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TESINA FINAL DE CARRERA

DETERMINATION OF DEFORMATION PROPERTIES BY TESTING AND CALCULATION OF SETTLEMENTS

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NOTATION AND SYMBOLS:

σ_0: *Total ground stresses*
u_0: *Pore water pressure*
γ_s: $Ground\ specific\ gravity$
σ'_0: *Effective stress*
γ_w: *Water specific gravity*
Δ: *Settlement*
δ_∞: $Settlement\ in\ a\ infinite\ time$
Q: $Compression\ index\ in\ a\ \varepsilon, log\sigma'\text{-}diagram$
Δz_i: $Layer\ deep\ increase$
$\Delta\sigma'$: $Effective\ stress\ increase$
$\sigma_0{}'_i$: $Initial\ effective\ stress$
C_v: *Consolidation coefficient [m^2/s]*
E_m: *Oedometer modulus [Pa]*
K: *Permeability [m/s]*
a_v: *Compression coefficient [Pa^{-1}]*
C_c: *Compression index*
C_s: *Swelling index*
P_c: *Preconsolidation pressure [Pa]*
T: *Adimensional time*
H: *Minimum distance of water outlet*
t_c: *Time of consolidation*
V_t: *Total volum*
R: *Radium of the ring*
H_o: *Height inicial of the sample*
V_v: *Volume of water*
M_w: *Massa of water*
e_o: *Porosity initial*
H_s: *Height before the deformation*
e: *Porosity*
D_{100} : *Deformation in 100% consolidation*
D_s : *Deformation in each load.*
D_{90}: *Deformation in 90% consolidation*
ε_c: *Final Deformation*

INTRODUCTION

The aim of this project is to characterize the organic soil and calculate the settlements produced as a result of the construction of a fish farm.

The fish farm will be placed in an organic soil in the harbor of Hvide Sande. Soil samples have been obtained through drilling in the area. Frank Geoteknik Company has carried out a report from where the soil properties have been obtained, parameters which will be used to estimate the settlements generated by the fish farm construction.

Organic soil is known to have problems in consolidation associated during and after construction, to ensure that the report from Frank Geoteknik is consistent, it has been decided that samples from similar soil and from other parts of Denmark (Vejle and Aabenraa) have also been tested.

The new oedometer test was used to study the samples; this test amends some errors that the normal oedometer test has. The soil properties obtained with this new test where then used to calculate the deformations caused by the fish farm construction.

Finally, measures necessary to reduce settlements and prevent the structure from having inadmissible deformations have been studied.

ABSTRACT:

Project about the deformation properties from samples of organic soils analized by new oedometer test and later the calculations about the possible settlements can provoke.

Focusing in the construction of a fish farm in Denmark, due to the variability properties of organic soil varius samples were taken to analize.

The problem in construct on organic soil is the consolidation, specially in the secondari consolidation, creep.

The purpose is calculate the settlements of the fish farm in differents cases, considering the dates from Aabenraa and Hvide Sande and all the properties.

Large settlements are produced in this type of soil, and very dangerous for the large time that it can be producing during the creep.

To reduce the possible consequences and problems caused by the settlements it can be proposed improvements for example preloads and foundations.

Keys words: Organic, settlements, creep, consolidation, oedometer, gyttja

PART I: THEORY

1. GEOLOGY:

To introduce the project, the first step is to situate the different samples and introduces the materials and properties of each location.

Then, a comparison between characteristics will be made to understand the different features that each situation.

1.1 Location:

Denmark is a country 40.000km^2 very flat, there are not mountains, the highest hill 172m above sea level.

Denmark is divided in zones:

- Jutland includes Nordjylland, Midtjylland, Syddanmark (Fynn is the island in the West where is Odense city)
- Sjaelland includes Hovedstaden and Lolland (Island in the south)

Considering the samples of the project only it will be explained the geology in Midjutland and Syddanmark in the Jutland region.

In Jutland, the sediments were deposited through fusion of the ice layer (moraine). Since the last ice age, in North of Jutland, the soils are sandy except the highest plateaus where there are materials loamy or clayey.

The caps from the Quaternary period are thick, 200 m at Skagen the north most side of Jutland, older layers are completely covered.

The project is based in 3 samples; in the first time the sample was from Vejle, but the sample had a slow procress of consolidation that the oedometer trial was stopped andit was started again with sample from Aabenraa to compare with sample with Hvide Sande.

All the calculations are from the Laboratory in VIA Collegue University, in Horsens.

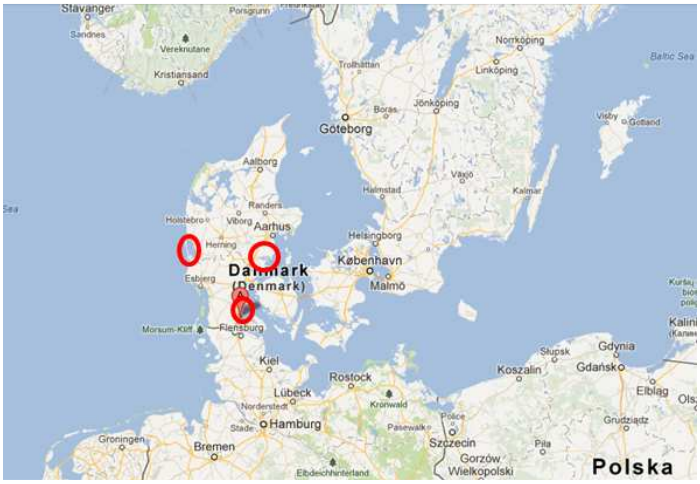

Fig. 2 Denmark map, the situation in Europe, with 3 locations.

1.1.1 Aabenraa's Port:

In Jutland, specifically in the region Sydannmark , close the fiord with the same name, where the food industry is very important in the economy, especially in the production of beer, it is founded the Aabenraa's Port which has 7,5 m of depth and important naval trade.

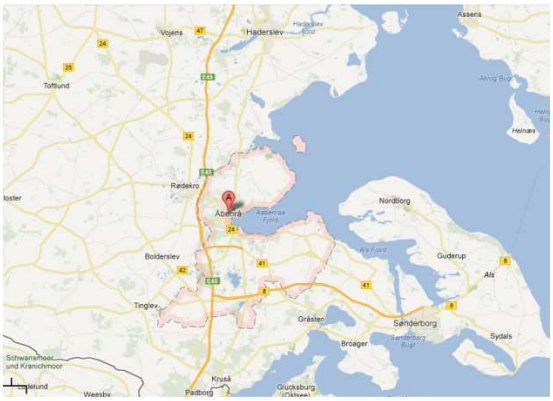

Fig. 3 Map of Aabenraa.

In the history, the port was an important place to keep the bigger commercial port in the Danish kingdom, after Copenhagen and Flensburg.

This city was known for her fish industry and production.

1.1.2 Vejle

In the Jutland, specifically in the region Midtjylland, her name, in English, is ford because in the past, it was an old wetland very important in the Viking age.

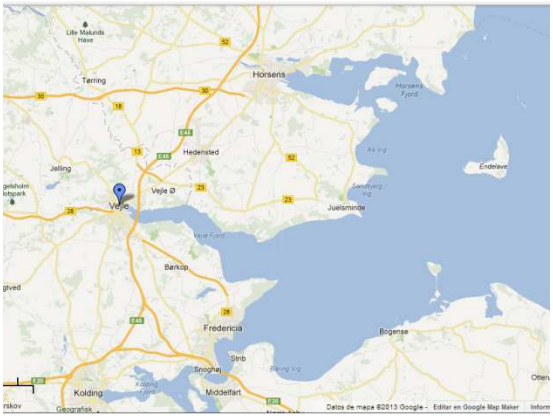

Fig. 4 Map of Vejle

Industrial, trade and services center, with the second highest hill 172m are in this zone, the first highest hill is in the north of Horsens, city where is the Laboratory of VIA, Vejle is the bigger economy side. In the century, XIX, was made an infrastructure like a new port in the fiord (1827) and rail station (1868).

1.1.3 Hvide Sande

Hvide Sade is surrounded between the fiord Ringkøbing, in the Jutlandia region,and the North Sea. The city is the entry in the fiord which has 30 km^{2}and 2-3 m of depth.

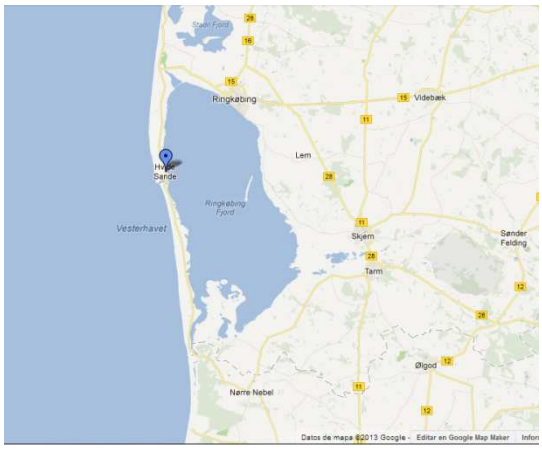

Fig. 5 Map of Hvide Sande

A surge in 1911 made an overture of 230m and bigger floods. So, in 1915 it was open other enter in the south and was close in Hvide Sande, but later in 1931 was open again in Hvide Sande.

1.1.3.1 After fish farm constructed:

The fish farm is representated in the Fig.6, to see the dimensions and the shape of the built.

Fig. 6 Actually situation of the fish farm, in Hvide Sande, Langsan 34.

1.2 Materials:

The materials in this small country are very various, there are limestone deposits, clayey, mostly sandy and loamy soils. Only in the Island Bornholm, at East in Denmark, where solid rock lies full exposed.

The landscapes are modified by great glaciers in the last ice age where the ice covered all the country.

To Study the settlements in these zones is very important to know the geology, the materials and the properties to learn the possible problems in the future.

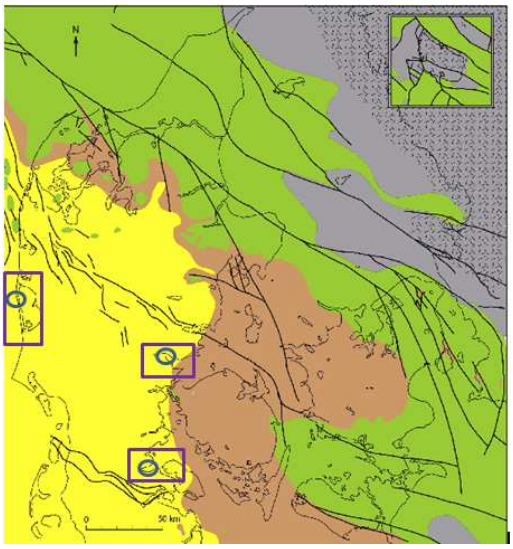

Fig.7 Denmark geological map with the 3 locations from the project.

The mostly part of Jutland in Denmark is from Upper Tertiary, the Fynn Island is from Lower Tertiary and the Sjaelland is from Lower Tertiary and Upper Cretaceous

In general, all the points are from 2-32 millions of years old in the Upper tertiary: Pliocene, Miocene and Upper Oligocene.

Fig. 8 All the samples are from Upper Tertiary (Pliocene, Miocene and upper Oligocene).

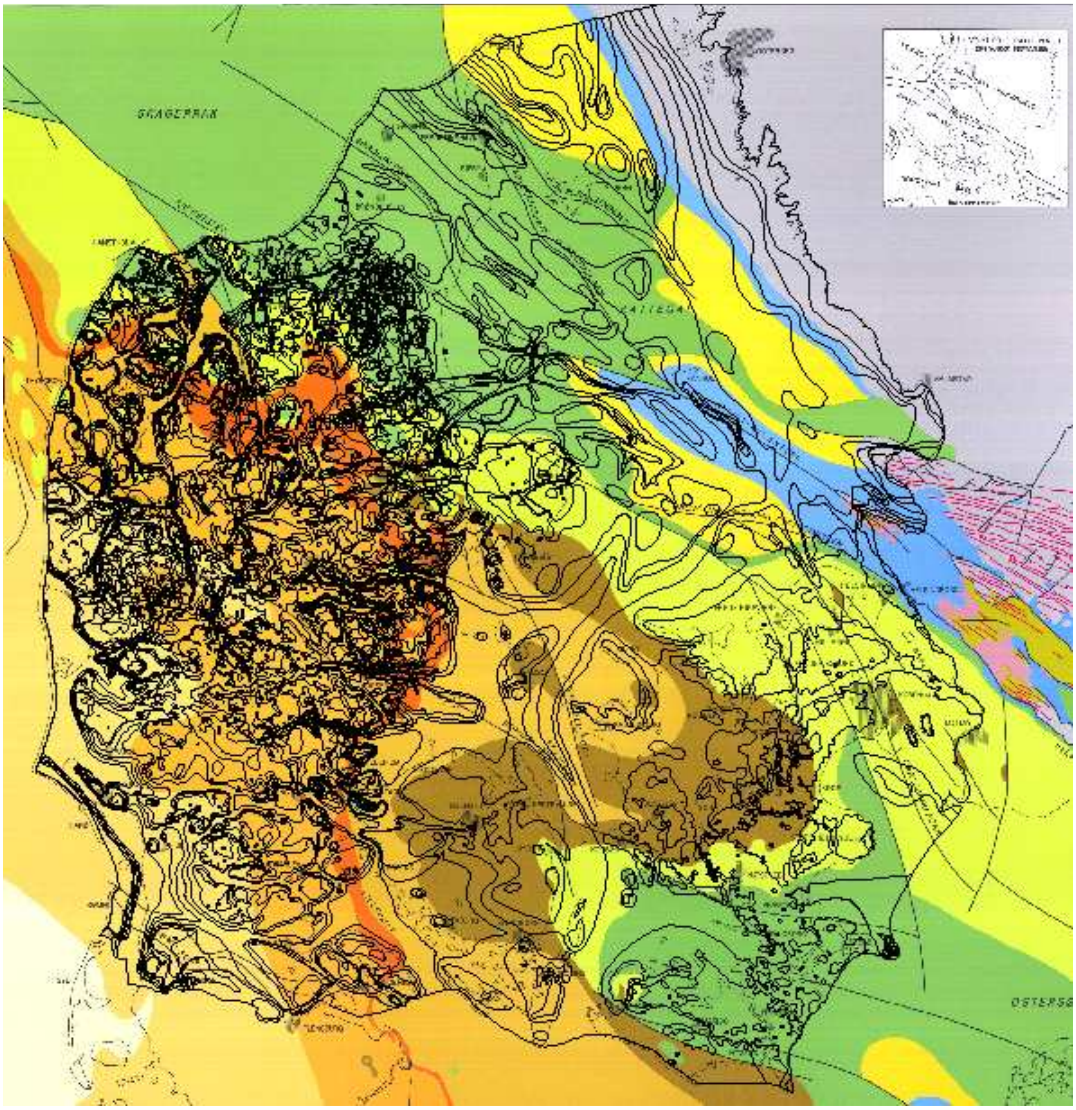

Fig. 9 Geological map from Denmark with boundaries.

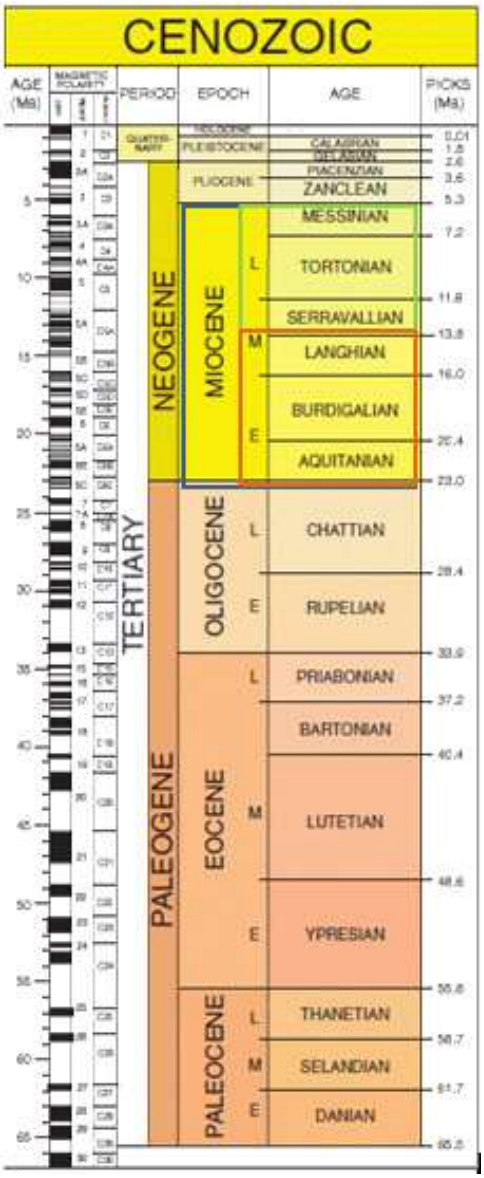

Fig.11 Geological Times Scale Cenozoic.

All the zones are from Cenozoic age, the period is Tertiary, specifically in the Upper Cenozoic; in the epoch Miocene that is divided in 3 parts in the Upper: Messianian and Tortosian; in the Medium: Serravallian and Langhian and in the Lower: Burdigalian and Aquitanian.

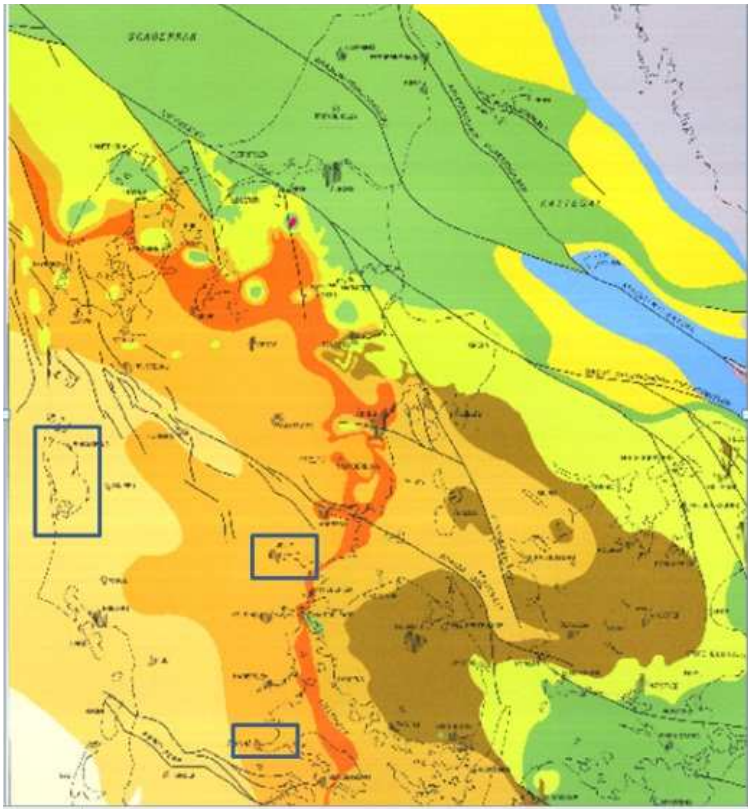

Fig.10 Geological map from Denmark with the locations from the project but without boundaries from the country.

Hvide Sande is from Upper Miocene. Vejle is from Lower Miocene. Aabenraa is from Lower Miocene.

Fig. 12 Legend from Fig.5

1.3 Problem in the North-East of Europe:

1.3.1 Moraine

Denmark is a country with cold weather; there are seasons with under zero degrees which provoke ice formation. So, lower temperatures influence in the creation of moraine. Moraine is any glacially formed accumulation of unconsolidated glacial debris (soil and rock) which can occur in currently glaciated and formerly glaciated regions, such as those areas acted upon by a past glacial maximum[1]. This debris may have been plucked off a valley floor as a glacier advanced or it may have fallen off the valley walls as a result of frost wedging orlandslide. Moraines may be composed of debris of different sizes which ranging from glacial flour to large boulders. The debris is typically sub angular to round in shape. Moraines may be on the glacier's surface or deposited as piles or sheets of debris where the glacier has melted. Moraines may also occur when glacier transported rocks fall into a body of water as the ice melts.

Types:[2]

Depending on they are deposited, the moraines can be:

Type 1: **in the front** when the moraine is deposited in the break time of glacial, formed for irregular blokes.

Type 2: **background** the materials is deposited in the bottom of the glacier, usually clay and in the same time they can be deposited in the retreat of the glacier and of different thickness depend if it has been slow or fast progress, or a moraine dispersed.

Type 3: **Accumulation** from rock material excavated and accumulated by the front of the glacier when it moved it, often the material is folded.

Materials:

- Silty, Sand or clay.

1.3.2 Gyttja

Gyttja is a freshwater deposit (mud) consisting of organic and mineral matter found at the bottom or near the shores of lakes, solid component from lake and liquid component from saline water. It is very common in East of Europe but complex as regards their mineral, chemical and biological composition[3].

Gyttja sediments were originally deposited during the Late Pleistocene and the Early Holocene.

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[1] *Benn, D. I. and Evans, D. J. A. (1998) Glaciers & Glaciation. Oxford University Press, New York, NY.*

[2] *Easterbrook, D. J. (1999) Surface processes and landforms. (Second Ed).Prentice Hall, Upper Saddle River, New Jersey.*

[3] *AndrzejLachacz, Monika Nitkiewicz, (2009) WlodzimierzPisarekSoil conditions and vegetation on gyttia lands in the Masurian Lakeland*

It is normally greenish color, but can be brown or red. In beaches, usually is a grey color, as in the project. In the wet state, gyttja has an elastic consistency and it has a brittle rupture. It shrinks strongly on drying to form hard lumps with low density.

This material is formed from small parts of shells. In Spain, it is most typical the Peat is an organic material, dark brown and rich in carbon. It consists of a spongy mass and light in which the components are still appreciated vegetable that originated. It is formed by plants. It is used as fuel and in the production of organic fertilizers. Aerobic digestion of the peat reduces the oxygen, so there is onset of degradation and the anaerobic digestion can produce gyttja that is formed of fish excrement.[4]

Accumulation takes place underwater, following the deposition of suspended matter contained; Organic gyttja is formed in hollows and depressions with no inflow or outflow, fed by nutrient-poor water.

Key role is played by the local climate, in amount of factors, like intensity and distribution of precipitation, temperature and wind, other parameters are surface features geological structure of the catchment hydrological and hydrographic relations and vegetation development.

Now, a study about geology for each location will be realized according the materials, the depths and water level.

1.4 Characteristic for each Location:

Using the program JUPITER, from the website http://www.geus.dk/digital_data_maps/, it is possible check borings made in Denmark, you can know the materials with characteristics like age, components and depths and thickness, the ground water level…

At the end, a comparison of all locations will be realized.

Scale of colors in general in our drillings:

Green borings: made by a companies, like The Company Geo Heat, application is geothermal.

Blue borings: it is to know the water level; one application is in fish farms, in the project's case.

Orange borings: made by Geo Geoteknisk institute, purpose: geotechnical.

Dark pink borings: made by private companies, application: dropped/ abandoned a plan.

Light pink borings: application is mining and drillings.

All the borings used to make the comparison are in the ANNEX2: Geology

[4] *Myœliñska E. (2003) —Classification of organic soils for engineering geology.Geol. Quart., 47 (1): 39–42. Warszawa.*

1.4.1 Aabenraa Port:

In the South- East of Denmark, inside a fiord there is port of Aabenraa, 6 borings were considered:

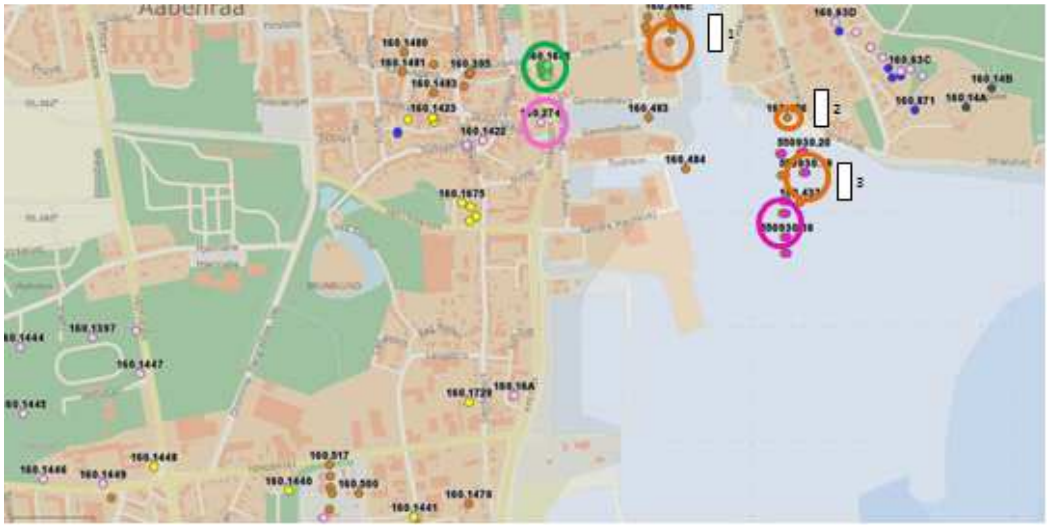

Fig. 13 Map of the borings of Aabenraa. The port is in the west zone but there are not borings, it is taken the information from the East.

All the references about the notifications and colors from the geology from Jupiter are in the ANNEX 2. Geology.

In the city zone, the green point is 200m of drill, there are not layer of gyttja in all ground, all clay and sand. It is good, because the gyttja provoke a lot of problems.

In the light pink point, near green point, the drill indicates there are a lot of layer of glacial moraine intercropped between sand and clay.

In the pink point in the water, inside of fiord, 23 m of drill gives information about the glacial layer, since 5m of depth is a glacial moraine.

The orange point gives the information about the postglacial, glacial layers, and the different glacial moraines: silty, sand and clay.

In the superficial layer in the point 1 and 2 is filling, in the point 1 later there is a postglacial layer and in the point 2, there is layer of gyttja in the 2.6 m of depth above the postglacial layer and the point 3 the superficial is postglacial with gyttja.

In the point 3, there is postglacial gyttja layer, different glacial layers: clay, fusion water and the last is interglacial layer.

The blue borings had not been considered because in the city there is other level of water different than in the water of the port.

0 Fill	0 Fill	
	2.6 gyttja	**0** postglacial **gyttja**
5 Postglacial marine mud	4 postglacial clay	2.5 postglacial
8.3 small layer	7 postglacial sand	4.4 glacial clay
9.4 glacial moraine silty	8.7 silty	7.3 *glacial fusion water*
13.3 glacial moraine clay	9.3 glacial moraine clay	10.1 clay glacial
20 glacial sand	14.9 glacial moraine sand	11.8 interglacial
24.4 clay	15.2 glacial moraine clay	

Table: 1 Sequence of materials in each orange boring to see the glacials layers.

There are a lot of glacial layers and glacial moraine, it is very important to know where they are and the type. Depending of the material the consequences is different.

1.4.2 Hvide Sande Port:

In the west, within fiord Ringkobing, 5 boring have been considered.

It is taken information from the report, to complete the description; a lot of fill in this area underneath postglacial layer is found and it is supposed that the layer of postglacial is 19-20m under Geological Surface.

Fig. 14 Geological boring from Hvide Sande. In the zone inside fiord.

Green boring is not considered because is in a zone different that the study's zone.

The sample wasbetween 7-9m of depth, blue point in the city indicates the groundwater level is between 1-2 m of depth. But in all project, it is respected the water level in the top, in the surface of the ground, to make all the calculations.

Points pink in the port zone gives the information about climate, chronological and lithology:

Climate		chronological	lithology
0 – 20m	marine	postglacial	
20 – 25m	glaciofluvial	glacial	
25 - 95m	marine		Miocene
0 - 21m	marine	postglacial	Holocene (upper Quaternary)
21 - 24.5m	glaciofluvial	glacial	Quaternary
24.5 - 25m	glaciolacustrine	glacial	Quaternary

Table: 2 Relation between the lithology obtained depending of the depth that determine the climate.

Hvide Sande is girdedin one side the North Sea, it influences in the climate marine, with saltwater and in the other side there is the fiord with fresh water that provoke climate fluvial and lacustrine. And the chronological is normal,deeper there are the glacial materials and over they were deposited latest materials postglacial.

0 sand
1.1 gyttja
1.55 gravel
1.6 gyttja
3.95 sand
4.4 gyttja
4.7 sand
5.5 gyttja
7.7 sand
8.5-9.4 gyttja

Table: 4 Sequence boring 2 from Hvide Sande

0 sand
2 gyttja
2.05 sand
2.3 Gyttja
3.1 sand

Table: 3 Sequence boring 1 from Hvide Sande

In this zone, there are a lot of layers of gyttja between layers of other materials. It is very important study these layers to avoid problems with the building of fish farm.

1.4.3Vejle Port:

In the East part of Denmark, near Horsens, city where the laboratory from VIA is, the sample is from the water part of Vejle. The groundwater level is in 8.5m in the city and 2.85 in the fiord zone.

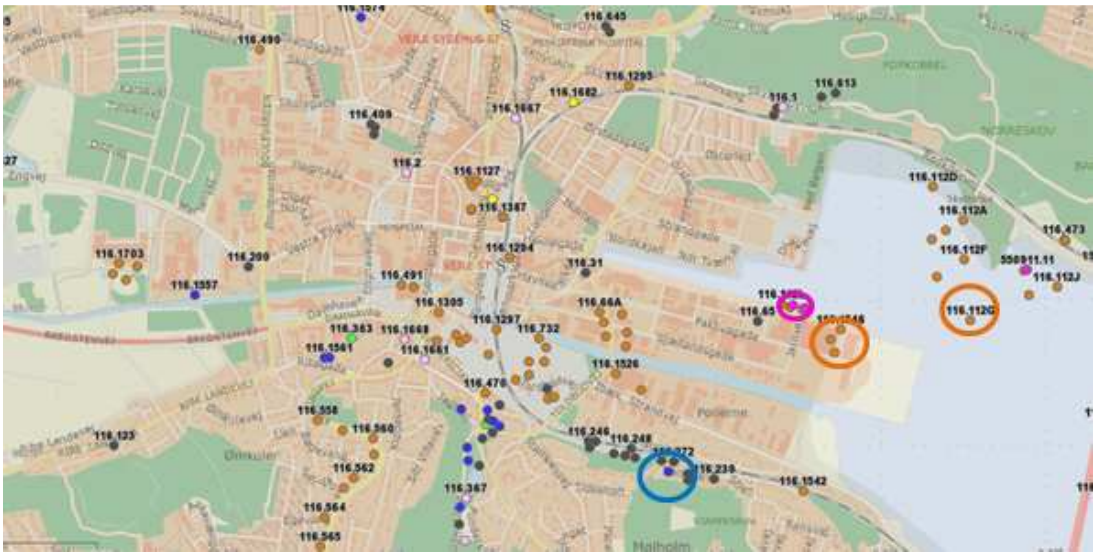

Fig. 15 Map of Vejle with the borings.

In the city zone, there are not layers of gyttja, there are alternation sand layers and clay layers, and in all the case the layers of gyttja are in the water zones, it is very good new to the security in the constructions in the city.

In the port zone, the point pink has 8 m of gyttja above sand and clay and the orange point

0 Gyttja
8.5 sand
13.5 clay

0 Gyttja
6 gravel/sand
7.5 sand
13.5 clay

Table: 5 Sequence of materials in Vejle from the orange drillings.

All the layers of gyttja are in surface of the ground; it can be removed or studied to improve the consistence.

1.5 Comparison:

SIDE	Aabenraa	Hvide Sande	Vejle
age	Lower miocene	Upper miocene	Lower miocene
materials	Gyttja, silty, clay, sand	Gyttja gravel, sand	Gyttja, sand, clay
Water level	-	1-2 m	8.5 m city, 2.85 m fiord
phase	Postglacial, glacial, interglacial	Postglacial, glacial	-
moraine	Sand, silty, clay	-	-
clima	-	Marine, fluvial, lacustrine	-

Table: 6 Comparison between the characteristic of three locations

Only there are moraines in Aabenraa, but in all locations there are the same age and materials.

2. ORGANIC SOIL

"Organic soil" is a type of soil formed for mineral sediments and organic matter. This organic matter can be formed from animals, plants organisms, carrying out biogenic matter by mixing with mineral matter.

The organisms and aquatic animals play an important role in the formation of the organic soil; the plants also form the basis of soil with an organic content as a significant amount of organic soil is formed directly or indirectly by plants. By the other hand, terrestrial animals have not usually an important role in relation to geological themes.

This kind of soil has always created some doubts about its division into different groups.

One of the identification processes utilized it is by its combustibility, they are created by the decomposition of the plants and animals dead. The process mainly it takes place through bacterial activity and is intensified by a suitable humidity, a hot climate and access to oxygen from the air.

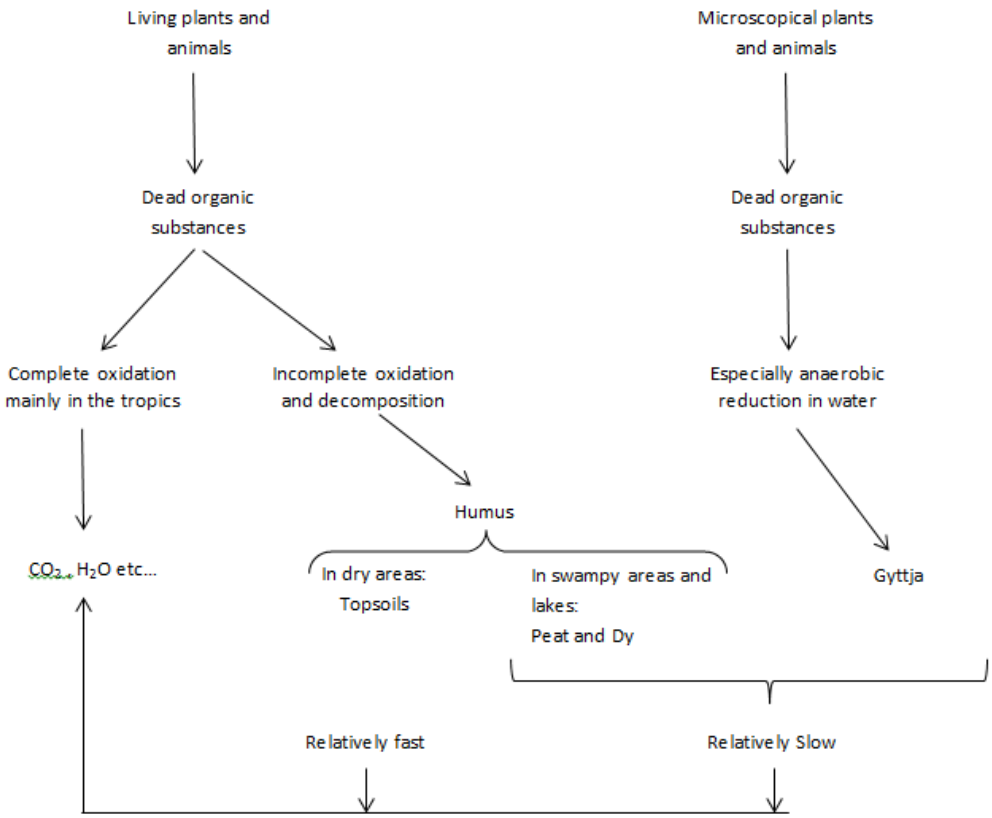

Fig. 16 Diagram with the sequences of formation of the organic soil.[5]

- [5] *Peter A. Larsen,(1978) INITIAL DEVELOPMENT (RIPENING) OF SOME RECLAIMED GYTTJA SOILSIN KOLINDSUND, DENMARk University of Aarhus, Denmark*

This project is focused in Gyttja, so solely is going to explain with detail this kind of soil.

How is Gyttja formed?

As it has explained in the chapter 1, Geology,Gyttja is formed by a flocculent substance whichis originated from plants and animal rich in fats and proteins. This flocculent substance is generated through the decomposition and dissolution of dead microscopic aquatic animals and in that the minerals and remains of plants and animals less decomposed are embedded in the soil. Gyttja is completely formed when it is produced a fermentation processes that generate sulphureted hydrogen and methane.

Engineering properties in organic soil.

The engineering properties in organic soils are a function of the amount and kind of organic matter content in the soil. When a pure mineral soil is compared with another which contains organic matter in its structure, it can be observedthatorganic soil produces changes in geotechnical properties. The organic substance decreases permeability and has tendency to increase creep.

As in mineral soils, the loading history and the effective stress acting in the soil are the parameters which determine the organic soil strength. The soils which contain organic matter are as a rule fairly recent deposits in waterlogged areas. Furthermore, most of them do not have a dry crust and have only resisted the load of the overlying soil, without having been subjected to any other load so that organic soils have no a significant load history. By being an uncompressed soil and have a large ground water level, the organic soils have a low density and effective stress. Consequently, the soils with organic matter content have a low strength and are extremely compressible. They also exhibit large creep effects.

Some organic soils can contain remains of undecomposed plant, this generates an anisotropic structure. These remains of plants are usually in horizontal position so that it produces shear strength higher than mineral soils and permeability higher in horizontal axe than vertical.

In summary, the geotechnical properties about the organic soil depend of amount and type of organic matter. Furthermore it has to consider structural anisotropy of the soil if it contains remains undecomposed plants, as even with a low content of organic matter it can produce significant effects in its properties.

The most important problems caused by organic soil in relation with engineering are itscreepeffects, low strength and effective stress, its high compressibility caused by low density, as even with small loads might occur that settlements which decrease more than half of the original thickness of organic soil layer.

These settlements are produced by a processcalled consolidation.

3. CONSOLIDATION:

The consolidation is a coupled process of deformation and flow, itmeans,there is flow if there is deformation and conversely. It always depending in the effective stresses variation.

This process happens in saturated soils, it means, the soil is formed solely by solid and liquid matter, and therefore there will be deformation when itoccurswater outlet, as water and solid matter are considered uncompressible. It can differentiate various processes in the consolidation in apply a load. Initially,it is producedan instantaneous deformation in which there is not water outlet, so there are an increment of interstitial pressure and total stresses, equal to the value of the load applied. Due to there are not water outlet in this phase, the settlements are recoverable. Later, a deformation deferred in time is produced, in which occursa volumetric decrease caused by the water outlet through the pores. In this phase it can be observed a decrease of the porewater pressure due to de water leakage, therefore there is an effective stress increased. This phase has unrecoverable, it won't have the same characteristics, and recoverable deformations, the first moment that the material is "elastic" and it will have the same characteristics, although the unrecoverable settlements are higher. This process is called **primary consolidation** and is coupled with the **second consolidation**, a longer and complex phase which is composed by unrecoverable deformations.

In the following figure, fig.17, it can be observed in a schematic mode the deformation evolution in a saturated soil.

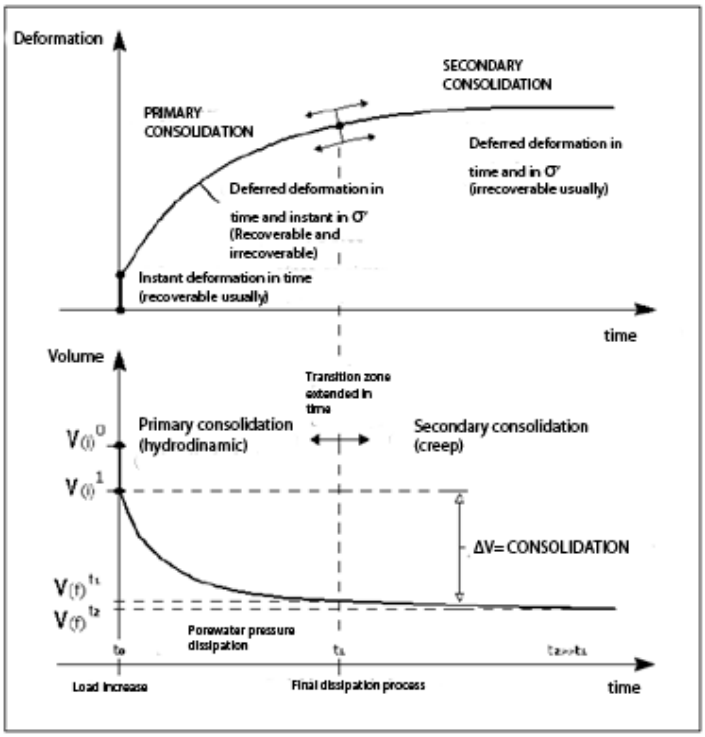

Fig. 17 Evolution about the deformation in time, in a load process of a saturated soil sample.

In figure 17, it represents a section of a saturated ground where it will be applieda load and it could analyze the consolidation process steps, since initial state to final state when it has produced the interstitial pressure dissipation.

In initial situation [0], it can observe the total ground stresses (σ_0) and pore water pressure (u_0) distribution. In this case, the interstitial pressure is equal to hydrostatic pressure, where:

$$\sigma_0(z) = \gamma_s \cdot z \qquad \gamma_s\ (ground\ specific\ gravity) \qquad (1)$$
$$u_0(z) = \gamma_w \cdot z$$
$$(2)\sigma'_0(z) = \sigma_0(z) - u_0(z) \qquad \sigma'_0(z)\textit{(effective stress)} \qquad (3)$$

After the load application, it is produced an instant increase of stresses. This instant situation [1] it resembles to undrained conditions, it means, there is not water outlet, consequently, both stresses, total stress and interstitial pressure, they increase with an equal value to the increased load (Δσ). Therefore, there will not be effective stress variations.

$$\sigma_1(z) = \sigma_0(z) + \Delta\sigma$$
$$(4)u_1(z) = u_0(z) + \Delta\sigma \qquad (5)$$
$$\sigma'_1(z) = \sigma_1(z) - u_1(z) = \sigma'_2(z) \qquad (6)$$

Thereafter, it will begin the consolidation process [2], the water will start to be expelled, producing interstitial stress dissipation and generating an increase in the effective stresses and a volume decrease of the soil until reaching again the hydrostatic pressure.

$$\sigma_2(z) = \sigma_0(z) + \Delta\sigma = \sigma_1(z) \qquad (7)$$
$$u_2(z,t) = u_1(z) + \Delta u_2(z,t) \qquad (8)$$
$$\sigma'_2(z,t) = \sigma_2(z) - u_2(z,t) \qquad (9)$$

3.1 Primary consolidation process:

The primary consolidation is a coupled process of flow and deformation in which there is an alteration of effective stress differed over time, it is caused by the interstitial pressure dissipation. These effective stresses variations induce deformations.

The pictures 19 and 20 represent in a schematic way the corresponding trajectories to the values of interstitial pressure and effective stresses, and total and effective stresses, respectively.

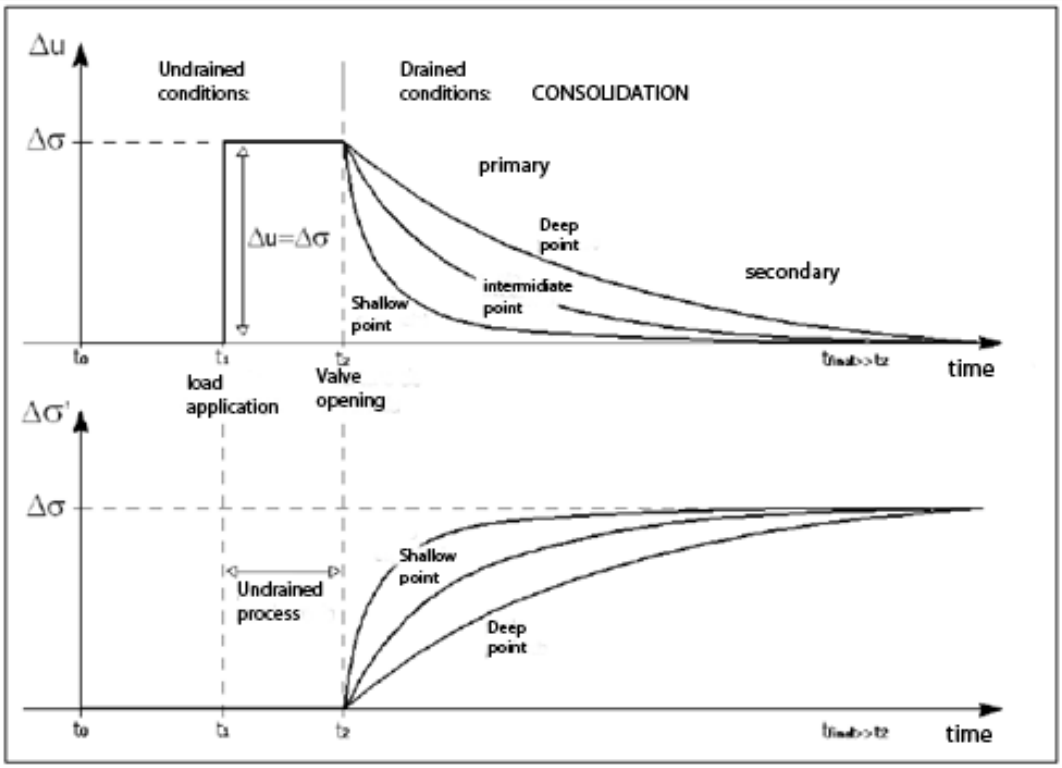

Fig. 19 Trajectoriesof the values of"interstitial pressure" and "effective stress" as a functionof time.

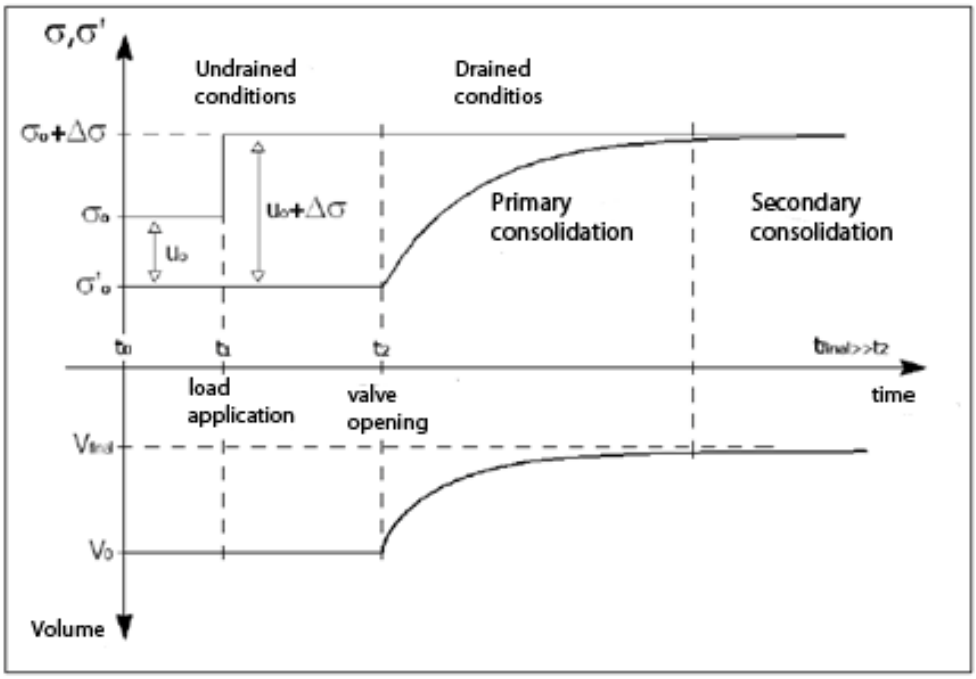

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Fig. 20 Trajectories of pressure values and volumetric deformation in function of time.

In undrainedconditions, interstitial overpressures and recoverable deformations are appeared.

In drained conditions, if the load is such big that the soil structure cannot bear it (the particles contacts are not enough to bear the load), the soil reorders its particles

decreasing pores and increasing contact surface among its particles, this means an increase in its density.To make this happen, water retained in pores must be expelled. The loss of volume produced in the soil, it produces in a non-recoverable fashion.

To calculate the settlements in an organic soil, where it is usually a normal consolidated soil, it is used the below equation (10).

$$\delta_\infty = \sum_{i=1}^{i=n} Q \cdot \Delta z_i \cdot \log(1 + \frac{\Delta\sigma'}{\sigma_0{}'_i}) \quad (10)$$

Where;

$\delta_\infty = Settlement\ in\ a\ infinite\ time$
$Q = \ compression\ index\ in\ a\ \varepsilon, log\sigma'\text{-}\ diagram$
$\Delta z_i = layer\ deep\ increase$
$\Delta\sigma' = effective\ stress\ increase$
$\sigma_0{}'_i = initial\ effective\ stress$

3.2 Second consolidation:

The secondary consolidation is a non-recoverable deformation which is produced of deferred form to the effective stress change; therefore it does not follow the effective stresses principles of Terzaghi. These deformations are usually irrecoverable and which occurs very slowly. The secondary consolidation process does not occur consecutively to primary consolidation, if not that it occurs simultaneously and it can begin right at the time of the load application.

In the secondary consolidation mightexist overpressure in the smaller pores as well, but as this kind of consolidation is slower, these overpressure are negligible. In this process, the soil ends adapting to its final terms and it does not follow the elastic model.

4. OEDOMETRIC TEST:

To study the soil behaviour in a specific soil, it utilized an oedometer trial, which is a common trial utilized in the geotechnical laboratories. This is a trial in which is obtained a good relation between the data obtained and the cost, although the time spent to obtain the results usually take long time.

The oedometer test is carried out with a rigid ring, so it does not allow horizontal deformations, it means, the volumetric deformation is equal to vertical deformation.

The oedometer trial results are interpreted from the consolidation theory which has already been referred in chapter 3, in which it treats in a coupled process of flow and deformation. Therefore, the process is represented by the Terzaghi equation (11):

$$C_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{dt} - \frac{\partial \sigma}{\partial t} \quad (11)$$

Where operating, it can be obtained the following formula (12), which it is related with the permeability and the oedometer modulus.

$$C_v = \frac{K \cdot E_m}{\gamma_w} \quad (12)$$

4.1 Test process:

For this project it has been utilized the new oedometer apparatus. This new apparatus has been fabricated to be able to avoid some of the standard apparatus errors, as:

1. Crushing of grains situated between two parts of the apparatus.
2. Closing of gaps between different parts of the apparatus because the surface is not perfectly plane or due to faults in the arrangement of the apparatus.
3. Deformations in threads or bending of the support.

The fig.21 shows the new oedometer apparatus.

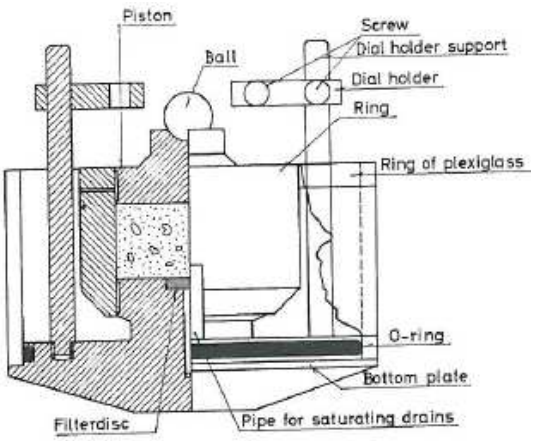


Fig. 21 New Oedometer test

The specimen utilized in this project has a diameter of 70 mm and a height of 35 mm. It is only allowed the water outlet by the bottom of the ring through a drained formed by a porous stone. The porous stone allows the pore water drain to the water deposit which is located around the ring and which has the mission to keep the soil saturated at all times. At the top of the ring there is a ball which is responsible to

transfer the vertical load applied by a lever arm and weights. Also there are two cells at the top, to measure the produced strain into the sample.

After calibrate the equipment and put the sample in it, it proceeds to apply the different load cases. To this test they have been utilized the followings vertical weights: 250, 500, 1000, 2000, 4000, 9000, 19000 and 2900g. As was explained in chapter 2, the organic soil has not a significant load history, it means, the soil is normally consolidated, therefore the project has only been carried out by the loading phase, without make the discharging phase, which is made to obtain the preconsolidationpressure.

4.2 Obtaining parameters from oedometer test:

From the oedometer test it is possible to obtain the following parameters:

- C_v : Consolidation coefficient [m^2/s]
- a_v : Compression coefficient [Pa^{-1}]
- E_m : Oedometer modulus [Pa]
- K : Permeability [m/s]
- C_c : Compression index
- C_s : Swelling index
- P_c : Preconsolidation pressure [Pa]

Nevertheless, in this project it is only explained how to obtain the consolidation coefficient (C_v) and the Oedometer modulus (E_m), as they are the necessary parameters for the project.

The Terzaghi equation (13) contains the Consolidation coefficient (C_v), which related interstitial pressure dissipation with time. The determination of C_v is made from the experimental curve which related the porosity, or volume deformation, with time.

$$C_v = \frac{T \cdot H^2}{t_c} \qquad (13)$$

Where;
T= Dimensionless time
H= Minimum distance of water outlet
t_c= time

4.2.1 Casagrande's method:

Casagrande's method or also called logarithmic method plotted the time in logarithmic scale for doing more visible the changes. It begins by the curve shown in fig. 22, which has values obtained by oedometer test. At first, it must make a parabolic extrapolation to obtain the initial porosity (e_0). Afterwards, it necessary obtains (e_{100}), which is can be obtained, by the two slopes indicated in the chart. The point where both lines intersect shows the moment, in which has been produced e_{100}. The midpoint between both deformations is the deformation produced when it is occurs the 50% of consolidation. Having obtained e50, it is possible to obtain t_{50} through the curve, which belongs to a consolidation stage of 50% ($U_{0.5}$), thereby and looking in the table $U=f_{(T)}$ (table 7), it calculates the dimensionless time (T_{50})=0.197 and by the equation,

$$C_v = \frac{0.197 \cdot H^2}{t_{c50}} \quad (14)$$

it is possible to calculate the Consolidation coefficient, where H is the sample height in each load case, as the oedometer test only has drain at the bottom of the ring.

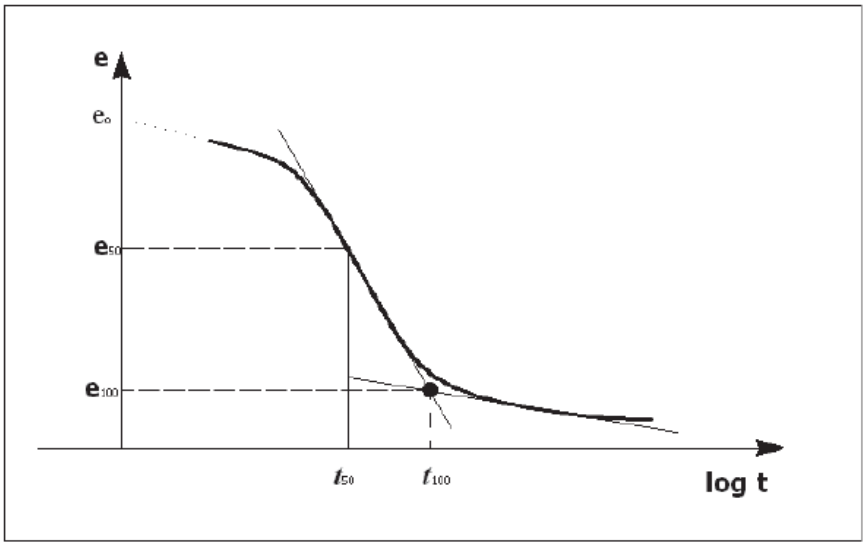


Fig. 22 Chart e-log(t) for a determinate load case. Casagrande's method.

The dimensionless time it is related with the consolidation grade. This relation it is shown in the following equation.

$$T < 0.2 \qquad U(T) = \sqrt{\frac{4T}{\pi}} \quad (15) \qquad ; T < 0.2 \qquad U(T) = 1 - \frac{8}{\pi^2} \cdot e^{-\frac{\pi^2}{4}T} \quad (16)$$

U(%)	T
0	0.000
10	0.008
15	0.018
20	0.031
25	0.049
30	0.071
35	0.096
40	0.126
45	0.159
50	0.197
55	0.238
60	0.287
65	0.342
70	0.405
75	0.477
80	0.656
85	0.684
90	0.848
95	1.127
100	2

Table: 7 Theroric relation U(%) = $f_{(T)}$

4.2.2Taylor's method:

The alternative method to obtain the Consolidation coefficient, it is the method based in Taylor theory, which plotted the relation between deformation and time in root square scale as it is represented in chart 23.

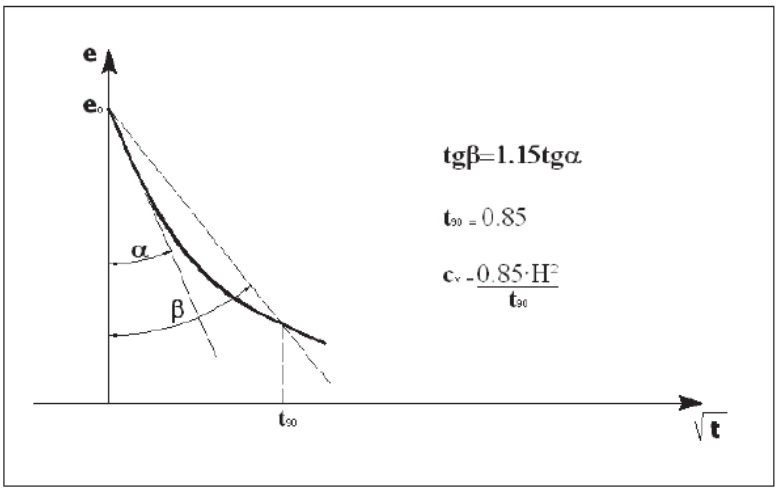


Fig. 23 Chart e-$\sqrt{t}$ for a determinate load case. Taylor's method.

For obtaining Consolidation coefficient in this method it must draw a line into the first straight stretch of the curve as it is shown in the above chart, Fig23. This line has a degree which is called (α). From the point in which alpha line cross the y axis, it must draw another line with a degree equal to 1.15α, this degree is called (β), β=1.15α (this relation is not arbitrary and it is demonstrated). According to Taylor, the point in which the line beta intersects with the curve t allows to estimate t_{c90} which is produced when de sample reach the 90% of consolidation ($U_{0.9}$). By the same way as in Casa Grande method, it is possible to calculate C_v by the equation:

$$C_v = \frac{T_{90} \cdot H^2}{t_{c90}} \qquad (17)$$

Where T_{90} is taken from the table U=F(T), T_{90}=0.848.

4.2.3 Oedometer Modulus:

For obtaining Oedometer modulus it is necessary to make a chart which relates deformation with effective stress in logarithmic scale as it is shown in chart 1.

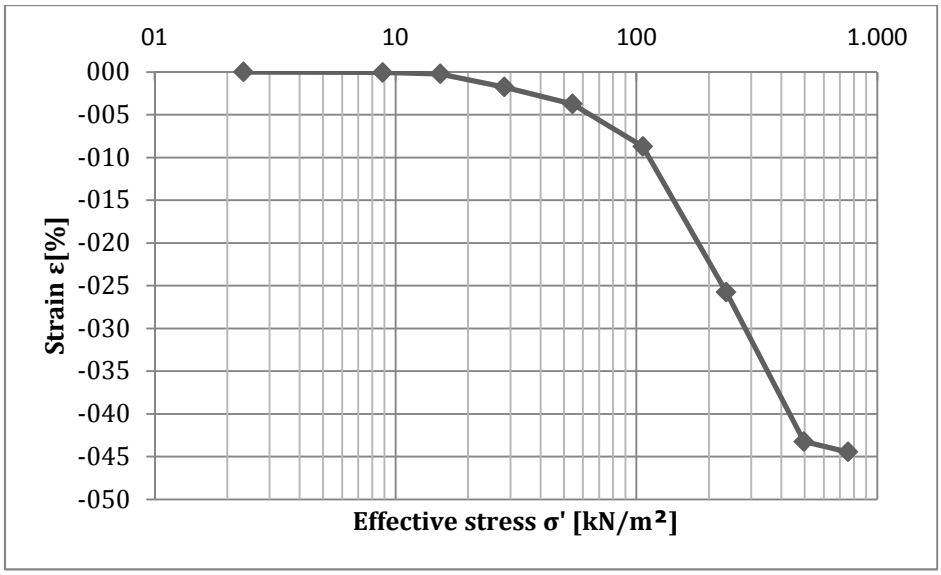


Chart: 1 Relation between strain-log(σ')

The oedometer modulus is represented by the slope in each stretch; therefore it can be obtained by:

$$E_m = \frac{\Delta\sigma\prime}{\Delta\varepsilon} \tag{18}$$

Using the increments of stress and deformations.

Also, it can be represented for the compression index in a ε,logσ'-diagram (Q), as:

$$Q(\%) = \frac{\Delta\varepsilon}{\log(\frac{\sigma_f}{\sigma_0})} \cdot 100 \tag{19}$$

Using the increments of deformations and logaritmics of stress. To be more accurate. Following the danish standards. It will be the formula used.

In conclusion, the inability that the oedometer test has to deform horizontally, makes the results obtained in this type of test are not entirely representative in the real behavior ground under the same load with unrestricted horizontal deformation, it means:

- The actual consolidation is generally not one-dimensional, so that deformations obtained for the test are lower than reality, because in reality the soil can deform laterally. In the typical case of an embankment, if it is large enough, it can assume that in the ground beneath, and in an area sufficiently far from the edges of the embankment, are restricted largely the horizontal displacements (completely in the symmetrical point), but not near the edges.

- Also, due to test conditions, pore pressure increases obtained (equal to the total vertical stress variationtest load, Δσ) are superior to real. In the 'real consolidation', Δu <Δσ. From this point of view, the settlements obtained in the test will be higher than the real (it is assumed Δu = Δσ) and in conjunction with the previous point it is usually obtained some overestimation of deformations of consolidation that leaves the side of safety.

PART II: LABORATORY

5. OBTAINED RESULTS:

The method is concerned mainly with the primary consolidation phase, but it can also be used to determine secondary compression characteristics. The soil sample is loaded vertically in set increments of applied stress. Water draining from the sample causes a decrease in volume; this is represented in a decrease in height and a rise in density. As the soil is fully saturated all water content can be represented as the void area of the soil.

5.1 Vejle test:

The sample from Vejle with sells and organic components, dark grey, wet and very soft, the results had a lot of problems, it needed a lot of time for arrives to the final of the first consolidation, 3 loads were put:

[kN/m^2]	[g]
2,3	0
8,8	250
15,3	500
28,3	1000

Table: 8 Loads in the process

With these loads, the next parameters were obtained using method from VIA University:

σ' [kN/m^2]	ε_c [%]	ε_c - ε_{slut} [%]	σ_m [kN/m^2]	K [kPa]	c_k [m^2/s]
2,3	0,0				
8,8	0,0	0,0	5,6	18558,8	9,8E-07
15,3	-0,1	0,0	12,1	11252,8	1,1E-06
28,3	-0,3	0,1	21,8	7443,8	2,4E-06

Table: 9 Information from the method used from VIA.

University uses the formulas (20) and (21) based in the diameter (D) that in the project's case is 0.7m and t_c is the time obtained from t_c= a + b*x.

a=point intersection, b= slope, x= (intersection final- intersection initial)/ (slope final-slope initial)

Following the formulation for the new oedometer test, explained in the chapter 4.

$$k = \frac{\pi}{4} * \frac{\gamma_w * H^2}{E * \mathrm{tc}} \quad (20)$$

$$c_k = \frac{\pi}{4} \cdot \left(0{,}7 \cdot D\right)^2 / t_c \quad (21)$$

Deformation was very small in all the loads, so the decision was to change the sample to have bigger results.

Chart: 2 Properties Vejle sample.

The chart is correct, the relation and the parameters are good, but there is the limitation of the time to make the project.

stage	P1 (kN/m2)	P2 (kN/m2)	H1 (mm)	H2 (mm)	ΔH	average H
1	2,3	8,8	35	34,98	-0,02	34,99
2	8,8	15,83	34,98	34,93	-0,05	34,955
3	15,83	2803	34,93	34,798	-0,132	34,864

Table: 10 Settlements for 3 loads in the Vejle sample is 0.202mm

Considering 3 loads, total settlement was 0.202 mm. The value is very small for 1 kg. To avoid the slow time in the progress the diameter of the filter was changed of 0.35m to 0.7m to evacuate more water.

5.2 Aabenraa, test 1:

Test started on 3rd of April of 2013 and it finished on 17th of May of the same year, it lasted 44 days; the oedometer test worked with a ring of diameter D=70mm and H_o =35mm, and a filter of the same size that the diameter.

Weight (g)	Before	After
bowl	116,11	114,49
bowl+ dry soil	177,06	245,67
bowl+soil+water	208,6	309,83
water	31,54	64,16

soil wet	92,49	195,34
dry soil	60,95	131,18

Table: 11 Weight before to dry the sample and after, the test from case 1 Aabenraa

After the process of dried, the water content decrease from 51.75% to 48.91% and the specific gravity changes from 15.30 [kN/m3] to 18.2 [kN/m3].

Water before: $w = \frac{M_{wet-M_{dry}}}{M_{dry}} = \frac{92.49-60.95}{60.95} = 0.51747$ (22)

Water after: $w = \frac{M_{wet-M_{dry}}}{M_{dry}} = \frac{195.34-131.18}{131.18} = 0.4891$

Quantity of water decreases a little bit because the soil is more compact and oedometer test went out percent of water from the soil.

5.2.1 Description of the charts:

The sample is with mud, some skulls, some tender plant residues, gray, calcareous, obtained from 7-9m of depth.
Rod (t) and Log (t) will be explained for each load.

In the firsttest of Aabenraa 5 loads were put:

[kN/m^2]	[g]	t(days)
2,3	0	
8,8	250	1
15,3	500	5
28,3	1000	8
54,3	2000	6
106	4000	24

Table: 12 Duration for each load in the Aabenraa test 1.

For the last load, the time needed was long, because the sample looks that was not arriving to the primary consolidation, but after it was checked that it had arrived days ago.

stage	P1 (kN/m2)	P2 (kN/m2)	H1 (mm)	H2 (mm)	ΔH	average H
1	2,3	8,8	35	34,67	-0,33	34,835
2	8,8	15,83	34,67	34,189	-0,481	34,4295
3	15,83	2803	34,189	33,579	-0,61	33,884
4	28,03	54,3	33,579	32,809	-0,77	33,194
5	54,3	106	32,809	31,269	-1,54	32,039
				3,731	-3,731	

Table: 13 Values of the heights it depends of the loads.

5.2.1.1 Load 250g (8.8 $KN/_{m^2}$)

The first load, 250g, needed one day to arrive to the consolidation stage.

Chart: 3 Rod (t) for 250g **Chart: 4 Log (t) for 250g**

Shape looks correct, in the initial moment: it is an immediate settlement very small and then it is started the primary consolidation and at finally, the secondary consolidation, creep. Sample is very soft, for this reason, shape is irregular.

It is the only case with form of Log (t) correct, with the 3 phases: immediate, primary and secondary.

5.2.1.2 Load 500g (15.3 $KN/_{m^2}$)

Chart: 5 Rod(t) for 500g, test 1 **Chart: 6 Log(t) for 500g, test 1**

In the second load, the sample is less irregular, but in the chart 6, Log(t) shows a problem in the initial moment there is a swelling, it is not common so maybe it is a mistake or someone touched the load. Besides, Log (t) needs turn in the secondary consolidation, it is not appreciable.

5.2.1.3 Load 1000g (28.3 $KN/_{m^2}$)

Chart: 7Rod(t) for 1000g, test 1 **Chart: 8 Log(t) for 1000g, test 1**

For this load, it is obtained 0.1mm of immediate settlement, 0.45mm of primary consolidation and then begging the second consolidation.

In chart of Log (t), in the final part it stars to turn for the secondary consolidation, moment to change the load. The straight line in the log (t) is represented by one step in the Rod(t), maybe someone touched the load. It is not common this type of steps in a consolidation progress.

5.2.1.4 Load 2000g (54.3 $KN/_{m^2}$)

Chart: 9 Rod (t) for 2000g, test 1 **Chart: 10 Log (t) for 2000g, test 1**

In this load, it is observable that the Log (t) has not the common form. The Log (t) does not turn to see easily the secondary consolidation. Immediate settlements is 0.18mm.

5.2.1.5 Load 4000g (106 KN/m^2)

Chart: 11 Rod (t) for 4000g, test 1

Chart: 12 Log (t) for 4000g, test 1

Immediate settlement is 0.28mm. As the last load, the Log (t) does not turn in the secondary consolidation. This problem is because the type of sample, in organic soils the chart of Log (t) not is representative to define the consolidations.

Laboratory from Via

Taking into account all the loads, the next chart represents the relation between Strain-Stress,

Chart: 13 The straight line is the main trend in primary consolidation and the dashed line is the secondary consolidation.

The results obtained from the oedometer trial can be graphed for determining consolidation coefficient, consolidation modulus, void ratio and effective stress of the given soil.

σ' [kN/m2]	εc [%]	εc -εslut [%]	σm [kN/m2]	E [kPa]	ck [m2/s]
2,34	0,00				
8,83	-0,71	0,24	5,59	914,23	6,7E-07
15,33	-1,40	1,13	12,08	939,46	3,1E-06
28,32	-2,08	1,07	21,83	1927,30	9,8E-06
54,31	-3,03	1,24	41,32	2727,91	9,2E-06
106,28	-5,43	1,99	80,29	2162,99	2,1E-06

Table: 14 Results of the method used from VIA

Following the formula (21) explained in the beginning of this chapter, using these values it will be calculated consolidation 100%.

Before starting to calculate the consolidation coefficient, some parameters will be calculated with these values.

The first thing that needs to be calculated is the initial volume of the simple, which can be easily obtained from the initial height (*35 mm*) and diameter (*70 mm*).

$V_t = \pi * R^2 * Ho$ (23)

$V_t = \pi * (\frac{70mm}{2})^2 * 35mm = 134696.1\ mm^3$

The next step is to get the volume of voids, to do it is necessary take into account that the soil tested was fully saturated; hence all the voids were full of water. The water mass can be obtained ($M_t - M_s = 31.54\ g$) and the density of water is known (*1 g/cm*^{3}).

$V_v = \frac{M_w}{\gamma_w}$ (24)

V_v=31.54 (g) / 0.001 (g/mm3) V_v=31540mm^3

To obtain the initial void ratio (e_0), the next formula is used:

$e_o = \frac{V_v}{V_t}$ (25)

e_o =31540mm^3/134696.1 mm^3=0.2342

Another value that it is needed for the calculations is the equivalent height of solid particles (H_s), it is obtained from the equation: H_o=35mm

$H_s = \frac{H_o}{1+e_0}$ (26)

Void ration at the end of a loading stage:

$e = \frac{H-H_s}{H_s}$ (27)

	Before O.T	After O.T
Pv	0,441881505	0,47094
H	0,035	0,03126
Vt	0,000134696	120,3372
Vv	0,00003154	64,16
Hs	0,028359432	0,020998
eo	0,23415729	0,53316

Table: 15 Values calculate before and after of Oedometer Test.

Using the different H for each step and the Hs calculate, the next table is obtained:

σ' [kN/m2]	E
2,33860325	0,2341
8,83472337	0,2225
15,3308435	0,2055
28,3230838	0,1840
54,3075643	0,1568
106,276525	0,1025

Table: 16 Values of the porosity.

Relation between voids and loads is proportional, like it was expected; other loads could be good to check to determinatemoment the *e* is established and decreases very slowly. For example, between the load 2nd and 4th the reduction is 0,05mm with 20KN/m^2 of the differential load and between the 4th and 5th is the same decrease 0.05 but with 52KN/m^2.

It decreases much more slowly until a fixed value which does not decrease further.

Chart: 14 Relation between the loads and the voids.

With more loads, the material is more confined andthere are fewer voids, so the value of the void decreases.

The results used to obtain the time at which 90% of consolidation has occurred for each of the individual loading stages is as follows:

$$Cv = \frac{T_{90} * H^2}{t_{90}} \quad (28)$$

Using the charts for each load, the Consolidation modulus can be obtained from 2 possible methods: Casagrande or Taylor. In organic soils, the secondary consolidation is hard to know when it start because the loads were not left a long times.

Taylor is the best method considering the charts the Log (t) and Rod (t)and Taylor is based in these charts. The method is explained in the chapter 4, Oedometer test.

5.2.2 Taylor application:

The method chosen is Taylor due to the chart deformation- Rod(t) are good to obtain the consolidation time 90%, determining the time later you can know the consolidation coefficient.

Values tabulated for the time: T_{90}= 0.848 T_{100}= 1.129

The method has been explained in chapter 4, oedometer test.

5.2.2.1 Load 250g (8.8 KN/m^2)

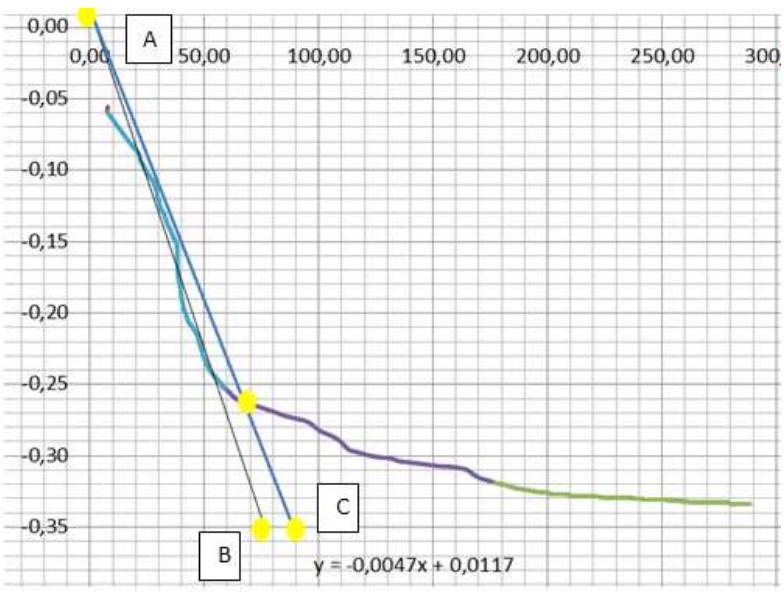


Chart: 15 Load 250g, the point for 90% consolidation is $\sqrt{t_{90}}$=68 s is a settlement for -0.26mm.

The last chart 15, it is one example how calculate the values. The others load are in the *ANNEX 2 Test.*

√t90	y (D90)
68	-0,26
42	-0,261
41	-0,3

56	-0,43
72	-0,8

Table: 17 Results for 90% consolidation

D_{90} is the settlement in the $\sqrt{t_{90}}$. Values obtained in the intersection point.

$$D_{100} = D_s - \frac{10}{9} * (D_s - D_{90}) \quad (29)$$

When the D_{100} is obtained, returning to the tables it is possible calculate $\sqrt{t100}$

D100	√t100
-0,277189	108
-0,28	52
-0,320978	55
-0,452222	64
-0,838	85

Table:18 Results for 100% consolidation

In the table 18, there is the relation between rod square of the time and the deformation in the 100% of consolidation obtained following the formula number 29.

The last values are when the consolidation is 100%. Now, it is possible to know if the load was left a lot of time or not.

εslut	**D100**
0,33	-0,277
0,481	-0,280
0,61	-0,320
0,77	-0,452
1,54	-0,838

Table: 19 Last value from the oedometer test and the completely consolidation.

In the table 19, there is the list of the deformation in the last moment that the process was left and the deformation in the 100%.

$\varepsilon slut > |D_{100}|$ this relation mains that the load were left running a lot of time. The first load was enough, it was stopped after to arrive 100% consolidation. The last load was left a lot of time more than the necessary to arrive 100% consolidation.

Following the next formulas:

$\varepsilon_c = D_{100} + \varepsilon_{c-1}$ (30) H_o=35mm, ε_c - εslut [%]= $\frac{D100-(\text{Last value})}{Ho} * 100$ (31)

The next table (20) is obtained from the values of each deformation for each load:

εc [%]	εc - εslut [%]
0	
-0,277188889	0,150888889
-0,557188889	0,574285714
-0,878166667	0,825777778
-1,330388889	0,907936508
-2,168388889	2,005714286

Table: 20 ε_c is considering the D_{100} and the other considering ε_c and the last value of oedometer test.

The next chart (16) is obtained representing the last values,

Chart: 16 The new relation between Strain- Efective Stress with the news values from Taylor.

The chart 16 represents the relation between strain and stress, it is possible to confirm that the progress not is finished; it would have been good put minimum 2 loads more. But the limitation of the time was an impediment

Relation is correct, when the load increase the deformation increase too, the time is very important because the deformation increase with the time with the same load, but increase faster with more load.

εc [%]	D100
-0,2771	-0,2771
-0,5571	-0,28
-0,8781	-0,3209
-1,3303	-0,4522
-2,1683	-0,838

In the last chart, it is possible check with the last loads influenced by the time and the bigger loads the deformations are bigger.

Table: 21 Relation of the deformations, considering D_{100},

Chart: 17 Consolidation coefficient depends of each stress average.

Consolidation coefficient depends of the t_c, it is not an exactly parameter because depends of the method that you use to calculate the consolidation and the time that the load was left.

The time to arrive at 100% consolidation each time is less, because it needs less time insomuch as in the last load it consolidated a little bit more.

Chart: 18 Relation between consolidation modulus with average stress. The K is oedometer modulus E in the tables.

Consolidation modulus changes depending on the loading stages; this is useful when the expected stress changes are due to a particular construction (Knappett and Craig 2012). The values obtained are those expected for a normally consolidated ORGANIC SOIL. An easy value to try to understand if the results are consistent is to look at the void ratio, as it is expected to decrease as the loading process continues. The values of C_v expected to organic soil of high plasticity (Head 1998).

σ' [kN/m2]	εc [%]	εc - εslut	σm [kN/m2]	E	Hlab^2	C_v
2,33860325	0					
8,83472337	-0,277	0,151	5,586	2343,57	0,0012	1,163E-07
15,3308435	-0,557	0,574	12,083	2320,04	0,0012	4,880E-07
28,3230838	-0,878	0,826	21,827	4047,71	0,0011	4,208E-07
54,3075643	-1,330	0,908	41,315	5745,95	0,0011	2,967E-07
106,276525	-2,168	2,006	80,292	6201,55	0,001	1,527E-07

Table: 22 Recap of values applying method Taylor.

In the last table 22, there are represented all the values after apply method Taylor in each loads.

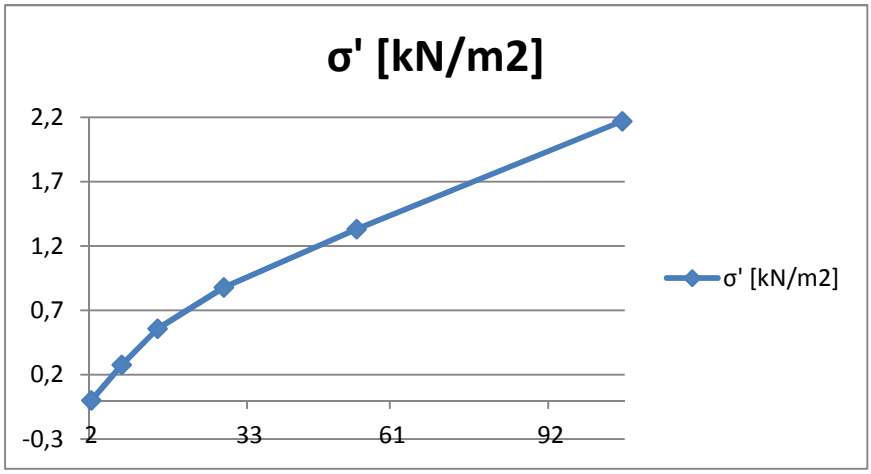


Chart: 19 Relation between total deformation and the stress.

The deformation will be established when it arrives a determinate value. The values are:

σ' [kN/m2]	εc [%]
2,33860325	0
8,83472337	0,277189
15,3308435	0,557189
28,3230838	0,878167
54,3075643	1,330389
106,276525	2,168389

Table: 3 Values of the last chart 19.

It is represented a curve that it would need more loads to understand the purpose, the main significances is that in a determinate moment you continue put loads but the deformation does not increase more, it is stable . But it is not clear in the chart 19 because needs more points to see the horizontal line.

The results of the stress-strain graphical representation, as seen in chart 19, show sample follow the trend of the theoretical curve. This shows that the soils are acting in typical soil behavior and therefore the test has been successful in a graphical representation. Sample represents linear behavior between stress and strain values. This showsthat the stress and strain values are proportional but the values in the final part would be straight line only when the secondary consolidation finishes, and it is very difficult because the organic soil has big secondary consolidation.

5.3 Aabenraa, test 2:

To contrast the results in the test 1, it isdecided to take the oedometer test carried out for another group, which was realized with a soil from the same place and an

oedometer apparatus with the same characteristics. The ring with a 70mm of diameter, 35mm of height and a drain only in the bottom of the ring.

At the following table 24 is showing the water content and the density of the sample before and after of the oedometer test.

	BEFORE TRIAL	AFTER TRIAL
Water content (%)	163.12	76.38
Specific gravity (KN/m^2)	12.44	28.60

Table: 24 Soil characteristics.

The applied loads are shown in the table 25.

Stress[kN/m^2]	weight[g]
2.3	0
8.8	250
15.3	500
28.3	1000
54.3	2000
106	4000
236	9000
496	19000
756	29000

Table: 25 Load steps

The test started on 20th of March and it finished on 9th of May of 2013, therefore, the test took one month and 20 days.

Below it could see the graphics obtained in the oedometer test; of each applied load it could see two graphics, in square root and logarithmic scale in the time (t) axe.

5.3.1 LOAD 250g (8.8$^{KN}/_{m^2}$):

Chart: 20 Rod (t) for 250g, test 2

Chart: 21 log (t) for 250g, test 2

5.3.2 LOAD 500g (15.3 $KN/_{m^2}$):

Chart: 22 Rod (t) for 500g, test 2

Chart: 23 log (t) for 500g, test 2

5.3.3 LOAD 1000g (28.3 $KN/_{m^2}$):

Chart: 24 Rod (t) for 1000g, test 2

Chart: 25 log (t) for 1000g, test 2

5.3.4 LOAD 2000g (54.3 $KN/_{m^2}$):

Chart: 26 Rod (t) for 2000g, test 2

Chart: 27 log (t) for 2000g, test 2

5.3.5 LOAD 4000g (106 $^{KN}/_{m^2}$):

Chart: 28 Rod (t) for 4000g, test 2

Chart: 29 log (t) for 4000g, test 2

5.3.6 LOAD 9000g (236 $^{KN}/_{m^2}$):

Chart: 30 Rod (t) for 9000g, test 2

Chart: 31 log (t) for 9000g, test 2

5.3.7 LOAD 19000g (496 $^{KN}/_{m^2}$):

Chart: 32 Rod (t) for 19000g, test 2

Chart: 33 log (t) for 19000g, test 2

5.3.8 LOAD 29000g (756 $^{KN}/_{m^2}$):

Chart: 34 Rod (t) for 29000g, test 2 **Chart: 35 log (t) for 29000g, test 2**

From this graphics it is possible to obtain the Consolidation modulus (Cv) using the Casagrande or Taylor theory for the logarithmic and square root scale respectively, as explained in chapter 4, Oedometer test.

As it can appreciate, the logarithmic graphs do not show clearly where the consolidation passes from the primary to the secondary consolidation. This is due the load was not left enough time. For this reason it decided to obtain the Consolidation coefficient with the Taylor theory, when it hasbeenproduced the 90% of consolidation, and the dimensionless time (T_{90}) it is equal to 0.848.

The chart 36 shows the graphic of the load 19000g with the abscissa axis in square root ($\sqrt{t}$) scale, in which it can be seen the process to obtain the Cv with the Taylor theory. First it is necessary draw a straight line which has to pass for the straight zone before the curve and then it has to be extended to the Y axe. This line has a grade which is called alpha (α), and it must draw another line with a slope 1.15 smaller than alpha, and it is called beta (β), therefore, β=1.15α.

The point in which the beta line crosses the graph of settlements, it is the point in which is produced the 90% of consolidation. In this graph, the line crosses in the time 325.96 $\sqrt{s}$ and with a deformation of 3.27mm, so it can obtain the Cv with the next equation:

$$C_v = \frac{T_{90} \cdot H_b^2}{t_{c_{90}}} \qquad (32)$$

Where:

$$C_v = Consolidation\ coefficient$$
$$T_{90} = 0.848, dimensionless\ time\ in\ t_{c_{90}}$$

H_b= *The whole sample height for this step load, as the ring used in the test only has drain at the bottom.*

$$t_{c_{90}} = time\ in\ which\ is\ produced\ the\ 90\%\ of\ consolidation$$

For the case of the load 19000g:

$$T_{90} = 0.848$$

H_b= 0.0219 mm

$$t_{c_{90}} = 325.96\sqrt{s} = 106249.92\ s$$

$$C_v = \frac{T_{90} \cdot H_b^2}{t_{c_{90}}} = 3.96\ EXP - 9\ {}^{m^2}/_{s}$$

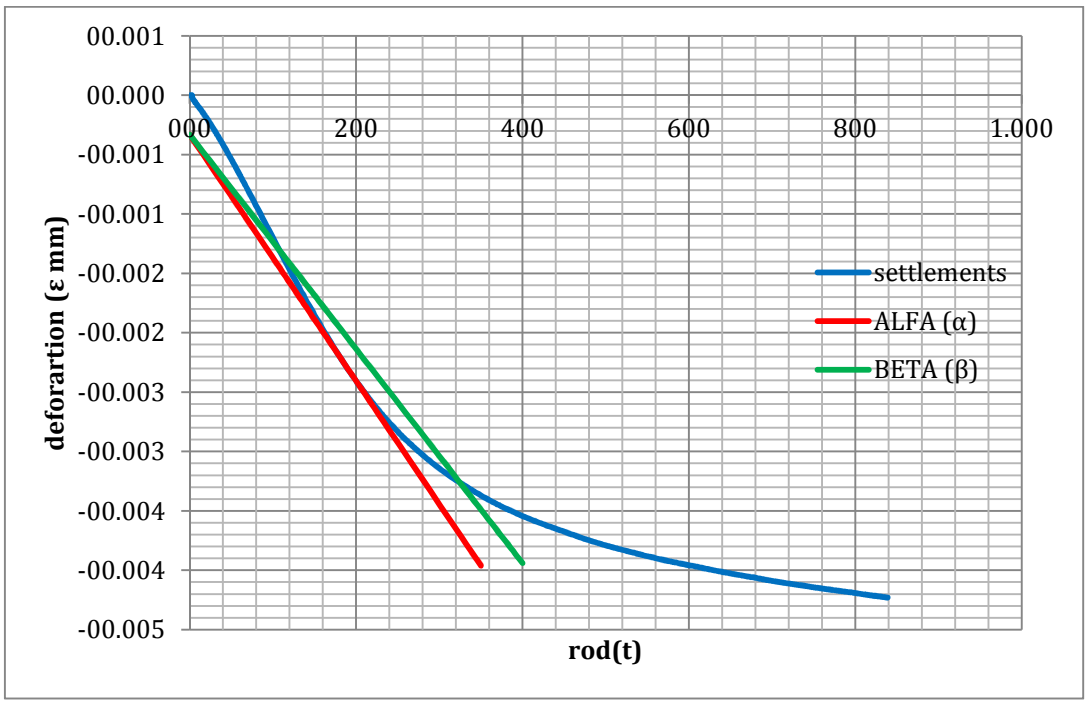


Chart: 36 Relation deformation-$\sqrt{t}$ in the case of 19000g case, Taylor's method.

Having obtained the Consolidation coefficient, it is possible to find the point in which the 100% of consolidation is produced in the same graphic. However, in this case, it decided to locate the point from another way to be sure that the obtained results have coherence.

From the Cv calculated above, it can obtained the time in which is produced the 50% of consolidation, therefore, looking in the logarithmic graph, it can obtain the deformation produced in the 50% of consolidation. Inversely to the logarithmic theory, it obtains the deformation of the 100% of consolidation. This deformation is the double of the 50% of consolidation. The chart 37 shows the process.

Besides, it shows the following steps to reach the results:

1. *<u>Obtaining of tc50:</u>*

$$C_v = \frac{T_{50} \cdot H_b^2}{t_{c_{50}}} \qquad (32)$$

Where:

$C_v = 3.96\ EXP - 9\ {}^{m^2}/_{s}$ (Obtained with Taylor theory from $t_{c_{90}}$

$$T_{50} = 0.197$$

H_b= 0.0219 mm

$$t_{c_{50}} = \frac{T_{50} \cdot H_b^2}{C_v} = 2.468\ EXP + 4\ s = 4.39 \log(t)$$

2. *Location of tc50 in the logarithmic graph:*

To locate $t_{c_{50}}$ in the graph, it has to take in count that this graph is represented in logarithmic scale, so it is necessary pass the $t_{c_{50}}$ in log(t). Having located this point, the ε_{50} is known.

$$\varepsilon_{50} = -1.923$$

3. *Obtaining ε_{100}:*

Inversely to the logarithmic method, in which it obtains ε_{50} from ε_{100}, knowing the deformation of 50% of consolidation it can be obtainedthe deformation in which the primary consolidation passes to be the secondary consolidation (ε_{100}).

$$\varepsilon_{100} = (\varepsilon_{50} - \varepsilon_0) \cdot 2 + \varepsilon_0 \qquad (33)$$

ε_0 is considered the point in which the (β) crosses the Y axis in time 0, therefore:

$$\varepsilon_0 = -0.332$$
$$\varepsilon_{100} = -3.514$$

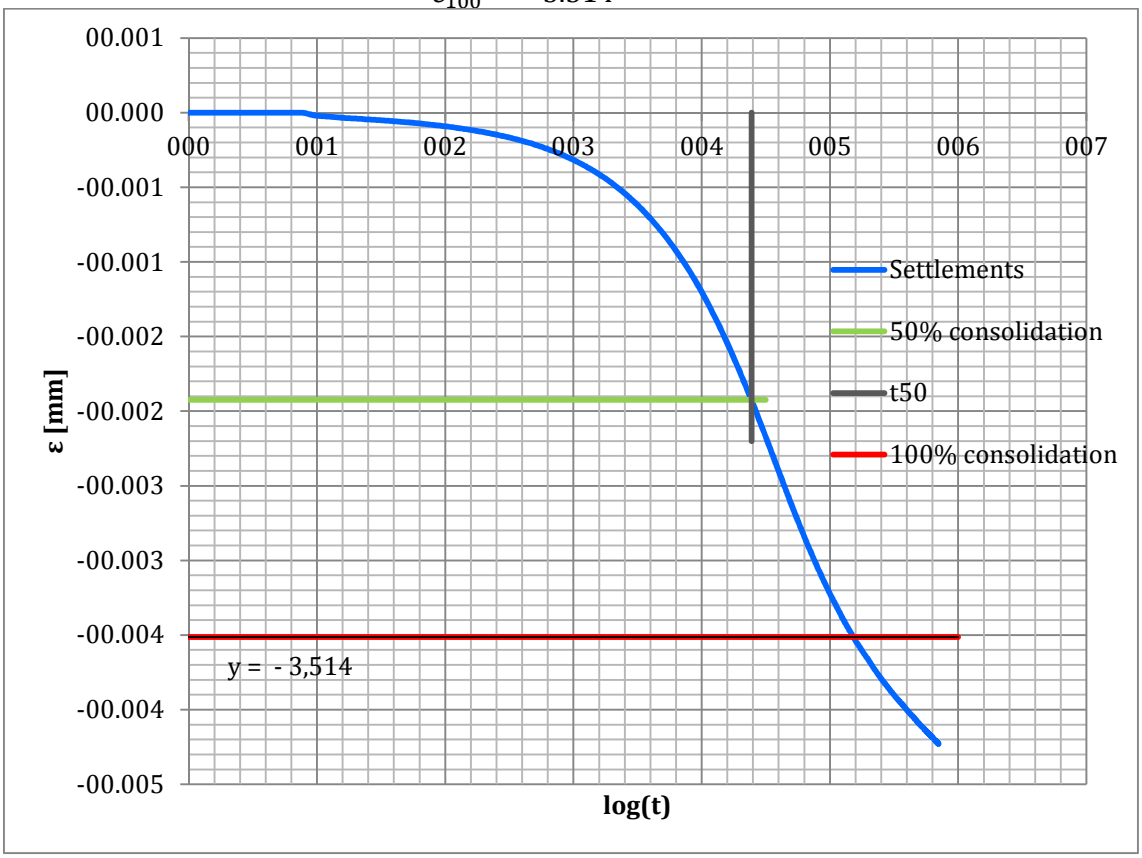


Chart: 37 Relation deformation-log(t) in the case of 19000g case, Casagrande's method.

The necessary data from oedometer test for the different load cases it can see them in the *ANNEX 3,Test.*

Below, in the table 26 are shown the obtained results for all the load cases:

Kg	σ' [kN/m2]	Heightof sample(mm)	εc [90%]	tc(90%)	Cv	tc(50%)	log(t)	εc[50%]	εc [100%]	εc [%]	εc - εslut [%]
250	8.83	34.991	0.0088	18.00	3.2046E-06	75.27	1.88	-0.0113	-0.0369	-0.105	-0.034
500	15.33	34.945	0.0387	94.00	1.172E-07	2052.70	3.31	-0.0275	-0.0390	-0.217	0.014
1000	28.32	34.545	0.3865	150.13	4.4897E-08	5236.06	3.72	-0.2642	-0.4358	-1.479	0.085
2000	54.31	33.823	0.5871	123.97	6.3125E-08	3570.16	3.55	-0.3357	-0.6482	-3.395	0.395
4000	106.28	31.956	1.4112	156.03	3.557E-08	5655.62	3.75	-0.7954	-1.5191	-8.149	1.248
9000	236.00	26.685	3.9153	283.11	7.5339E-09	18619.52	4.27	-2.2733	-4.3820	-24.570	0.957
19000	496.04	21.989	3.2717	325.96	3.859E-09	24682.85	4.39	-1.9233	-3.5142	-40.552	0.716
29000	755.89	20.896	0.1349	266.73	5.2044E-09	16528.02	4.22	-0.1278	-0.1503	-41.271	0.010

Table: 26 Obtained result in test 2.

Having obtained the data it proceeds to make the following graphics to get the Oedometer modulus and study in detail the results.

The chart 38 shows the relation between effective stresses (σ') – deformation (ε), it can be observed that increasing the vertical stress, the sample it becomingstiffer. The confinement stress increase with the vertical effective stress, therefore its behavior is stiffer. This is possible to see clearly in the last stretch of the graph, where it has produced a small deformation due to excessive confinement of the sample.

The dashed line represents the deformation produced after the 100% of primary consolidation. As it can be observed, there is not so much deformation, because the test was not left a longtime, so it is difficult to see the problematic in the secondary consolidation that the organic soil has.

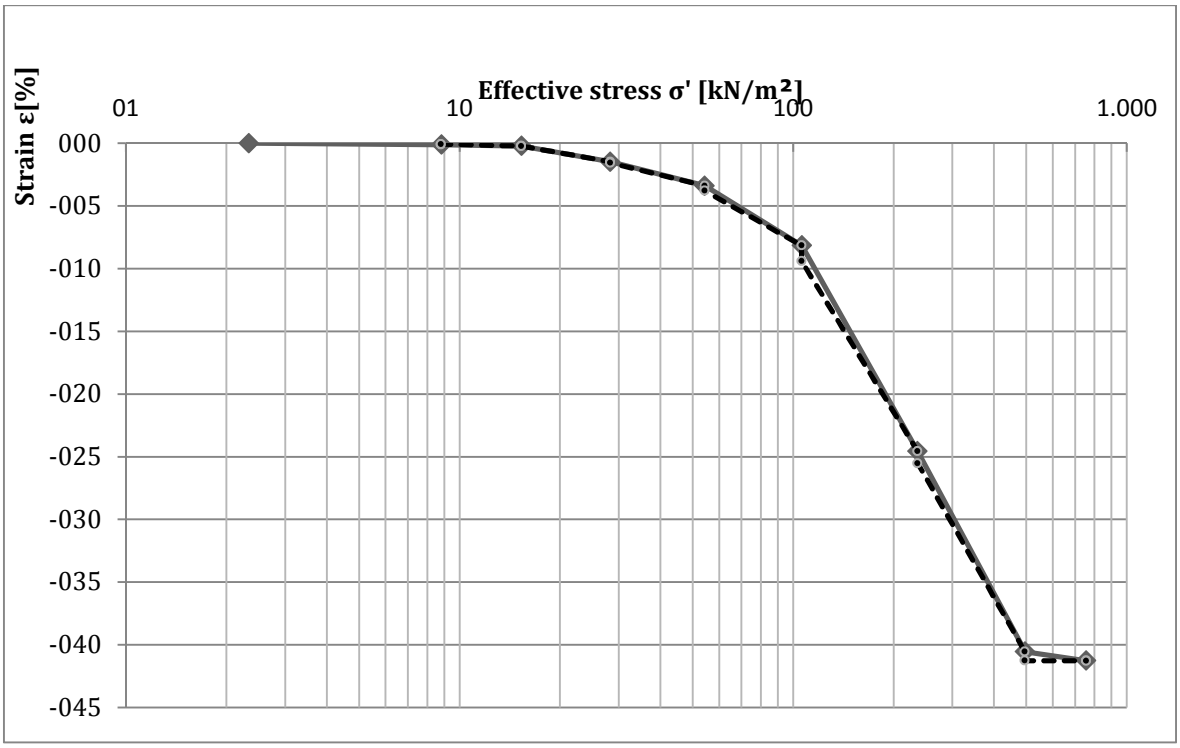


Chart: 38 Relation strain-log(σ') in test 2.

In the char 38, it is possible to see in the last step the consolidation is establishing, (estabiliza) and it is not decrease more.

Such relation between the effective load increase and the reached deformation in each load step allows knowing the soil compressibility through the Oedometer modulus (E_m) or in logarithmic scale with the compression index (Q). The chart 39 allows seeing the relation between vertical effective stress (σ') - Oedometer modulus (E_m).

$$E_m = 1129\ KPa \qquad\qquad Q = 51.6\%$$

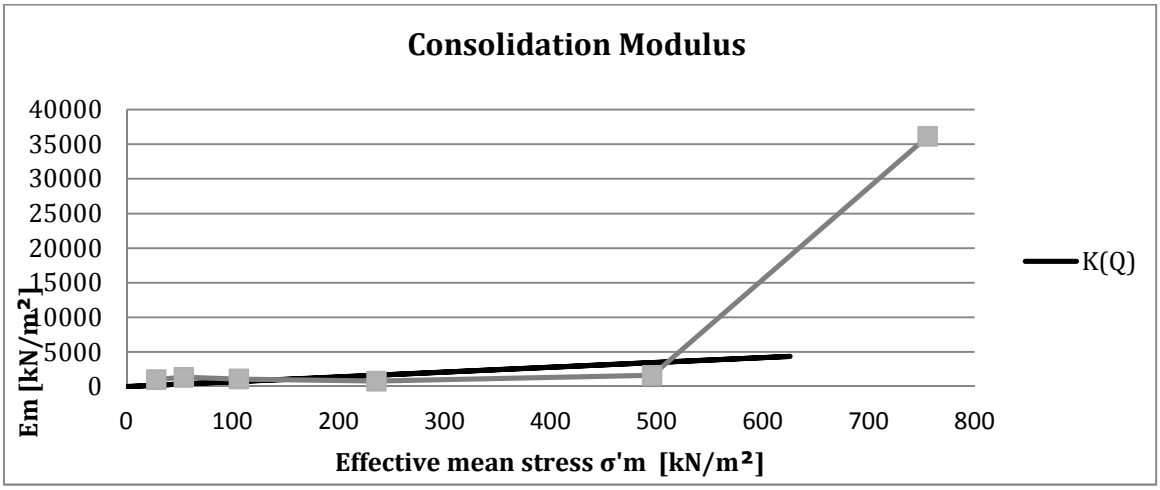


Chart: 39 Relation Em-σ' in test 2

The interstitial pressures speed dissipation or the settlements velocity can be estimate by the consolidation coefficient, which has been calculated previously by

Taylor's method. This coefficient has a variation depending on each load case and besides, it tends to vary between samples of the same soil. Therefore, the consolidation coefficient (C_v) is used to estimate the necessary time, of an approximate form, to reach the total consolidation. Nevertheless, it is good to know that the real consolidation is usually faster than the consolidation estimated from the C_v obtained in the laboratory.

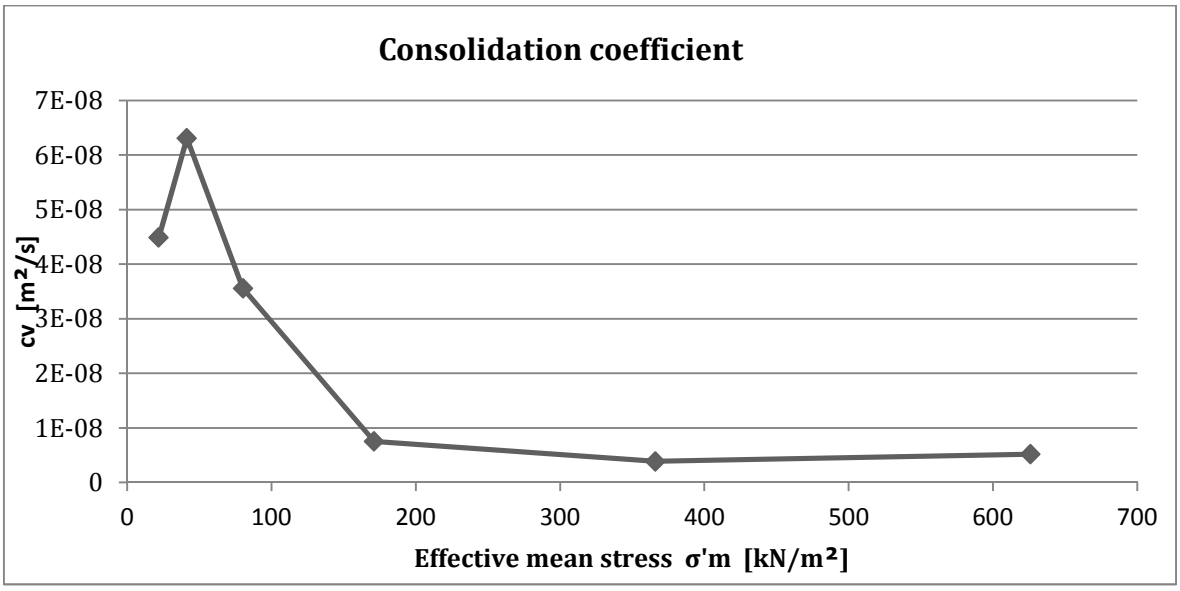


Chart: 40 Relation Cv-σ' in test 2

In the table 27 can be seen the data of the charts and the Oedometer modulus in each load case:

σ' [kN/m2]	εc [%]	εc - εslut [%]	σm [kN/m2]	E [kPa]	cv [m2/s]
2.3	0.00				
8.8	-0.11	-0.034	5.6	6161.2	3.2E-06
15.3	-0.22	0.014	12.1	5820.9	1.2E-07
28.3	-1.48	0.085	21.8	1029.9	4.5E-08
54.3	-3.39	0.395	41.3	1355.9	6.3E-08
106	-8.15	1.248	80.3	1093.2	3.6E-08
236	-24.57	0.957	171.1	790.0	7.5E-09
496	-40.55	0.716	366.0	1627.1	3.9E-09
756	-41.27	0.010	626.0	36125.3	5.2E-09

Table: 27 Test 2 data.

5.4 Hvide Sande:

The properties from Hvide Sande are taken from the report, in the next table 28, the parameters of the gyttja and sand which will be used to calculate the settlements in the next chapter.

	ϕ	γ/´Y	C'	Q	K
SAND	34-36º	17/8 kN/m^3			
Gytje	25º	14/4 kN/m^3	0 N/m^2	25%	1500kN/m^2

Table: 28 Parameters from the sample of Hvide Sande.

Density of soil	**γ**	**14 KN/m2**
Liquid limit	w_l	103%
Plastic limit	W p	54%
Plasticity index	I_p	49%
Undrained shear strength	τ	9 Kpa
Initial void ratio	e_o	2.8
Compression index	C_c	0.8
Recompresion index	C_r	0.08
Swelling index	C_s	0.05
Coeficient of consolidation	C_v	2*10^-8
Secondary compression	C_α	0.044
After surcharging	C^s_α	0.010
Undrained modulus*	E_v	944

Table: 29 Parameters from Hvide Sande, taken from the report of Franklin Geoteknik.

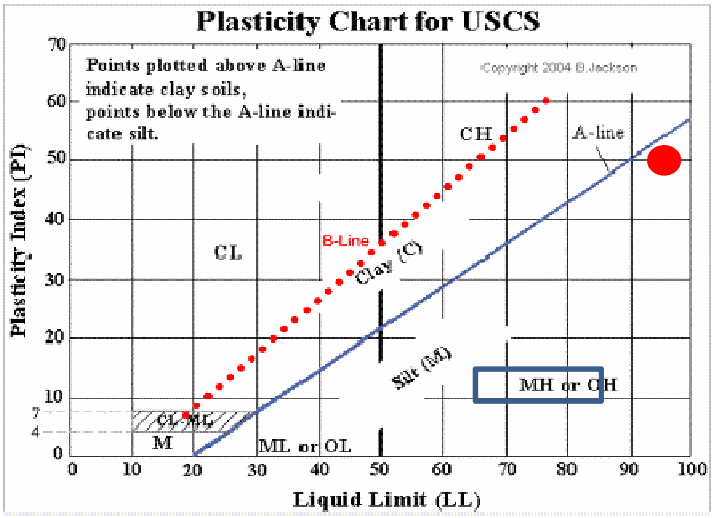


Fig. 24 Chart with the classification of the differents types of materials depen of the liquid limit and plasticity index.

Note: this figure is used in Spain, not in Denmark. For USCS.

The type of the sample, according the plastic index and liquid limit, it is OH= Organic High because is very plastic and the liquid limit is upper 100%.

The compression index from Hvide Sande, obtained from the oedometer test, it is Cc=0.8, so it is possible obtain the secondary compression following the next formula:

$C\alpha$=0.05Cc (34)

$C\alpha$ = 0.05*0.8 = 0.04

In organic soils, the secondary compression has be $C\alpha > 0.03$, in Hvide Sande the $C\alpha$ =0.044 is obtained by other method and it is a correct value, inside in the ranking admissible.

Having all the parameters is possible calculate the consolidation coefficient is $9 \cdot 10^{-8}$.

In general, for a gyttja's sample the parameters are in a ranking of possible values,

	σs	γm	w	w_l	w_p	I_p	Q
gyttja	1.3-2.5	10-19	50-500	70-350	30-170	40-180	15-50

Table: 4 Values from general gyttja from the book: Teknis, Nyt Teknisk

All the parameters are in the possible ranking except the Q in the case Aabenraa 2, the Q is a little bit big and the percent of water in Aabenraa 1 is a little bit small because w=51% it means the quantity of water is low.

6. Results comparison from 2 oedometer tests:

After having obtained the results, it is realized that both tests had a large variety between themselves; therefore it has been studied and argue such differences.

First of all, it has to be pointed the initial conditions obtained before running the test, as it can be significantly in their behaviours. As it can be seen in Table 1, the sample from test 2 has higher water content and lower specific gravity than sample from test 1. This can be significant forthe soil compressibility, it means more water content more porosity (saturated soil) and therefore less contact between soil particles.

	Aabenraa 1		Aabenraa 2	
	before	**after**	**before**	**after**
w (%)	51,74	48,91	163,12	76,38
ϒ (KN/m2)	15,3	18,2	12,44	28,6

Table: 31 Comparison between the two samples of Aabenraa.

Looking the obtained results in the oedometer test it can be observed that both tests give results absolutely different. To compare the results it is going to take as a benchmark the charts which relate deformations with effective stresses and the results obtained in the oedometer test.

Chart: 41 Oedometer test 1

Chart: 42 Oedometer test 2

	TEST 1			TEST 2		
σ' [kN/m2]	**E (KPa)**	**Cv (m^2/s)**	**εc - εslut [%]**	**E (kPa)**	**cv (m^2/s)**	**εc - εslut [%]**
8.83	2343.57	1.2E-07	-0.330	10066.74	3.2E-06	-0.020
15.33	2320.04	4.9-07	-0.481	4122.90	1.2E-07	-0.003
28.32	4047.71	4.2E-07	-0.610	849.25	4.5E-08	-0.007
54.31	5745.95	3.0E-07	-0.770	1309.13	6.3E-08	0.372
106.28	6201.55	1.5E-07	-1.540	1043.98	3.6E-08	1.176
236.00	-	-	-	761.37	7.5E-09	0.793
496.04	-	-	-	1486.53	3.9E-09	0.383
755.89	-	-	-	21243.79	5.2E-09	-0.115

Table: 32 Characteristics

In the test 1 has obtained values of oedometer modulus higher than the test 2, consequently the consolidation coefficient is also higher as they are related directly. The Q value it is also related with oedometer modulus, but indirectly, so the Q value in the test 1 is lower than in test 2.

	TEST 1	TEST 2
Q (%)	2.90	51.60

Table: 33 Compression index

From this data, it can be said that the sample 1 is stiffer than 2, thereby it will produce less deformations and in a shorter time than the soil from the sample 2. Although the sample 1 was only loaded until the step 5 (4000g), if it is compared the values in the same stretch, the obtained results do not suffer any change, it means, the sample 2 has lower stiffnes than sample 1. This could be consequently from the soil confinement, where the sample 2 is less confined than sample 1, therefore it has higher compressibility.

It is focused in the produced deformation in each load case, it can be observed in table 34 that in sample 1 the test has higher deformation than sample 2, and then the sample one begin to decrease its settlements. This fact could be due to some organic matter pieces or any characteristic of the soil, but it can be affirmed at any concrete theory as it have not enough data to make a good comparison.

εc [%] 1	εc [%] 2
-0.28	-0.11
-0.56	-0.22
-0.88	-1.48
-1.33	-3.39
-2.17	-8.15

Table: 34 Deformations

Referent to the creep, which is shown by a dashed line, it is can be seen that in sample 2 they have not been produced large settlements, this does not means the soil does not have a secondary consolidation, this is due to the test was not left the enough time to apreciate this phenomenom. In the test 1 it can be observed that consolidation of the settlements from the secondary consolidation has an important effect and it can apreciate in the charts from the test that they did not stabilize, it means, if the test had been left longer time the secondary settlements would have been bigger.

As it is said in chapter 2, organic soils, it is possible to obtain results absolutely differents, as the organic soil has different properties depending on the amount of organic matter, the difference between the historic load that the soil has borne or the results of the degradation to make organic provoke different properties, therefore, it is not uncommon obtaining such differences.

Due the sample 1 is having less reliable values, the test 2 has been chosen to calculate the settlements.

At the sample 2 have been obtained results about Oedometer modulus quite high, nevertheless, the results are reasonable if it looks the initial charracteristics of the soil, where the soil has a high water content and a low specific gravity, therefore it is deductible that the soil has high compressibility and it will produce large settlements.

PART III CALCULATIONS

7. SETTLEMENTS CALCULATIONS:

This chapter consists in the calculations of the settlements provoked by the construction of the fish farm, comparing a sample from Aabenraa and the parameters obtained from report of Hvide Sande.

As the second test of Aabenraa has more information and it is more accurate, it will be chosen to obtain the settlements in Aabenraa.

There are different types of settlements which are explained in the chapter 3, consolidation;

$\delta total = \delta immediate + \delta consolidation + \delta creep$ (35)

In organic soil, chapter 2, it has been commented it is very difficult to study the creep in the chart Log (t), because the curve in the secondary consolidation sometimes not is clear.

In the case of Aabenraa 1, only in the load 250g the Log (t) has the correct shape, it is shown in the chart (44), with the 3 parts of the settlements: immediate, primary and secondary.

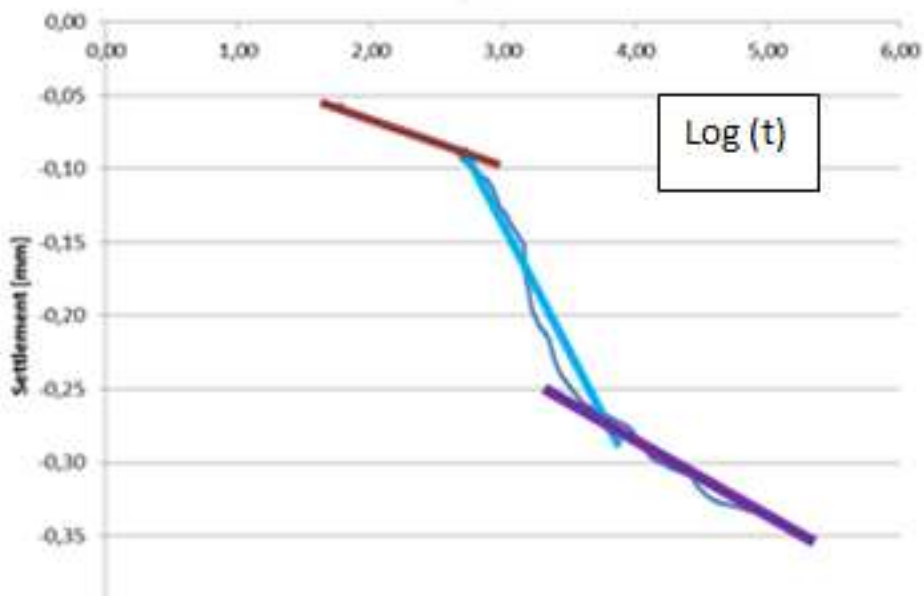


Chart: 43 Example from Aabenraa 1, Load 250g.

For this reason, it was chosen Taylor's method and all the references were taken from the Rod (t) chart.

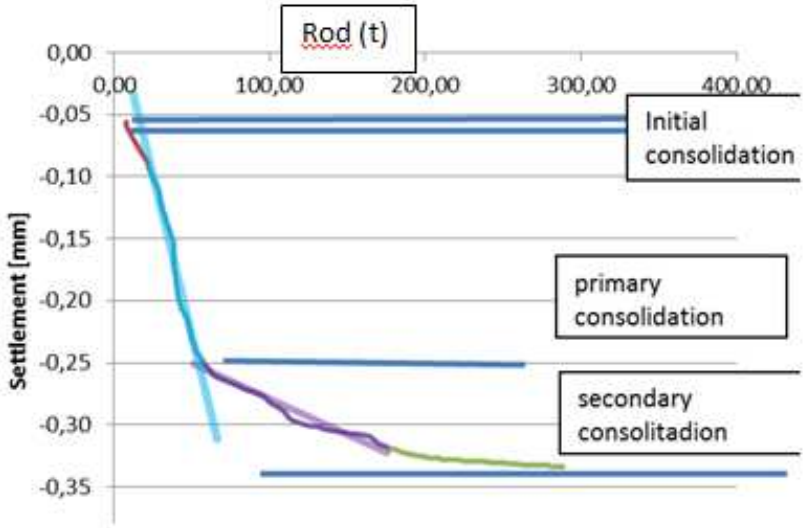


Chart: 44. Vertical displacement for the first stage of the consolidation (8.8 kN/m2).

To calculate the settlements it is necessary consider the unfavorable zone for that purpose it is decided to focus in the boring number 9, because there are 3 layers of gyttja and it is one of the drills made by Franck Geotekniks have got more information and more depth.

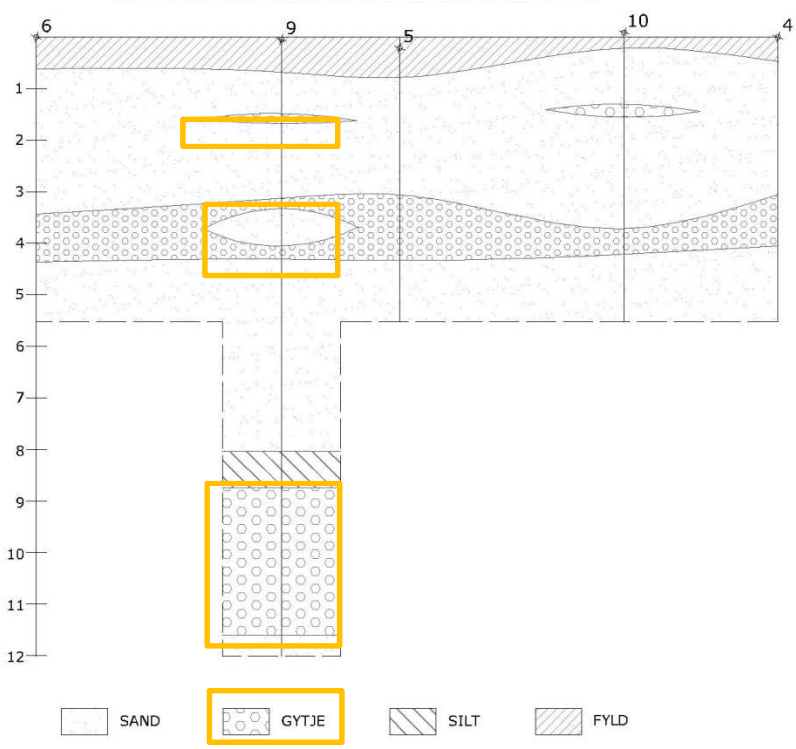


Fig. 25 Profile with the borings in the line 1, in the North.

All the borings are from the *ANNEX 4, Report from Franck Geoteknick Company*, and the profiles made by Autocad in the *ANNEX 2, Geology.*

The calculations of settlements will be realized using the specific gravity, the thickness of each layer and the Q in each case. Following the formulation explained in the chapter 3, consolidation.

7.1 Design of pool:

For starting the calculations at the settlement it is necessaryto define the pool, the construction of indoor fish farm will be with 6m deep basins and the button of the basins is expected to be established in the level +0.0. The meters of the figure 26 are estimated.

The number 9 has surface in +0.80, it is necessary removed 0.7m of fill and 0.1 m of sand and Ground Water will be assumed placed in level +0,0 before to make the pool and the button of the basins in calculation of settlements.

Fig. 26 Measures of the Fish Farm

The boring number 9 is in the line NW, in the middle of the fish Farm.

7.2 Settlements calculations:

It is important to be able to predict the settlements occurring by constructional building. In this way, it can be ensured that the settlements will not damage the construction.

The task requires to compute the immediate and the consolidation settlements for the studied pile foundation and to check if the total amount is admissible or not.
The total settlement of a foundation can be divided in three components, in organic soils the largest proportion of it will take place over a number of years (consolidation), while in sands is expected to happen faster (immediate).

The weight decided for the pool is 60 KN/m^2, approximately, 6m of depth of water and 10 KN/m^3 neglecting concrete, or it is taken the same 60 KN/m^2 but considering 5.5 m of water and 0.2m of concrete the weight of concrete is 25 KN/m^3.

7.2.1 Aabenraa Case.

First calculation is about Aabenraasample, the C_v chosen is the lower because the final load 60 KN/m^2 is between 54.31 KN/m^{2}and 106.28 KN/m^2 so it is selected the bigger load because it is the most unfavorable solution. The result will be excessive, but it is preferable consider the worse situation to be in the safety conditions.

σ' [kN/m^2]	ε_c [%]	ε_c - ε_{slut} [%]	σ_m [kN/m^2]	E [kPa]	c_v [m^2/s]
8.83	-0.06	-0.02	5.6	10066.7	3.2E-06
15.33	-0.22	0.00	12.1	4122.9	1.2E-07
28.32	-1.75	-0.01	21.8	849.3	4.5E-08
54.31	-3.74	0.37	41.3	1309.1	6.3E-08
106.28	-8.71	1.18	80.3	1044.0	3.6E-08
236.00	-25.75	0.79	171.1	761.4	7.5E-09
496.04	-43.25	0.38	366.0	1486.5	3.9E-09
755.89	-44.47	-0.12	626.0	21243.8	5.2E-09

Table: 35 Values of case 2 of Aabenraa.

Case 2 of Aabenraa is used to make all the calculations about the settlements, and the parameters used are the next:

C_v= 3.6*10^{-8} , γ_w=10 KN/m^2, Q=51.6%, σ=60KN/m^2,δ = Q*Z*log $(1+\frac{\Delta\sigma}{\sigma})$ (36)

In the *ANNEX 2, Geology*, there is the profile with the deepths of boring 9.

before
0,7
0,8
0,2
1,5
1,2
3,69
0,71
2,7
0,41

after
60
0
0,7
0,2
1,5
1,2
3,69
0,71
2,7
0,41

sand	silt	gyttja
17	19	12,44

Fig. 27 Sequence of materials before to build the pool and after. Table: 5 weights of each material in KN/m^3.

The color classification of each material is in the last table (36) and the blue grids are the pool.

As the ground water table is in ground surface, the organic soil will be considered saturated for the settlement calculations.

The consolidation settlement will be calculated using a one-dimensional method based on the oedometer test. The assumptions of this method are: no lateral strain and

initial excess pore pressure is equal to the increase in total vertical stress. The process followed consists on dividing the deposit in layers and calculate the settlement in each one, to end adding all the contributions together to obtain the overall value. To determine the settlement at the center of the layer it has to be divided in three. Table 37 shows the calculations for each layer. The meaning of each column and how is calculated is as follows:

- **Layer**, number of the layer considered, starting at the top of the deposit, the Numeration.
- **Thickness** (H) of each layer.
- **Depth (d)**, value in meters of the depth of the middle point of the layer, with the origin being at ground surface.
- **Depth (z)**, value in meters of the depth of the middle point of the layer, with the origin being at the foundation level.
- **σvo,** effective vertical stress at a point in the middle of the layer considered.
- **σvo** = d·γs sigma 0 is the initial stress. (37)
- **σ'v**, sigma c, increment of the effective vertical stress at the center of each layer. Stress after of the built.
- **δ**, consolidation settlement for each layer. $\delta = Q*H*\log(\frac{\sigma'v,}{\sigma vo,})$(38)
- **ε_z**, deformation corresponding to each sublayer. $\varepsilon_z = \delta/H$ (39)

Q= 51.6% obtained from the oedometer test.

Layer	Thickness	Depth (m) d	Depth (m) z	sigma 0	sigma c	$\varepsilon_{z(\%)}$	δ(m)
1	0,2	1,6	0,8	10,744	57,144	37,4515193	0,07
2	1,2	3,24	2,44	22,952	69,352	24,7803043	0,30
3	2,86	10,15	9,35	59,93	106,33	12,8489953	0,37
3,1	0,9	9,25	8,45	57,856	104,256	13,1968369	0,12
3,2	0,9	10,15	9,35	60,296	106,696	12,7895572	0,12
3,3	0,9	11,05	10,25	61,394	107,794	79,9639424	0,11
							0,740
							0,720

Table: 37 Results of the settlements for each layer

More precision is expected if the soil is divided in more layers. In the Table 37 there are the option considering layer 3 as one layer the total settlement is 0.74mm and it has been repeated dividing the layer 3 in 3 layers, considering 3.1, 3.2 and 3.3, in total 5 layers, and the consolidation settlement has been found to be slightly different 72 cm (2mm more with 3 layers). But it is a small difference for a 10.15 m soil deposit.

The main conclusions are the values obtained for the different parameters are consistent for high plasticity organic soil that is normally consolidated. This test allows easy to estimate settlement and consolidation calculations for foundations and other loading structures.

cv	T95	H	t (year)
0,000000036	1,127	1,705	2,9258

Fig. 28 Values to calculate the time in the middle of bigger layer of gyttja in the 95% of consolidation.

$t_{(s)} = \frac{T_{95} * H^2}{C_v}$ (40) T_{95}=1.127 it is a tabulated value

To make the calculations about the time it is considered the bigger layer of gyttja which is 2.71 m and the layer of silt that it 0.7m, the total thickness is 3.41m, it is divided in two H = 1.705m because it is calculated in the middle of layer.

The layer will reach to the 95% consolidation in almost 3 years after build the fish farm.

7.2.2Case 2, Hvide Sande:

Hvide sample is used to make all the calculations about the settlements, and the parameters used are the next:

$C_v = 9 \cdot 10^{-8}$ $Q = 25\% \sigma = 60 KN/m^2$ $\gamma_w = 10\ KN/m^2$

sand KN/m3	silt KN/m3	gyttja KN/m3
17	19	14,00

Table: 38 Values of the weight of each material, the color clasification is the same like in the case Aabenraa

Layer	Thickness (m)	Depth (m) d	Depth (m) z	sigma 0 (KN/m2)	sigma c (KN/m2)	$\varepsilon_{z(\%)}$	δ(m)
1	0,2	1,6	0,8	10,9	57,3	18,018	0,04
2	1,2	3,24	2,44	24,2	70,6	11,624	0,14
3	2,86	10,15	9,35	64,22	110,62	5,9040	0,17
3,1	0,9	9,25	8,45	60,82	107,22	6,1557	0,06
3,2	0,9	10,15	9,35	64,82	111,22	5,8618	0,05
3,3	0,9	11,05	10,25	66,62	113,02	37,260	0,05
							0,344
							0,335

Table: 39 Settlements in the case of Hvide Sande in each layer.

The result is much lower than Aabenraa, but it is correct bearing in mind that the Q in Aabenraa is double than Hvide Sande.

cv	T95	H	t (years)
9E-08	1,127	1,705	1,1703

Table: 40 Values to calculate the time to arrive 95% of consolidation.

Time consolidation depends C_v and it is bigger in Hvide Sande, so, t_c is 1.17 years, less than half result of Aabenraa only because the C_v is a little bit big.

All the results have been exaggerated because it was considered the silt as it was gittja, the load bigger than the normal is 60 kN/m^2

8. SETTLEMENTS COMPARISON:

Afterward to make the calculations of the settlements in Aabenraa case 2, a comparison between the settlements in Hvide Sande will be realized to know if the results were correct and the differences between both cases.

Both cases have similar conditions for example the dimension of the fish farm, it will be a construction of indoor fish farm with 6m deep basins and with the button of the basins is expected established in the level +0,0m.

Franck Geotekniks has made 4 supplying, and the load from the basin has changed to 60kN/m2.

8.1 Hvide Sande:

Now, it is recompilated all the informations abou Hvide from the report, like a summary.

In the Hvide Sande case, all the information is from the report made by Franck Geotekniks, it is the information of all the parameters, settlements and explications about the construction of the fish farm.

As all the constructions, the structure has to be constructed to handle the settlements and differential settlements.

Some data obtained from the report is:

Geological data:

A lot of fill is found in this area and underneath postglacial layers. It's supposed that the layer of postglacial soil ends 19-20 m the GS.

Foundation conditions:

- plate footings or pile footing under strata producing settlements.
- due to the preload directly footing can be considered, if reinforcement can absorb.

Foundation can be done in qualified sustainable soil on well-compressed.

Settlements: Oedometer test on an undisturbed sample placed 3-4m under GS.

Time progress for the highest placed gyttja is nearly 80 days (3 months)

They recommend preloading with 3m of sand upon the level for the future bottom of the basins. And the preloading is expected to give 20-30 cm of settlement.

After remove of the preload a renewed loading will give 5-7 cm of settlements and the differential settlements will be around 3-4 cm, the preload will last 3 months.

Pre load of 3m sand with unit weight 18kN/m3 gives load 54 kN/m2.

The differential settlements without making substantial cracking

During the preloading measurements of settlement is established

1. A plate with a stick is placed on the Ground Surface at the start.

2. Then the sand is built in by steps of 0,5m and compressed afterwards the top-level is measured.

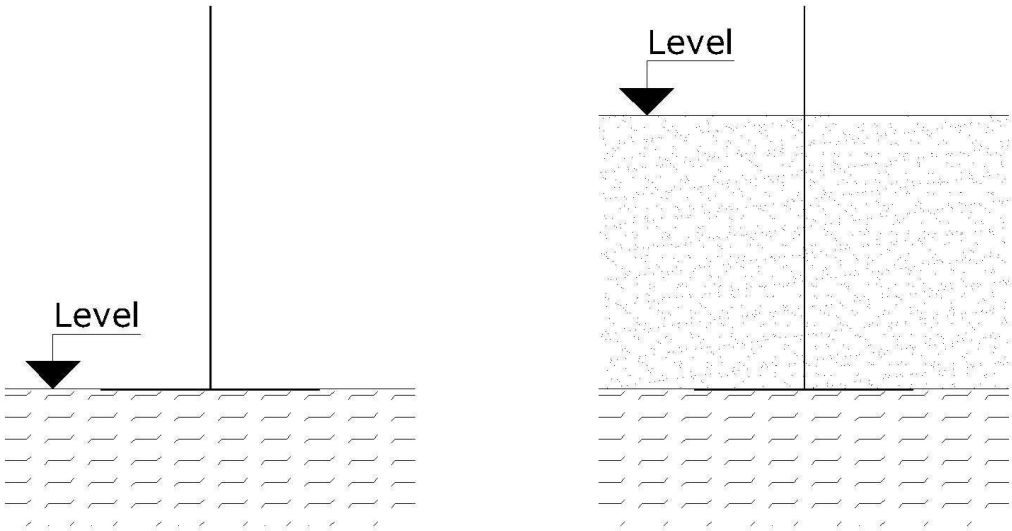


Fig. 29 Represention the consideration of diferent level of water in each case.

Data	Level establishing plates		75% of the pre-load	The preload is finished	Settlement during build in of the preload
	22.09.2011	**30.9.**	**13.10**	**20.10**	**20.10.2011**
1	-0.620	-0.650	-0.705	-0.725	10.5
2	-0.370	-0.400	-0.430	-0.450	8.0
3	-0.620	-0.660	-0.705	-0.745	12.5
4	-0.350	-0.380	-0.420	-0.445	9.5
5	-0.410	-0.450	-0.450	-0.490	8.0
6	-0.405	-0.450	-0.480	-0.450	4.5

Table: 6 Settlements considering all the steps of the construction. All in centimeters.

It's expected when the preloading is finished there will still occur 5-7 cm of settlements when the basin is established. And the differential settlement is estimated are to around 3-4 cm.

Floor must be done with reinforcements to prevent cracking from differentials settlements.

Drainage general water table in Ringkobing inlet is +0,0m. During the preload ground water table (GWT) is in level -1,5 because of the establishing drainage. Considering the level in -1.5 will be assumed a new preload provoked by the drainage.

8.2 Aabenraa

The boring number 9 was used to make the calculation and in calculation of settlements, ground water table (GWT) is assumed placed in level +0,80m, in the ground surface.

It is important consider, that it is not realible calculate the settlements with the drilling of one zone and the parameters from other.

Oedometer test on an undisturbed sample placed 7-9m under Ground Surface, giving parameters like Q =51.6 and the permeability is $3.18 \cdot 10^{-10}$ m/s.

Geological data:

It's supposed that the layer of postglacial soil ends 9-10 m the Ground Surface (GS), below there are different types of moraine.
About the differential settlements, they are considered unknown.
Time progress is only 44 days.
A plate with a stick is placed on the+ 0.0at the start.

8.3 Comparison:

	Hvide Sande	Aabenraa (2)
Oedometer test	VIA	VIA
Specific gravity	18 KN/m^3	12.44 KN/m^3
Depth	3-4m	7-9m
Q	25%	51.6%
E	1500 kN/m^2	1129 kN/m^2
k	$6 \cdot 10^{-10}$ m/s	$3.18 \cdot 10^{-10}$ m/s
SIDE	NUMBER 3	NUMBER 9
Settlement MAX	33.5 cm	72 cm
Preload	3 months	?
settlement	+ 5-7 cm	?
Time left	80 days	50 days
cv	$9*10^{-8}$	$3.6*10^{-8}$
t	1.17 years	2.95 years
	Eurocode 7	Eurocode 7

Table: 42 Recompilation of values of each case

In both cases, the results are obtained from VIA's laboratory and calculated following the formulation from Eurocode7 (1997).
The depth is an important concept because more depth is more weight above the sample, and more load to provoke settlements. The depth is similar in both samples.
The zone where the settlements are calculated are the same in the 2 case, the worse zone where is supposed it is the bigger settlement

As it is shown in the results, fish farm transmits a stress about 60 KN/m2 to the soil in the sample of Aabenraa analyzed in VIA laboratory; it will produce about 72cm of settlements in the ground. The results are too large if they are compared with the settlements calculated by Franck Geoteknik Company, which were estimated obtaining settlements about 30cm.

This is due to the parameters of the soil are different, the soil from Hvide Sande has a Q value of 25% and in the Aabenraa soil it has been obtained a value around 50%. This is not an odd difference, as the soil samples are from different place, and furthermore, as it has already explained in above chapters, the organic soil has highly variable properties.

Also, it is important to say that the Hvide Sande soil has a higher permeability than Aabenraa permeability, so the settlements will be produced in a shorter time. The settlements are bigger in Aabenraa than Hvide Sande, more than double value because the Q is more than double.

Both cases coincide in the worse zone is in the middle north-west where the fish farm will be constructed.

Settlements are bigger and it would be good ideas make some improvements in the constructions to try to reduce the magnitude of settlements.

9.IMPROVEMENTS:

Due that settlements are very large and the structure would be damaged, it has studied some alternatives to reduce the settlements:

9.1 Temporary Preloading:

The improvement consists in applying a load on the soil by an embankment to provoke the settlements which would be caused by the structure. Once the settlements are caused, it takes off the embankment and it begins building the final structure. Thereby, the final structure will only be affected by the settlements due to the secondary consolidation. The problematic of this method is that usually it takes a long time to finish the treatment.

The calculation process is explained below:

The fish farm will produce settlements in organic soil around 72cm, therefore the provisional embankment should produce the same settlements to avoid large settlements under the structure.

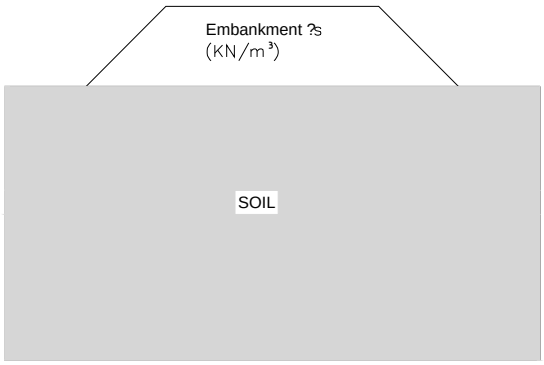


Fig. 30 TemporaryPreloads

In this project have been studied three cases of provisional embankments with different time durations.

9.1.1 Embankment of 6 month of duration.

It is necessary producing a settlement of 72 cm in 6 month of duration, for generating this settlements it must find the height of the embankment which will produce such deformation.

The embankment will only be 6 month, so it does not mean it will reach the end of the consolidation, therefore it has to find the final settlements which would be produced by the embankment.

To get this data it is needed to use the consolidation theory. It is necessary to use the formula which related settlements with consolidation grades for a determinate time.

$$\delta_{6month}^{embankment} = U_{(T)} \cdot \delta_{\infty} \qquad (41)$$

Using the equation of the time of consolidation it obtains the dimensionless time (T), and afterwards it is possible to obtain the consolidation grade.

$$T_{6month} = \frac{C_v \cdot t_c}{H^2} \qquad (42)$$

Where;

C_v = 3.6 EXP -8 m^2/s = 3.11 EXP -3 $m^2/days$

t_c = 6 month = 180 days

H= Distance which the water has to rove trough the soil (in that case it is half of the thickness) = 1.705 m

Therefore;

$$T_{6month} = \frac{3.11EXP - 3 \cdot 180}{1.705^2} = 0.192$$

Enforcing the formula mentioned in the chapter of Oedometer test which related dimensionless time with consolidation grade:

$T_{6month} < 0.2$ $\quad U(T) = \sqrt{\frac{4T}{\Pi}}$ (43) $\quad U(0.192) = 0.495$

Going back to the equation (1) and operating:

$$\delta_\infty = \frac{\delta_{6month}}{U(0.163)} = \frac{0.72}{0.495} = 1.45\ m \qquad (41)$$

Finally, by applying of the formula to obtain the settlements in a normally consolidated soil, it is found the load increase which should apply to generate the wanted deformation.

$$\delta_\infty = \sum_{i=1}^{n} Q \cdot \Delta z_i \cdot \log\left(1 + \frac{\Delta\sigma'}{\sigma'_o}\right) \qquad (44)$$

As the layer thickness of the organic soil is not too large or dividing the thick layers in thin layers, it is possible to use this formula, therefore the values are:

Q= 51% = 0.516
Δz_i = layer thickness for each organic soil strata
σ'_0 = initial effective stress
$\Delta\sigma'$ = load increase to apply

Iterating in the equation (3), it obtains:

$\Delta\sigma' = 160KN/m^2$, which would produce 1.47m of total settlements.

Therefore, with an embankment built with a sand of 20KN/m^3 of specific gravity, it will be necessary to make a temporary embankment with:

$$height^{embankment} = \frac{160}{20} = 8\ m$$

9.1.2 Embankment of 9 month of duration.

For the calculation of an embankment about 9 month of duration, the results are the followings:

The needed settlements are the same than the previous option, it means, it is needed 72cm of settlements.

$$\delta_{9month}^{embankment} = U_{(T)} \cdot \delta_\infty = 0.72m \qquad (41)$$

$$T_{9month} = \frac{C_v \cdot t_c}{H^2} \qquad (42)$$

Where;
C_v = 3.6 EXP -8 m^2/s = 3.11 EXP -3 m^2/days

t_c = 9 month = 270 days
H= 1.705 m

Therefore;

$$T_{9month} = \frac{3.11EXP - 3 \cdot 270}{1.705^2} = 0.288$$

Enforcing the formula mentioned in the chapter of Oedometer test which related dimensionless time with consolidation grade:

$T_{6month} > 0.2$ $\quad U(T) = 1 - \frac{8}{\pi^2} \cdot e^{-\frac{\pi^2}{4} \cdot T}$(43) $\quad U(0.288) = 0.603$

Going back to the equation (1) and operating:

$$\delta = \frac{\delta_{9month}}{U(0.163)} = \frac{0.72}{0.603} = 1.19\ m$$

Iterating in the equation (3), it obtains:

$$\delta_\infty = \sum_{i=1}^{n} Q \cdot \Delta z_i \cdot \log\left(1 + \frac{\Delta\sigma'}{\sigma'_o}\right) \qquad (44)$$

Where,
Q= 51% = 0.516
Δz_i = layer thickness for each organic soil strata
σ'_0 = initial effective stress
$\Delta\sigma'$ = load increase to apply
$\Delta\sigma' = 110KN/m^2$, which would produce 1.21m of total settlements.

Therefore, with an embankment built with a sand of 20KN/m^3 of specific gravity, it will be necessary to make a temporary embankment with:

$$height^{embankment} = \frac{110}{20} = 5.5\ m$$

9.1.3 Embankment of 1 year of duration.

In the same way, it obtains the height for an embankment which will be 1 year of duration.

$$T_{1year} = \frac{C_v \cdot t_c}{H^2} = \frac{3.11EXP - 3 \cdot 365}{1.705^2} = 0.391$$

Therefore,

$T_{6month} > 0.2$ $\quad U(T) = 1 - \frac{8}{\pi^2} \cdot e^{-\frac{\pi^2}{4} \cdot T}(43)U(0.391) = 0.691$

The final deformation will be:

$$\delta_\infty = \frac{\delta_{1year}}{U(0.391)} = \frac{0.72}{0.691} = 1.04\ m$$

Iterating in the equation (3), it obtains:

$$\delta_{\infty} = \sum_{i=1}^{n} Q \cdot \Delta z_i \cdot \log\left(1 + \frac{\Delta\sigma'}{\sigma'_o}\right) \qquad (44)$$

$\Delta\sigma' = 90KN/m^2$, which would produce 1.04m of total settlements.

Therefore, with an embankment built with a sand of 20KN/m^3 of specific gravity, it will be necessary to make a temporary embankment with:

$$height^{embankment} = \frac{90}{20} = 4.5\ m$$

In the below table it can be seen a summary of the obtained results to obtain 72cm of settlements for a determinate time with an improvement through embankment:

Taken time of embankment	γs (KN/m^3)	Δσ' (KN/m^2)	Embankment height (m)
6 month	20	160	8
9 month	20	110	5.5
1 year	20	90	4.5

Table: 43 Results about the different preloading options

It is very big the embankment necessary for 6 months, it is decision of the client to choose which solution he wants. For 9 months or 1 years is more factible.

9.2 Vertical drains:

The vertical drains is an improvement to accelerate the rate of consolidation, it is not an improvement to generate settlements over the soil to avoid the posterior settlements in the final structure. The vertical drains have a high permeability which facilitates the water outlet in soils with low permeability.

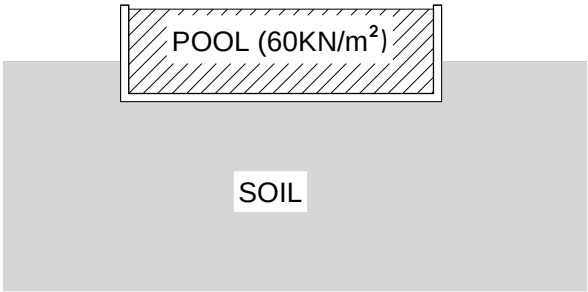
It was thought in the collocation of vertical drains working together with the embankments to accelerate the process of deformation. Thereby could be obtained the same deformation in a shorter time, however, due to the organic soil layers are not very large, this option was rejected, because they should be placed at a very close distance between them to reduce the water outlet path, not much time could be reduced and the placement of drains would not be workable.

9.3 Pumping wells:

Another possibility to produce settlements before build the structure, it is to make wells in the ground and pump the deep water, thereby it is reduced the hydrostatic pressure and consequently there is an effective stress increment.

This is a good system to generate settlements and it is also compatible with the embankment, so it would be possible to create higher settlements in shorter time. Nevertheless, the fish farm is situated close the sea, so it would be difficult to lower the water pressure in this case.

9.4 Floating foundations:



A way to completely avoid the settlements is to make a floating foundation. This method consists in removing the soil in an equivalent way to the stresses which are going to transmit the final structure.

As it is explained in chapter of consolidation, it is produced settlements when there is an effective stress variation. The using of the method about floating foundations, it is means, digging the soil to take off the same load that is going to transmit by the final structure, the soil will not be seen affected by an effective stress variation, therefore the soil will not settle (primary consolidation). This is due to the soil load does not exceed the maxim historic stress and it will not be seen submitted to restructure their particles, it means, the secondary consolidation will not be produced.

It is not certainly true that is not going to produce any settlements, in the excavation phase, the soil is unloaded and a swelling is produced, if the load does not exceeds the maxim historic stress, the soil works in the elastic zone and the settlements are not significantly large in this zone.

Applying this theory in the project, and looking in chart 1, which relates the total stresses with depth, it can be observed that it is needed to make an excavation of 3.8m of deep, as the fish farm will transmit a stress about 60KN/m2.

Nevertheless, at 3.80m of depth it can be observed that there is an organic soil layer, therefore, it is recommended to make a deeper excavation, about 20cm more, and refill the excavation excess with sand to avoid the directly contact between the organic soil and the structure.

Having studied the options to improve the soil and avoid such deformations, the construction of temporary embankment is a simple and economic solution to avoid the primary consolidation settlements.

Among the studied options, it is rejected the options which has a duration about 6 month, as it would have to build an embankment with 8m of height and it is not workable. Between the options about 9 month or 1 year of duration, it should be discussed with the customer preference, as both options are workable in relation with their height, but the duration about 1 year is thought to be the best option, because it is more economic due the lower volume of sand, and it is considered that the duration difference is not too excessive as the former fish farm can be kept operating while the new indoor fish farm is building.

Nevertheless, this method does not avoid the secondary consolidation, and in organic soil, these settlements are very large, so the structure could not bear so many deformations and could be damaged. Therefore, the floating foundation is considered the best method to build the fish farm, as it avoids the settlements completely.

By other hand, the construction with deep foundation (pilings) is another method to avoid the deformations totally, although it has not been studied in the project, as would be necessary to make different test to obtain another sort of properties.

Final note: it is possible confirm that it is not correct consider the settlements from Aabenraa to calculate the improvements in Hvide Sande

PART IV: CONCLUSION

10. CONCLUSION

After obtaining and analyzing the results from the oedometer test, it has been checked that the organic soil has very variable properties depending on some factors such as water content and the creation progress in what the soil has been developed.

Large settlements are produced in the soil studied, depending on its compressibility grade the final height of the layer can be half of its original value. In an organic soil after finishing the first phase of consolidation (water expulsion) an important second phase takes place. In this phase restructuring of particles takes place which can produce large settlements.

After taking into account different solutions to improve the settlement problematic, the conclusion that can be extracted are: temporary embankments only avoid the settlements due the primary consolidation, but not the settlements due to the creep, and consequently the structure could be damaged; the method of floating foundations is believed that it is the best way to carry out the project, as it can avoid thus as the settlements due to the primary consolidation and due to the secondary consolidation.

The recommendations for a project which has to be built over organic soil are: careful and detailed study about the soil has to be planed and performed, organic soil has different properties depending on: organic matter content, depth in which it is found, mineralogy of this content and others facts. Therefore, it is necessary to carry out the study with samples from the interest zone and relevant to the type of construction that is planned.

11. OPINION

Now that the project is finished, I have good feelings about the knowledge acquired during it. I have focused the project in a known theory but with a specific soil that is not common in our country, Spain, therefore, the Universities do not focus in them.

The project is based on a real project, which is has been carried out by an external Company, therefore I had the feeling to be working on a real one and this transmits a great interest. Furthermore, it encompasses various study fields, as I have done a laboratory test and then the settlements calculation from the properties obtained in the laboratory.

In the laboratory, I had some problems at the beginning of the project, as the results obtained were not as I expected because of the organic soul behavior, and I were afraid about how the test was working. It was a useful experience as I have realized that sometimes the real projects are not so perfect like in the theory.

On other hand, the part about the settlements calculations taught us to see a real situation, where they are more difficult to calculate than the problems seen in our student life, and then solve the problems and find a solution when the results obtained are not admissible.

In our opinion, I think it have been an interesting project, in which I have enlarged our knowledge about soil mechanics.

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