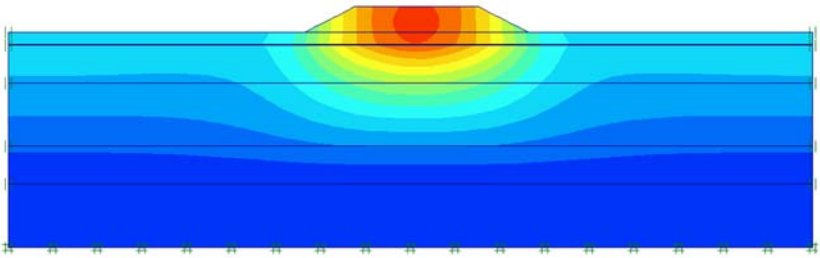

ANALYSIS OF SETTLEMENTS OF TEST EMBANKMENTS DURING 50 YEARS - A Comparison Between Field Measurements and Numerical Analysis

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Structural Mechanics

Master's Dissertation

Department of Construction Sciences
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Preface

This master thesis work is the final part of the Civil Engineering programme at the faculty of engineering at Lund University (LTH). The thesis represents 30 credits which comprises 20 weeks of studies.

This thesis was written at the Department of Construction Science at LTH and deals with numerical analysis of settlements. It was initiated by Lars Johansson, geotechnical engineering at Ramböll, Malmö. The major part of the work was carried out at Ramböll office in Malmö.

I would like to thank Lars Johansson at Ramböll for giving me the prospect of writing this thesis which without him would not have been possible. I am very happy to have had the chance to work with this exciting master thesis, which has been very special for me. I would like to express my gratitude for all help, support and advice during this work. I would also thank to Professor Ola Dahlblom at LTH for giving me advices and views of the written text.

Lund, December 2008
Bahatin Gündüz

Abstract

This report is dealing with time dependent settlements calculated with numerical methods and subsequent comparisons with field measurements. The numerical computations in this report have been performed using Plaxis, a two-dimensional numerical program based on the finite element method. Plaxis is very practical for solving complex geotechnical problems involving settlements or slope stability.

The compared objects in this report are test embankments at Lilla Mellösa and Skå-Edeby. At Lilla Mellösa two test fills were constructed by SGI in 1945 – 1947 while at Skå-Edeby four test fills were constructed in 1957. The background to the building of test fills was the search of a place for a new international airport outside Stockholm. The soil profile in both areas consists of very compressible soil layers with large thickness.

There are six different material models to choose between in Plaxis. The differ in models are how accurate they describe the mechanical behaviour of soils. The purpose of each model is to establish a relation between stresses and strains in the material. When modelling the test embankments, some different soil models of different complexity; Mohr Coulomb, Hardening soil (allows for the use of different deformation moduli for loading and reloading), and Soft soil creep (includes creep behaviour), respectively have been used. Because the work involves secondary compression for soft soil layers of clay, the use of Soft Soil Creep model has been considered reasonable. Mohr Coulomb and Hardening Soil have been used for other layers such as gravel, sand, dry crust and fills.

Plaxis gives fairly good results compared to field measurements for all the cases for both drained and undrained conditions. Calculations show that the Soft Soil Creep model matches better the field measurements than the Soft Soil model. The calculated excess pore pressure distribution with time however shows a significantly different behaviour than the corresponding field measurements.

Contents

Chapter 1

Introduction

In chapter 1 a short description of the test fields at Lilla Mellösa and Skå-Edeby is given. Also Plaxis is mentioned. It continues with an explanation of the purpose of this subject and finally the disposition of how the work will be run is given.

1.1. Background

1.1.1. General

Today the use of numerical calculation programs especially those based on the finite element method becomes more practical. Because those programs are relatively fresh and still under development a valuation of the program is recommended to give a fair rating of how precise the calculated results are. Sometimes it is also important to find out how close the program is following the behaviour of an object with time. In this thesis work field measurements of settlements and excess pore pressures at Lilla Mellösa and Skå-Edeby will be used and compared with calculated results from Plaxis.

1.1.2. Test Embankments at Lilla Mellösa and Skå-Edeby

When the *Swedish geotechnical institute (SGI)* was founded in 1944, the institute was immediately engaged in the search of a place for a new international airport outside Stockholm. The primary site to be investigated for the issue was Lilla Mellösa near Upplands Väsby. However the soil condition in the area was not suitable because of very compressible soil layers with large thickness. For that reason SGI decided to build test embankments to consolidate the soil in advance and to study the process of consolidation. Two test embankments were assembled at Lilla Mellösa. One which was vertical drained and the second one which was undrained. The field at Lilla Mellösa was by time dismissed as a possible place for a new airport but the test area has been left for further geotechnical investigation.

The need of a new airport became more urgent when Scandinavian Airlines ordered a fleet of new jets in 1956. There had been researches in Halmsjön (today Arlanda Airport) for the purpose but the location which lay 40 km north of Stockholm seemed too far away. A possible alternative to Halmsjön was Skå-Edeby which was a plain agriculture ground and was situated in an island about 25 km west of Stockholm. The soil conditions in Halmsjön were well-known but much less were known about the conditions in Skå-Edeby. In 1957 SGI was commissioned to perform field tests and investigate the possibility of building a new airport in Skå-Edeby. The building time had to be short and because the soil consisted of 15 meter soft clay, vertical drainage was the only possible and useable method at that time. Four circle shaped test fills were constructed to study the consolidation process of soft clay. The alternative of Skå-Edeby for the location of a new airport however was abandoned from economic reasons. Even if Skå-Edeby did not turn to be an airport construction site the research work in the area continued for the reason that the result could be useful for future projects. Two additional test embankments were built later. (Larsson 2007)

1.1.3. Plaxis

Plaxis is a numerical program based on the finite element method and is very useful for calculation of loads, deformations and water flow in soil and for geotechnical constructions. It was developed in the end of the ´80s at Delft University in the Netherlands. During the ´90s it was further developed for commercial purposes and is since 1998 available in Windows environment. Plaxis is mainly a two-dimensional program for statically computing but there are also additional versions of the program which can calculate dynamical models. Since some years, a three dimensional version is available commercially. In this thesis only the two dimensional version has been considered.

1.2. Purpose and objective

The purpose in this master thesis is to compare settlements calculated by Plaxis with field measurements from Lilla Mellösa and Skå-Edeby.

The main objective is to verify a numerical model which describes the current field situation as realistically as possible.

1.3. Disposition

The work will be carried out in several steps. The first part of the work will be a literature study in order to find relevant background information on the topic. Geotechnical literature data bases at Lund University and at the Swedish Geotechnical Institute will be used. One important phase of this step is to identify previous work adopting the same or similar approach. The main hypothesis with this work is to find out if it is possible to catch the observed behaviour by numerical calculations. Analyses will be carried out for embankments placed on different soil layer profiles. Different methodologies will be adopted, i.e. separation between the mechanical and the pore water phases, and incorporation of both at the same time (consolidation), respectively. Also different soil models can be adopted in order to describe the soil behaviour as correctly as possible. A presentation of the results without any interpretations or judgements will be given. In a separate conclusion section the results will be discussed and analysed. Some suggestions and ideas for further work and improved methodology will also be elaborated in this section.

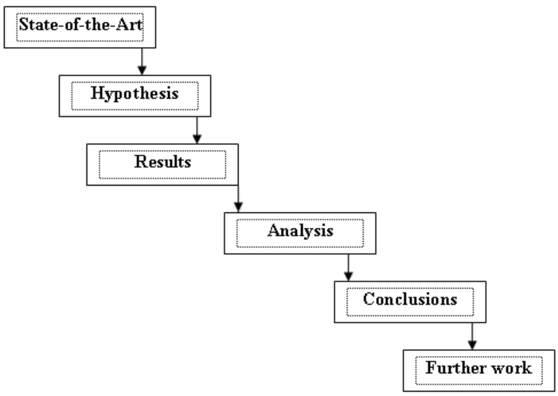

Figure 1.1. The disposition of how the work will be carried through.

Chapter 2

Clay behaviour and properties

In chapter 2 a brief description of the clay soils will be specified because it is of importance to understand the behaviour of clay soils.

2.1. Clay minerals

The term clay can be explained as a material composed of a mass of small mineral particles, which in association with water exhibits the property of plasticity. The geotechnical properties of clay materials depend largely on their chemical structure. Their chemical structure is composed of extremely small crystalline particles of one or more members of a small group of minerals that are commonly known as clay minerals. Studies of the crystal structure of clay minerals lead to a better understanding of the behaviour of clays under different loading conditions.

The chemical structures of clay minerals are essentially hydrous aluminium silicates, with magnesium or iron replacing wholly or in part for the aluminium, in some minerals.

There are two fundamental building blocks that are involved in the formation of clay minerals structure. Those are silicon–oxygen tetrahedron units and an aluminium or magnesium octahedron units. (Axelsson 2005). Clay minerals can be divided into three general groups on the basis of their crystalline arrangement. The three most common clay minerals are kaolinite, montmorillonite and illite which are divided into different groups, see table 1.

	Name of mineral	Structural formula
I.	Kaolin group	
	1. Kaolinite	$Al_4Si_4O_{10}(OH)_8$
	2. Halloysite	$Al_4Si_4O_6(OH)_{16}$
II.	Montmorillonite group	
	Montmorillonite	$Al_4Si_8O_{20}(OH)_4nH_2O$
III.	Illite group	
	Illite	$K_y(Al_4Fe_2.Mg_4.Mg_6)Si_{8-y}$ $Al_y(OH)_4O_{20}$

Table 1. Classification of clay minerals (Coduto 1999).

Kaolinite is produced from the weathering of the parent rocks that have orthoclase feldspar (e.g. granite). The layers are held together by hydrogen bonding giving a very stable structure to the mineral. The hydrogen bonding is a result of the attraction forces between the oxygen atoms of the silica sheet and the hydroxyl ions of the alumina sheet. (Svensson 2003).

Illite is the most common clay mineral in Swedish soils. It is a product of weathering of micas with the major parent rock of muscovite. It has a central aluminium or magnesium octahedron unit surrounding by two silicon–oxygen tetrahedron units. The bond between the two layers is made of potassium, and is not as strong as in kaolinite and has more space for water to enter between the elemental layers.

Montmorillonite has similar structure as illite but the bonding between these layers is very weak, so large quantities of water can easily enter and separate them, thus causing the clay to swell. Montmorillonite is formed from the weathering of volcanic ash in marine water under poor drainage conditions. The reason why clay minerals swell is their ability to substitute atoms or water molecules in their crystal structures which make them unstable and have a lower strength or swelling (Svensson 2003).

2.2. Phase relationships in soil

Soil has a three phase system consisting of solid particles, liquid and gas. The liquid and gas are in the voids or pores between the solid particles. The solid phase is always present in soil and consists usually of particles derived from rocks. Sometimes it can also include organic materials. The liquid phase is usually present and consists most often of water but sometimes it can also be gasoline and other chemical, sea water or natural petroleum seeps.

If the liquid phase does not completely fill the voids, then the remaining space is occupied by the gas phase. It is usually air but can also include other gases, such as methane and carbon dioxide from organic materials. (Coduto 1999).

The relative proportions of solids, water and air in a soil is very important to identify because their proportions have a significant effect on its behaviour. Of that reason there is quantitative methods developed by geotechnical engineering to understand the relationship between the phases in the soil. The phase-relationships in terms of mass-volume and weight-volume for a soil mass are shown by a block diagramme in Figure 2.1.

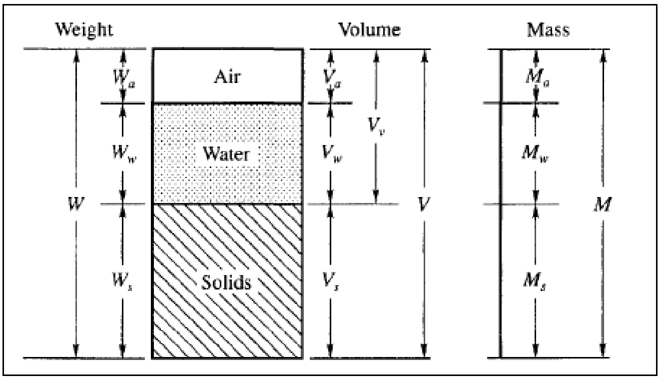

Figure 2.1. Three phases of the soil element (Coduto, 1999)

2.3. Settlements

Consolidation is a term used to describe the procedure of transferring the total stress to effective stress and increasing the pore water pressure by squeezing out the water. This happens when a load is applied or increased on saturated soil layers. At the beginning the increased force (total stress) will initially be carried by the water in the pores resulting thereby in an excess pore water pressure. If drainage is allowed, the resulting hydraulic gradients initiate a flow of water out of the clay mass and the mass begins to compress. A part of the applied stress is transferred to the soil skeleton, which in turn causes a reduction in the excess pore pressure. When the excess pore water pressure is equal to zero the whole load will be carried by the soil skeleton. This process will generate both vertical deformations, settlements, and horizontal settlements. A settlement in the soil is impossible to avoid but what is more important is the magnitude of the settlement and its comparison with tolerable limits. (Axelsson 2005)

Consolidation may be due to one or more of the following factors:

- External static loads from structures.
- Self-weight of the soil such as recently placed fills.
- Lowering of the ground water table.

The settlement at the ground is the sum of three parts; consolidation settlement, secondary compression settlement and distortion settlement. (Coduto 1999).

2.3.1. Consolidation settlement

Consolidation settlement (or primary consolidation settlement) happens when an increase in the effective vertical stress, occurs which gives rise to a decrease in the volume of the voids. If the soil is saturated (S_r=100%) reduction in volume occurs only if some of the pore water is squeezed out of the soil. The volume of solids remains constant because the compression of individual particles is negligible. (Coduto 1999).

2.3.2. Secondary compression settlement

Secondary compression is supposed to start after the primary consolidation ceases which is after the excess pore water pressure approaches zero, but it has not to be zero. Secondary compression happens because of particle reorientation, creep and breakdown of organic materials. This part of consolidation does not require the removal of pore water. Secondary compression takes place mostly in highly plastic clays, organic soils and sanitary landfills. It is negligible in sands and gravels. Secondary compression does not depend on changes in vertical effective stress. (Coduto 1999)

2.3.3. Distortion settlement

Distortion settlement is a kind of settlement that develops from lateral movements of the soil because of changes in vertical effective stress. This happens when a heavy load is applied over a small area which resulting in a lateral deformation. The value of distortion settlement is much smaller than consolidation settlement and is generally ignored. (Coduto 1999)

Chapter 3

The test embankments

In this chapter the result from the test fields at Lilla Mellösa and Skå Edeby will be given without any interpretation. This chapter includes a short background to the construction of the test fills, the soil condition and the soil behaviour.

3.1. The test field at Lilla Mellösa, Upplands Väsby

3.1.1. General

At the farm of Lilla Mellösa near Upplands Väsby (Figure 3.1), two test fills were constructed by SGI in 1945 – 1947. Those fills had a dimension of 30x30 and a height of 2.5 m. The first one was installed with vertical drains. The second one was installed without vertical drains. Also a third one was constructed consisting of the removed surcharge material from the drained fill. The material in those fills consisted of gravel. After about 200 days 0.7 m of the upper part of the drained fill was removed.

The follow-up of the test fills was continued regularly but since new test fills with better vertical drains and instrumentation were constructed at Skå-Edeby, the interest in the fills at Lilla Mellösa declined and the follow-up of the fills became more irregular despite two major investigations that were held in 1966 and 2002, with a new and better method (Larsson 2007).

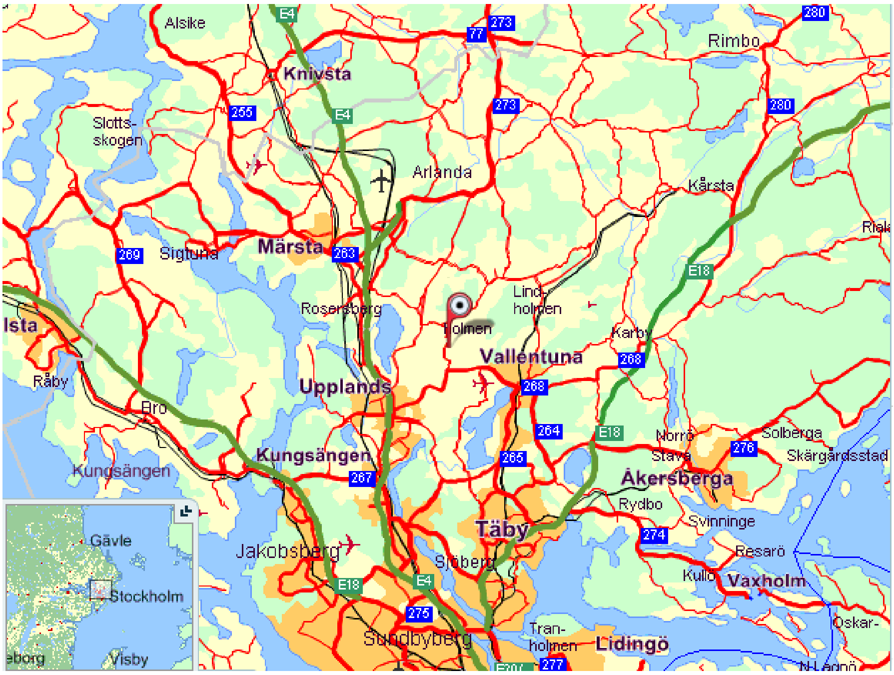

Figure 3.1. The location of Lilla Mellösa near Upplands Väsby, Stockholm.

3.1.2. Soil Condition

The soil profile at Lilla Mellösa consists at the top of a 0.3 m organic subsoil which was removed before the banks were constructed. In the upper part there is a 0.5 m overconsolidated dry crust composite with a significant part of organic soil. Under the dry crust there are several layers of soft soil with a major amount of organic soil. The amount of organic soil is decreasing with depth. (Larsson 2007). The natural water content is approxotimately equal to the liquid limit and decreases from a maximum of 130% under the dry crust to about 70% in the bottom layers. The bulk density increases from about 1.3 t/m^3 to about 1.8 t/m^3 at the bottom. The pore water pressure is hydrostatic with a ground water level around 0.8 metres below the ground surface. The first 2 metres of the soil profile is overconsolidated due to dry crust effects and the rest of the the soil profile is considerably overconsolidated. (Larsson 2007).

The soil profile at Lilla Mellösa can be seen in Figure 3.2.

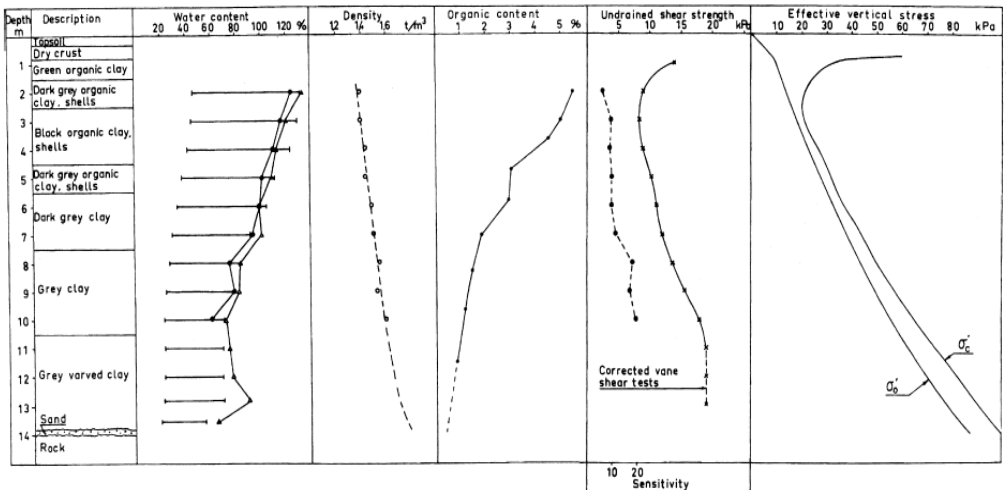

Figure 3.2. Soil conditions at Lilla Mellösa (Larsson 2007).

3.1.3. Observed behaviour

Drained test embankment

The test embankment with vertical drains had a settlement of about 0.7 m after 200 days. At this time the upper part of the test embankment (0.8 m) was removed according to the original plan. However the settlements did not stop because of the removal of surcharge, instead they continued but with a lower velocity than before. In 2002 the settlement was about 1.6 m and had a velocity about 6 mm per year. The major part of the compression before the removal of the surcharge took place in the uppermost 5 metres at the soil profile were the vertical drainage was installed. The compression in the upper part has now stopped while the settlement in the lower part continues still without an end yet. (Larsson 2007)

Undrained test embankment

The initial settlement during construction was about 0.065 m. In 1966 the settlement without regarding creep effects was about 1.4 m and there was an excess pore pressure in an order of 30 kPa. In 1979 the total settlement was 1.65 m and a remaining excess pore pressure of 20 kPa. In 2002 the total settlement was over 2 m while there was still an excess pore pressure in an order of 12 kPa. The settlement is still continuing but with a lower rate. The current settlement rate is about 10 mm per year. (Larsson 2007)

3.2. *The test field at Skå-Edeby*

3.2.1. General

The construction of test embankments at Skå-Edeby started in July 1957. The experience from Lilla Mellösa was very valuable when new equipment was constructed and tested for the first time. At the start four test fills with diameters from 35 to 70 metres were constructed. The first one (Area 1) had a diameter of 70 meter and was installed with vertical sand drains. The sand drains were divided into three segments with a drain spacing of 2.2, 1.5 respective 0.9 meter. The height of fill was 1.5 meter. The second one (Area 2) had the same height, a diameter of 35 meter and vertical sand drains with a drain spacing of 1.5 meter. The third one (Area 3) was also installed with vertical drains and a diameter of 35 meter but had an additional load of 0.7 meter fill. The fourth one (Area 4) had a diameter of 35 meter, a height of 1.5 meter but no vertical drains (Larsson 2007).

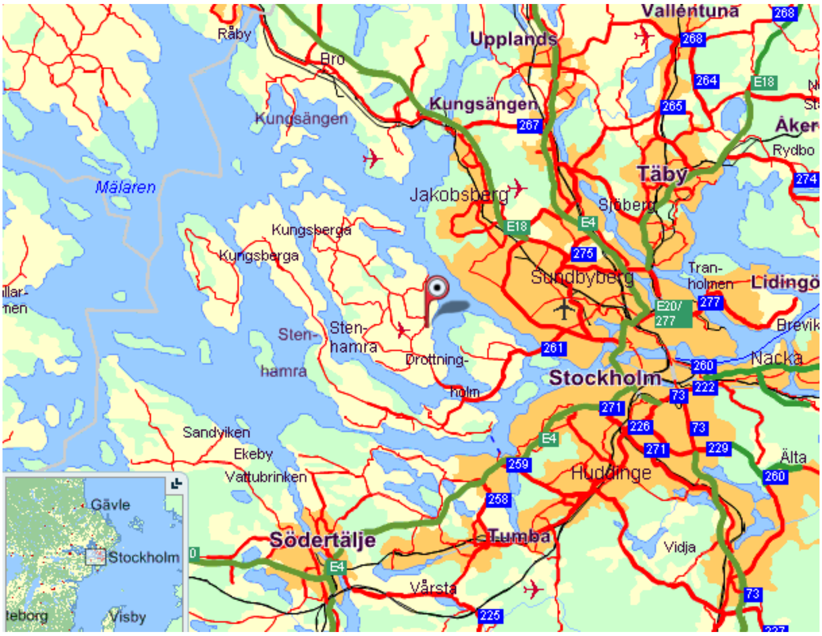

Figure 3.3 The location of Skå-Edeby,Stockholm.

In 1961 the test fill with a height of 2.2 meter was partially removed. The removed part was used for construction of an additional test fill.

During 1972 another test fill was created (Area 5). This was installed with fabricate vertical band drains of type Geodrain.

Figure 3.3 shows the location of Skå-Edeby and Figure 3.4 shows the map over the test areas in Skå-Edeby. In the map the distances between each embankment and also the depth to rock layer can be seen.

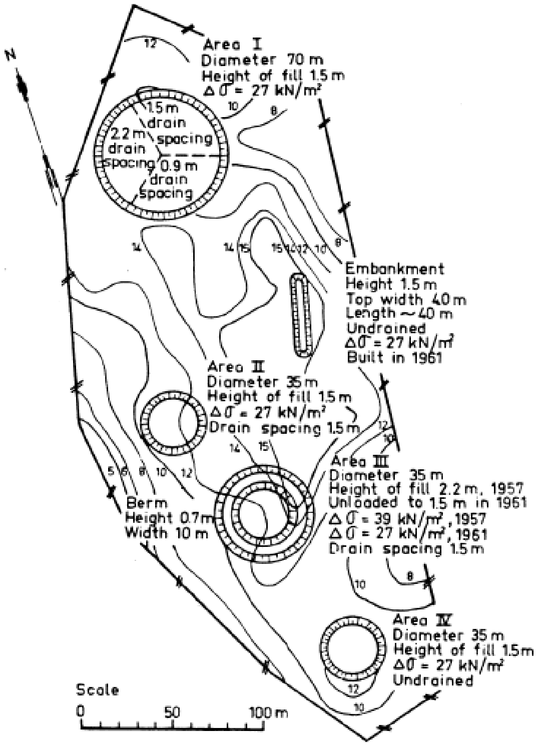

Figure 3.4. The test areas at Skå-Edeby (Holtz and Broms 1972, from Larsson 2007).

3.2.2. Soil condition

The soil under the test embankments is soft and has a thickness of 12 to 15 metres overlaying till or rock. At the top the soil consist of 0.5 meter thick overconsolidated dry crust. Underneath the dry crust, there is a layer of high-plastic organic clay. The organic clay rests on a soil layer consisting of postglacial clay with low organic content. Below the postglacial clay there is another layer of organic clay. The organic clay is varved. (Larsson 2007)

The natural water content is above the liquid limit except for the upper two metres which are affected by dry crust effects. The water contents decreases from about 100 % at the top to about 60 % in the lower layers. The bulk density increases from about 1.3 t/m^3 to about 1.8 t/m^3 at the bottom. (Larsson 2007)

The pore water pressure is hydrostatic and the groundwater level varies from the ground surface to 1 metre below. The ground water is measured to vary seasonally with a maximum variation of ±0.5 metres. (Larsson 2007)

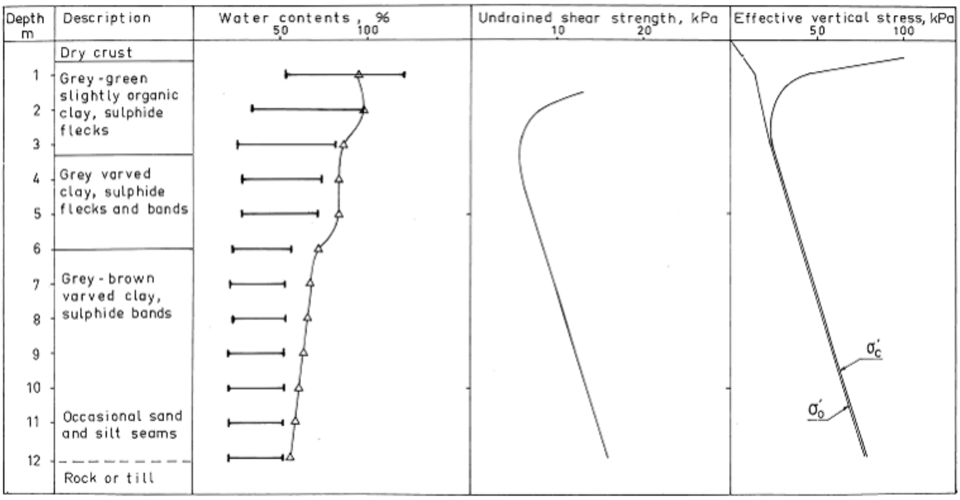

Figure 3.5. Soil condition at Skå-Edeby. (Larsson 2007)

3.2.3. Observed behaviour

Test embankments with vertical drains

The test embankment of area 3 with vertical drains had a settlement of about 1.55 m after 4 years. At the same time the upper part of the test bank (0.7 m) was removed. The removal of surcharge gave rise to a heave that reached its maximal value 8 mm after one year. After the removal of upper part the settlements continued linearly with time. In 2002 the settlement was about 1.6 m. The embankments of area 1, 2, and 5 had a total settlement of 1.25 m, 1.20 m, and 0.75 m, respectively, in 2002.

Test embankments without vertical drains

The initial settlement during construction was about 0.06 m. In 1972 the settlement without regarding creep effects was about 0.75 m and there was still an excess pore pressure in order of 20 kPa. In 1982 the total settlement was 0.95 m and still an excess pore pressure in order of 12 kPa was left. In 2002 the total settlement was 1.10 m while the remaining excess pore pressure was about 8 kPa.

Chapter 4

Numerical analyses, FEM

In chapter four the Plaxis software will be described. The procedure of using the program will be explained briefly. Also some common material models in the program will be introduced in detail.

4.1. General

Calculation with conventional methods can be useable if the problem has linear elastic behaviour. This is a simplification. Most of the problems do not have linear elastic behaviour. Soil has non-linear elastoplastic behaviour. To solve advanced numerical problems it is often recommended to use a computer program based on the finite element method (FEM). A numerical analysis program is reasonable when the problem can not be solved by conventional methods based on analytic solutions. The basic idea in the finite element method is to divide a complicated model into a finite number of elements for which stresses and strains can be solved numerically.

Without going deep into the world of FEM it can be mentioned that FEM is a technique to find approximate numerical solutions for partial differential equations as well for integral equations. This can be done by eliminating differential equations completely or rendering it to ordinary differential equations which can then be solved by other techniques (Euler's method etc.). The basic concept of the FEM is that a complicated model of a body or structure is divided into a number of smaller elements. Those elements are then connected by nodes. At every node there are one or more degrees of freedom where the quantity of functions is described. By solving the values at the nodes the stress and strains in every element can be calculated. (Ottosen and Petersson 1992)

4.2. Plaxis

Plaxis is a program based on the finite element method. The program was originally developed at the University of Delft in Netherlands where research in geotechnical design based on FEM in the ´70s resulted in a commercial version of the program in 1987 and since 1998 it is available in a Windows version. The program can simulate problems with the most common construction elements such as beams and struts. Today, the program is practical for solving complex geotechnical problems involving settlement or slope stability. The program is divided into four sub programs; *Input, Calculations, Output and Curves*.

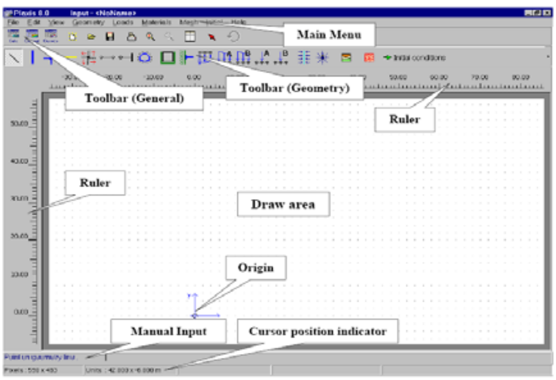

Figure 4.1. Main window of the input program. (Brinkgreve 2006)

In the input program (Figure 4.1) the soil model can be created. Soil layers, loading, struts and plates are all drawn by geometry lines available in the toolbar. In the next step the user enters the material data for each material in the *material sets*. In material sets all information; *name*, *material model*, *material type* (drained/undrained), *permeability*, *unit weight*, *stiffness* and *strength* must all be entered before continuing to further steps. When the geometry model is complete, the finite element model or *mesh* can be generated. There are several options depending on how coarse or fine mesh the user wishes to adapt. Selecting a finer mesh is recommended in the parts that are of interest or where most faulting can happen. The use of a finer mesh however requires a longer calculation time. When the mesh is generated the program continues to establish the *initial conditions*. The initial conditions cover the initial values for *effective stress*, *tension* and *pore pressure*. The initial pore pressure can in the simplest case be determined by drawing the ground water level and assuming hydrostatic pore pressure increase. When the initial conditions have been generated the program continues to *calculation*.

The calculation sub program can be used to define calculation steps. The steps can be defined in the same order as it would be done in reality. There are three different calculation types for the user to choose; *plastic*, *consolidation* and *φ/c-reduction* where the last one is helpful for computing *safety factors*. Once all steps have been defined the calculation process can start by clicking on the <Calculate> button. During the calculation a small window appears which gives information about the progress of each calculation phase. The information is continuously updated and shows a load-displacement curve, iteration process (plastic points, global errors, etc) and the level of the load systems. When the calculation has ended the result for each phase can be evaluated in the Output sub programme. In the output window the user can view the result of displacement, stresses, pore pressure and excess pore pressure. This can be listed or visualised. Also shear stress and bending moment for construction elements can be seen. A pre-established point can be studied in the Curve program.

4.2.1. Elements

The soil layers can be modelled by selecting either 6-node or 15-node triangular elements, as illustrated by Figure 4.2. The 15 - node triangle has a fourth order interpolation for displacements and twelve stress points. The 6-node triangle has a second order interpolation for displacements and three stress points. The 15-node triangle is more useful for complicated problems but demands more memory consumption and results in a slightly slower calculation and operation performance. The 6-node triangle gives good result in standard deformation analyses but is not recommended in problems where failure plays an important role. In those cases the 15-node elements is recommended. (Brinkgreve 2006)

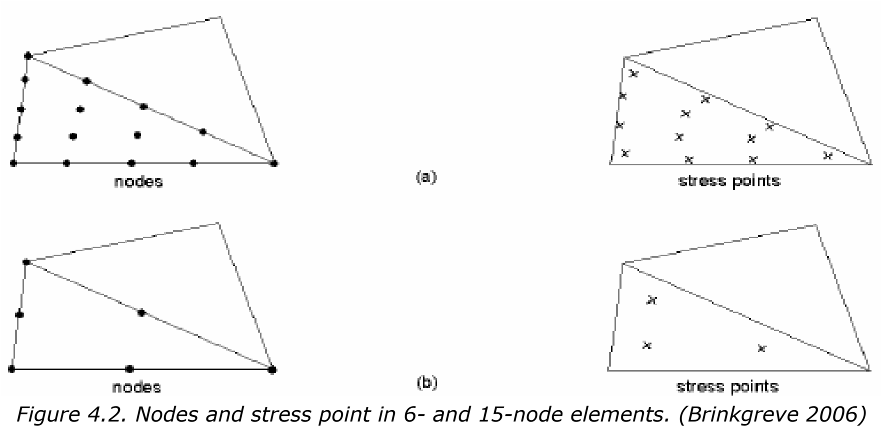

Figure 4.2. Nodes and stress point in 6- and 15-node elements. (Brinkgreve 2006)

4.2.2. Calculation types

There are as previously mentioned three types of calculation to choose between in Plaxis; *Plastic* calculation, *Consolidation* analysis and *Phi-c reduction*.

A *Plastic* calculation can be selected when the user is interested in an elastoplastic deformation analysis in which it is not essential to take into account the magnitude of excess pore pressures with time. A plastic calculation does not take time effects into account. A plastic calculation can also be used with soft soils but the loading history and consolidation cannot be followed, instead a reasonably accurate prediction of the final situation will be given.

The *Consolidation* analysis should be used when it is of interest to follow the development of excess pore pressure with time in soft soils.

Phi-c reduction is a safety analysis in Plaxis which is desired to use when the situation in the problem needs a calculation of the safety factor. A safety analysis can be made after each individual calculation phase but it is recommended to use a safety analysis at the end, when all calculation phases have been defined. Especially it is not advisable to start the calculation with a safety analysis as a starting condition for another calculation phase because this will end up in a state of failure. (Brinkgreve 2006)

4.3. Material models

There are six different material models to choose between in Plaxis. The difference between those models is in how accurate they present the mechanical behaviour of soils. The design for each model is to describe the relation between *stress* and *strain* in the material.

A short presentation of each model will be given before explaining in more detail those models that have been used.

4.3.1. Linear isotropic elasticity

The linear, isotropic elasticity is the simplest stress-strain relation that is available in Plaxis. This model has only two input parameters, Young´s modulus, E, and Poisson’s ratio, ν. Such a model is not appreciable to explain the complex behaviour for soil but it is suitable for modelling massive structural elements and bedrock layers.

4.3.2. Mohr Coulomb model

Mohr Coulomb is an elastic plastic model involving five input parameters, E and ν for soil elasticity, φ and c for soil plasticity and ψ as an angle of dilatancy. To get a first assessment of deformations it is according to Brinkgreve (2006) advisable to use the Mohr Coulomb. This because other advanced models need further soil data than Mohr Coulomb.

4.3.3. Jointed Rock model

The Jointed Rock model is an anisotropic elastic plastic model which is especially compliant for generating layers of rock involving stratified and specific fault directions. Plasticity can occur in a maximum of three shear planes where each plane has its own strength parameters, φ and c. If the material has constant stiffness properties such as E and ν then the intact rock will behave fully elastic while reduced elastic properties may be defined for the stratification direction.

4.3.4. Hardening-Soil model

The Hardening Soil model is similar to the Mohr Coulomb model but it is more advanced. As for Mohr Coulomb, the input parameters of the Hardening Soil model are the friction angle, the cohesion and the dilatancy angle. The difference from Mohr Coulomb model is that Hardening Soil model uses three different input stiffnesses: the trixial loading stiffness, E_{50}, the trixial unloading stiffness, E_{ur}, and the oedometer loading stiffness, E_{oed}. For many soil types it can be assumed that $E_{ur} \approx 3E_{50}$ and $E_{oed} \approx E_{50}$ although very soft and very stiff soils can give other ratios. Another difference from the Mohr Coulomb model is that stiffnesses in the Hardening Soil model increase with pressure. The Hardening-Soil model is suitable for all soils, but does not account for viscous effects such as creep and stress relaxation (generally all soils exhibit some creep).

4.3.5. Soft Soil model

Soft Soil model is a Cam–Clay type model that is used for computing primary compression of near normally-consolidated clay-type soils. According to Brinkgreeve (2006) the Hardening Soil model outshines the Soft Soil model but it is still retained because older users of Plaxis might be comfortable with this model.

4.3.6. Soft Soil Creep model

The secondary compression mostly happens in soft soil such as normally consolidated clays, silts and peat why this model has been specially developed for this purpose. Brinkgreve (2006) mentions that Soft Soil Creep model is not much better than the Mohr Coulomb model in unloading problems such as tunnelling and excavation.

4.3.7. Modified Cam Clay model

This model is well known from international modelling literature. It has been added to Plaxis recently to compute with other codes. Primary the aim with this model is to enable modelling of near normally consolidated clay soils.

4.3.8. Discussion

In the modelling of the test fills at Lilla Mellösa and Skå-Edeby the models of Mohr Coulomb, Hardening Soil and Soft Soil Creep have been used. Because the work involves secondary compression for soft soil layers of clay, the use of the Soft Soil Creep model has been reasonable instead of Cam Clay and Soft Soil. Mohr Coulomb and Hardening Soil have been used for other layers such as gravel, sand, dry crust and fills.

4.4. Mohr Coulomb model

Mohr Coulomb (MC) model is by Brinkgreve (2006) referred to as an elastic perfectly plastic model. By explaining this, a yield function is introduced. The yield function, f, can be explained as a function of stress and strain and is presented as a surface in a stress space. The yield function is the boundary line between elastic and plastic behaviour. A perfectly plastic model is a model that has a fixed yield surface. Moreover the stress points that can be found inside the yield surface have an elastic behaviour and all strains are reversible.

The Mohr Coulomb yield condition consists of six yield functions. The friction angle (φ) and the cohesion (c) are appearing in the six yield functions and together they form a hexagonal cone in the stress space (Figure 4.3). In addition to the six yield functions there are also six plastic potential functions that are defined for the Mohr-Coulomb model. The plastic potential functions contain a third plasticity parameter, the dilatancy angle (ψ).

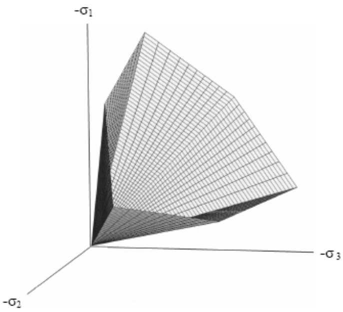

Figure 4.3. Mohr Coulomb yield functions forming a hexagonal cone. (Brinkgreve 2006)

In geotechnical literature there is also mentioned a Mohr Coulomb failure criterion. Figure 4.4 shows the two dimensions state of the Mohr Coulomb failure criterion. When the Mohr circle touches the Mohr envelope the failure state is reached.

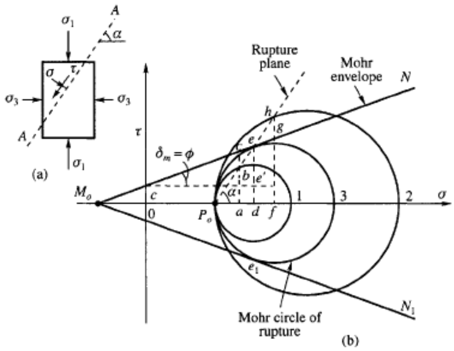

Figure 4.4. The two dimension state of Mohr Coulomb failure criterion. (Murty 2003)

There is a possibility to include the amount of tensile stress that can establish in some practical problems. Soil has generally no or very small tensile strength and of that reason Plaxis by default selected with a tensile strength of zero in models of Mohr Coulomb and Hardening Soil. Because the tensile stress can increase with the cohesion this behaviour can be included in the model by selecting the "tension cut - off" parameter. (Brinkgreve 2006).

The MC model requires as mentioned before, five parameters; Youngs modulus (E), Poisson's ratio (ν), friction angle (φ), cohesion (c), and dilatancy angle (ψ). Those parameters can be obtained from basic tests of soil samples. The basic stiffness modulus that is used in Plaxis is Young's modulus (E). Because many geotechnical materials have a non linear behaviour special attention should be taken to the stiffness parameter during a calculation. The MC model uses a constant stiffness parameter under the whole calculation. In reality soils have not a constant stiffness. The stiffness depends significantly on stress level and the stress level generally increases with depth. There are different stiffness moduli that can come to advantage. There is the initial modulus (E_o) and the secant modulus at 50% strength (E_{50}). E_o works better in materials that have a large linear elastic range while E_{50} fit better in problems involving loading of soils. In unloading problems such as tunnelling and excavations it is beneficial to apply E_{ur}. Both E_{50} and E_{ur} increase with the pressure which gives deep soil layers a larger stiffness than shallow layers. There is also the opportunity to selecting the $E_{increment}$ –value that is available to select in Plaxis in the MC model. By selecting it the stiffness can follow the stress development in the soil. $E_{increment}$ –value is the increase of the Young's modulus per unit of depth. (Brinkgreve 2006)

Poisson's ratio (ν) is evaluated by matching K_0 since $K_0 = \sigma_h / \sigma_v = \nu/(1-\nu)$. For loading conditions ν can range between 0.3 and 0.4. For unloading conditions values in the range between 0.15 and 0.25 should be used. (Brinkgreve 2006). The cohesive strength (c) should have small values (c < 0.2 kN/m2) for cohesionless sands (c=0) for avoiding complications. There is also a special option in Plaxis, $c_{increment}$, that makes the cohesion value changing with the depth. The dilatancy angle (ψ) shows little dilatancy (ψ ≈ 0) for clay soils that are not heavily overconsolidated. For sand layers the dilatancy depends on both the friction angle and the density (ψ ≈ φ-30).

4.5. The Hardening Soil model

The Hardening Soil (HS) model is an advanced model for simulating different kinds of soils, both soft and stiff soils. There are two big distinctions between the HS model and the MC model. One is that HS uses a hyperbolic stress – strain curve instead of a bi-linear curve and second the stress level can be controlled better with the HS. The HS model works in a way that its stiffness modulus changes when the stress level changes. The yield surface of the HS model is not fixed in a stress space like an elastic –plastic model, instead it can expand due to plastic strain.

Plaxis uses three different stiffness moduli in the HS model to describe the hyperbolic stress – strain curve. Those are the secant stiffness in standard drained triaxial test (E_{50}^{ref}), tangent stiffness from oedometer test (E_{oed}^{ref}) and a stiffness modulus for unloading/reloading (E_{ur}^{ref}). (Brinkgreve 2006). Those stiffness moduli are calculated by:

$$E_{ur} = E_{ur}^{ref}\left(\frac{c \cdot \cos\varphi - \sigma_3' \cdot \sin\varphi}{c \cdot \cos\varphi + p^{ref} \cdot \sin\varphi}\right)^m \tag{4.1}$$

$$E_{oed} = E_{oed}^{ref}\left(\frac{c \cdot \cos\varphi - \sigma_1' \cdot \sin\varphi}{c \cdot \cos\varphi + p^{ref} \cdot \sin\varphi}\right)^m \tag{4.2}$$

$$E_{50} = E_{50}^{ref}\left(\frac{c \cdot \cos\varphi - \sigma_3' \cdot \sin\varphi}{c \cdot \cos\varphi + p^{ref} \cdot \sin\varphi}\right)^m \tag{4.3}$$

The power m defines the stress dependency. To simulate a logarithmic compression the power should be equal to 1.0. Also for soft soils this value should be used as 1.0. There has also been testf where the power m varies between 0.5 and 1.0.

The HS model can be formulated by a hyperbolic relationship between the vertical strain, ε and the deviatoric stress, q in primary loading for a triaxial loading;

$$-\varepsilon_1 = \frac{1}{E_i}\frac{q}{1-q/q_a} \text{ for } q < q_f \tag{4.4}$$

where q_a is the shear strength and E_i is the initial stiffness that also are equal to

$$E_i = \frac{2E_{50}}{2-R_f} \tag{4.5}$$

The ultimate deviatoric stress, q_f and q_a is defined as following:

$$q_f = \left(c \cdot \cot\varphi - \sigma_3^{\prime}\right)\frac{2\sin\varphi}{1-\sin\varphi} \tag{4.6}$$

$$q_a = \frac{q_f}{R_f} \tag{4.7}$$

where R_f is a failure ratio <1. In Plaxis, the magnitude of R_f is by default 0.9.

The failure parameters according to the MC model (friction angle, cohesion and dilatancy) are also formulated in the HS model. There are also some advanced parameters in the HS model that have default values in Plaxis. The Poisson's ratio for unloading-reloading, v_{ur} (default v_{ur} = 0.2), reference stress for stiffness, p_{ref} (default p_{ref} = 100kPa) and K_0-value for normal consolidation (default $K_0^{nc} = 1-\sin\varphi$). The hyperbolic relationship between ε and q can be seen in Figure 4.5.

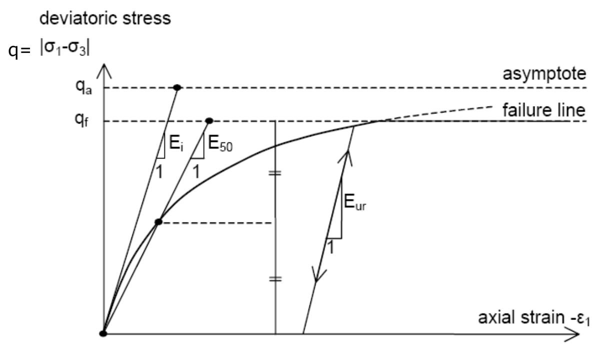

Figure 4.5. Hyperbolic stress-strain relation in standart trixial loading. (Brinkgreeve 2006)

Figure 4.6 shows that the triaxial moduli (E_{50}^{ref} and E_{ur}^{ref}) control the shear yield surfaces and oedometer modulus (E_{oed}^{ref}) controls the cap yield surfaces. E_{50}^{ref} controls the magnitude of plastic stress while E_{ur}^{ref} controls the magnitude of elastic strains. In the same way E_{oed}^{ref} controls the magnitude of plastic strains that come from the yield cap. The magnitude of the yield cap is determined by the isotropic pre-consolidation stress, p_p.

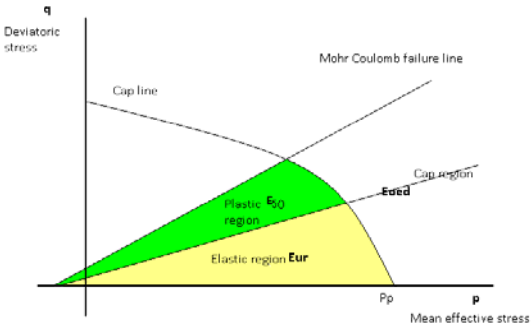

Figure 4.6. Two-dimension yield –cap surface of HS model in a q-p plane. (Edmark&Sandberg 2005, Brinkgreve 2006)

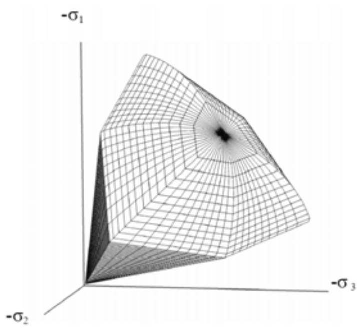

Figure 4.7. Yield cap surfaces of Hardening Soil in a stress space. (Brinkgreve 2006)

4.6. Soft Soil Creep model

The Soft Soil Creep (SSC) model is as its name indicates a model that takes into consideration creep (i.e. secondary consolidation) which is very common for soft soils (normally consolidated clays, clayey silts or peat). The SSC model is the only model in Plaxis that computes problems involving secondary consolidation.

The special feature for soft soils is their high compressibility. In oedometer test normally consolidated clays behave ten times softer then normally consolidated sands. This because clays have Eoed=1-3 MPa comparing with sands that have around 10-50 MPa (Janbu 1985). Another feature of soft soils is that their oedomer stiffness has linear stress dependency. In a stress–stiffness curve a line of the form $E_{oed} = \sigma / \lambda^*$ can be plotted. This leads to the well known logarithmic compression law

$$\varepsilon - \varepsilon_0 = \lambda^* \cdot \ln\left(\frac{\sigma}{\sigma_0}\right) \qquad (4.8)$$

where the parameter λ^* is the modified compression index. Those considerations are valid for normal consolidated stress states and do not include secondary compression but since all soils exhibit some creep primary consolidation is always followed by some amount of secondary compression. If secondary compression is a small part of primary consolidation it is definitely understandable that creep plays an important role in problems involving large primary consolidation. (Neher & Wehnert and Bonnier 2001)

The SSC-model is an extension of the Soft Soil (SS) model that accounts for creep. The SS model is based on the modified Cam Clay model that use Mohr Coulomb criterion to describe failure. In the SS model a logarithmic relationship between the volumetric strain, ε_v, and the mean effective strain, p', is assumed. For virgin isotropic compression it can be written as

$$\varepsilon_v - \varepsilon_{v0} = \lambda^* \cdot \ln\left(\frac{p'}{p'_0}\right) \tag{4.9}$$

In situations involving isotropic unloading/reloading, the elastic volume strain is formulated as

$$\varepsilon_v^e - \varepsilon_{v0}^e = \kappa^* \cdot \ln\left(\frac{p'}{p'_0}\right) \tag{4.10}$$

The parameter κ^* is the modified swelling index that determines soil behaviour during unloading/reloading.

A constitutive law for creep was beginning to establish in 1930's when it was observed that soft soils settlements could not be explained by classic consolidation theory. Butterfield proposed a creep equation in 1979 of the form

$$\varepsilon^H = \varepsilon_C^H + \mu^* \cdot \ln\left(\frac{\tau_c + t'}{\tau_c}\right) \tag{4.11}$$

where ε_C^H is an expression for deformation during consolidation and μ^* is an expression for creep index describing secondary compression per logarithmic time increment. The parameters, τ_c and t', are time parameters. Consolidation and creep behaviour in standard oedometer test can be seen in Figure 4.8.

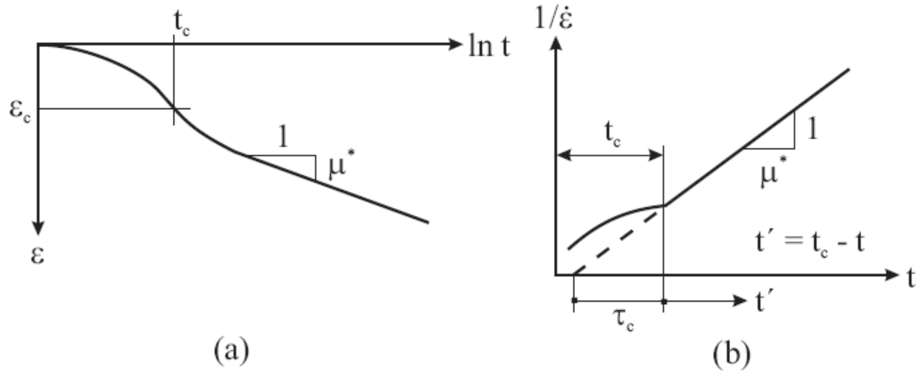

Figure 4.8. Consolidation and creep behaviour in oedometer tests which can be used to determine the creep index. (Brinkgreve 2006)

By combining equation 4.9, 4.10 and 4.11 the total volumetric strain in an isotropic stress state can be expressed as

$$\varepsilon_v = \kappa^* \cdot \ln\left(\frac{p'}{p'_0}\right) - (\lambda^* - \kappa^*) \cdot \ln\left(\frac{p'_{pc}}{p'_{p0}}\right) + \mu^* \cdot \ln\left(\frac{\tau_c + t'}{\tau_c}\right) \tag{4.12}$$

where ε_v is the total volumetric strain because of an increase of the mean effective stress from p'_0 to p' during a time period of $\tau_c + t'$. The parameters p'_{p0} and p'_{pc} represent the pre-consolidation pressure corresponding to before-loading and end-of-consolidation states respectively. The relation in equation 4.12 can also be seen in Figure 15. The Figure illustrates that the isotropic consolidation line (IC-line) is not reached after the end of the consolidation but after some creep has occurred.

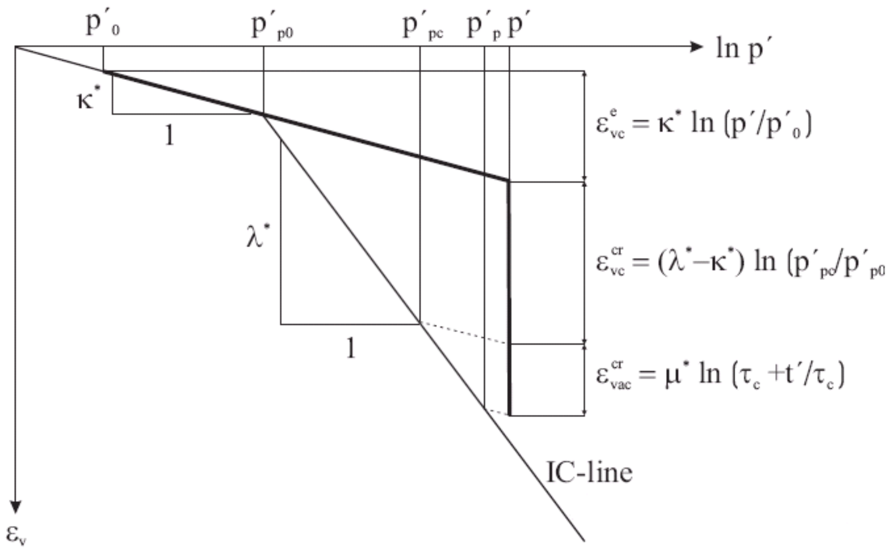

Figure 15. Figure showing the logarithmic relation between volumetric strain and mean effective stress. (Neher & Wehnert and Bonnier 2001)

The model parameters involved in the SSC model are failure parameters as in MC model (c, φ, ψ), basic stiffness parameters $(\kappa^*, \lambda^*, \mu^*)$ and advanced parameters with default values (ν_{ur}, K_0^{NC}, M). The stiffness parameters can have relation to both Cam Clay parameters, to A,B,C parameters and to internationally normalized parameters. The parameters A,B,C and C_r, C_c, C_a are also the swelling index, compression index and creep index respectively. The relations between those parameters are following:

- *Relation to Cam-Clay parameters*

$$\lambda^* = \frac{\lambda}{1+e}, \; \kappa^* = \frac{\lambda}{1+e}, \text{---} \tag{4.13}$$

- *Relation to A,B,C parameters*

$$\lambda^* = B + \kappa^*, \kappa^* = 2A \;, \; \mu^* = C... \tag{4.14}$$

- *Relation to internationally normalized parameters*

$$\lambda^* = \frac{C_c}{2.3(1+e)}, \; \kappa^* \approx \frac{2}{2.3}\frac{C_r}{1+e}, \; \mu^* = \frac{C_a}{2.3(1+e)} \tag{4.15}$$

For a rough estimate of the model parameters the ratio λ^* / μ^* is in the range 15 to 25 and the ratio λ^* / κ^* is in the range around 5 to 10.

The M -value from the advanced parameters can be seen in Figure 16. The Figure clarifies the yield surfaces of the SS model in the p’-q plane. The parameter M represents the slope of the so-called *critical state line*, which describes the stress states at post peak failure. The MC-line in the Figure is fixed but the cap (ellipse with the p_p^{eq}) can change due to primary compression.

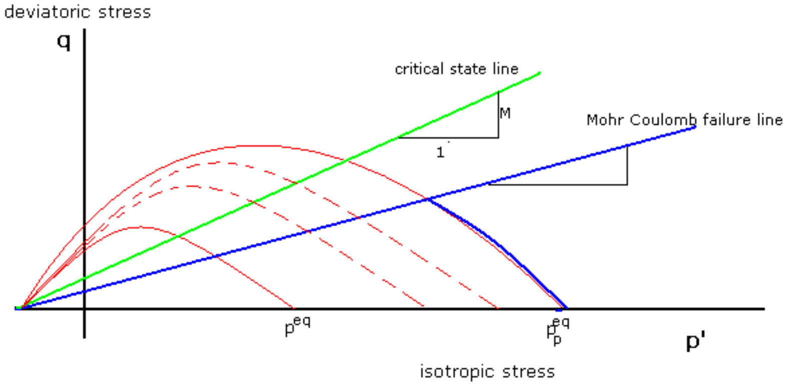

Figure 4.10. The yield surfaces of the SS-model in p'-q plane

The M-value can be calculated automatic by entering a value of K_0^{NC} since the relation between M and K_0^{NC} is

$$M = 3\sqrt{\frac{\left(1-K_0^{NC}\right)^2}{\left(1+2K_0^{NC}\right)^2} + \frac{\left(1-K_0^{NC)}\right)\left(1-2v_{ur}\right)\left(\lambda^*/\kappa^*-1\right)}{\left(1+2K_0^{NC}\right)\left(1-2v_{ur}\right)\lambda^*/\kappa^* - \left(1-K_0^{NC)}\right)\left(1+v_{ur}\right)}} \tag{4.16}$$

The Poisson's ratio (ν_{ur}) in SS model is an elasticity constant with a value in the range 0.1 to 0.2. The default value is 0.15.

4.7. Pre consolidation stress

In advanced material models such as the HS-model, the SS-model, and the SSC-model there are two ways to determine the preconsolidation stress, σ_p, which is important for computing the cap-type yield surface. Those are Over-Consolidation Ratio (*OCR*) and the Pre-Overburden Pressure (*POP*) and are formulated by;

$$OCR = \frac{\sigma_p}{\sigma_{yy}^{'0}} \tag{4.17}$$

$$POP = \left|\sigma_p - \sigma_{yy}^{'0}\right| \tag{4.18}$$

The parameter $\sigma_{yy}^{'0}$ is the in-situ effective vertical stress. When σ_p has been determined it can be used to compute p_p^{eq} which is the equivalent isotropic preconsolidation stress that determines the initial position of a cap-type yield surface in the advanced soil models. The comparison can be observed in Figure 4.11.

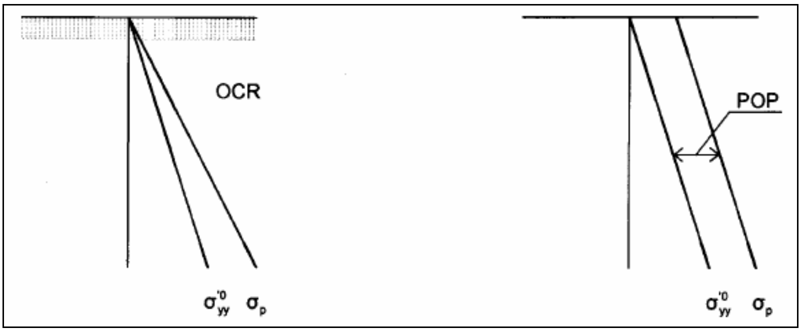

Figure 4.11. Vertical preconsolidation stress compared with the in-situ effective vertical stress. (Brinkgreve 2006)

4.8. Results from other work with Plaxis

Neher & Wehnert and Bonnier (2001) studied the Plaxis models SSC-model and SS-model on two test embankments in order to assess their performance. The first embankment was Boston trial Embankment which was built in Boston in 1965. The soil condition at the location consisted of sand underlying 41 m thick clay layer (Boston Blue Clay). Other soils in the analysis were fill and peat. The constitutive models that were applied on sand and fill were HS-model. The peat was computed with the MC-model. About half of the Boston Blue Clay (BBC) has an OCR-value of at least two, decreasing at the bottom of the layer to a value near one. The clay layers were divided into 12 sublayers. The chosen ratio between the modified soil parameters was $\lambda^* / \kappa^* = 5$ and $\lambda^* / \mu^* = 35$.

The result shows that the SSC-model matches the measurements better in the upper clay layers where the OCR-value is high. At the bottom of the clay profile however the SS-model reflects better the field data. This is underneath the central part of the fill. At the toe of the fill however the SS-model fits better the data all the way. The SS-model agrees better with the measurements especially at the bottom of the profile. Also the development of excess pore pressure has been done. The study shows that the calculated pore-water pressures are somewhat higher than those measured especially result from calculation with SSC-model gives high values of excess pore pressures.

The second embankment that was studied was the undrained test embankment at Skå-Edeby. The soft layers are divided into 9 sub-layers. The chosen ratio between the modified soil parameters is $\lambda^* / \kappa^* = 6$ and $\lambda^* / \mu^* = 15$. The results from the SSC-model and the measured data agree reasonably well. The settlements and also the excess pore pressures were strongly underestimated by the SS-model.

Another study (Indraratha et al, 2007) deals also with calculation by Plaxis for comparision with field data. In this study the SS-model and the SSC-model have been used to analyse the behaviour of settlements and excess pore pressures at Sunshine Embankment with and without prefabricated vertical drains. The embankment was located in Maroochy Shire, Queensland, Australia. The subsoil at this site consisted of very soft, highly compressible, saturated organic marine clays of high sensitivity. The soil profile was divided into three sublayers. The mesh discretization with 6-node triangular elements was used in Plaxis for calculations. The settlements calculated with both SS-model and SSC-model agree well with the field measurements. Same study compares the excess pore pressures between the field data and numerical analysis data. The study shows that results from SSC-model are much better than from the SS-model after about 50 days. The result is calculated at a point, 3 m below the surface and 1 m from the centreline.

Vepsäläinen et al (2003) studied the consolidation on the subsoil in the Haarajoki test embank-kment at Haarajoki, near Helsinki. The aim with the study was to compare measured and calculated settlements. The measured data was taken three years after the embankment was built. The subsoil consisted of 23 m thick, slightly overconsolidated clay underlying a layer of dry crust. Half of the embankment was built on vertical strip strains. The calculations in the part without vertical drains were done with Plaxis and for the part with vertical drains, the program SAGE-CRISP was used. In Plaxis the MC-model was used for the embankment and the dry crust. Soft clay layers under the dry crust were modelled with SSC-model. The calculations were made below the centre line of the embankment. The compatibility between the observed and calculated settlements in Plaxis was rather good after three years.

Chapter 5

Establish material parameters

The establishment of material parameters that will be used in the calculations will be presented in this chapter. The estimating of some parameters that are not specified but are also of importance will also be evaluated.

5.1. Test fill

5.1.1. Lilla Mellösa

The material used as embankment is gravel. The model used for this is the HS model because the problem is involving unloading/reloading, even if the distinction between HS and MC is less for friction soil than cohesion soil. (Johansson 2008). Furthermore it will be simulated as drained. The elastic module according to Bergdahl et al (1993) should be around 40 MPa assuming it is relative very firm. This value is corresponding to E_{50}^{ref} in HS model. Furthermore E_{50}^{ref} is equivalent to E_{ur}^{ref} ($E_{ur}^{ref} = 3E_{50}^{ref}$). To choose a relevant value for E_{oed}^{ref}, Plaxis have a built-in procedure for establishing a correct value of E_{oed}^{ref} with regard to other parameters in the model. The best way to go is to put in the values of E_{ur}^{ref} and E_{50}^{ref} in a first step and thereafter put in the number of E_{oed}^{ref}. Giving a small value of E_{oed}^{ref} will give a warning suggesting "*Use Eoed>.1*E50"* and a too high value of E_{oed}^{ref} will give another warning. In that way a correct value of *Eoed* according to Plaxis can be chosen. The established values will be $E_{ur}^{ref} = 120MPa$ and E_{oed}^{ref} =35MPa. The Poisson's ratio, suggested by Plaxis manual will have values in the range between 0.15 and 0.25 in problems involving unloading. The value of 0.20 will be chosen. The friction angle will be around 37 degrees for gravel (Bergdahl et al 1993). Dilatancy will be 7 (ψ =φ-30). The cohesion will be 0.5 for avoiding numerical failures (correctly c=0 for friction soil). According to Larsson (2008) the fill has a unit weight of 18 kN/m³. Permeability for gravel is around 0.01 to 1 m/day. (Axelsson 2005)

5.1.2. Skå-Edeby

The same parameters and model type that has been used at Lilla Mellösa for test fills will also be used for modelling the test fills at Skå-Edeby.

5.2. Dry Crust

5.2.1. Lilla Mellösa

The values to describe the deformation properties of the dry crust are difficult to find, giving the ability to assume from geotechnical experience (Johansson 2008). The dry crust has a thickness of 0.5 m and consists largely of organic soils. The unit weight presumes 15 kN/m³ (unsaturated) and 17 kN/m³ (saturated). Permeability assumed to be $1.18 \cdot 10^{-4}$ m/day (Johansson 2008). The value of E_{50}^{ref} assumes to be 5 MPa (and $E_{ur}^{ref} \approx 3 \cdot 5 = 15$ MPa) giving us to choose E_{oed}^{ref} =4.6MPa. The select of E_{50}^{ref} =5MPa is because soft soils have mostly the elastic module less than 10 MPa. Poisson's ratio presumed to be 0.20. The friction angle will be 30 and cohesion will be 1 kPa.

5.2.2. Skå Edeby

Also the parameters that have been chosen for the dry crust at Lilla Mellösa will be used at Skå-Edeby because there are very small distinctions between the two sites when it comes to the dry crust.

5.3. Clay layers

5.3.1. Lilla Mellösa

Because the simulation problem is involved with secondary compression, the SSC model will be used for clay layers. For normal consolidated clay the friction angle is 30 degrees and the cohesion is assumed to be 4 kPa. The permeability varies between $5.18 \cdot 10^{-5}$ m/day and $8.64 \cdot 10^{-5}$ m/day. The unit weight varies between 13 and 18 kN/m³. To estimate the stiffness index $\kappa^*, \lambda^*, \mu^*$ we have two opportunities to determine the index parameters. The first option is to choose evaluated experimental parameters according the Norrkoping clay that has been evaluated by Westerberg (1999). The compression index λ according to Westerberg (1999) is between [0.70-0.80] in isotropic experiment and [0.5-1.0] in oedometer experiment and the swelling index κ has values of [0.04-0.07]. Those are Cam Clay parameters that have to be re-calculated according to the SSC model parameters. The second alternative to determine the index parameters is that we know the compression index (C_c) at Lilla Mellösa. Larsson (2007) says that the compression index is 2.3 in the upper and 1.3 in the bottom layers. This gives us two ways to estimate the stiffness index for SSC model. The first one with relation to the Cam-Clay parameters and the second one with relation to internationally normalised parameters. However we don't know the values of C_r and C_a. But we can estimate those roughly because the relation λ^* / κ^* is in interval 5-10 and λ^* / μ^* is in the interval 15-25. By calculating λ^* from C_c we can in the next step calculate κ^* and μ^* and those values can be compared with the values according to Norrkoping clay. Another parameter that has to be known is the void ratio, e. According to Axelsson (2005) and Coduto (1999) the void ratio varies in the interval [0.7–3.0] for clay layers where the high values of e are equivalent to a high water content (around 120-150%) and the low values of e are equivalent to a lower water content (around 50-70%). The soil condition at Lilla Mellösa has water content around 130% in the upper layers and 70% in the bottom layers. The relation to Cam clay parameters has been calculated first and those parameters with relation to normalised parameters have been used for comparison. The results show that there are very small distinctions between the two types of parameters. The comparison to normalised parameters was made at the site at Lilla Mellösa. Because the difference between those two alternatives was small only the relation to the Cam Clay parameters was made in the remaining sites.

The established parameters at Lilla Mellösa can be seen in following tables:

Vertical drains

Relation to Cam-Clay					
Layer no.	layer4	layer5	layer6	layer7	layer8
material	org. clay1	org. clay2	clay1	clay2	varved clay
water content,w(%)	130	110	100	80	70
void ratio,e(-)	3,0	3,0	2,8	2,5	2,2
compression index,Cc	2,3	2,1	1,8	1,5	1,3
$\lambda^*=Cc/(2.3+e)$	**0,26**	**0,23**	**0,21**	**0,19**	**0,18**
$\kappa^*=\lambda^*/10$	**0,026**	**0,023**	**0,021**	**0,019**	**0,018**
$\mu^*=\lambda^*/15$	**0,017**	**0,015**	**0,014**	**0,013**	**0,012**

Relation to normalised parameters					
Layer no.	layer4	layer5	layer6	layer7	layer8
material	org. clay1	org. clay2	clay1	clay2	varved clay
water content,w(%)	130	110	100	80	70
void ratio,e(-)	3,0	3,0	2,8	2,5	2,2
λ	1,00	0,90	0,80	0,63	0,57
κ	0,07	0,07	0,07	0,07	0,07
$\lambda^*=\lambda/(1+e)$	**0,25**	**0,23**	**0,21**	**0,18**	**0,18**
$\kappa^*=\kappa/(1+e)$	**0,018**	**0,018**	**0,018**	**0,020**	**0,022**
$\mu^*=\lambda^*/15$	**0,017**	**0,015**	**0,014**	**0,012**	**0,012**

Without vertical drains

Relation to Cam-Clay					
Layer no.	layer4	layer5	layer6	layer7	layer8
material	org. clay1	org. clay2	clay1	clay2	varved clay
water content,w(%)	130	110	100	80	70
void ratio,e(-)	3,0	3,0	2,8	2,5	2,2
compression index,Cc	2,3	2,1	1,8	1,5	1,3
$\lambda^*=Cc/(2.3+e)$	**0,26**	**0,23**	**0,21**	**0,19**	**0,18**
$\kappa^*=\lambda^*/10$	**0,026**	**0,023**	**0,021**	**0,019**	**0,018**
$\mu^*=\lambda^*/25$	**0,010**	**0,009**	**0,008**	**0,008**	**0,007**

5.3.2. Skå-Edeby

The permeability at Skå-Edeby is decreasing from the $8.64 \cdot 10^{-5}$ m/day at the top of the profile to $4.32 \cdot 10^{-5}$ m/day at the bottom. The unit weight varies from 13 kN/m³ at top to 18 kN/m³at 12 meters depth below ground surface. The compression index is 1.5 in the upper and 1.0 in the bottom layers. The water content decreases from around 100% in the upper part to around 60% in the lower part of the soil model. However the test fills are located with some distance from each other (Figure 3.4) which means that they can have very different material parameters. The calculated stiffness parameters are:

Area 1,2,5 (Vertical drained behaviour)

Layer no.	layer3	layer4	layer5
material	org. Clay	varved clay1	varved clay2
water content,w(%)	100	80	60
void ratio,e(-)	2,8	2,3	1,5
compression index,Cc	1,5	1,3	1,1
λ*=Cc/(2.3x(1+e))	**0,17**	**0,17**	**0,19**
κ*=λ*/10	**0,017**	**0,017**	**0,019**
μ*=λ*/15	**0,011**	**0,011**	**0,013**

Area 3 (Vertical drained behaviour with unloading)

Layer no.	layer3	layer4	layer5
material	org. Clay	varved clay1	varved clay2
water content,w(%)	70	70	60
void ratio,e(-)	1,8	1,8	1,5
compression index,Cc	1,5	1,4	1,3
λ*=Cc/(2.3x(1+e))	**0,23**	**0,22**	**0,23**
κ*=λ*/10	**0,023**	**0,022**	**0,023**
μ*=λ*/15	**0,016**	**0,014**	**0,015**

Area 4 (Undrained behaviour)

Layer no.	layer3	layer4	layer5
material	org. Clay	varved clay1	varved clay2
water content,w(%)	70	70	60
void ratio,e(-)	1,8	1,8	1,5
compression index,Cc	1,5	1,4	1,3
λ*=Cc/(2.3x(1+e))	**0,23**	**0,22**	**0,23**
κ*=λ*/10	**0,023**	**0,022**	**0,023**
μ*=λ*/25	**0,009**	**0,009**	**0,009**

5.4. Undrained behaviour

In Plaxis there is the possibility to simulate an undrained behaviour with effective model parameters in an effective stress analysis. This can be done because Plaxis can transform the effective parameters G and v into undrained parameters E_u and v_u. This can be seen in the equations 5.1 to 5.4.

$$E_u = 2G(1+v_u) \tag{5.1}$$

$$v_u = \frac{v' + \mu(1+v')}{1+2\mu(1+v')} \tag{5.2}$$

$$\mu = \frac{1}{3 \cdot n} \frac{K_w}{K'} \tag{5.3}$$

$$K' = \frac{E'}{3(1-2v')} \tag{5.4}$$

The undrained Poisson's ratio v_u must be 0.5, but since this is not possible because of singularity of the stiffness matrix the value of v_u is by default set to 0.495 by Plaxis. Plaxis suggests also that the bulk modulus of the water (K_w) must be higher than the bulk modulus of the soil skeleton (K') to have realistic results. The $K_w \geq K'$ relation can be guaranteed by having $v' \leq 0.35$. A warning will turn up if a too high value of Poisson's ratio has been chosen.

The use of an undrained behaviour with effective material parameters is available for all material models that are present in Plaxis and such a combination gives a clear distinction between effective stresses and excess pore pressure in the output part after calculation. In general the effective soil parameters are not always available, especially soft soils. This can be solved by using obtained undrained soil parameters from in situ tests and laboratory tests and thereafter convert the measured undrained Young's moduli into effective Young's moduli by the equation:

$$E' = \frac{2(1+v')}{3} E_u \tag{5.5}$$

This works only for the MC model and the HS models. For other advanced models no such direct conversion is possible. In that case the required effective stiffness parameter can be estimated from the measured undrained stiffness parameter.

Plaxis introduces three ways of modelling undrained behaviour. Those will be called Method A, B and C respectively hereafter. The first one (method A) is the most recommended and the soil behaviour is ruled by effective stresses. Method A can be used for calculating advanced models such as the HS model, SSC model and SS model. The method A is analysed in terms of *effective stresses* and the *Material type* has to be set to U*ndrained* in Plaxis. By selecting this way of analysis, the effective strength parameters c', φ', ψ' and the effective stiffness parameters E'_{50}, v' will be used. The advantage of this method is that PLAXIS distinguishes between effective stresses and (excess) pore pressures.

The second method (method B) is also analysed in terms of effective stresses and can be used when no information on effective strength parameters is available. This method works as an undrained effective stress analysis (*Material type* = *Undrained*) with direct input of the undrained shear strength, i.e. $c = c_u, \varphi = \varphi_u = 0, \psi = 0$. The stiffness parameters will also be entered as $E_{50}^{'}, v^{'}$ because whenever the *Material type* parameter is set to *Undrained*, the effective values of stiffness parameters must be entered. This method works best for MC- model and can not be used with the SS- model and the SSC- model.

The third method (Method C) to model undrained behaviour is analysed in terms of total stresses. This method can be selected when it is not desired to use *Undrained* option in Plaxis to perform an undrained analysis. Instead *Material type* will be set to *Drained* and the stiffness will be modelled by selecting an undrained Young's modulus E_u and an undrained Poisson's ratio v_u, and strength will be modelled by using an undrained shear strength. The disadvantage of this method is that no information on excess pore pressure distribution will be given. This means that only a total stress analysis will be given and a change in loading will not give a change in excess pore pressure. A consolidation analysis with this method is of that reason not possible.

	Undrained behaviour	
Method A	**Method B**	**Method C**
Recommended	-	Not recommended
works with advanced models	can not work with SS- and SSC-model	works with MC-model only
Effective strength parameters c',φ',Ψ'	**Undrained** strength parameters c=cu,φ=0,Ψ=0	**Total** strength parameters c=cu,φ=0,Ψ=0
Effective stiffness parameters E50',v'	**Effective** stiffness parameters E50',v'	**Undrained** stiffness parameters Eu,Vu=0,495
Undrained behaviour	Undrained behaviour	Drained behaviour !!

Table 5.1. A Compendious table over the different methods for analyzing undrained behaviour.

5.4.1. Lilla Mellösa and Skå-Edeby

The model that will be used in the undrained behaviour is Method A because this model works well for advanced material models and secondly because we are also interested in distinction between effective stresses and excess pore pressures.

5.5. Established parameters

5.5.1. Lilla Mellösa

The established material parameters for embankments at Lilla Mellösa are given in Table 5.2, and 5.3, respectively.

With vertical drains

Layer no.	layer1	layer2	layer3	layer4	layer5	layer6	layer7	layer8	layer9
material	gravel	loading	dry crust	org. clay1	org. clay2	clay1	clay2	varved clay	sand
model	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain
elements	15-node	15-node	15-node	15-node	15-node	15-node	15-node	15-node	15-node
mtrl model	HS	HS	HS	SSC	SSC	SSC	SSC	SSC	HS
type	drained	drained	drained	drained	drained	drained	drained	drained	drained
γunsat(kN/m³)	18	15	15	13	14	16	18	19	18
γsat(kN/m³)	21	17	17	15	16	18	20	22	21
kx(m/day	0,01	0,01	1,18E-04	8,60E-05	1,70E-04	2,60E-04	3,45E-04	4,55E-04	5,00E-01
ky(m/day)	0,01	0,01	1,18E-04	8,60E-05	1,70E-04	2,60E-04	3,45E-04	4,55E-04	5,00E-01
cref	0,025	0,025	4	4	4	4	4	4	0,025
phi	37	35	30	30	30	30	30	30	40
psi	7	5	0	0	0	0	0	0	10
lambda(λ*)	-	-	-	0,250	0,220	0,210	0,190	0,180	-
kappa(κ*)	-	-	-	0,050	0,044	0,042	0,038	0,036	-
mu(μ*)	-	-	-	1,70E-02	1,50E-02	1,40E-02	1,30E-02	1,20E-02	-
Vur	-	-	-	0,15	0,15	0,15	0,15	0,15	-
K0nc				0,569	0,635	0,635	0,635	0,635	-
Eref(kN/m²)	-	-	-	-	-	-	-	-	-
v	-	-	-	-	-	-	-	-	-
E50ref(kN/m²)	40000	32000	5000	-	-	-	-	-	25000
Eoedref(kN/m²)	35000	25000	4555	-	-	-	-	-	20000
Eurref(kN/m²)	120000	98000	15000	-	-	-	-	-	75000
m	0,5	0,5	0,5	-	-	-	-	-	-
strength	rigid	rigid	rigid	rigid	rigid	rigid	rigid	rigid	rigid

Table 5.2. Established parameters at Lilla Mellösa with vertical drains.

Without vertical drains

The material parameters at the embankment without vertical drains have the same material parameters as the one with vertical drains except for the stiffness parameters κ^* and μ^*.

Layer no.	layer1	layer2	layer3	layer4	layer5	layer6	layer7	layer8	layer9
material	gravel	loading	dry crust	org. clay1	org. clay2	clay1	clay2	varved clay	sand
model	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain
elements	15-node	15-node	15-node	15-node	15-node	15-node	15-node	15-node	15-node
mtrl model	HS	HS	HS	SSC	SSC	SSC	SSC	SSC	HS
type	drained	drained	drained	drained	drained	drained	drained	drained	drained
lambda(λ*)	-	-	-	0,250	0,228	0,206	0,186	0,177	-
kappa(κ*)	-	-	-	0,025	0,023	0,021	0,019	0,018	-
mu(μ*)	-	-	-	1,00E-02	9,13E-03	8,24E-03	7,45E-03	7,07E-03	-
Vur	-	-	-	0,15	0,15	0,15	0,15	0,15	-
K0nc				0,569	0,635	0,635	0,635	0,635	-

Table 5.3. Established parameters at Lilla Mellösa without vertical drains.

5.5.2. Skå-Edeby

The established material parameters for embankments at Skå-Edeby are given in Table 5.4, 5.5, and 5.6, respectively.

Vertical drained behaviour

Layer no.	layer1	layer2	layer3	layer4	layer5	layer6	layer7
material	gravel	loading	dry crust	org. clay1	varv. clay 1	varv. clay 2	sand
model	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain
elements	15-node	15-node	15-node	15-node	15-node	15-node	15-node
mtrl model	MC	MC	MC	SSC	SSC	SSC	MC
type	drained	drained	drained	drained	drained	drained	drained
γunsat(kN/m	18	15	15	13	14	16	18
γsat(kN/m³)	21	17	17	15	16	18	21
kx(m/day)	0,01	0,01	1,18E-04	8,60E-05	7,03E-04	6,20E-04	5,00E-02
ky(m/day)	0,01	0,01	1,18E-04	8,60E-05	7,03E-04	6,20E-04	5,00E-02
cref	0,025	0,025	4	4	4	4	0,025
phi	37	35	30	30	30	30	40
psi	7	5	0	0	0	0	10
lambda(λ*)	-	-	-	0,172	0,171	0,191	-
kappa(κ*)	-	-	-	0,017	0,017	0,019	-
mu(μ*)	-	-	-	1,14E-02	1,14E-02	1,28E-02	-
νur	-	-	-	0,15	0,15	0,15	-
K0nc				0,635	0,635	0,635	-
Eref(kN/m²)	40000	32000	8000	-	-	-	25000
ν	0,35	0,35	0,35	-	-	-	0,35
strength	rigid	rigid	rigid	rigid	rigid	rigid	rigid

Table 5.4. Established parameters at Skå-Edeby with vertical drains

Vertical drained behaviour with unloading

Table 5.4 is also valid for the test Area 3 except for the stiffness parameters which are presented in table 5.5.

Layer no.	layer1	layer2	layer3	layer4	layer5	layer6	layer7
material	gravel	loading	dry crust	org. clay1	varv. clay 1	varv. clay 2	sand
model	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain
elements	15-node	15-node	15-node	15-node	15-node	15-node	15-node
mtrl model	HS	HS	HS	SSC	SSC	SSC	HS
type	drained	drained	drained	drained	drained	drained	drained
lambda(λ*)	-	-	-	0,233	0,217	0,226	-
kappa(κ*)	-	-	-	0,023	0,022	0,023	-
mu(μ*)	-	-	-	1,55E-02	1,45E-02	1,51E-02	-
νur	-	-	-	0,15	0,15	0,15	-
K0nc				0,635	0,635	0,635	-
Eref(kN/m²)	-	-	-	-	-	-	-
ν	-	-	-	-	-	-	-
E50ref(kN/m	40000	32000	5000	-	-	-	25000
Eoedref(kN/	35000	25000	4555	-	-	-	20000
Eurref(kN/m	120000	98000	15000	-	-	-	75000
m	0,5	0,5	0,5	-	-	-	-
strength	rigid	rigid	rigid	rigid	rigid	rigid	rigid

Table 5.5. Established parameters at Skå-Edeby with vertical drains.

Undrained behaviour

The established material parameters corresponding to the embankment without vertical drains are presented in Table 5.6.

Layer no.	layer1	layer2	layer3	layer4	layer5	layer6	layer7
material	gravel	loading	dry crust	org. clay1	varv. clay 1	varv. clay 2	sand
model	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain	plain strain
elements	15-node	15-node	15-node	15-node	15-node	15-node	15-node
mtrl model	MC	MC	MC	SSC	SSC	SSC	MC
type	undrained	undrained	undrained	undrained	undrained	undrained	undrained
lambda(λ*)	-	-	-	0,233	0,217	0,226	-
kappa(κ*)	-	-	-	0,023	0,022	0,023	-
mu(μ*)	-	-	-	9,32E-03	8,70E-03	9,04E-03	-
vur	-	-	-	0,15	0,15	0,15	-
K0nc				0,635	0,635	0,635	-
Eref(kN/m²)	40000	32000	8000	-	-	-	25000
v	0,35	0,35	0,35	-	-	-	0,35
strength	rigid	rigid	rigid	rigid	rigid	rigid	rigid

Table 5.6. Established parameters at Skå-Edeby without vertical drains.

5.6. Calculation stages

5.6.1. Mesh generation

The mesh generated can be seen in Figure 5.1. The user has the option of choosing 6-node or 15-node triangular elements. We have chosen 15-nodes elements in all calculations because there are chances to get disadvantage values with 6-node elements. The *global coarseness* will be chosen to *very fine.* Further some clusters, especially those containing soft soils have been refined locally for the sake of having meshes that are accurate enough to produce acceptable numerical solutions.

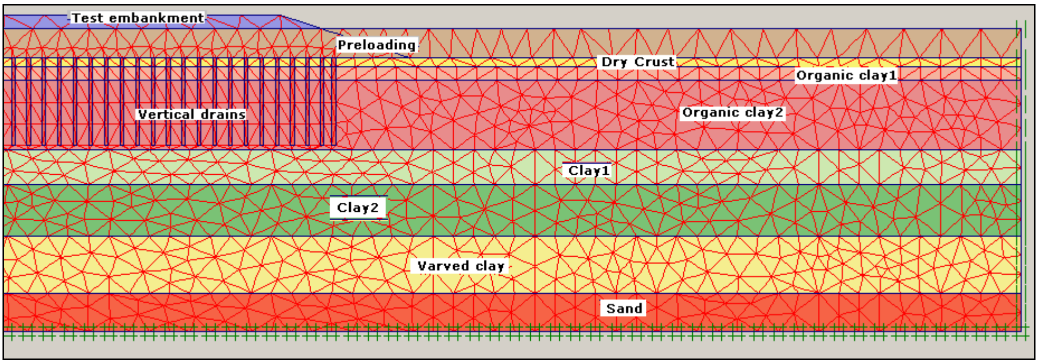

Figure 5.1. 15-node mesh generation of the drained test fill at Lilla Mellösa

5.6.2. Generated initial pore pressures

The phreatic level (groundwater table) is 0.5 m below the surface. Over the groundwater table the pore pressures are zero and from the groundwater table the pore pressure variation is assumed to be hydrostatic i.e. the water pressure will increase linearly with depth according to the specified water weight. The highest pore pressures will therefore be seen in the lower parts.

5.6.3. Generated initial effective stresses

The initial effective stresses in a soil body are caused by the uppermost soil layers. The stresses will increase with the depth. The stress state is characterised by an initial vertical effective stress, $\sigma'_{v,0}$ and by an effective horizontal stress, $\sigma'_{h,0}$. The initial vertical effective stress is calculated by

$$\sigma'_{v,0} = \sum_i^n \gamma_i \cdot h_i - p_w \tag{5.6}$$

where γ_i is the unit weight of individual layers, h_i is the layer depth and p_w is the initial pore pressures at the stress point. The initial horizontal effective stress is related to $\sigma'_{v,0}$ by the coefficient of lateral earth pressure, K_0 and are calculated as

$$\sigma'_{h,0} = K_0 \cdot \sigma'_{v,0} \tag{5.7}$$

where the default K_0 -value is based on the Jaky's formula:

$$K_0 = 1 - \sin\varphi \tag{5.8}$$

5.6.4. Consolidation phases

The consolidation phases can be illustrated as in Figure 5.2. The phase 1 in the consolidation analysis was to unload the loading we had in the beginning. The reason we applied a loading was because the result gave a huge amount of displacement during the first 25 days when embankment was installed. The value of displacement during the first 25 days was around 400-600 mm. To decrease this value we applied as mentioned a loading and also preceded the analysis as an undrained analysis in the beginning. After the incorporation of those two changes, the displacement decreased to around 100 mm during the first 25 days. The incorporation of those two modifications can be understandable because in reality the soil must have had a small settlement because the upper part (dry crust) is heavier than the lower part. The model has as far been modelled as undrained but it will be designed as drained from phase 3 and further. The change of model behaviour can also be seen in the colour change of the model. This is only adopted in the test embankments with vertical drains. The Figure exemplifies the calculation stages at test embankments with vertical drains that have a phase of unloading after some time.

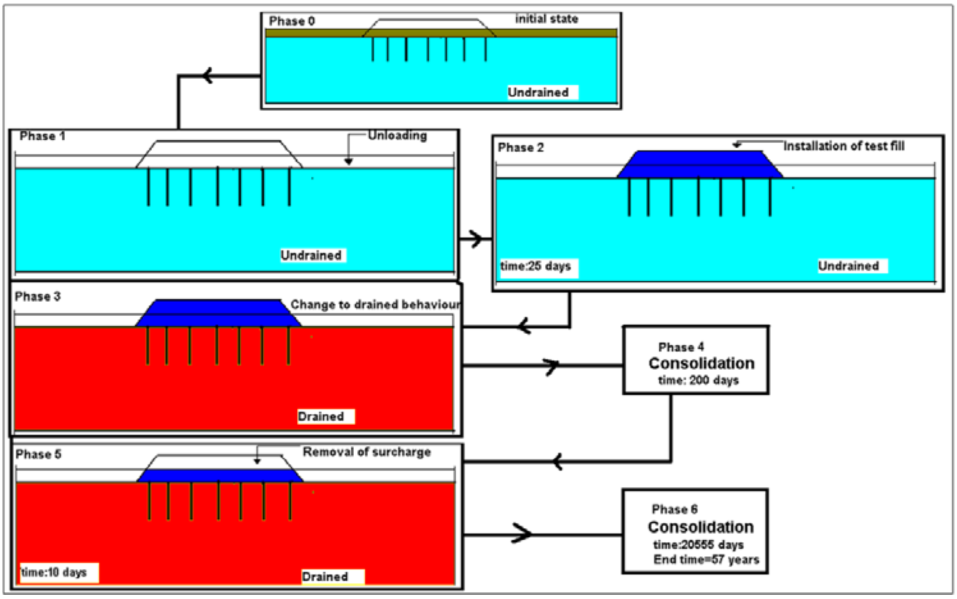

Figure 5.2. Calculation phases at embankment with vertical drains.

Chapter 6

Obtained results

6.1. Results obtained at Lilla Mellösa

6.1.1. Drained behaviour

The result from the test field at Lilla Mellösa can be seen in Figure 6.1. The solid lines indicate the values that have been received from analysis with Plaxis and the dashed line represents the result that has been obtained from field test. In 2002 the settlement is around 1.6 meters and there is no indication of settlement ending.

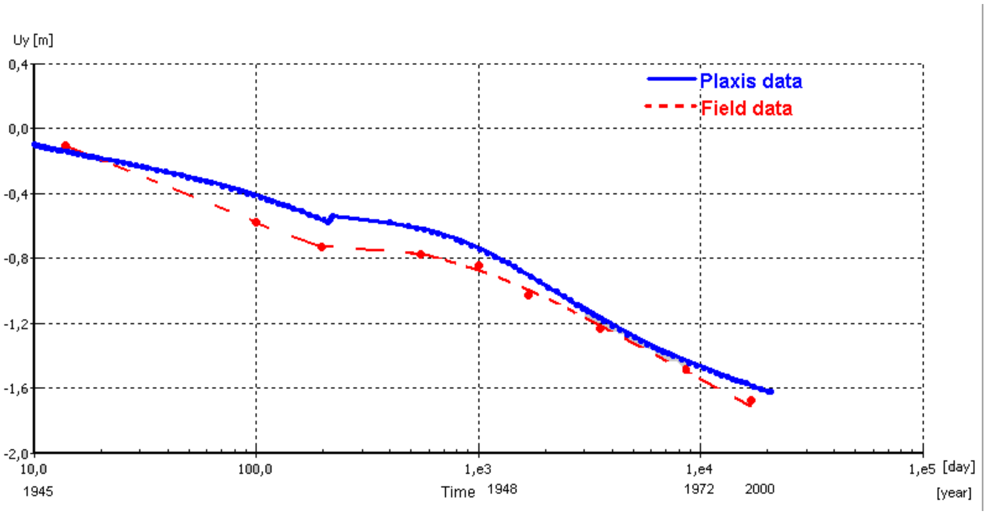

Figure 6.1. Comparison between settlements obtained from Plaxis and field data under the drained test fill at Lilla Mellösa, time in logarithmic scale.

6.1.2. Undrained behaviour

The result of the settlement in relation with logarithm time for the test fill without vertical drains can be seen in Figure 6.2. As before both the curve from field data and from established parameters are drawn in same diagram for comparison. The Figure 6.3 illustrates that the excess pore pressure from the field is decreasing from around 30kPa in 1966 to 10kPa in 2002. Same Figure spotlights that the excess pore pressure is decreasing from 4.0 kPa in 1966 to 0.9 kPa in 2002 from calculations with Plaxis. Another comparison between field data and result from Plaxis is Figure 6.4 where the diagramme is showing settlement in relation to linear time. Calculations have been done at depth 0 m, 2.5 m, 5 m, and 7.5 m in the centre line below the test embankment. The aim was to see if Plaxis results agree well with measured data at different depth.

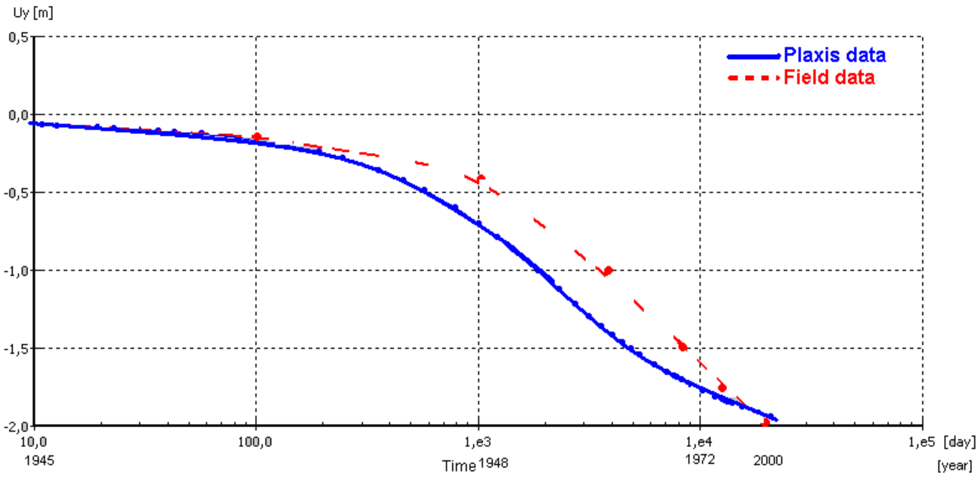

Figure 6.2. Comparison between settlements obtained from Plaxis and field data under the undrained test fill at Lilla Mellösa.

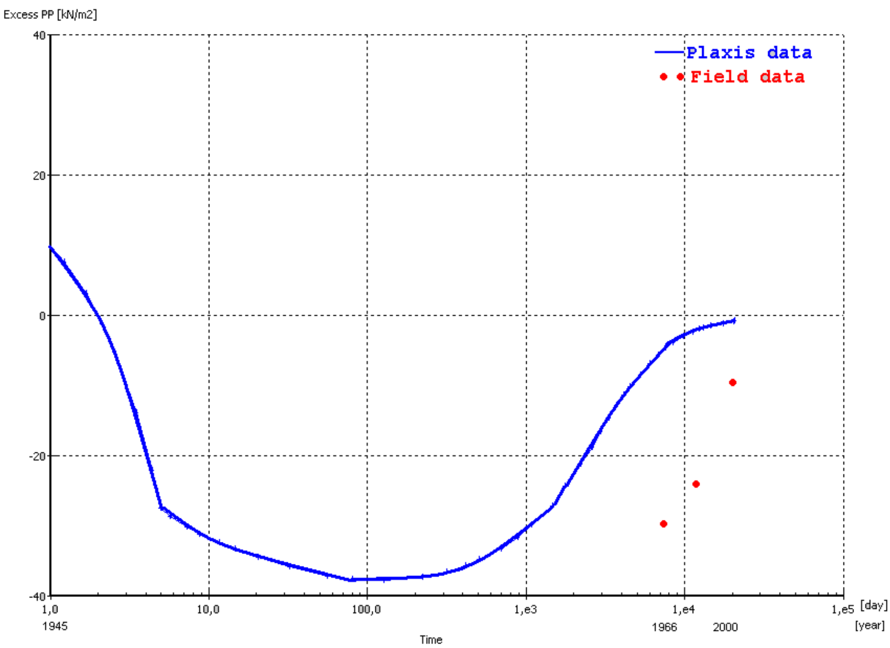

Figure 6.3. Compatibility between excess pore pressure from Plaxis and field under the undrained test fill at Lilla Mellösa.

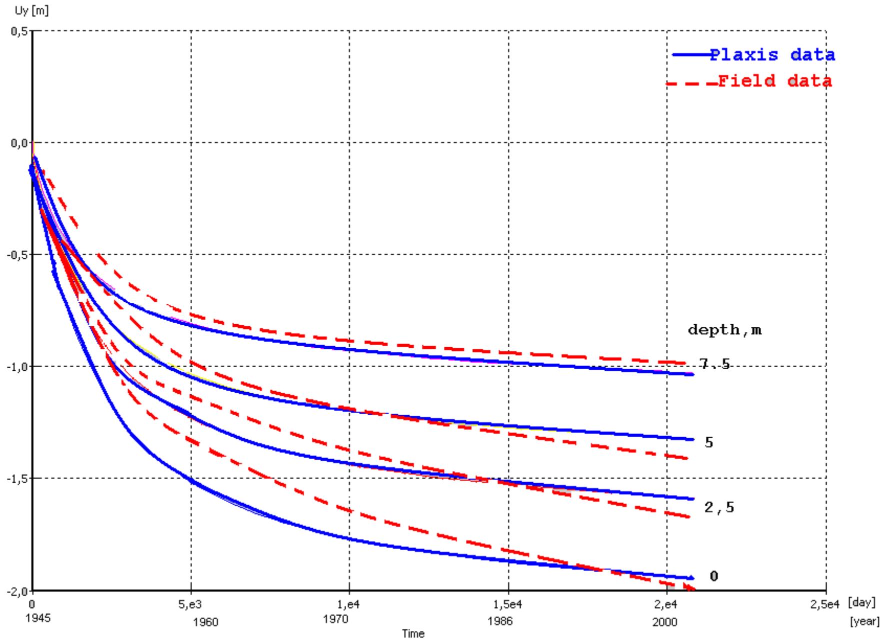

Figure 6.4. Measured settlements below the undrained test fill at Lilla Mellösa, time in linear scale.

6.2. Obtained result at Skå-Edeby

6.2.1. Area 1, 2, 5 (vertical drained)

The result from areas with vertical drains installed can be seen in Figure 6.5, 6.6, and 6.7 corresponding to Area 1, 2, and 5, respectively. The calculation of Area 1 has been made of the part with centre distance of 2.2 m. The calculation of other parts (centre distance 0.9 m and 1.5 m) has not been made. The result of area with centre distance 2.2 m is shown in Figure 6.5. The settlement distribution below the Area 2 in comparison with time is shown in Figure 6.6. The established curves are much similar with curves received from Area 1.

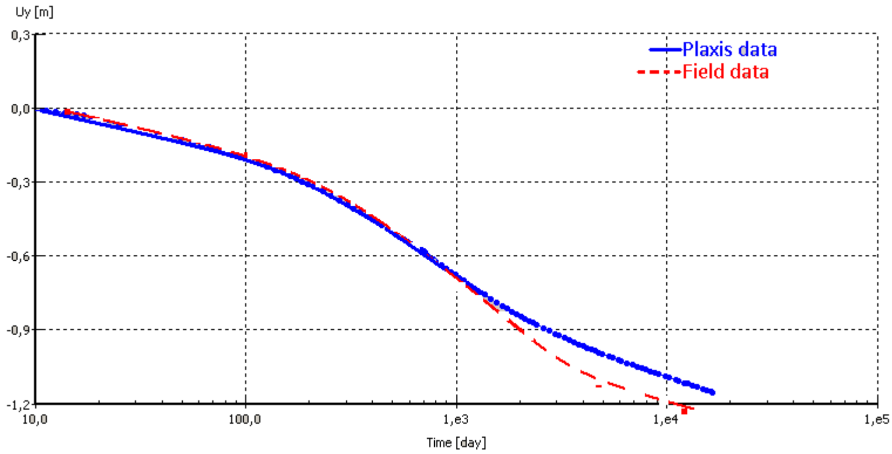

Figure 6.5. Measured settlements under the circular fill with vertical drains at Skå-Edeby, Area 1, time in logarithmic scale.

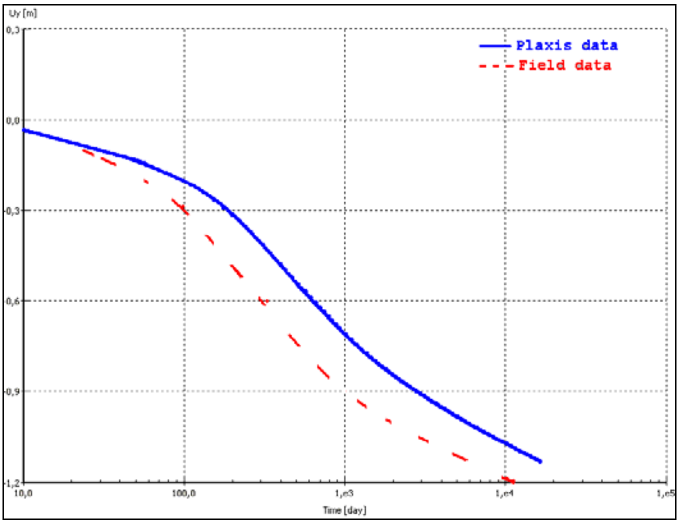

Figure 6.6. Measured settlements below Area 2 at Skå-Edeby, time in logarithmic scale.

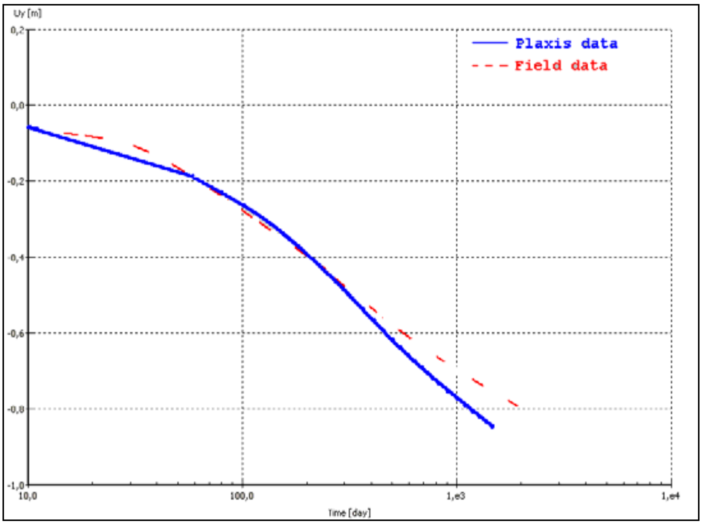

Figure 6.7. Measured settlements below the area 5 at Skå-Edeby, time in logarithmic scale.

6.2.2. Area 3 (vertical drain)

The result from Area 3 is shown in Figure 6.8. The curve established from Plaxis follows the same path as the one from field test. Both curves do get a temporally heave after removal of the upper part (0.7m) of the test fill. This happens around 4 years after the embankment was installed. The heave has a value of around 8 mm and 30 mm in measured respective Plaxis.

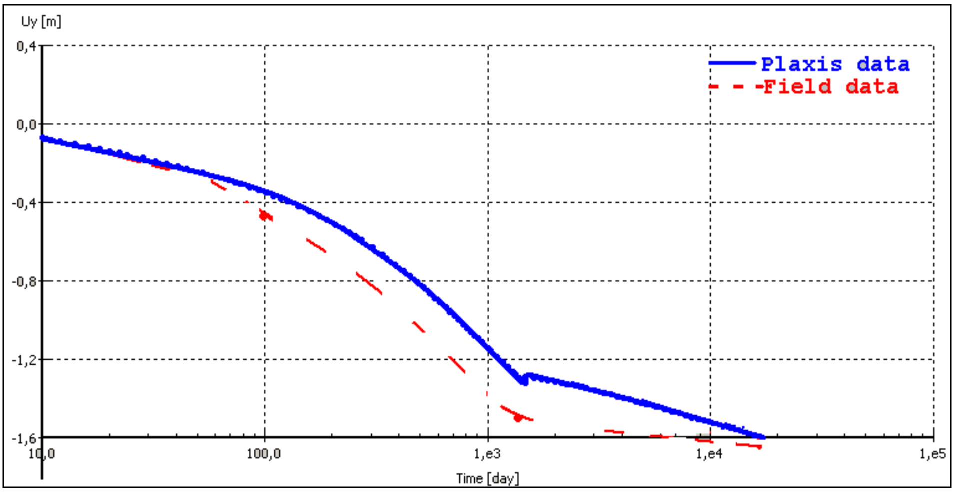

Figure 6.8. Measured settlements below Area 3 at Skå-Edeby, time in logarithmic scale.

6.2.3. Area 4 (undrained)

Figure 6.9 are showing the settlements received from the undrained test fill (Area 4). Analysis and comparison has been made at depth of 0, 2.5, 5, and 7.5 m below the central line of the embankment.

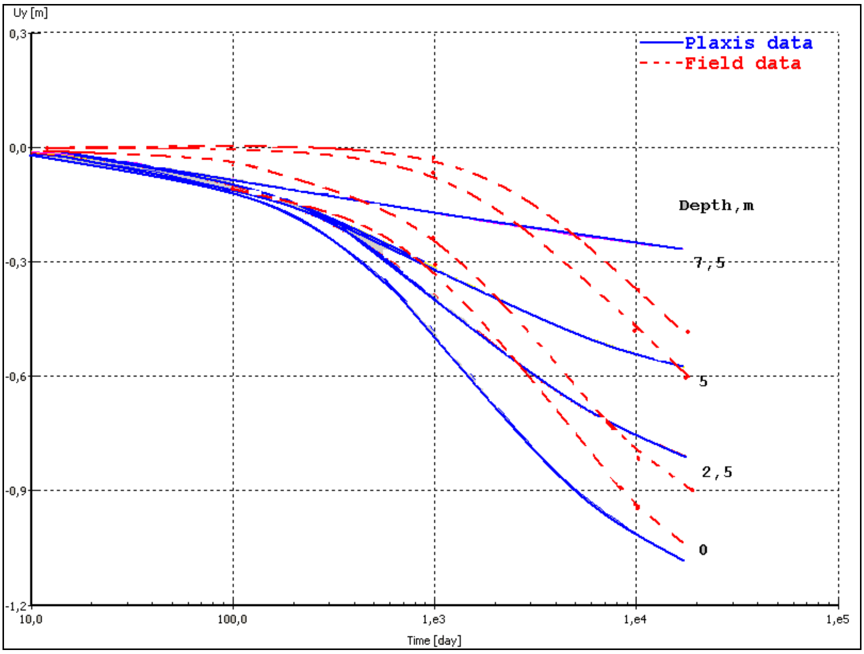

Figure 6.9. Measured settlements below the undrained area at Skå-Edeby, time in logarithmic scale.

Figure 6.10 clarifies the measured and calculated excess pore pressures at the undrained test embankment at Skå-Edeby. The excess pore pressures were measured at three different occasions only, as can be seen in the figure below.

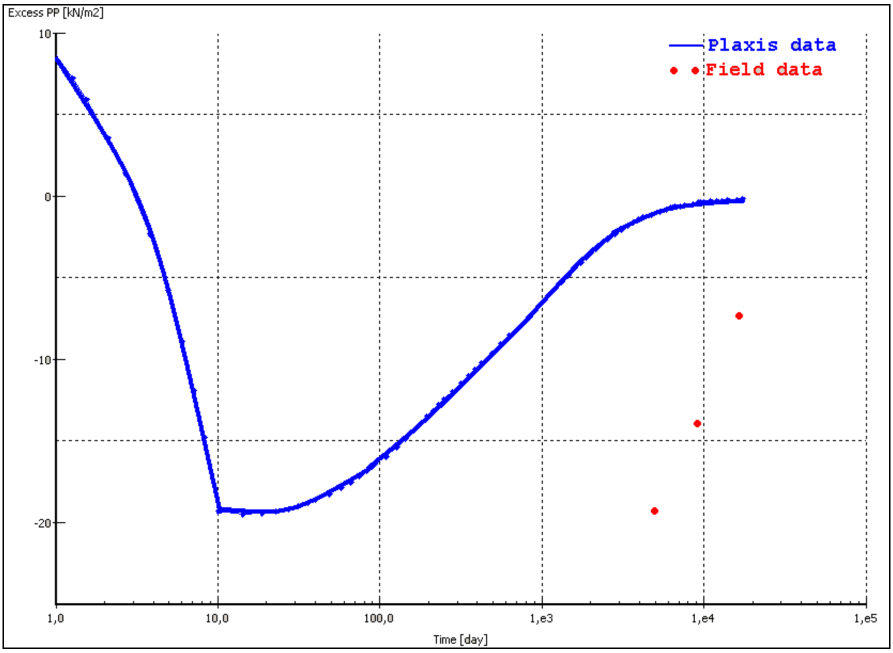

Figure 6.10. Compatibility between excess pore pressure from Plaxis and field under the undrained test fill at Skå-Edeby.

6.3. Comparison between constitutive models

6.3.1. Hardening Soil and Mohr-Coulomb

A comparison between the HS model and the MC model was made at Area 3 at Skå-Edeby. The experiment at this area involved unloading after some time. The purpose of this comparison was to see if there was any difference between those two models regarding having a stiffness module for unloading. The comparison is illustrated in Figure 6.11. The dashed line indicates results from the HS-model and dotted line represents the MC-model. The comparison was made from the surface and 5 m beneath the surface. The change of model did occur in the friction soils only, to see if the calculation gave different result. The soft soil layers had the same model (SSC-model) while sand, fill, dry crust and preloading clusters had MC-model and HS-model respectively.

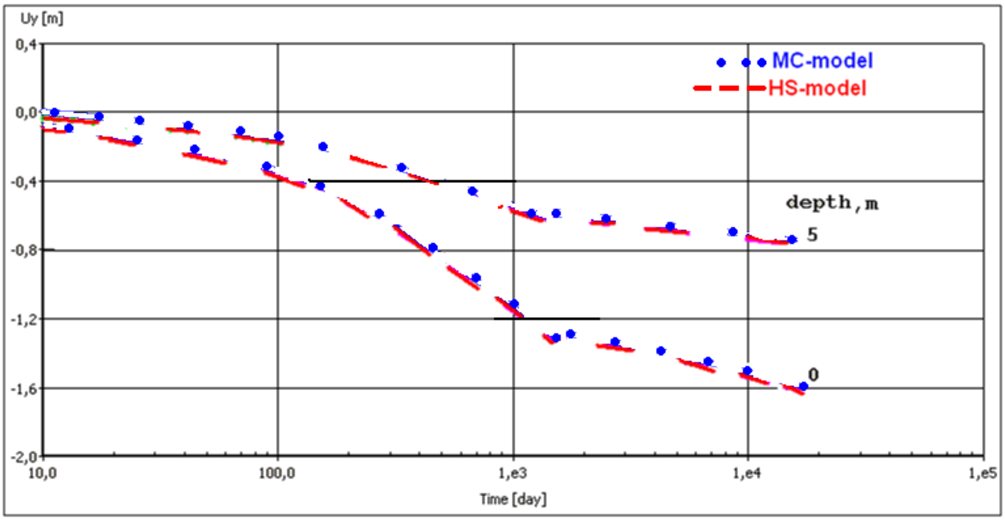

Figure 6.11. Comparison between HS-model and MC-model below the drained embankment at Skå-Edeby (Area 3)

6.3.2. SSC-model and SS-model at Skå-Edeby, Area 3

A comparison between the SSC model and SS-model was made at Area 3 at Skå-Edeby. The frictional soils had HS model as material model in both analysis while the soft clays had Soft Soil with creep in one analysis and one analysis without creep. The result can be seen in Figure 6.12. The dotted line indicates SSC model and the solid line indicates the SS-model. The analysis was made at 0 m depth, at the central part of the embankment. The result shows that settlement from SSC model increased while the SS model stays constant with time after unloading.

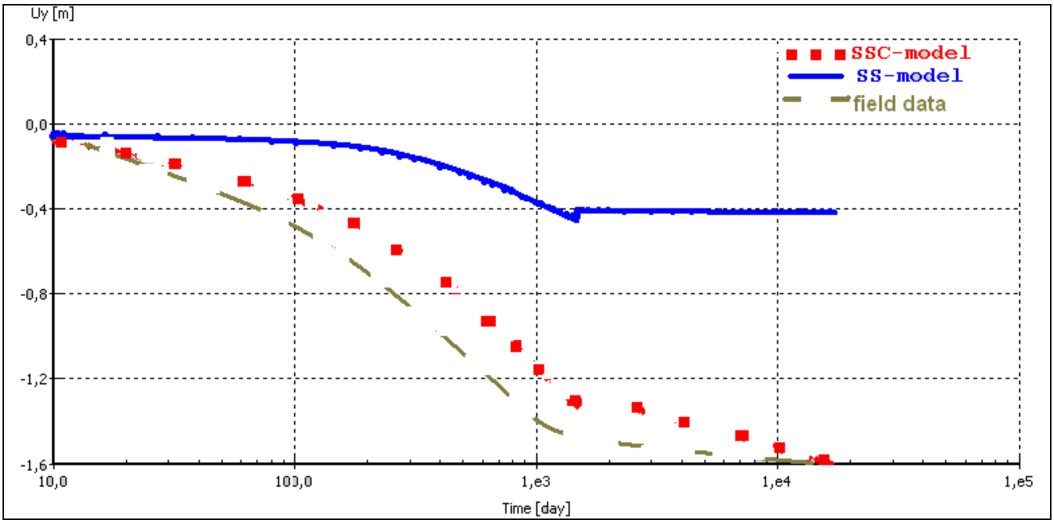

Figure 6.12. Comparison between the SS model and the Creep model below the drained embankment at Skå-Edeby (Area 3)

6.3.3. SSC-model and SS-model at Skå-Edeby, Area 4

Another analysis between the Soft Soil and Creep model was made at the undrained embankment at Skå-Edeby. The results are shown in Figure 6.13. The aim with this analysis was to find out if the curve from SS-model was turning into a constant displacement, as it was at Area 3.The Figure is showing the comparison at the depths of 0 m, 2.5 m, 5 m, and 7.5 m under the central part of the embankment. The figure is showing that there is a huge distinction between the Creep and Soft Soil model at calculation just under the surface at 0 m. However this difference is decreasing under calculation at bottom layers. The dotted line indicates field data at 0 respectively 7.5 m depth. This shows that the creep model satisfies better the field measurements both at the top and the bottom.

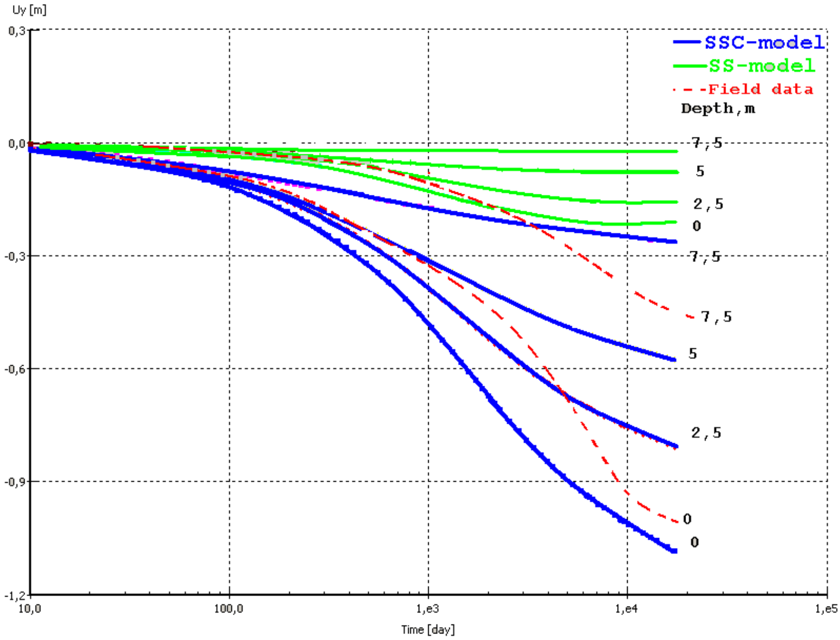

Figure 6.13. Comparison between SS-model and Creep-model below the undrained embankment at Skå-Edeby (Area 3)

Chapter 7

Analysis/Discussion

7.1. Choice of material model

Because all the simulations involved time dependent behaviour, the soft soil layers had the selection of Creep model in all the calculations. In the friction layers both the MC - model and HS - model have been applied.

7.1.1. Lilla Mellösa

Figure 6.1 shows that the field curve at Lilla Mellösa with vertical drains has a more scarpe slope than the Plaxis curve in the beginning. In reality the field curve should have a more crook similar to one from Plaxis because a linear curve is less possible. Both curves are showing a large settlement in the beginning until the first 200 days. This is because the test fill has vertical drains installed and of that reason a large settlement in the beginning is expected. The curve from numerical calculations indicates a heave in connection with removal of 0.8 m of the bank. The amount of the heave is around 35 mm. The curve from field data doesn't indicate any heave during the removal of 0.8 m fill. However a heave in connection with unloading is more expectable. After the removal of the upper part of test fill, the settlement velocity continues with a much lower rate than before. This is expected because the water content is much less in bottom layers and also there are no vertical drains installed in the bottom part which makes the consolidation process go slower than at the beginning. The curves are also indicating that no ending of settlement is in approach.

When there are no drains installed the settlement doesn't go quicker as can be seen in Figure 6.2. Both curves are following the same path during the whole consolidation process. In Figure 6.4 the Figures illustrates that the major part of consolidation during the first years did occur in the upper layers. In the study of excess pore pressure the Figure 6.3 gives compatibility between measured and calculated excess pore pressure. The excess pore pressure obtained from Plaxis starts to decrease much earlier than the field data. While the measured excess pore pressures in 1972, 1982 and 2002 are 30kPa, 22kPa and 12 kPa the corresponded values with Plaxis at the same time are 8kPa, 5kPa and 1 kPa. An explanation to this is that a numerical solution program has difficult to achieve excellent result for two different items at the same time, such as settlements and excess pore pressure. By using the parameters that are given by Larsson (2007) in our models we get good result in settlements but not in excess pore pressure. There was also the possibility to get corrected values in excess pore pressures but we would then be forced to choose parameters that do not agree with parameters given by Larsson (2007). This shows that our models achieve good results in settlements but not in excess pore pressures.

7.1.2. Skå-Edeby

Figure 6.5 clarifies that the Plaxis curve and field curve are much close to each other in the calculation of test area 1. In the Figure it can be seen that both curves become linear after around 2000 days. The reason of this is that the primary consolidation settlement ends and secondary consolidation starts. Figure 6.6 and 6.7 fortify that the curves obtained from test areas with vertical drains follow much the same path as the field curves.

As in the case of embankments installed with vertical drains, Figure 6.8 explains that the installation of vertical drains speeds up the settlements. In the Figure it can be seen that both

compared curves do get a temporally heave after removal of the upper part (0.7m) of the test fill. The curve from numerical calculations indicates a heave in order of 35 mm, while the heave in the measured data has a magnitude of 8 mm.
Figure 6.9 demonstrates the calculation at the undrained fill. The Plaxis curve follows much similar the curve from field measurements. However a deviation between them appears in the bottom layers. The measured excess pore pressure that was made during three occasions in 1971, 1982 and 2002 was 20 kPa, 13 kPa and 8 kPa. At the same occasions the calculated pore pressures were near zero. This means that the primary consolidation from numerical calculations must have gone much quicker than from the measured. However if it was so, the calculated settlements will also show an earlier primary consolidation than measured but by studying Figure 6.9 this does not appear.

The comparison between the HS-model and MC-model in Figure 6.11 demonstrates that there were almost no differences between those two models. Because the calculation of this case had an unloading phase, some difference between those models was expected. An explanation to why there was almost not any deviation between those two models is that the comparison was applied only on the friction soils in the test. We would probably get another result if the soft soil layers also had been analysed with MC - and HS-models.

The comparison between the Creep and Soft Soil model in Figure 6.12 and 6.13 shows that the Creep model is superior to the Soft Soil model. In calculations at 0 m depth below the embankment the Creep model matches much better the field data than the Soft Soil model does. The differences between those two models become less for deeper layers. Still the Creep model is superior here but the difference from the Soft Soil becomes smaller. A explanation to this is that the a low value of *OCR* does better match the Creep model then the Soft Soil model as have been presented by Neher & Wehnert and Bonnier (2001).

7.2. Choice of parameters

The stiffness parameters which are used for the clay layers have been evaluated on the basis of an assumption of the compression index, λ^*. The two other parameters, the swelling index, κ^* and creep index, μ^* have been estimated from λ^*. In this assumption the ratios of $\lambda^* / \mu^* = 15 - 25$ and $\lambda^* / \kappa^* = 5$ have been used in our calculations. However Neher, Wehnert and Bonnier (2001) used the ratio $\lambda^* / \mu^* = 35$ in simulations with Boston trial Embankment, which shows that the interval λ^* / μ^* could be much longer. The problem in our study by assuming parameters could be bothersome if the study demanded a ratio of $\lambda^* / \mu^* = 35$. Another problem is that we have estimated parameters that are not specified to a specific soil layer but is rather specificed in intervals for the whole soil profile. For example the compression index, λ^* has been evaluated from the compression index, C_c, void ratio, e and water content, w. To our adaptation those parameters were given in intervals applied for the whole soil profile and not for a specific layer. Neher, Wehnert and Bonnier (2001) used a value of compression index in the interval of [0,106-0,069] in their soil profile at Skå-Edeby. Those are much smaller values than what have been evaluated in this report. Any explanation to their assumption of compression index is not given, instead a reference to Larsson (1997) is specified. However with our estimation of earlier mentioned parameters, a value of compression index in the interval of [0,106-0,069] has not been possible to estimate at Skå-Edeby.

There has been made several simplifications of the calculations. Some simplification that has been made besides the evaluating of parameters is that we use the same parameters in every fill that belongs either to Skå-Edeby or Lilla Mellösa. Because the fills are located at some distance from each other some differences can be expected. Another simplification is that we have estimated that all the fills have the same depth to the bedrock layer where they in reality have different depth to the rock layer. This is of the reason why the calculated curves at Skå-Edeby of Area 1, 2 and 5 is mostly the same while they are different from the field.

Chapter 8

Conclusions

8.1. Conclusions

The purpose with this Thesis was to calculate settlements with the FEM program Plaxis and compare them with measured data from the field at Skå-Edeby and Lilla Mellösa. The result shows that Plaxis gives fairly good result in all cases that have been calculated, in both drained and undrained models. Some differences do occur if the fill is included in an unloading phase. In such case Plaxis gives a little different result.

Calculations show that the Soft Soil Creep model matches better the field measurements than the Soft Soil model. Another comparison between Hardening Soil model and Mohr Coulomb model shows almost not any distinction in analysis with friction soils.

The result of excess pore pressure in relation with time that have been obtained from Plaxis shows a much different behaviour than the corresponding result obtained from field.

8.2. Further work

In this thesis work only vertical settlements have been calculated. Therefore a study of horizontal settlements calculated with Plaxis and comparison with field measurements could be an idea for further work.

Another suggestion for further work is to study the compatibility of calculated settlements at a point in the central line of the embankment and at a point at the extremities of the embankment. The study can be compared with measured data. For example at both attempts at Lilla Mellösa and Skå-Edeby the settlements from the field at the extremities gave a magnitude of 80% of the settlements measured at the central part of the embankments. Does also Plaxis give such a relationship?

Also the analysis of the three different methodizes of undrained behaviour that are mentioned in Plaxis could be an idea for further work. The reason of this is to compare the variant methodizes with each other.

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