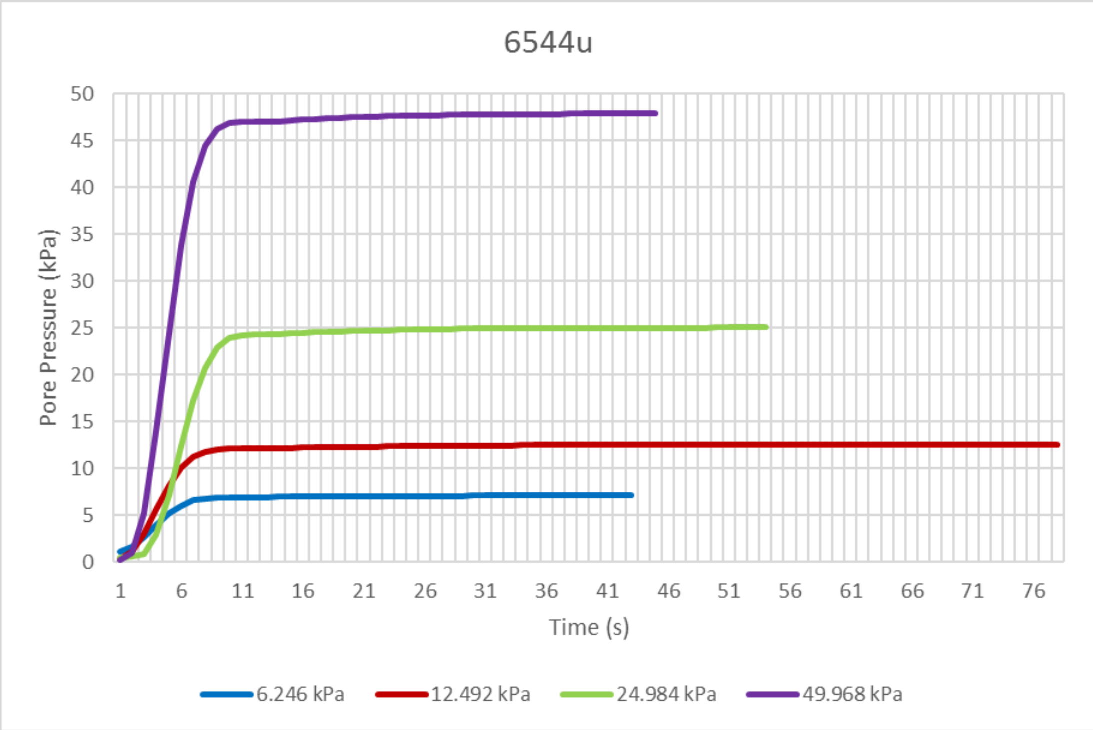
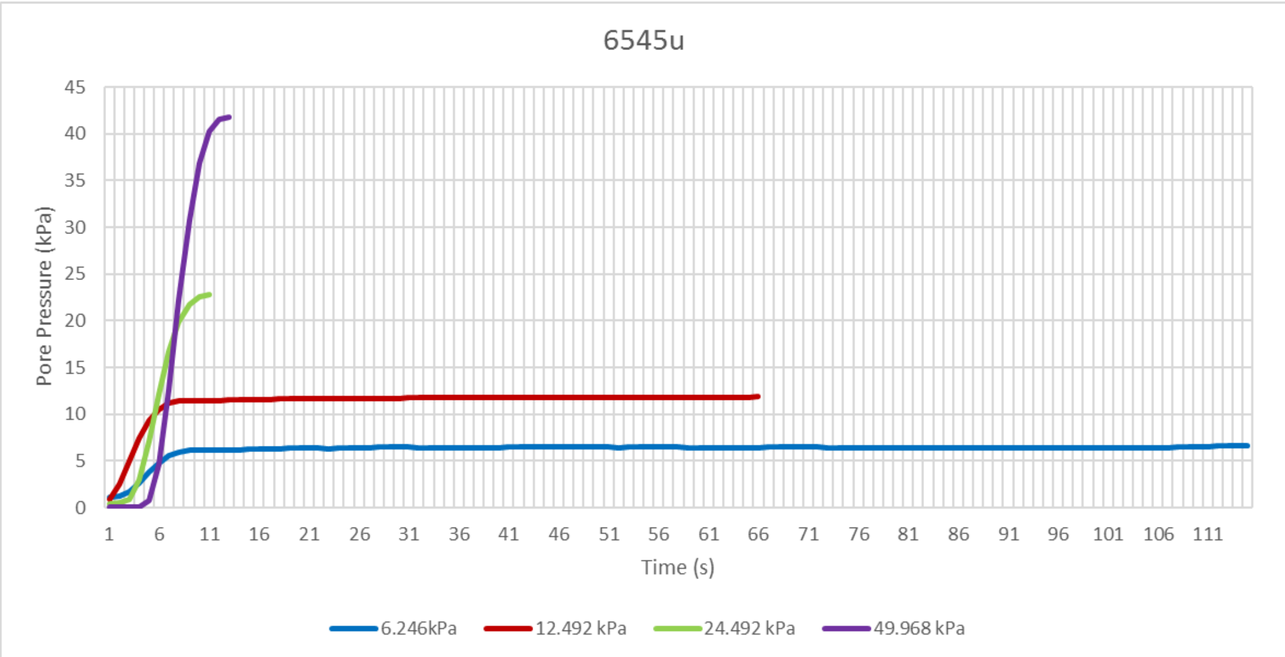

6535u
Pore Pressure (kPa)
Time (s)
Δσ=6.246 kPa
Δσ=12.492 kPa
Δσ=24.984 kPa
Δσ=49.968 kPa

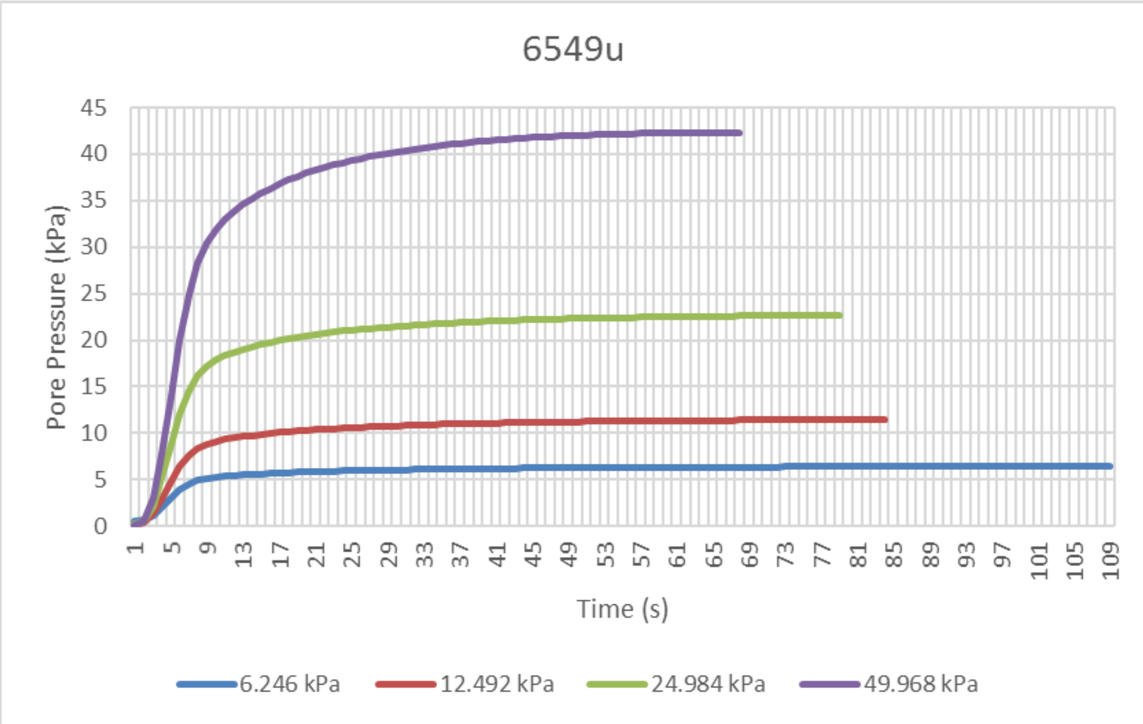

6536u
Pore Pressure (kPa)
Time (s)
Δσ=6.246 kPa
Δσ=12.492 kPa
Δσ=24.984 kPa
Δσ=49.968 kPa

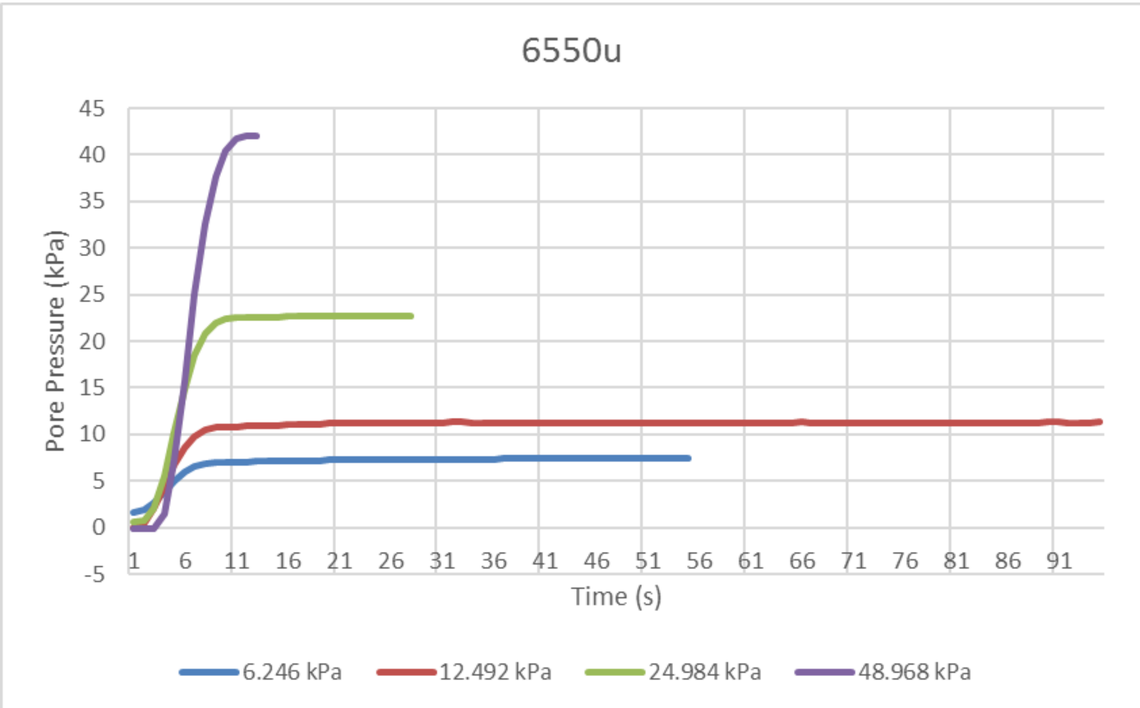


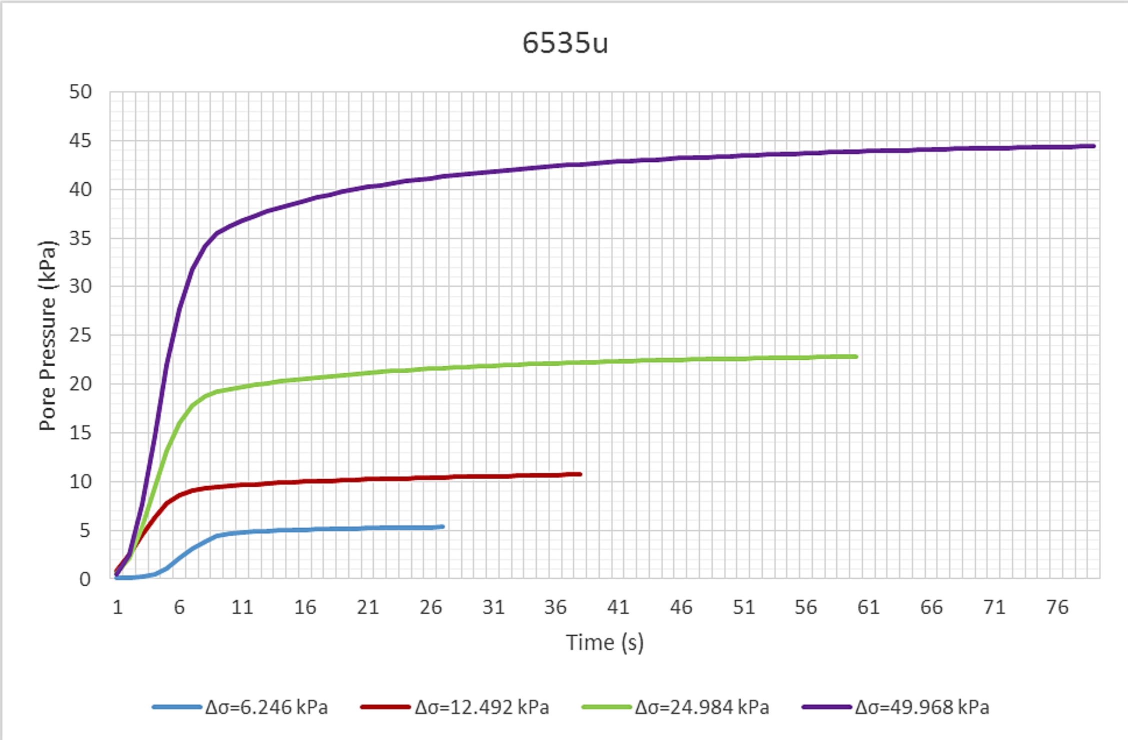
6544u
Pore Pressure (kPa)
Time (s)
6.246 kPa
12.492 kPa
24.984 kPa
49.968 kPa

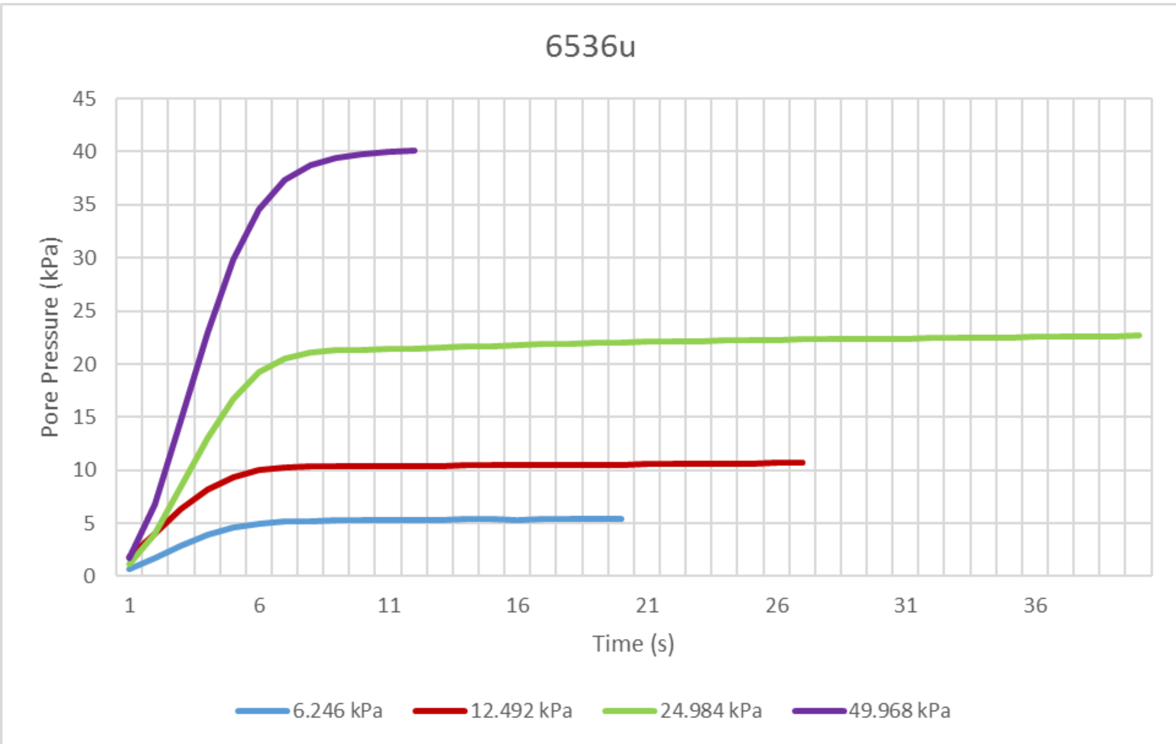


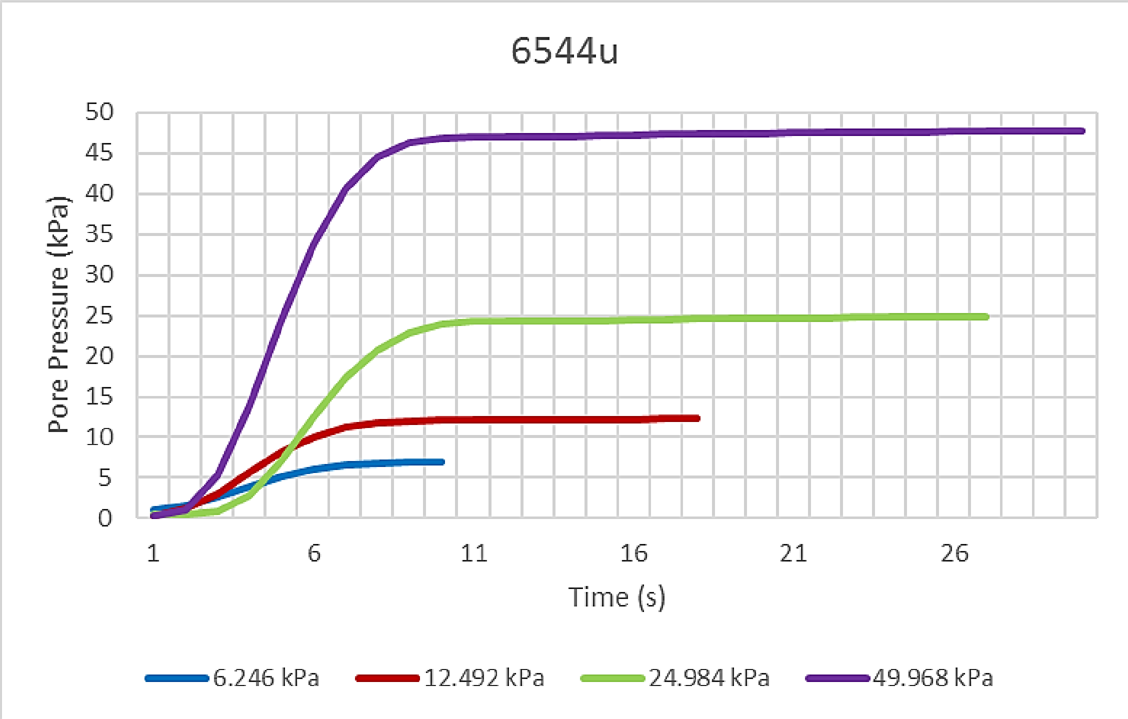
6545u
Pore Pressure (kPa)
Time (s)
6.246kPa
12.492 kPa
24.492 kPa
49.968 kPa

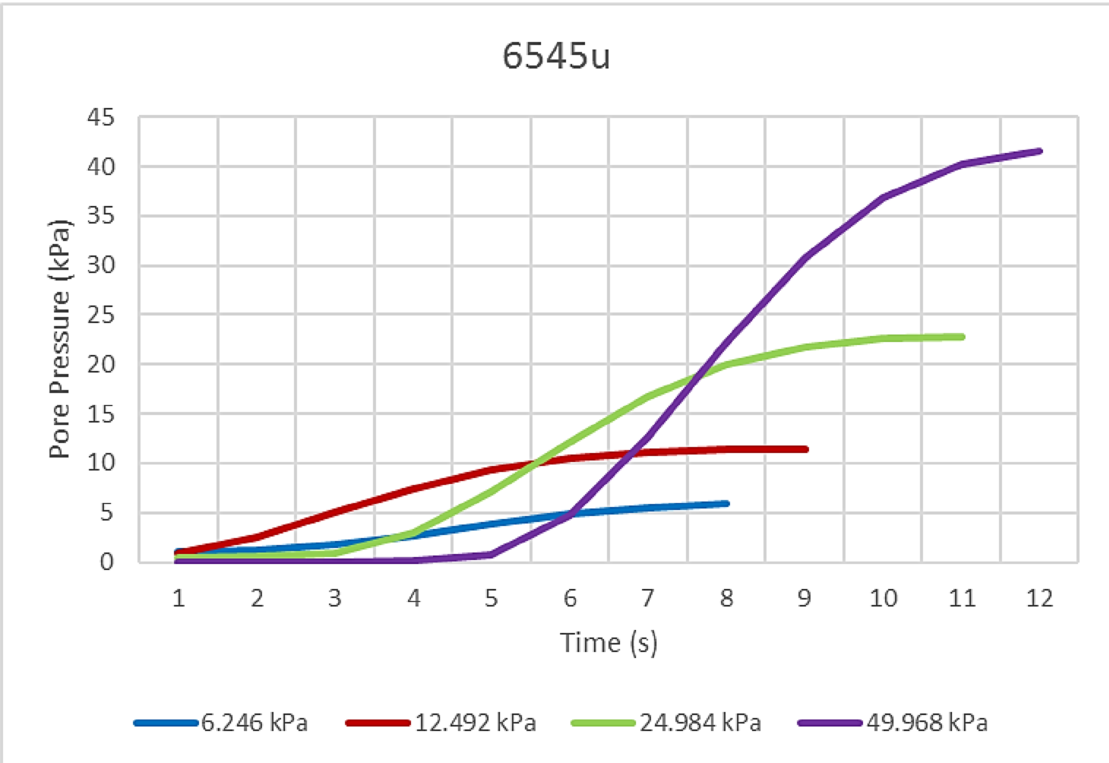

6549u
Pore Pressure (kPa)
Time (s)
6.246 kPa
12.492 kPa
24.984 kPa
49.968 kPa

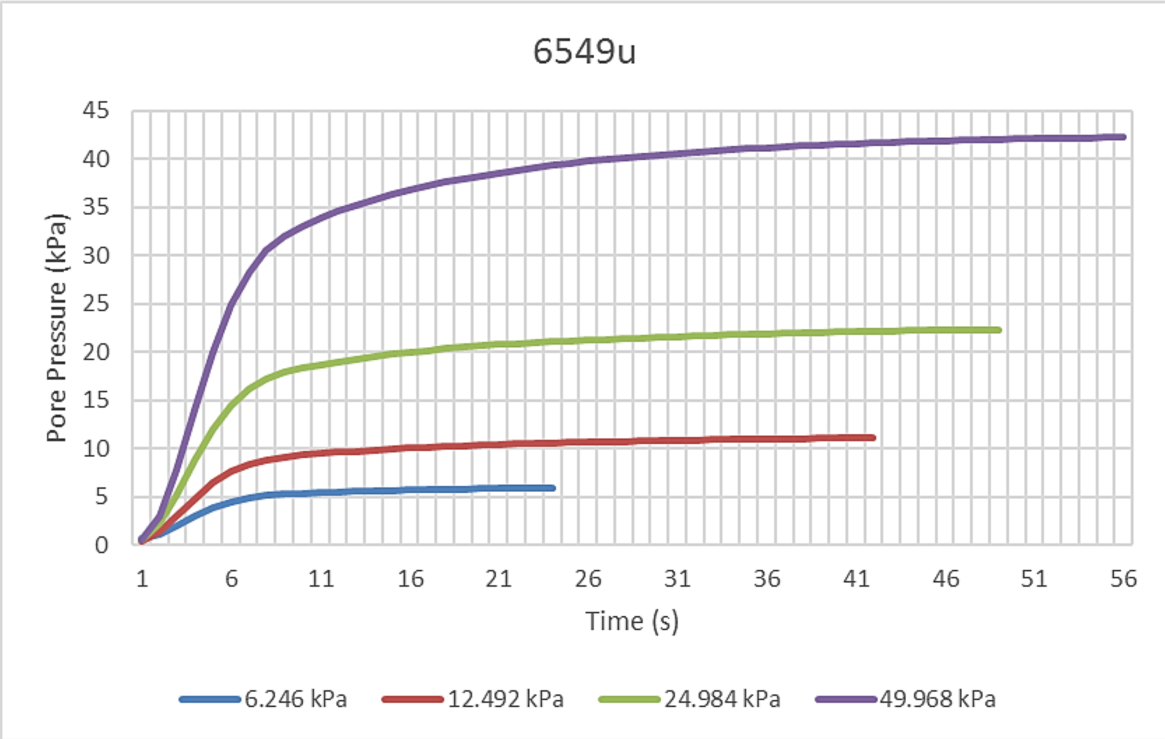

6550u
Pore Pressure (kPa)
Time (s)
6.246 kPa
12.492 kPa
24.984 kPa
48.968 kPa

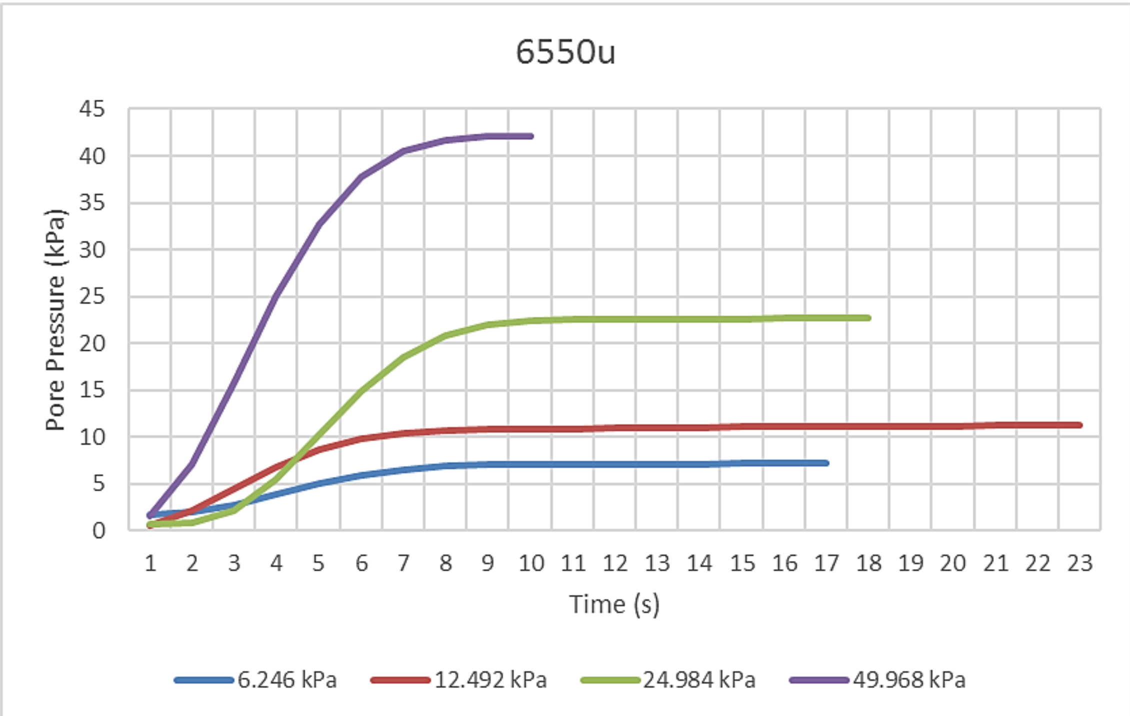

6535u
Pore Pressure (kPa)
Time (s)
Δσ=6.246 kPa
Δσ=12.492 kPa
Δσ=24.984 kPa
Δσ=49.968 kPa

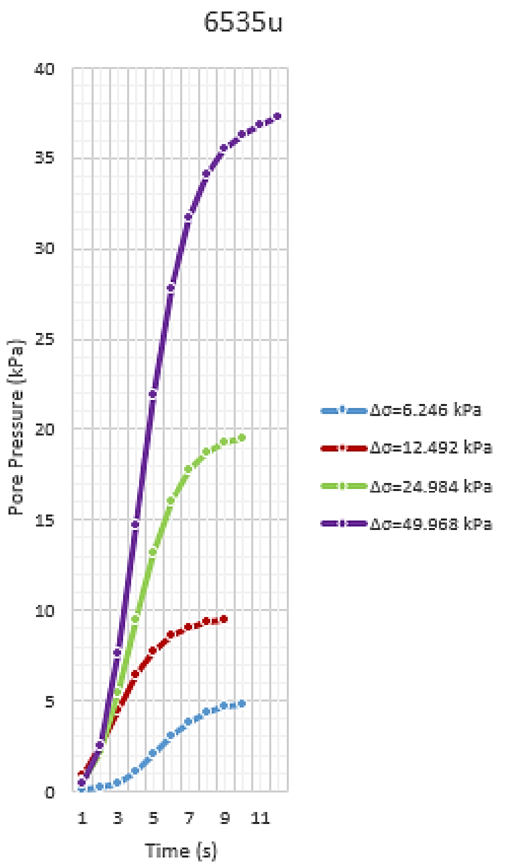

6536u
Pore Pressure (kPa)
Time (s)
6.246 kPa
12.492 kPa
24.984 kPa
49.968 kPa

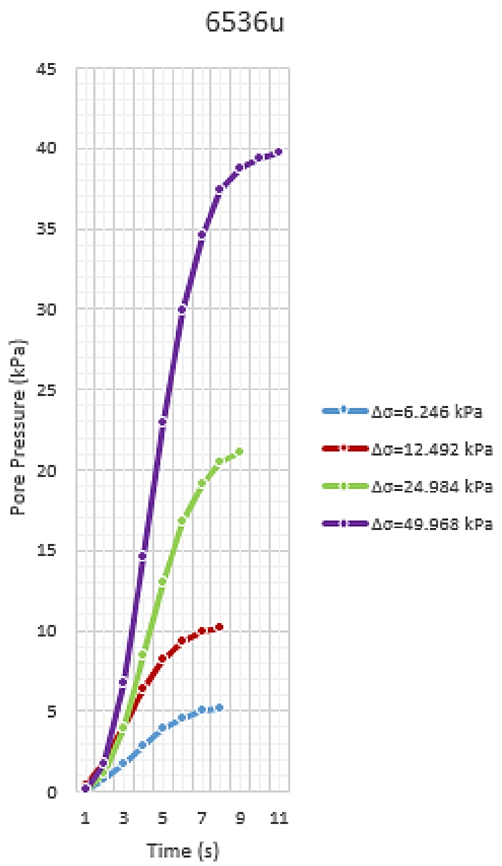

6544u
50
45
40
35
30
25
20
15
10
5
0
Pore Pressure (kPa)
1
6
11
16
21
26
Time (s)
6.246 kPa
12.492 kPa
24.984 kPa
49.968 kPa

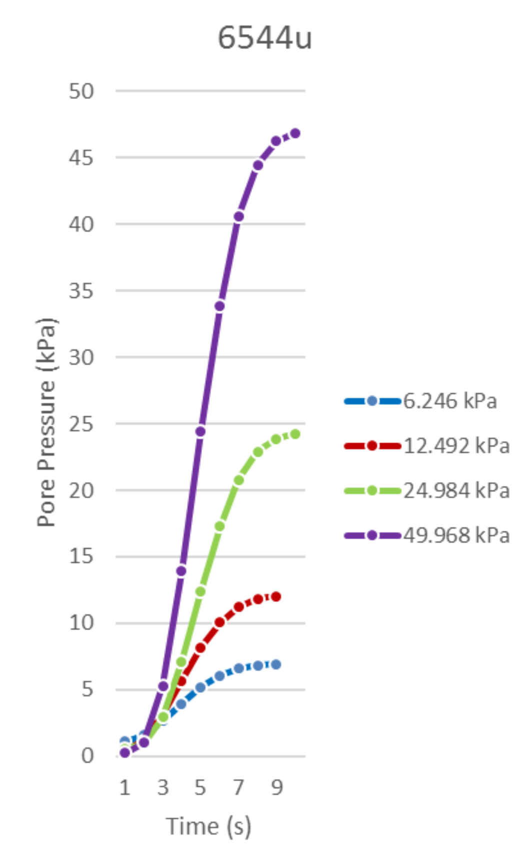

6545u
45
40
35
30
25
20
15
10
5
0
Pore Pressure (kPa)
1
2
3
4
5
6
7
8
9
10
11
12
Time (s)
6.246 kPa
12.492 kPa
24.984 kPa
49.968 kPa

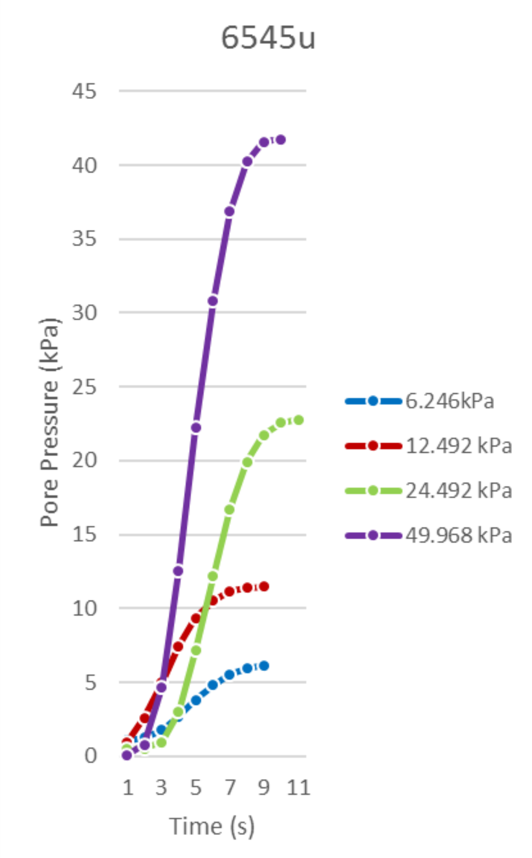

6549u
Pore Pressure (kPa)
45
40
35
30
25
20
15
10
5
0
1
6
11
16
21
26
31
36
41
46
51
56
Time (s)
6.246 kPa
12.492 kPa
24.984 kPa
49.968 kPa

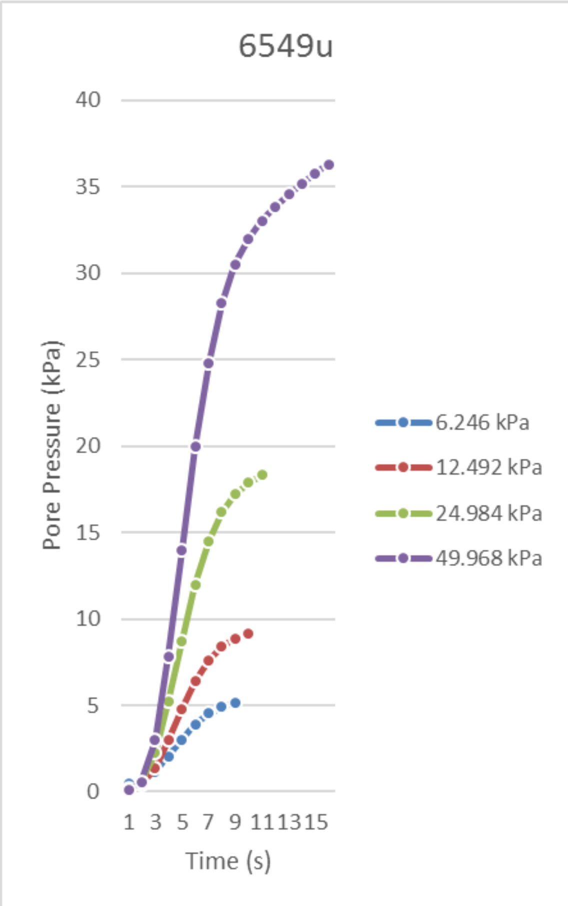

6550u
Pore Pressure (kPa)
45
40
35
30
25
20
15
10
5
0
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23
Time (s)
6.246 kPa
12.492 kPa
24.984 kPa
49.968 kPa

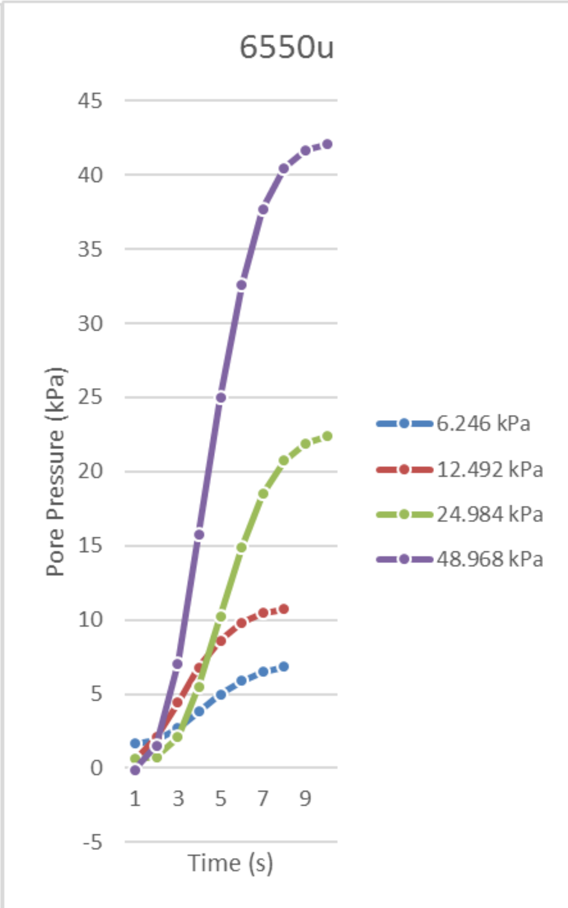

6535u
Pore Pressure (kPa)
Time (s)
Δσ=6.246 kPa
Δσ=12.492 kPa
Δσ=24.984 kPa
Δσ=49.968 kPa


6536u
Pore Pressure (kPa)
Time (s)
Δσ=6.246 kPa
Δσ=12.492 kPa
Δσ=24.984 kPa
Δσ=49.968 kPa


6544u
Pore Pressure (kPa)
Time (s)
6.246 kPa
12.492 kPa
24.984 kPa
49.968 kPa


6545u
Pore Pressure (kPa)
Time (s)
6.246kPa
12.492 kPa
24.492 kPa
49.968 kPa


6549u
Pore Pressure (kPa)
40
35
30
25
20
15
10
5
0
1 3 5 7 9 111315
Time (s)
6.246 kPa
12.492 kPa
24.984 kPa
49.968 kPa


6550u
Pore Pressure (kPa)
45
40
35
30
25
20
15
10
5
0
-5
1 3 5 7 9
Time (s)
6.246 kPa
12.492 kPa
24.984 kPa
48.968 kPa

Site:	**OSSINLAMPI**					**Test No**	**6544u**
Point:	V4			Depth:	2.06-2.03		
Oedometer:							
Apparatus No	10	h_0, cm	2,0	A, cm^2	20	V, cm^3	40,00	σ_1, **kPa**	**24,61**
Remarks:							
Sample:							
Soil	Clay	Cl-%		Humus, %		ρ_s, t/m^3	2,78	γ_0, kN/m^3	15,3
w_0, %	75,1	e_0	2,127	n_0, %	68,0	s_u, kPa		S_t	
S_r, %	98	F		w_L, %		w_P, %		I_P	

Coefficients of consolidation:

Taylor's method (T): $c_v = 0{,}848 * H^2 / t_{90}$, $H = H_{50}/2$

FIGURE 1:

H_0 (mm)	H_{90} (mm)	H_{100} (mm)	H_{50} (mm)	t_{90} (h)
19,31	18,83	18,78	19,04	3,95

Coefficient of consolidation c_v:

2,16 * 10^{-4} cm^2/s

0,68 m^2/a

Load increment [kPa]: 12,3 ======> 24,61

Secant modulus M_s = **466** **kPa**

Water permeability k= **0,4637** *** 10^{-9} m/s**

Casagrande's method (C): $c_v = 0{,}196 * H^2 / t_{50}$, $H = H_{50}/2$

FIGURE 2:

H_0 (mm)	H_{100} (mm)	H_{50} (mm)	t_{50} (h)
19,30	18,85	19,07	0,71

Coefficient of consolidation c_v:

2,77 * 10^{-4} cm^2/s

0,87 m^2/a

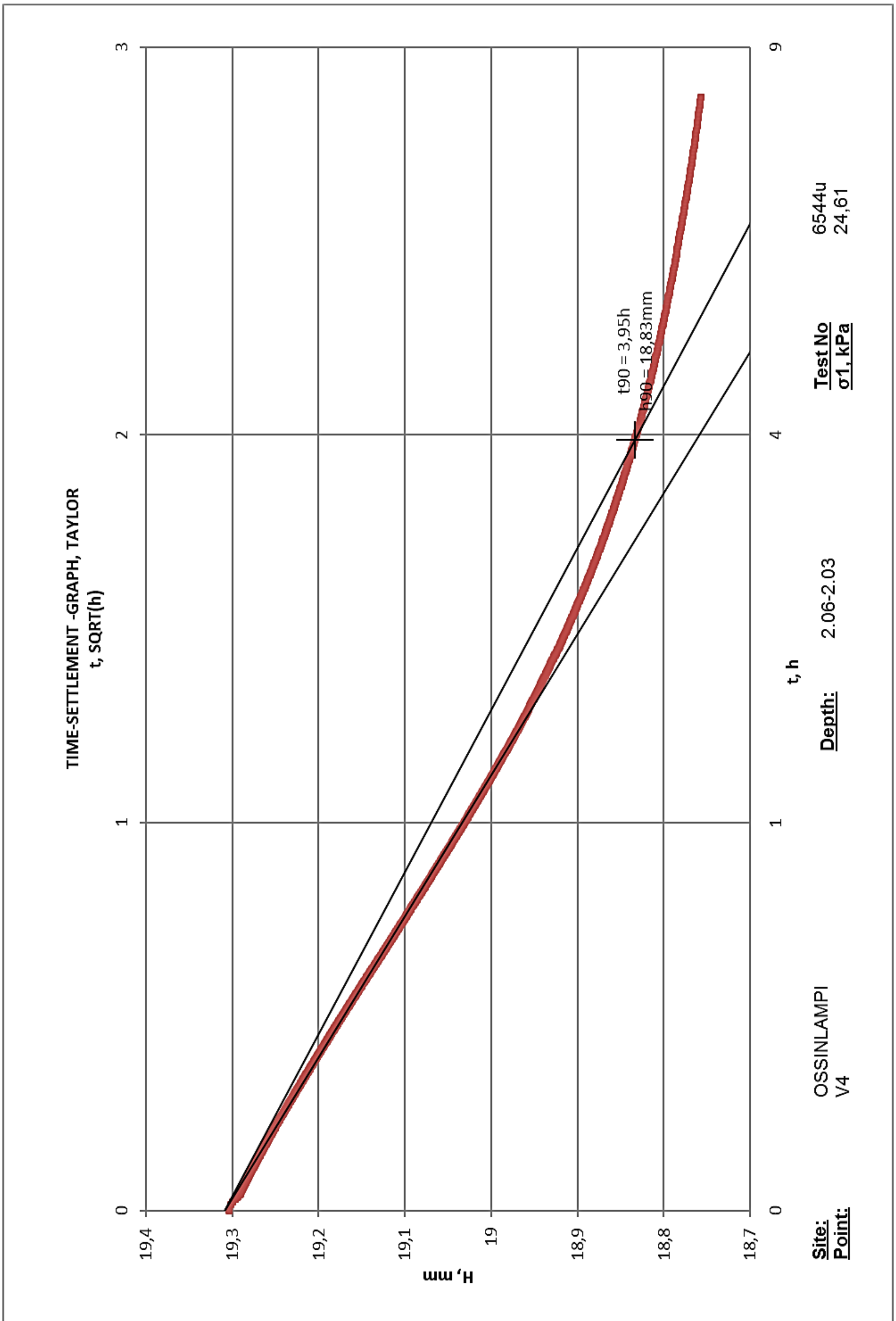
Load increment [kPa]: 12,3 ======> 24,61

Secant modulus M_s = **543** **kPa**

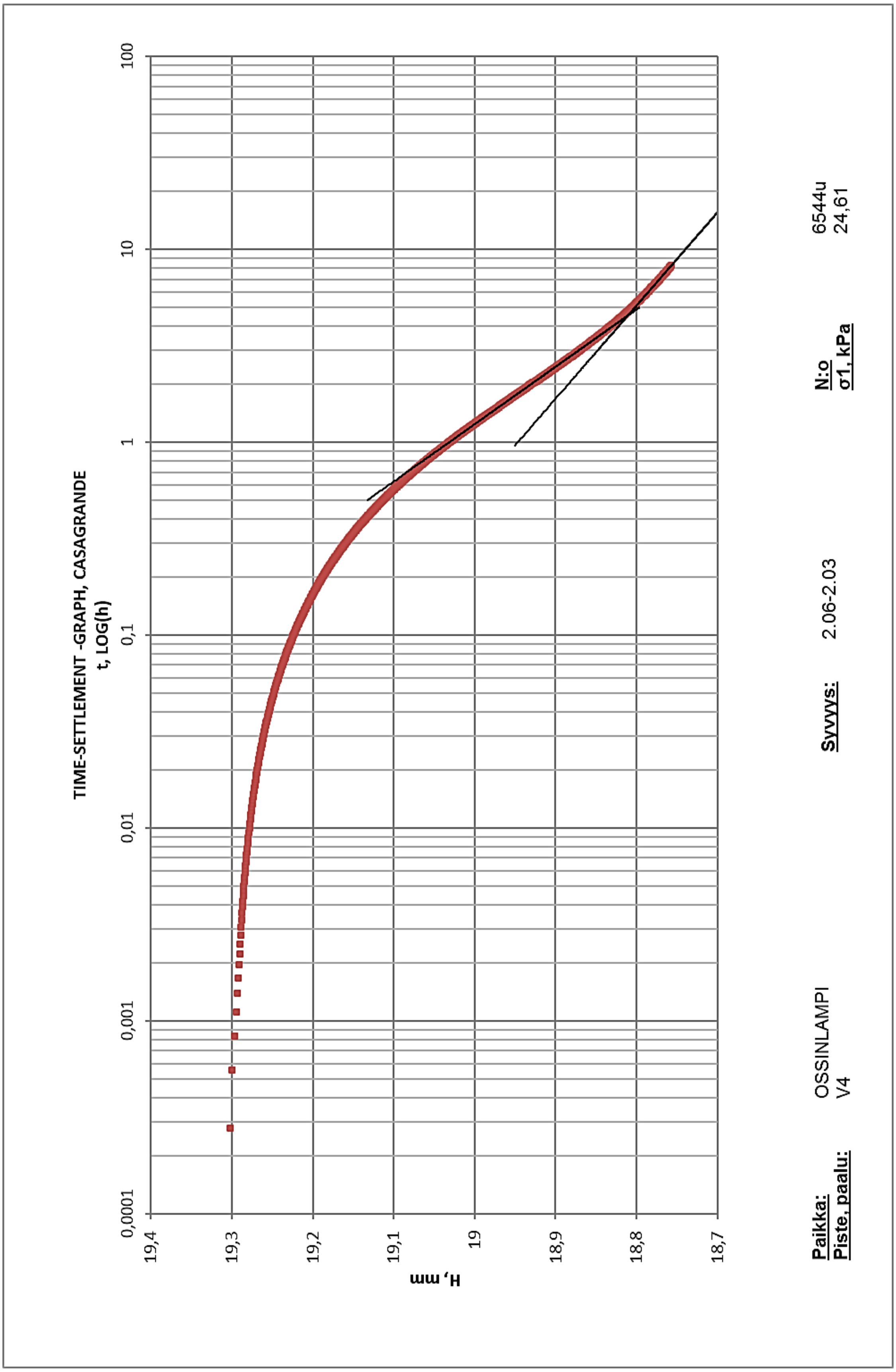
Water permeability k= **0,5113** *** 10^{-9} m/s**

Coefficient of creep

$\varepsilon_\alpha = c_{\varepsilon\alpha} =$ **1,27** %

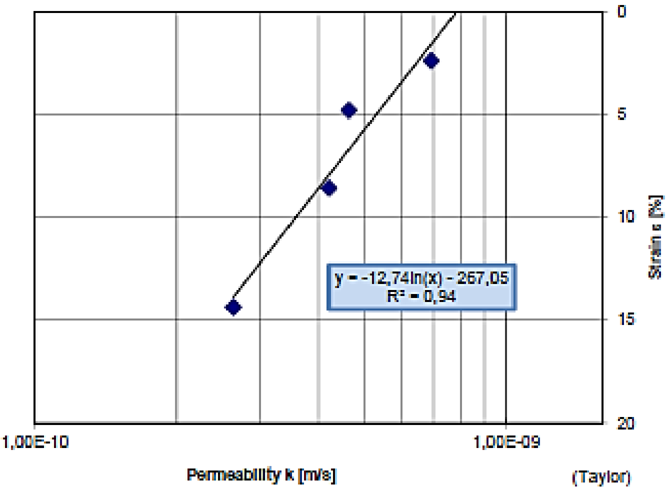


TIME-SETTLEMENT -GRAPH, TAYLOR
t, SQRT(h)
H, mm
19,4
19,3
19,2
19,1
19
18,9
18,8
18,7
0
1
2
3
0
1
4
9
t, h
t90 = 3,95h
h90 = 18,83mm
Site: OSSINLAMPI
Point: V4
Depth: 2.06-2.03
Test No 6544u
σ1, kPa 24,61

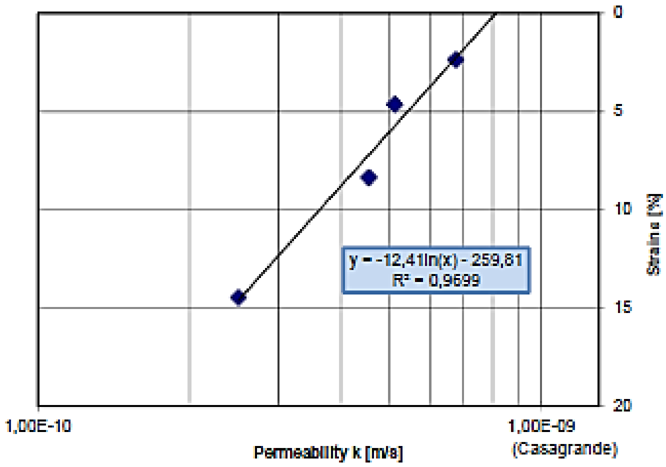

TIME-SETTLEMENT -GRAPH, CASAGRANDE
t, LOG(h)
0,0001
0,001
0,01
0,1
1
10
100
H, mm
19,4
19,3
19,2
19,1
19
18,9
18,8
18,7
Paikka:
Piste, paalu:
OSSINLAMPI
V4
Syvyys:
2.06-2.03
N:o
σ1, kPa
6544u
24,61

TEKNILLINEN KORKEAKOULU ÖDOMETRIKOE 16.6.2017
Rakennus- ja ympäristötekniikan osasto
Pohjarakennuksen ja maamekaniikan laboratorio

						Date		30.3.2017	
Site:	**OSSINLAMPI**							Test No	**6544u**
Point:	V4				Depth	2,12-2,09			
Oedometer:									
Apparatus No	10	h_0, cm	2,0	A, cm^2	20,0	V_0, cm^3	40,0		
Remarks	-								
Sample:									
Soil	Clay	Cl-%		Humus,%		ρ_s, t/m^3	2,78	γ_0,kN/m^3	
w_0, %	75,08436	e_0	2,127	n_0, %		s_k, kPa		S_t	
S_r, %		F		w_L, %		w_P, %		I_P	

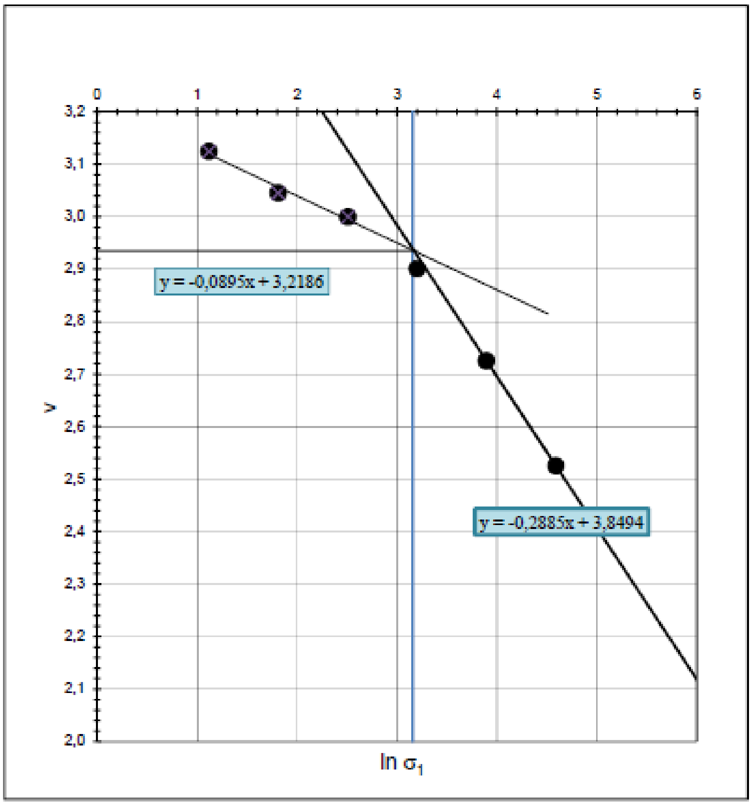


TAYLOR (trendline)			
k_1	0,7880	$* 10^{-9}$	m/s
β_k	3,41		
(user-defined)			
k_1		$* 10^{-9}$	m/s
β_k			



CASAGRANDE (trendline)			
k_1	0,8087	$* 10^{-9}$	m/s
β_k	3,50		
(user-defined)			
k_1		$* 10^{-9}$	m/s
β_k			

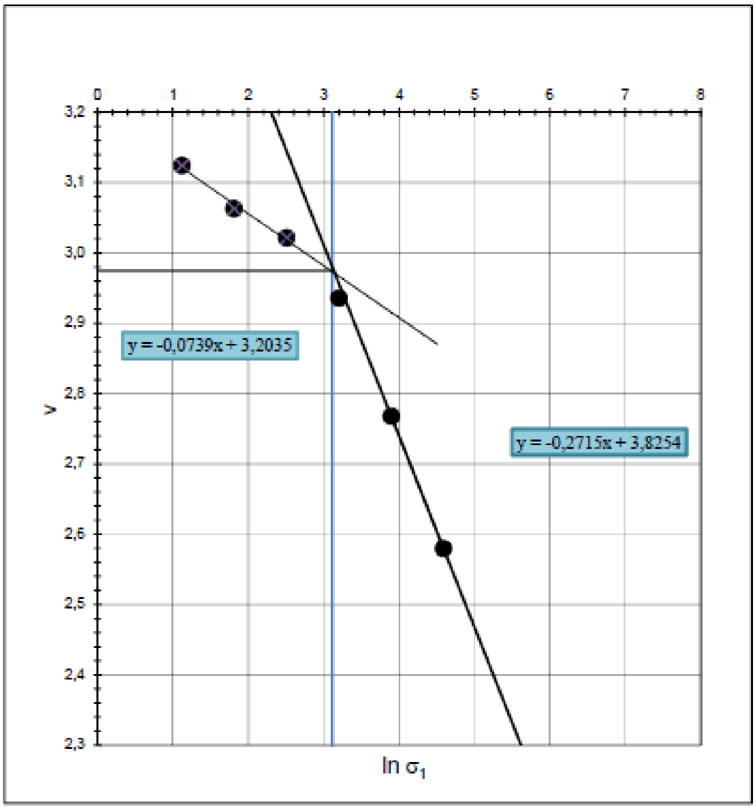
Site:	OSSINLAMPI			Test No	6535U
Point:	V4	Depth:	2,12-2,09 m		



σ_1 kPa	v= e+1	ln σ_1
3,1	3,124	1,118
6,1	3,045	1,812
12,3	3,000	2,510
24,6	2,901	3,203
49,2	2,725	3,896
98,4	2,525	4,589

λ	κ	
0,289	0,0895	
C_c	C_r	C_c/C_r
0,664	0,206	3,223

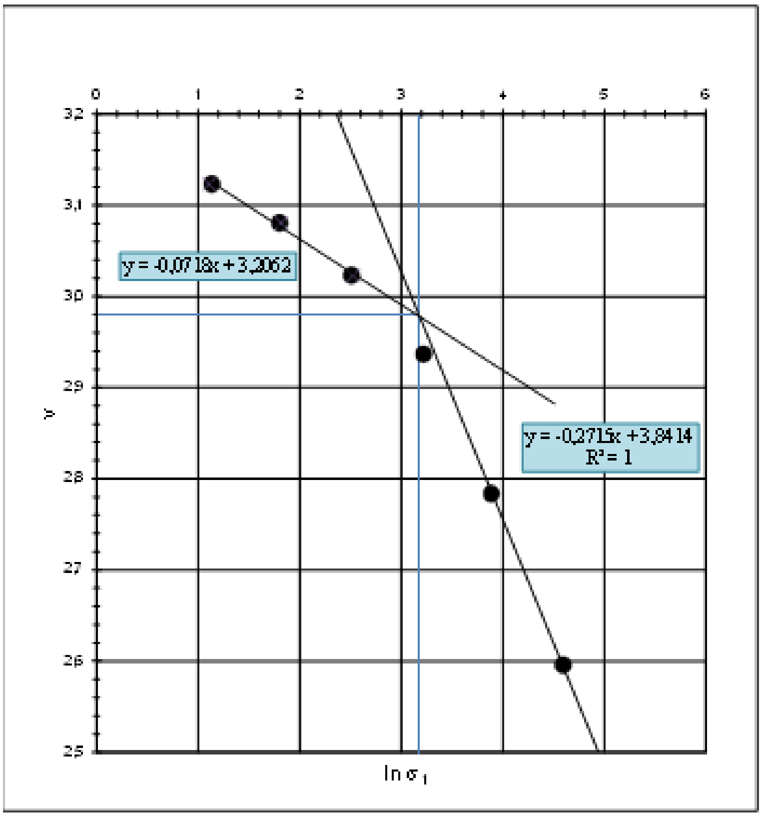
Site:	OSSINLAMPI			Test No	6536u
Point:	V4	Depth:	2,09-2,06 m		



σ_1 kPa	v= e+1	ln σ_1
3,1	3,124	1,118
6,1	3,063	1,812
12,3	3,021	2,510
24,6	2,936	3,203
49,2	2,767	3,896
98,4	2,579	4,589

λ	κ	
0,271	0,0739	
C_c	C_r	C_c/C_r
0,623	0,170	3,667

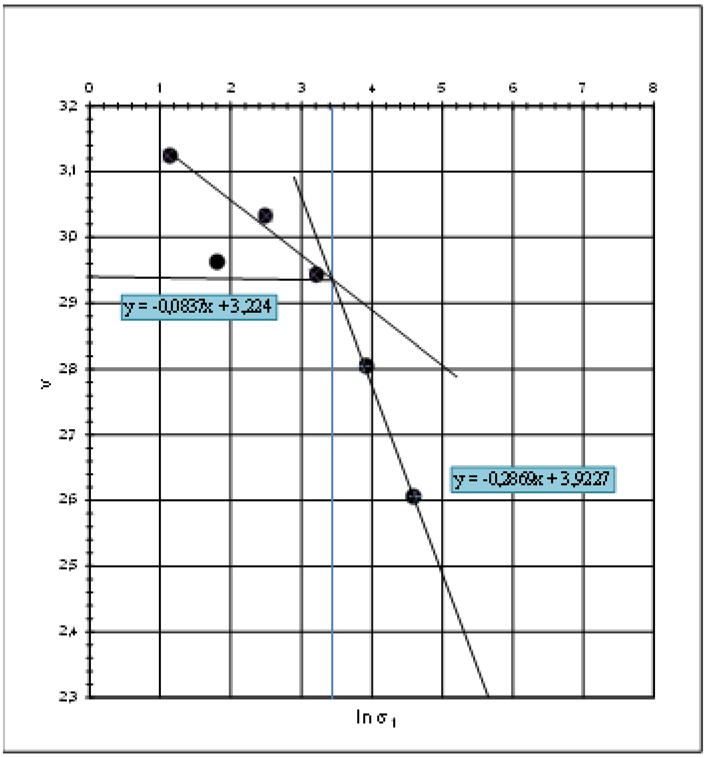
Site:	OSSINLAMPI			Test No	6544u
Point:	V4	Depth:	2,06-2,03 m		



σ_1 kPa	v= e+1	ln σ_1
3,1	3,123	1,118
6,1	3,081	1,812
12,3	3,024	2,510
24,6	2,936	3,203
49,2	2,784	3,896
98,4	2,595	4,589

λ	κ	
0,272	0,0718	
C_c	C_r	C_c/C_r
0,624	0,165	3,781

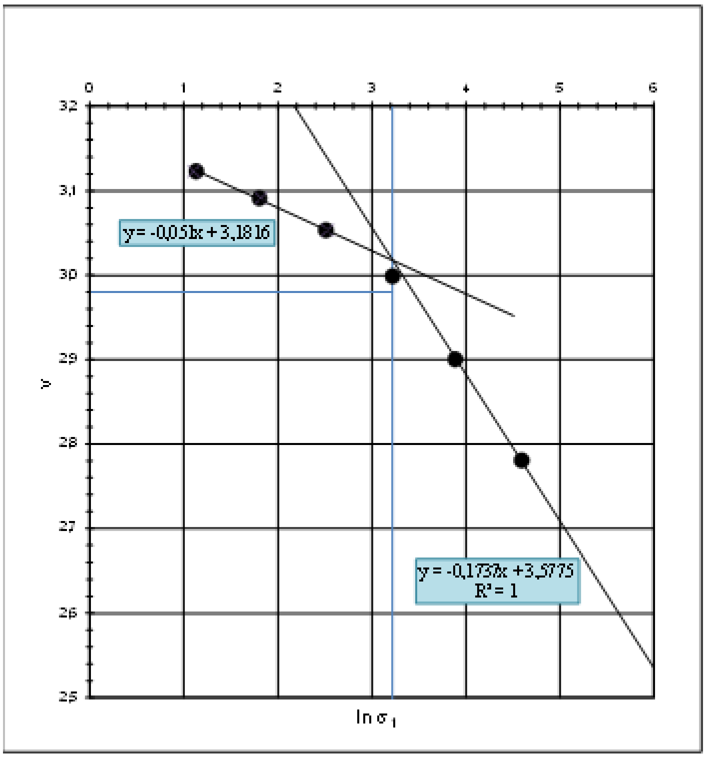
Site:	OSSINLAMPI			Test No	6545u
Point:	V4	Depth:	2,03-2,00 m		



σ_1 kPa	v= e+1	ln σ_1
3,1	3,124	1,118
6,1	2,962	1,812
12,3	3,032	2,510
24,6	2,943	3,203
49,2	2,805	3,896
98,4	2,606	4,589

λ	κ	
0,287	0,0837	
C_c	C_r	C_c/C_r
0,660	0,193	3,428

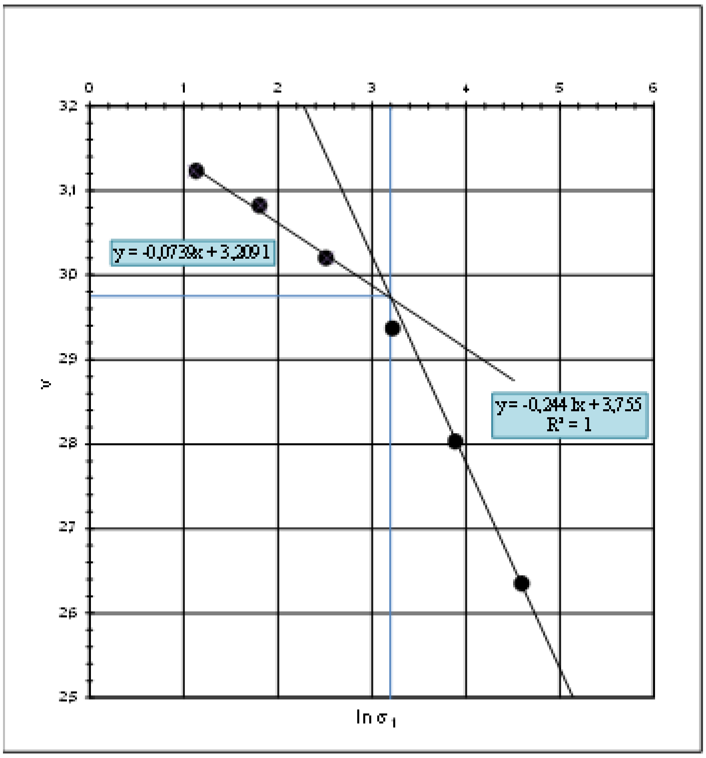
Site:	OSSINLAMPI			Test No	6549u
Point	V4	Depth:	2,00-1,97 m		



σ1 kPa	v= e+1	ln σ1
3,1	3,123	1,118
6,1	3,092	1,812
12,3	3,053	2,510
24,6	2,999	3,203
49,2	2,901	3,896
98,4	2,780	4,589

λ	κ	
0,174	0,051	
C_c	C_r	C_c/C_r
0,400	0,117	3,406

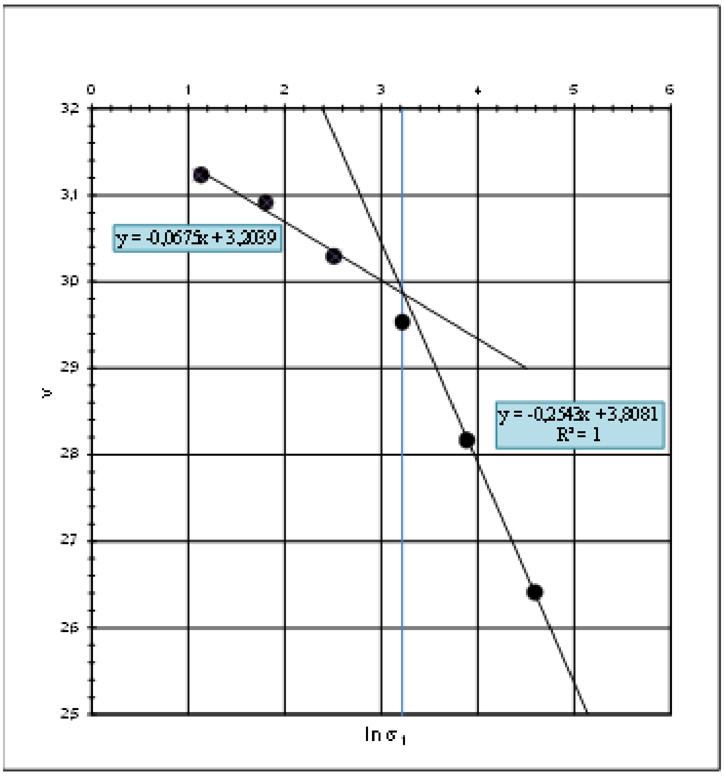
Site:	OSSINLAMPI			Test No	6550u
Point	V4	Depth:	1,97-1,94 m		



σ1 kPa	v= e+1	ln σ1
3,1	3,123	1,118
6,1	3,082	1,812
12,3	3,020	2,510
24,6	2,937	3,203
49,2	2,804	3,896
98,4	2,635	4,589

λ	κ	
0,244	0,0739	
C_c	C_r	C_c/C_r
0,561	0,170	3,303

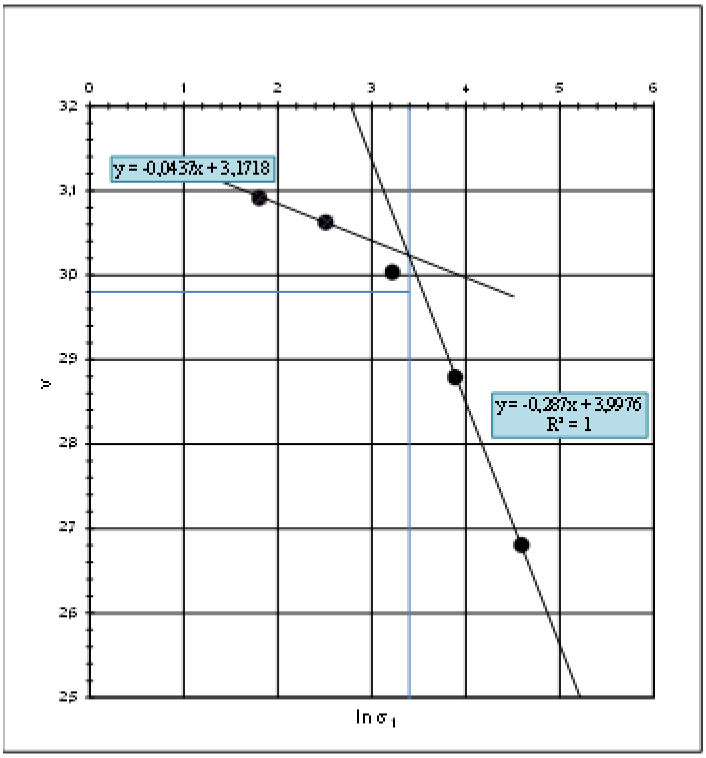
Site:	OSSINLAMPI			Test No	6557u
Point	V4	Depth:	1,94-1,91 m		



σ_1 kPa	v= e+1	ln σ_1
3,1	3,123	1,118
6,1	3,092	1,812
12,3	3,030	2,510
24,6	2,963	3,203
49,2	2,817	3,896
98,4	2,641	4,589

λ	κ	
0,254	0,0675	
C_o	C_r	C_o/C_r
0,585	0,155	3,767

Site:	OSSINLAMPI			Test No	6559u
Point	V4	Depth:	1,88-1,85 m		



σ_1 kPa	v= e+1	ln σ_1
3,1	3,123	1,118
6,1	3,092	1,812
12,3	3,063	2,510
24,6	3,004	3,203
49,2	2,879	3,896
98,4	2,680	4,589

λ	κ	
0,287	0,0437	
C_o	C_r	C_o/C_r
0,660	0,101	6,568