EU Creep (PIAG-GA-2011-286397)

Creep analysis of Onsøy test fill

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Creep of Geomaterials

Preface

This report is part of documentation of work package 1, WP2, (Benchmarking) in the EU CREEP project (PIAG-GA-2011-286397).

Summary

This report presents back-calculations of Onsøy test fill with advanced soil models with focus on the effect of creep. The objective of the report is to give recommendations for use of these creep models on settlement problems in soft clay. Different models that include creep are used for simulating the Onsøy test fill. The results are compared to field measurements and results from a calculation without accounting for creep. Overall all the creep models give results that are satisfactory. The report tries to address any deficiencies in the models and suggest future modifications. A description of some of the issues found in using the different models is also given.

The main conclusion is that: For engineering purposes all models, if used in the right way, can be used with satisfactory results. However, to be able to predict all measurements at Onsøy, the soil model should ideally account for small strain stiffness, anisotropy, and destructuration.

Finally the report gives some recommendations for creep calculations in clay.

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NGI Rapport / Report

CREEP – Creep of Geomaterials

Settlement analysis of Onsøy test fill

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Summary

This report presents back-calculations of the Onsøy test fill with advanced soil models with focus on the effect of creep. The objective of the report is to give recommendations for how to use these creep models on settlement problems in soft clay. The results from the simulations are compared with field measurements and results from a calculation without accounting for creep. Overall, all the creep models give results that are satisfactory. The report addresses any deficiencies in the models and suggest future modifications. Some main issues found in using the different models are also discussed.

The main conclusion is that for engineering purposes all models, if used in the right way, can be used with satisfactory results. However, to be able to predict all measurements at Onsøy, the soil model should account for small strain stiffness, anisotropy, and destructuration.

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1 Introduction

This report presents back-calculations of the Onsøy test fill with advanced soil models with focus on the effect of creep. The objective of the report is to give recommendations for how to use these creep models on settlement problems in soft clay. The work is financed by the CREEP project which is an EU funded Industry-Academia Partnerships and Pathways (IAPP) project.

In 1972 the Norwegian Geotechnical Institute (NGI) decided to build a test fill in Onsøy near Fredrikstad in Norway. The main purpose of the fill was to study the effect of time on strength and deformation characteristics of soft, plastic clays (Berre, 2013). The amount of measurements on the fill during settlement and failure together with the numerous advanced, high quality field and laboratory tests carried out makes the fill an excellent choice for studying the performance of advanced soil models.

1.1 CREEP – Creep of Geomaterials

The work presented in this report is part of the IAPP project "CREEP – Creep of Geomaterials". The project aims at establishing a consensus in creep modelling and developing new design tools for creep in soft soils, frozen soils and hard (granular) soils. This will be achieved through build-up of new knowledge and transfer of knowledge between industry and academia.

This report is part of the documentation of work package 2 (presented in Annex I – "Description of work" for the CREEP project). A survey of benchmark field tests has been carried out in the project (Dijkstra og Karstunen, 2014), and the Onsøy test fill was pointed out as one out of three that should be used for further benchmarking of creep models. The description of work package 2 states that common soil models (with creep formulations) should be used to back-calculate benchmark field tests and their capabilities of capturing aspects of real soil behavior will be assessed. This will be the basis for the combination, integration and unification of existing creep concepts into a more general model. This is the background for the scope of this report, which is a part of deliverable D2.

1.2 Scope

The scope of this report is as follows.

- A prediction of ground deformation and stress distribution for the Onsøy test fill will be carried out with the Soft Soil Creep (SSC) model available in the finite element program PLAXIS (www.plaxis.nl). The approach is as follows:
  - o Interpret relevant data from available field and laboratory tests
  - o Back-calculate oedometer tests with SSC and adjust parameters to get the best fit
  - o FE analyses with SSC of the test fill with parameters gathered from field and laboratory tests
  - o The results will be analyzed with focus on vertical and horizontal deformations and pore pressure response.

- When the results from the prediction is analyzed, the soil parameters are adjusted to get a best fit for ground deformations and stress distributions. A new back-calculation ("best-fit") will be done with the new adjusted parameters. The necessary adjustments will be analyzed and discussed. This will lead to recommendations for the use of SSC
- Back calculation of the test fill will also be carried out with other available soil models with creep formulations. Similar soil parameters as found in the "best fit"-calculation will be used. The soil models used in these calculation are:
    - Critical State Soft Soil Creep with non-linear shear stiffness G (CS-SSCG) (Ashrafi, 2014)
    - Non-associated creep model for Structured Anisotropic Clay (n-SAC) (Grimstad and Degago, 2010)
    - Sekiguchi-Ohta, Viscid (Sekiguchi and Ohta, 1977, Iizuka and Ohta, 1987)
    - KRYKON (Svanø, 1986)

The analyzed results will make a basis for recommendations when using advanced soil models for predicting settlements in creep sensitive soils. Possible improvements of the models will be pointed out.

2 Onsøy Test Fill

2.1 Background

The recent paper of Berre (2013), describes the Onsøy test fill in detail. This chapter summarizes some important details.

As mentioned in the introduction the objective of the Onsøy test fill was to study the effect of time on strength and deformation characteristics of soft, plastic clays. The fill was laid out in five layers to a total height of 2.3 meters during 14 days. The dimensions of the fill were 20 m x 60 m, and it was assumed that a cross section at the middle could be approximated by plain strain conditions. The estimated factor of safety was 1.35 using undrained shear strength from in situ vane tests. The fill was left to settle for approximately 3 years before it was brought to failure by raising the height rapidly.

2.2 Soil conditions

The fill was situated about 100 m from the Seut River (Figure 1). The terrain around the test fill had a slope of 1:200 towards the river, which is at about sea level. The elevation of the ground surface under the fill was at approximately +0.7.

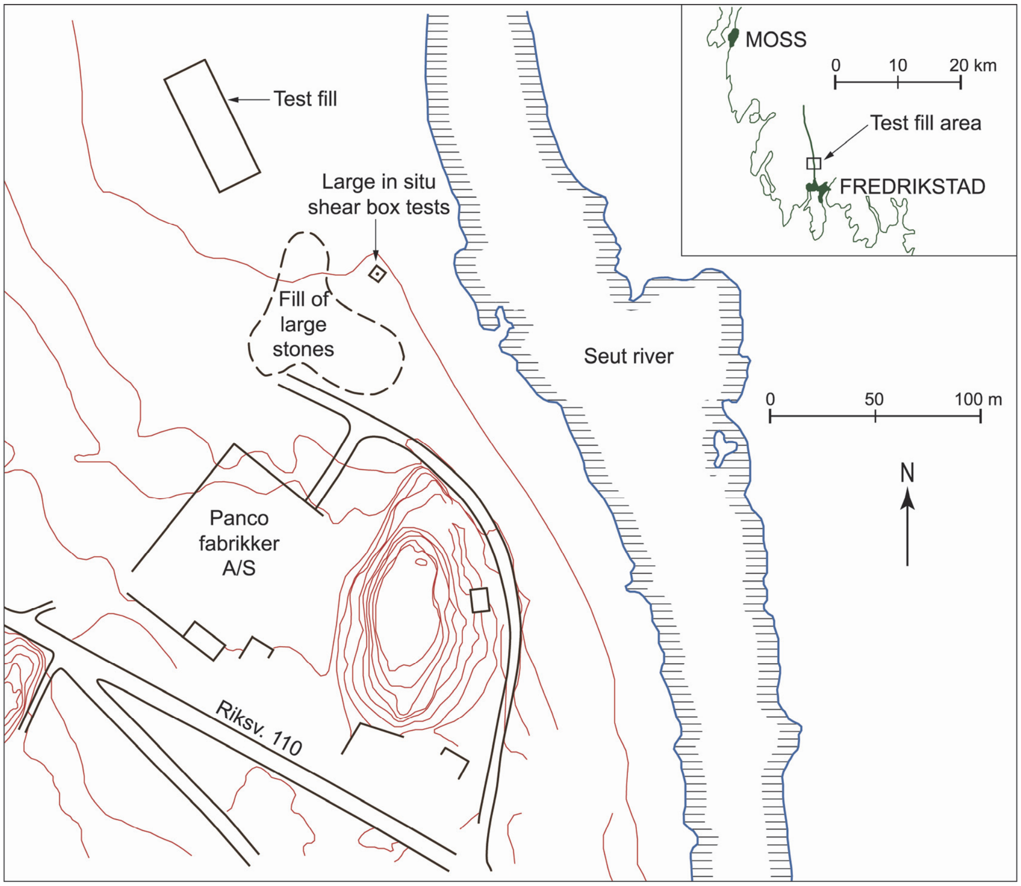

Figure 1: Map of Onsøy test field (Berre, 2013)

The ground water level at the location of the fill was about 0.2 to 0.3 m under the ground surface before start of construction, and the pore pressure was artesian (ca. 5% at 20 m depth). Bedrock was found at 53 m depth below the middle of the fill, and the thickness of the weathered crust varied from 0.8 to 1.5 meters. Above the bedrock it is a thin layer of moraine. The remaining soil profile consists of soft sensitive clay. Figure 2 shows a typical boring profile from the area. The clay has a natural water content varying from 57% to 67% and the plasticity index varies from 34 to 51.

An extensive program of field and laboratory test has been carried out on the clay over the years. In this study the laboratory tests on block samples reported in Berre (2010) are used to interpret soil parameters for the advanced soil models. Some parameters found in Lunne, et al. (2003) are also used in this study.

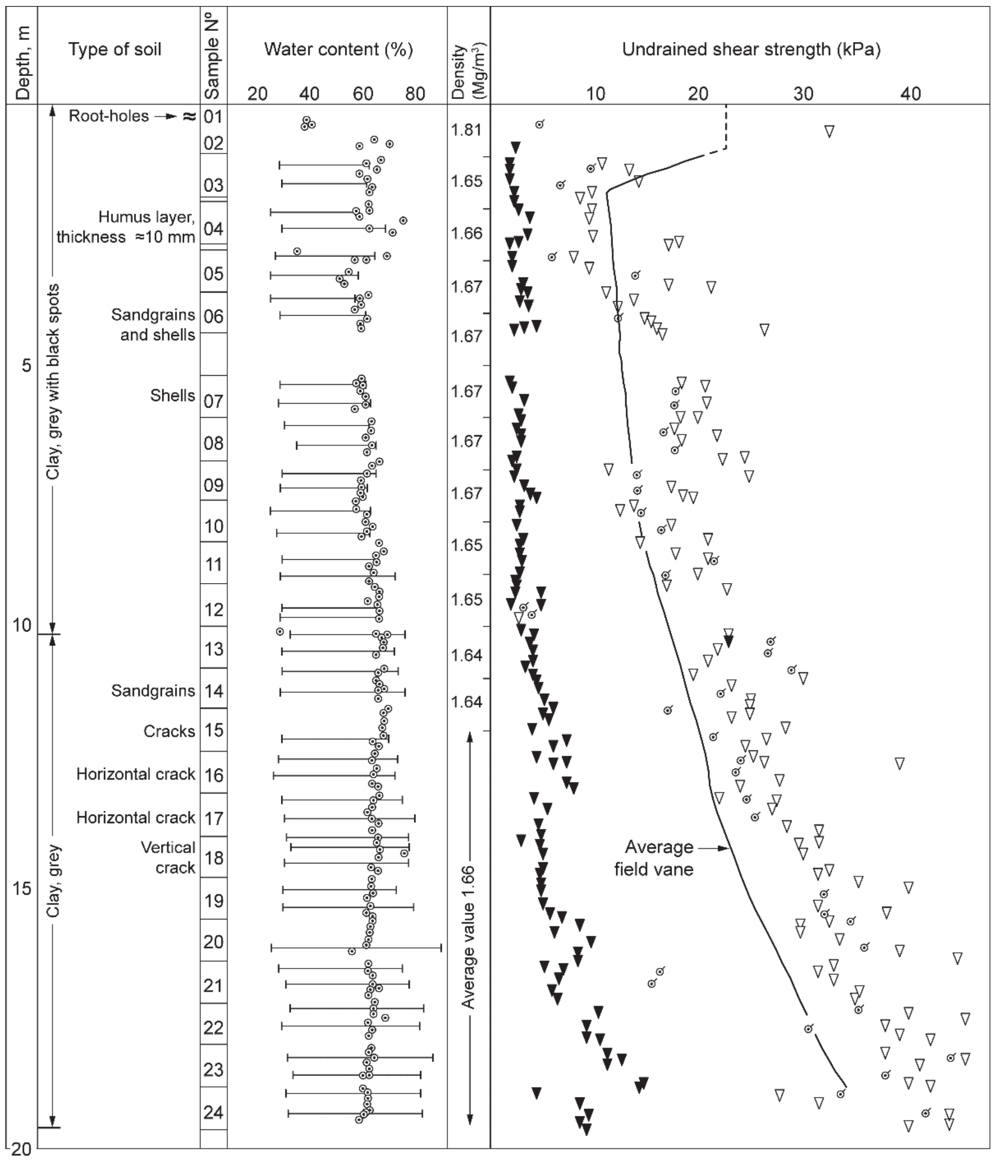

Figure 2: Boring profile from 54mm tube samples (Berre, 2013)

2.3 Instrumentation

Figure 3 shows the planned instrumentation of the fill. In addition to what is show in this figure, NGI also had the following instrumentation:

1. Ring magnets along a horizontal plastic tube in the fill.
2. Fifty wooden poles in the area around the fill. On top of the poles a copper plate with a cross mark was mounted.
3. "Brittle sticks" of wood in the ground for determination of any developing failure surface.

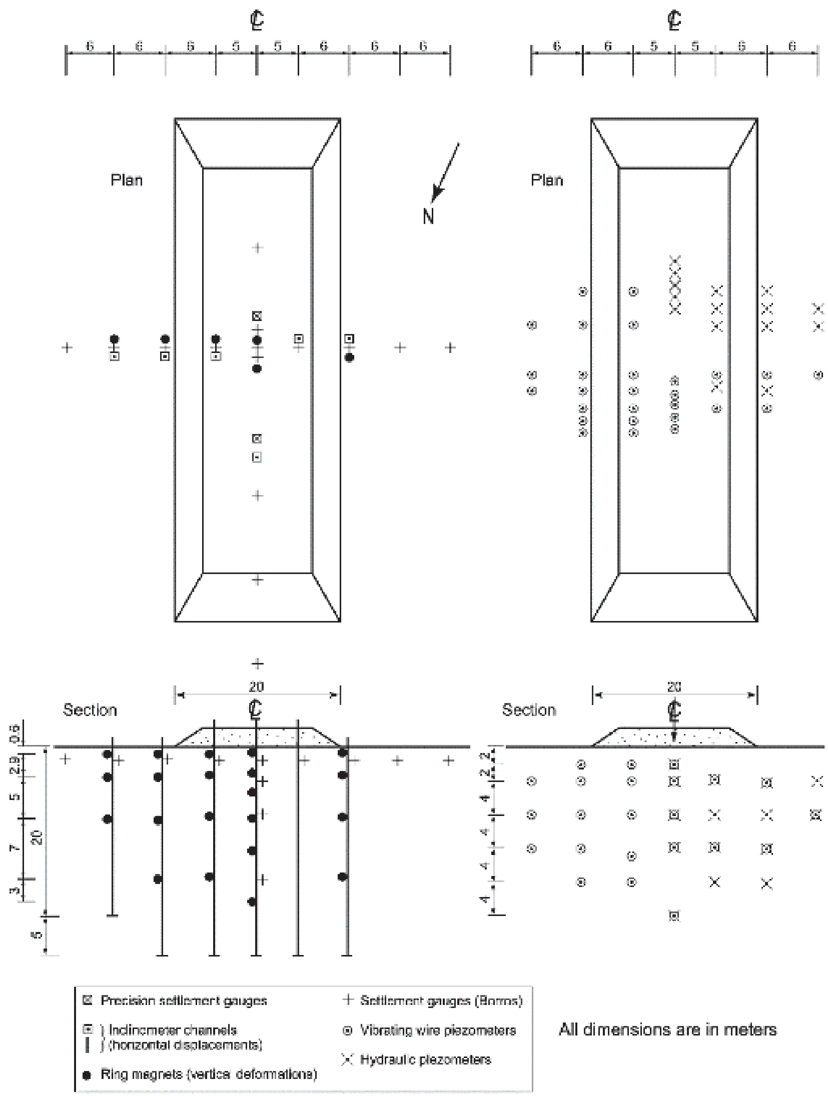

Figure 3: Instrumentation for measuring displacements and pore pressure (Berre, 2013)

2.4 Construction sequences

Figure 4 shows a plan over 0.7 to 0.8 meter deep sand filled trenches below and around the fill. The purpose of the trenches was to reduce the effect of the strength of the weathered crust on deformations and stresses below the crust. The trenches outside the fill were left open and those below the fill was filled with sand.

The construction of the fill started at day 22 with a 0.5 m thick layer of sand. Three more layers with the same thickness were laid out with some waiting time in between. The fifth layer was approx. 0.25 m thick and was finished on day 36. The fill was then left to settle until day 1120. The same construction sequences are modelled in Plaxis as shown in Table 1.

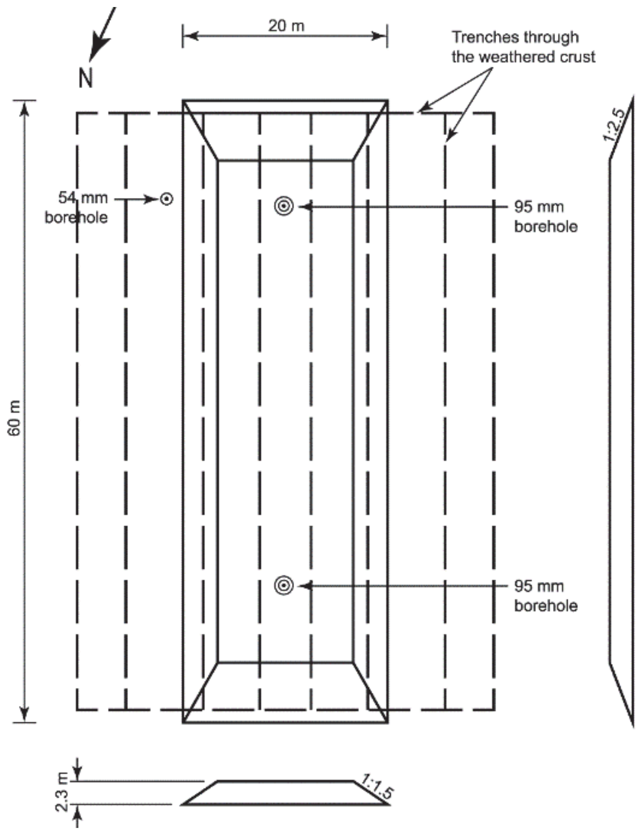

Figure 4: Plan with dimensions of fill (Berre, 2013)

Table 1: Construction sequences

Phase	Description	Phase duration [days]	Accumulated time [days]	"Real time" [days]
0	Initial phase – K_0-procedure	-	-	22
1	Fill 0,5m	1	1	23
2	Wait	1	2	24
3	Fill 0,5m	1	3	25
4	Wait	3	6	28
5	Fill 0,5m	1	7	29
6	Wait	3	10	32
7	Fill 0,5m	1	11	33
8	Wait	2	13	35
9	Fill 0,3m	1	14	36
10	Wait	6	20	42
11	Wait	24	44	66
12	Wait	38	82	104
13	Wait	43	125	147
14	Wait	37	162	184
15	Wait	145	307	329
16	Wait	227	534	556
17	Wait	235	769	791
18	Wait	274	1043	1065
19	Wait	43	1086	1108
20	Wait	12	1098	1120

2.5 Measurements

The most important measurements are reported and discussed in (Berre, 2013). The measurements used to validate the advanced soil models are shown in Figure 5 to 12.

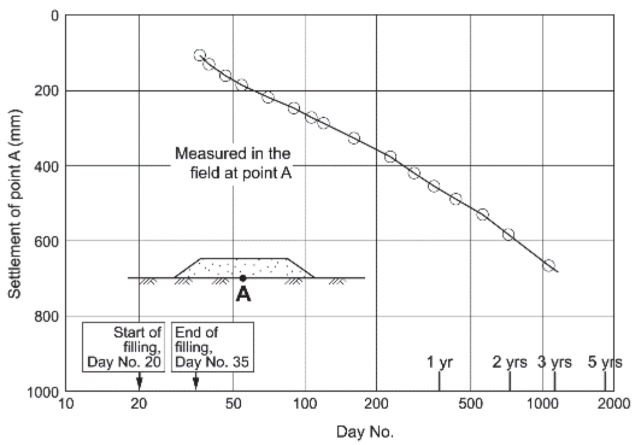

Figure 5: Settlement of ground surface below the middle of the fill (Berre, 2013)

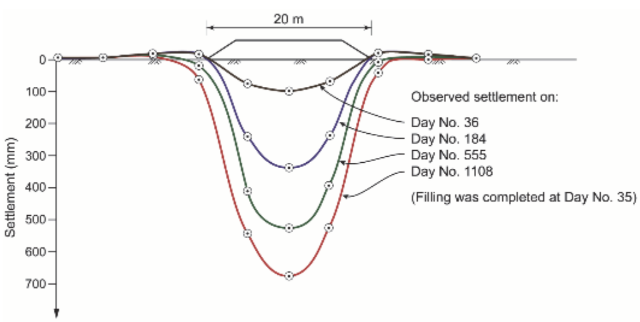

Figure 6: Settlement just below the weathered crust (Berre, 2013)

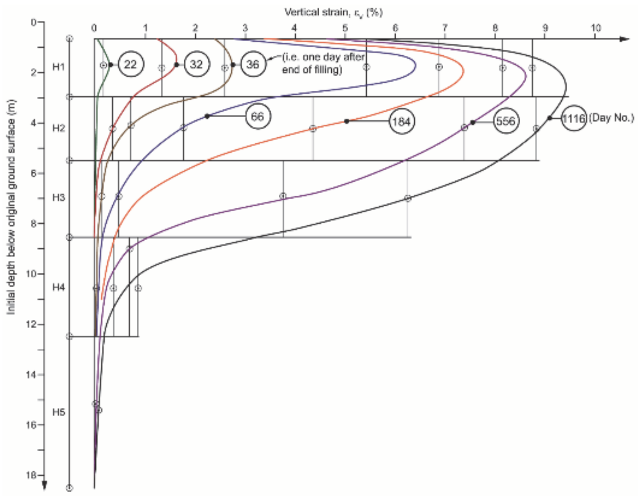

Figure 7: Vertical strain contours below the centreline of the fill (Berre, 2013)

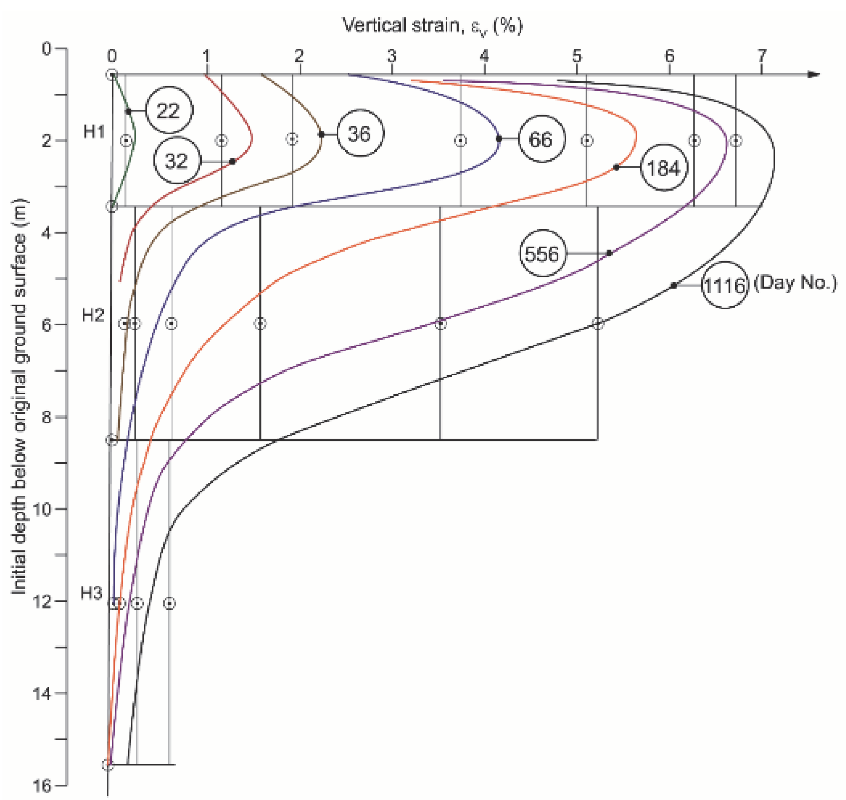

Figure 8: Vertical strain contours 5 m east of the centreline (Berre, 2013)

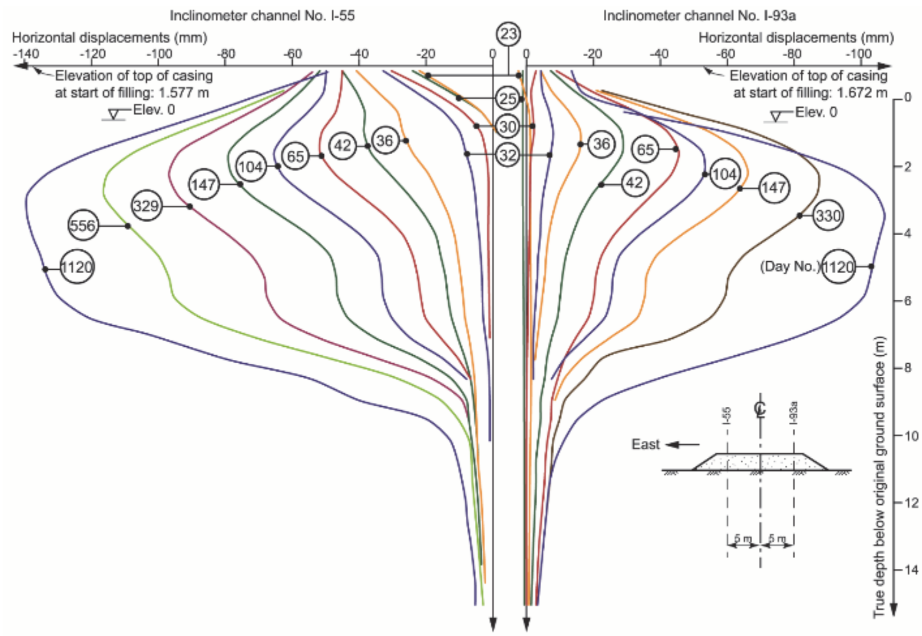

Figure 9: Horizontal displacement 5 m west and 5 m east of the centreline (Berre, 2013)

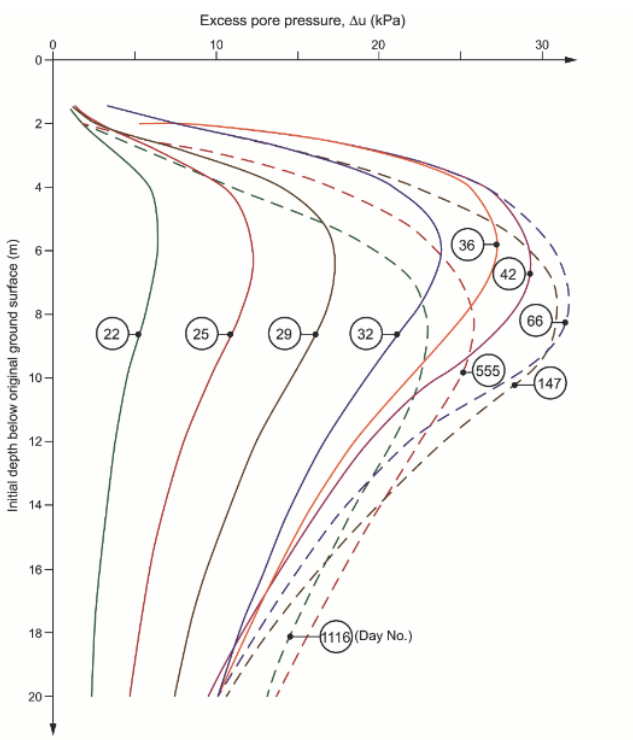

Figure 10: Excess pore pressure below the centreline of the fill, according to hydraulic piezometers, corrected for settlement of piezometers(Berre, 2013)

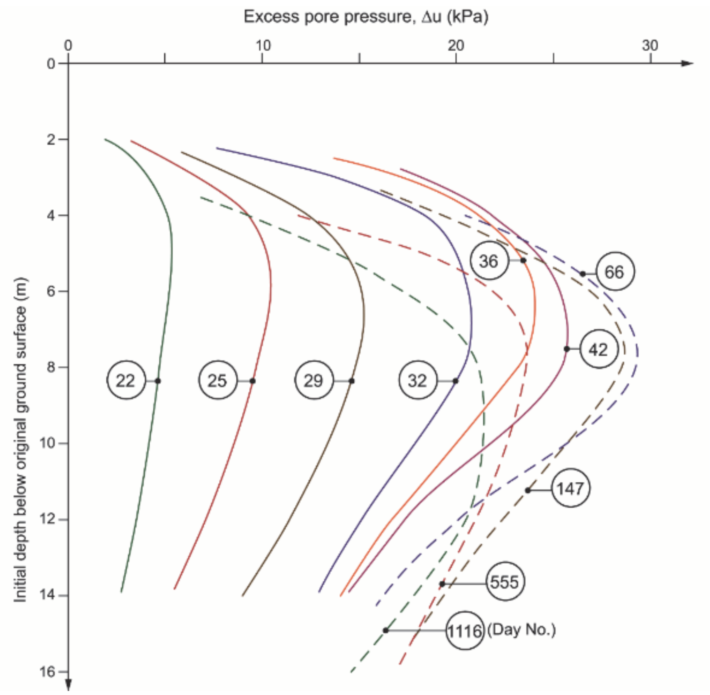

Figure 11: Excess pore pressure 5 m west of the centreline, according to hydraulic piezometers, corrected for settlement of piezometers(Berre, 2013)

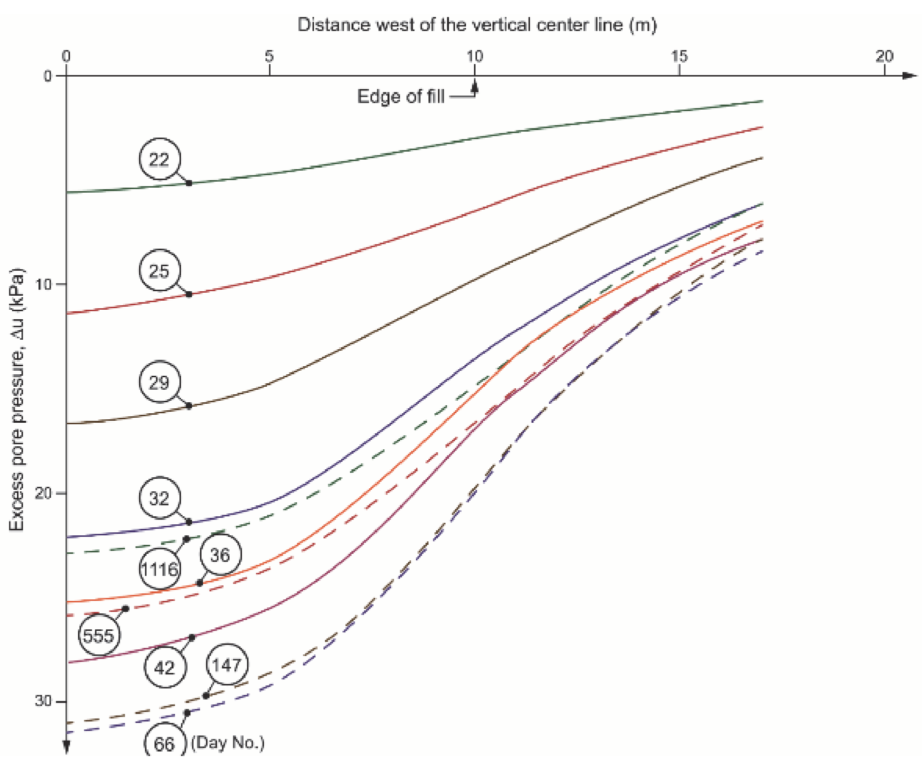

Figure 12: Excess pore pressure 8 m below the original ground surface, according to hydraulic piezometers, corrected for settlement of piezometers(Berre, 2013)

3 Creep models

3.1 General

In this work, four different 3D constitutive models where supposed to be tested by back-calculation of the Onsøy test fill. In addition calculations with the "classical" creep approach, used in conventional projects in Norway, was done. The theoretical backgrounds of these models are not presented in this report, but key references are given. However, an overview of the input material parameters is given.

It is not the scope of this report to give a detailed theoretical background for creep, se Chapter 7 and 8 for recommended literature.

3.2 Soft Soil Creep

A description of the SSC model is given in the PLAXIS user manual (2012), (Stolle *et al.*, 1999a)and(Stolle *et al.*, 1999b). The material parameters in the model are listed in Table 2.

Table 2: Soil parameters for the Soft Soil Creep model

Parameter	Description
c	Cohesion
φ	Friction angle
ψ	Dilatancy angle
κ^*	Modified swelling index
λ^*	Modified compression index
μ^*	Modified creep index
υ_{ur}	Poisson's ratio for unloading-reloading
K_0^{NC}	Stress ratio in a state of normal consolidation
OCR	Overconsolidation ratio
POP	Preoverburden pressure
M	K_0^{NC}-related parameter

3.3 Critical State Soft Soil Creep Model with Non-linear Shear Stiffness

A description of the CS-SSCG model is given in(Ashrafi, 2014). The material parameters in the model are listed in Table 3.

Table 3: Soil parameters for the Critical State Soft Soil Creep model with non-linear shear stiffness

Parameter	Description
κ^*	Modified swelling index
λ^*	Modified compression index
μ^*	Modified creep index
η_{K0}	Stress ratio at K_0'. $\eta_{K0} = q/p'$ at rest $\rightarrow \eta_{K0} = 3(1-K_0')/(1+2K_0')$
ζ	Shear stiffness degradation factor, $G_M = G_0\,(1 - \xi \cdot f)^2$, $f = (\eta - \eta_{K0})/(M_\theta - \eta_{K0})$
K_0^{NC}	Stress ratio in a state of normal consolidation
OCR_τ	Overconsolidation ratio at reference time
POP_τ	Preoverburden pressure at reference time
M_c	Slope of the critical state line
τ	Reference time
y_{ref}	Reference depth
G_{ref}	Shear stiffness at reference depth
G_{inc}	Shear stiffness increase per meter depth

3.4 *Sekiguchi – Ohta (Viscid) Model*

A description of the Sekiguchi-Ohta model is given in the PLAXIS user manual (2012), (Sekiguchi og Ohta, 1977)and(Iizuka og Ohta, 1987). The material parameters in the model are listed in Table 4.

Table 4: Soil parameters for the Sekiguchi-Ohta model

Parameter	Description
κ^*	Modified swelling index
λ^*	Modified compression index
α^*	Coefficient of secondary compression
$\dot{v}_0$	Initial volumetric strain rate
K_0^{NC}	Stress ratio in state of normal consolidation
OCR_0	Initial overconsolidation ratio
POP_0	Initial preoverburden pressure
M_c	Slope of the critical state line

3.5 *n-SAC*

A description of the n-SAC model is given in(Grimstad og Degago, 2010). The material parameters in the model are listed in Table 5.

Table 5: Soil parameters for the n-SAC model

Parameter	Description
υ	Poisson's ratio for unloading-reloading
K_0^{NC}	Earth pressure coefficient at rest in normally consolidated stress state (for remoulded material) This value would typically be a bit smaller than what is normally measured for natural clay
E_{ref}	Elastic Young's modulus at p_{ref}
E_{oed}^{ref}	Intrinsic oedometer modulus at p_{ref}
p_{ref}	Reference stress. Typically 100 kPa
$r_{s,min}$	The minimum time resistance number
$r_{s,i}$	The intrinsic time resistance number
ω	Gives the contribution of viscoplastic shear strain to destructuration
φ_T	Friction angle at peak of undrained stress path
φ_{CS}	Critical state friction angle
OCR_τ	Together with the reference time this parameter defines the position of the reference surface to the initial stress condition. POP can also be used.
t_{max}	Is a time for which significant reduction in creep rate occurs.

3.6 KRYKON – "classical Norwegian practice"

Svanø (1986) implemented a model for creep into a code for 1D consolidation called KRYKON. The KRYKON model uses the time resistance concept after Janbu (1969), where a reference curve for a given strain rate (= 1 / R_c) is defined, using the oedometer modulus, M_{OC}, for overconsolidated state and the oedometer modulus number, *m*, and stress intersection value, p_r,. The time resistance number, r_s, is given as a function of stress. More details are given in Svanø (1986) and Table 6.

Table 6: Soil parameters for the KRYKON model

Parameter	Description
M_{OC}	Oedometer modulus in OC state (from σ_0' to p_c')
m	Oedometer modulus number in NC state
p_c'	Pre-consolidation stress
p_r'	Intersection stress
Rc	Time resistance for the reference curve
r_0	The time resistance number at σ_0'
r_{pc}	The time resistance number at p_c'
m_r	Increase in time resistance with stress above p_c'

3.7 Comparison of the different models

The considered four creep models have different approaches when it comes to modelling creep and describes different behaviour observed on clay. The main differences between the models are listed in Table 7. Some different characteristic behaviours of clay are explained below.

Creep modelling
When implementing a model with *volumetric creep* (SSC) one assumes contours of constant volumetric creep strain rate for constant equivalent mean stress. This approach gives strain rates that are always positive, which consequently does not allow a stress path to reach the left side of the Cam-Clay yield surface. Because of this, swelling behaviour is not correctly simulated. By applying the creep term on the *plastic multiplier* (the rate of the total strain vector including shear terms as in n-SAC), this issue is resolved.

Anisotropy
Natural clay normally has a significant anisotropy when it comes to undrained shear strength. Critical state models with anisotropy (e.g. n-SAC) uses a rotated ellipse to simulate the different undrained shear strength observed in compression and extension of intact real clays.

Destructuration
Destructuration is the loss of structure when undisturbed clay experience plastic deformations (yielding). The structure of natural clay comes from bonding between clay particles, which give additional resistance to yielding. This structure is lost in remoulded and reconstituted samples, and partly lost in disturbed samples. When destructuration is included in a soil model, the undrained post peak strain softening of the soil can be simulated (e.g. n-SAC).

Lode angle
Lode angle dependency means that the slope of the critical state line is a function of the intermediate effective principal stress. This will take in-to account some anisotropy effect where the critical state line has a different inclination in triaxial compression and extension.

Small strain shear stiffness
Initially (at very small strains) soft clay is a rather stiff material, but the shear stiffness will degrade quickly and non-linearly when exposed to increasing shear strains / mobilization (e.g. CS-SSCG).

Table 7: Comparison of some creep constitutive models (Ashrafi, 2014)

Model	Plasticity					Elasticity
	Creep		Anisotropy	Destructuration	Lode angle	Small strain shear stiffness
	Vol. creep	Pl. multiplier				
SSC	✓	-	-	-	-	-
CS-SSCG	-	✓	-	-	✓	✓
Sekiguchi-Ohta	✓	-	✓	-	-	-
n-SAC	-	✓	✓	✓	✓	-
KRYKON	"✓"	-	-	"✓"	-	-

4 Back-Calculation of Onsøy test fill

4.1 Finite Element model

The embankment is modelled in plain strain. Due to symmetry, only half of the fill is modelled. The dimensions of the fill is taken from (Berre, 2013). The FE model shown in Figure 13 consists of 1595 triangular 15-noded soil elements. PLAXIS AE (2012) is the calculation tool used in this study.

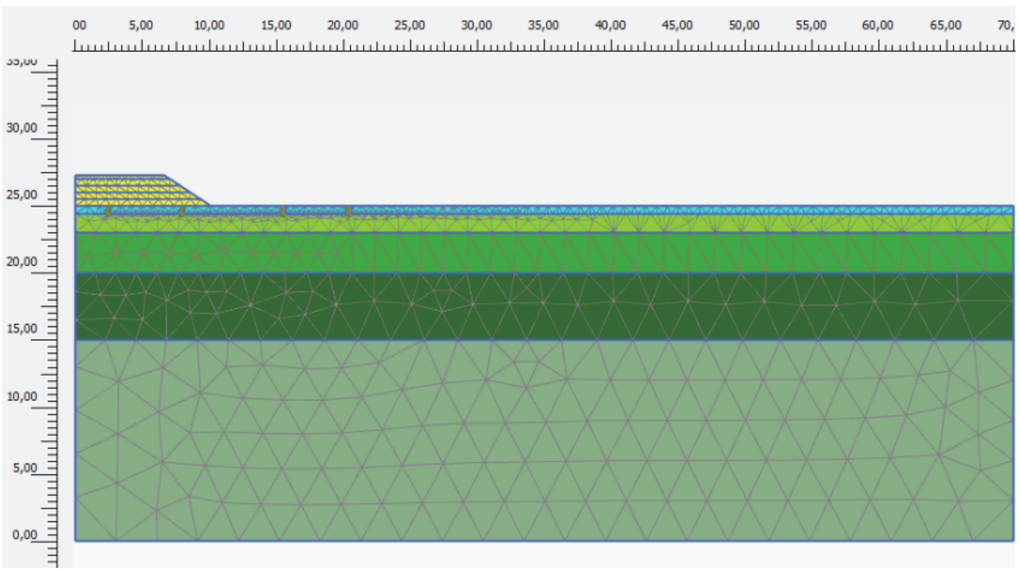

Figure 13: Finite element model used in all calculations

The centerline of the fill is due to symmetry, a closed consolidation boundary. The bottom and the right side are open consolidation boundaries. The choice of open consolidation boundary at the bottom can be discussed, because this will affect the amount of excess pore pressure building up in the lower part of the considered part of the clay layer. The correct behaviour will probably be somewhere in between open and closed boundary. Alternatively, one could have modelled the clay all the way down to bedrock (i.e. to 53 m depth). However, this will over predict the settlements because the assumption of plan strain condition is less correct at large depths. Therefore, the choice of open consolidation boundary at the bottom is assumed to give the most correct answer in this case.

4.2 Calculations with Soft Soil Creep

4.2.1 Prediction and best fit

The objective of the prediction calculation with SSC is to interpret soil parameters in the same way as in a real project without any information of measured behaviour. The results from this calculation will be compared with the measurements. Then, possible reasons for the differences are analyzed. In the next

SSC-calculation, the relevant parameters are adjusted to give a "best fit" to the measurements.

4.2.2 *Interpretation of laboratory tests*

The compressibility parameters in the SSC model are interpreted from CRS and IL oedometer tests. In the CRS oedometer tests, it is recommended to have a creep phase after passing the pre-consolidation pressure, followed by an unloading reloading loop. Soil strength parameters are interpreted from undrained triaxial tests, but this is not addressed in this report.

Compressibility parameters λ*, κ* and μ*

λ* and κ* are Modified Cam-Clay parameters in strain compressibility and are defining the slope of the normal compression line and the unloading reloading line in a ln(p')-ε_v-plot, where p' is the effective mean stress and ε_v is the volumetric strain. The star (*) denotes a difference from the normal Cam-Clay parameters, which are defining the same lines, but in a ln(p')-v-plot, where v = 1+e and e is the void ratio. μ* is the modified creep index and defines the time dependent volumetric strains.

Modified compression index: $\lambda^* = \frac{\lambda}{1+e} = \frac{C_c}{\ln 10(1+e)} = \frac{1}{m}$

Modified swelling index: $\kappa^* = \frac{\kappa}{1+e} \approx \frac{2 \cdot C_s}{\ln 10(1+e)}$ $\quad or \quad$ $\kappa^* \approx \frac{2 \cdot C_r}{\ln 10(1+e)}$

Modified creep index: $\mu^* = \frac{C_\alpha}{\ln 10(1+e)} = \frac{1}{r_s}$

λ* and κ* are taken from the slope of straight lines fitted to a standard CRS or a 24-hour IL oedometer test plotted in ε-lnσ. When interpreting κ* the slope has to be multiplied by 2 because of the difference between the vertical effective stress, σ', and the effective mean stress, p'. The value of 2 is obtained by assuming an isotropic stress state during loading, which is not the case, but it gives a good approximation.

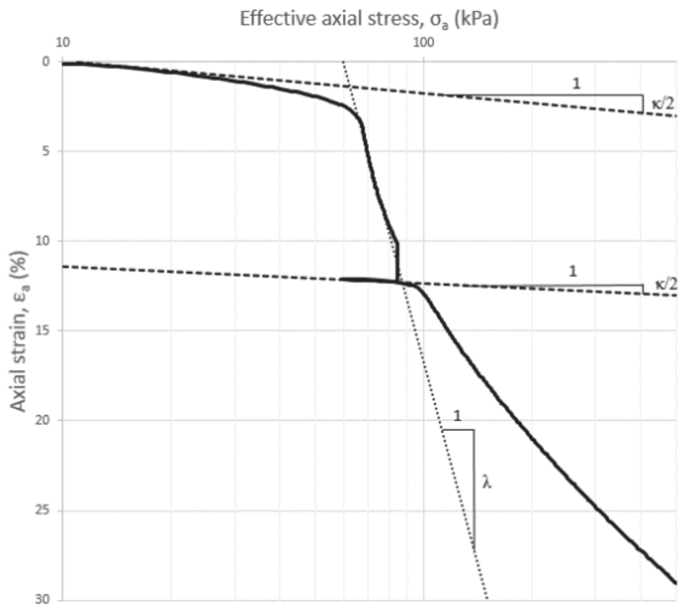

Figure 14: Oedometer test with straight lines for interpretation of modified swelling and compression index

From the particular oedometer test in Figure 14, the straight lines will give:

$$\lambda^* = \frac{\Delta\varepsilon}{\ln\left(\frac{\sigma_2}{\sigma_1}\right)} = \frac{(0{,}3-0)}{\ln\left(\frac{150}{60}\right)} = 0{,}33$$

$$\kappa^* = 2 \cdot \frac{\Delta\varepsilon}{\ln\left(\frac{\sigma_2}{\sigma_1}\right)} = 2 \cdot \frac{(0{,}03-0)}{\ln\left(\frac{500}{11}\right)} = 0{,}016$$

Alternatively, from the unloading reloading loop:

$$\kappa^* = 2 \cdot \frac{\Delta\varepsilon}{\ln\left(\frac{\sigma_2}{\sigma_1}\right)} = 2 \cdot \frac{(0{,}13-0{,}114)}{\ln\left(\frac{500}{10}\right)} = 0{,}008$$

It is essential to fit the straight lines to the stress range of interest for the particular problem at hand, because the SSC-model is not able to capture the non-linearity that undisturbed clay shows in the ln(σ')-ε-plot. Whether to use the initial part of the stress-strain-curve (from σ' = 0 – σ_c') or the unloading-reloading loop to interpret κ*, depends on the actual problem together with the assumed degree of sample disturbance. A too large κ* value will indirectly account for some creep which is not desirable when using SSC or other advanced soil models with creep. If the unloading-reloading-loop is too short, the value interpreted from this line could be too stiff. Since most loading problems operate in the stress area from σ_0' to somewhat slightly past σ_c', much care should be taken when choosing a suitable κ*-value.

μ* can be interpreted from different plots, and Figure 15 and Figure 16 show two different approaches. The curves on the two figures below are from the same oedometer test as above. The values are from the creep phase with constant effective vertical stress, which is larger than the preconsolidation pressure.

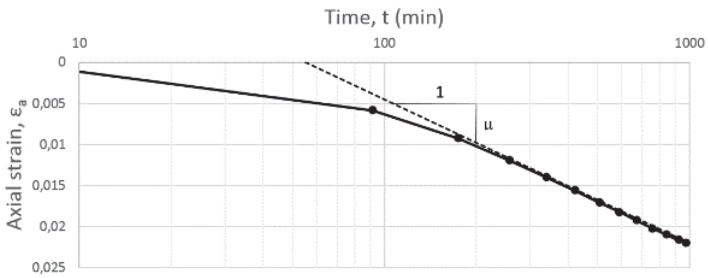

Figure 15: Interpretation of modified creep index

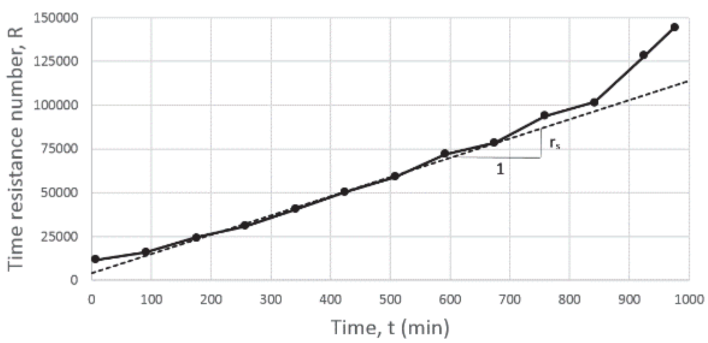

Figure 16: Interpretation of creep number

As seen in these two figures μ* and r_s are interpreted from the same type of data but with different presentation. The curve in figure 15 will give the following value:

$$\mu^* = \frac{\Delta\varepsilon}{\ln\left(\frac{t_2}{t_1}\right)} = \frac{(0{,}022-0)}{\ln\left(\frac{1000}{55}\right)} = 0{,}0076$$

Which gives,

$$r_s = \frac{1}{\mu^*} = \frac{1}{0{,}0076} \approx 130$$

Alternatively, Figure 16 gives the following value:

$$r_s = \frac{\Delta R}{\Delta t} = \frac{(114000-4000)}{(1000-0)} = 110$$

Which gives,

$$\mu^* = \frac{1}{r_s} = \frac{1}{110} \approx 0.0091$$

The two set of curves should in fact give the same value, but this depends of course on how one select the straight lines.

The relationship $\frac{\mu^*}{(\lambda^* - \kappa^*)}$ together with OCR is very important when controlling the initial creep rate of your model. This is addressed later in this report.

Permeability related parameters, k_y, k_x, c_k and e_0

Initial void ratio, e_0, is the void ratio found in the soil under in situ stresses, before loading. In the ground, this will vary with water content, saturation and stress state. The change in void ratio is given by the volumetric strains.

$$\Delta\, e = (1 + e_0)\, \Delta\, \varepsilon_v$$

When S_r = 100%, the initial void ratio can be simplified to be a function of the water content and the solid unit weight.

$$e_0 = w_0 \cdot \rho_s$$

Change in permeability is modelled in the following way in PLAXIS.

$$\log\left(\frac{k}{k_0}\right) = \frac{\Delta\, e}{c_k} = \frac{-(1+e_0)\, \Delta\, \varepsilon_v}{c_k}$$

The initial permeability, k_0, is either the k_y or k_x given as an input in PLAXIS. Figure 17 shows an example on how one can find k_y and c_k from a standard CRS oedometer test where void ratio is assumed to be 2.02 at σ_{v0}'. The straight line in the log(k)-ε-plot gives the following value.

$$c_k = \frac{(1+e_0)\, \Delta\, \varepsilon_v}{\log\left(\frac{k}{k_0}\right)} = \frac{-(1+2{,}02)(0{,}45-0{,}0195)}{\log\left(\frac{5{,}62E^{-11}}{2{,}61E^{-9}}\right)} = 0{,}78$$

If there is non-linearity in the log(k)-ε-plot, which is normal around the pre-consolidation pressure, the straight line must be fitted to the relevant strain area. Usually one would find small values for c_k, hence a bigger change in permeability for each strain increment, between σ_{v0}' and σ_c' compered to strains after σ_c'.

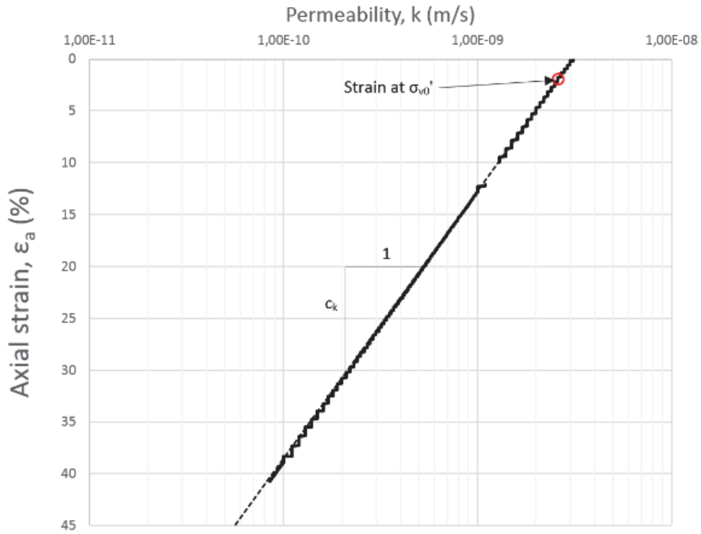

Figure 17: Interpretation of permeability and c_k

Parameters governing initial stresses, OCR, POP, υ_{ur}, K_0' and K_0^{NC}

These soil parameters are important when generating the initial stresses in the model. OCR and POP are also important when defining the initial strain rate in soil models with creep. OCR and POP define the relationship between preconsolidation pressure and in-situ vertical effective stress.

$$OCR = {\sigma_c'}\big/{\sigma_{v0}'}, \qquad POP = \sigma_c' - \sigma_{v0}'$$

When the over-consolidation ratio is defined, PLAXIS will calculate K_0' automatically:

$$K_0' = K_0^{NC} OCR - \frac{\upsilon_{ur}}{1-\upsilon_{ur}}(OCR - 1) + \frac{K_0^{NC} POP - \frac{\upsilon_{ur}}{1-\upsilon_{ur}} POP}{\sigma_{v0}'}$$

This corresponds to a K_0'-value created from preloading the terrain. This is not necessarily true if the soil has not been loaded before and the apparent preconsolidation pressure comes from aging of the material. If the latter is the case the K_0'-value will be closer to K_0^{NC}. K_0' can be set manually in PLAXIS. K_0^{NC} is often assumed to be:

$$K_0^{NC} = 1 - \sin\varphi$$

Experimentally K_0^{NC} can be estimated by performing an oedometer test with e.g. split ring where the horizontal stresses are measured. This is not standard equipment, but there is empirical data available. K_0' in-situ can be measured with hydraulic fracturing test, dilatometer or pressuremeter. Empirical data for the relationship between s_u^A/σ_{v0}', OCR and K_0' is also available.

OCR or POP can be obtained from oedometer test by estimating σ_c'. Techniques for doing this is known and will not be addressed in this report, but when doing this interpretation one should have in mind the effect of sample disturbance and strain rate of the test. This will highly affect the obtained OCR value.

Soil strength parameters

Failure is not an issue in this report, but soil strength determines the degree of mobilization in the soil and this will influence the calculated deformations. Interpretation of soil strength parameters is not addressed in this report.

4.2.3 Simulations of oedometer tests

There exists a large amount of laboratory data from the Onsøy test fill site, and the parameters defining the compressibility of the soil are interpreted from the most recent block samples. These samples were taken in 2009 from Onsøy. The focus has been on nine standard CRS oedometer tests with one or two unloading-reloading loops. These tests have first been interpreted as shown in the previous chapter, and then the parameters have been adjusted to make a stress-strain curve from PLAXIS to fit the test data.

The oedometer is modelled as a boundary value problem in PLAXIS. The model is axisymmetric with 15-noded elements. The height of the model is 0.02 meter and the radius is 0.01 meter. The pore water is free to dissipate through the top of the sample while the other boundaries are closed. The in-situ stress state is modelled by applying a linear elastic material with a height of 0.01 meters and a unit weight in the area of 1000 – 7500 kN/m^3, which gives an in-situ effective vertical stress of 10-75 kPa in the sample depending of sample depth. The sample is then unloaded to an isotropic stress in the order of 3 to 9 kPa before the loading starts. The oedometer curve in PLAXIS is fitted so that it hits the effective vertical stress which is hold constant for approx. 1000 min. The unloading-reloading-curve is included. Figure 18 show the results from the calculations.

Table 8: Interpreted oedometer results and some index parameters

Test	Depth [m]	σ_{v0}' [kPa]	ε_a at σ_{v0}' [%]	$\acute{\varepsilon}_a$ [%]	OCR	γ [kN/m^3]	e_i	$\lambda^*(1)$	$\lambda^*(2)$	$\kappa^*(3)$	$\kappa^*(4)$	μ^*
3_A1_Ø1	1,01	10,1	0,26	0,6	4,01	17,57	1,28	0,06	0,06	0,016	0,012	0,003
10_A2_Ø1	3,87	28,6	2,03	0,6	1,84	16,41	1,65	0,12	0,14	0,012	0,004	0,005
19_A1_Ø1	7,45	50,6	1,95	0,5	1,32	15,64	2,02	0,20	0,28	0,008	0,006	0,008
22_B_1-CRS	8,86	58,9	2,08	0,4	1,32	15,68	2,02	0,25	0,38	0,018	0,010	0,005
22_B_2-CRS	8,91	59,2	1,98	0,5	1,31	15,65	2,04	0,23	0,34	0,010	0,010	0,005
26_B1_0_B1_35	10,82	69,9	2,24	0,5	1,31	15,88	1,95	0,19	0,32	0,010	0,010	0,005
26_B1_0_B1_50	10,82	69,9	2,52	0,5	1,39	15,91	1,94	0,20	0,44	0,012	0,012	0,005
28_B_11	11,65	74,5	2,59	0,5	1,17	15,98	1,89	0,18	0,24	0,012	0,010	0,005
28_B_12	11,65	74,5	1,88	0,5	1,25	15,97	1,89	0,21	0,35	0,008	0,010	0,005

(1) From straight line between σ_c' to σ_c' + 50 kPa
(2) From most relevant stress after σ_c'
(3) Line from $\varepsilon_a = 0$ and $\sigma_a' = 1$ kPa crossing the oedometer curve at σ_{v0}'
(4) From unloading-reloading curve

Table 9: Input data from simulation of oedometer tests

Test	Depth [m]	σ_{v0}' [kPa]	e_i	$\acute{\varepsilon}_a$ [%]	OCR	λ^*	κ^*	μ^*
3_A1_Ø1	1,01	10,1	1,30	0,6	3,60	0,067	0,015	0,004
10_A2_Ø1	3,87	28,6	1,70	0,6	1,65	0,186	0,037	0,007
19_A1_Ø1	7,45	50,6	2,00	0,5	1,28	0,450	0,028	0,015
22_B_1-CRS	8,86	58,9	2,00	0,4	1,22	0,270	0,023	0,008
22_B_2-CRS	8,91	59,2	2,00	0,5	1,13	0,255	0,025	0,008
26_B1_0_B1_35	10,82	69,9	1,95	0,5	1,20	0,210	0,021	0,007
26_B1_0_B1_50	10,82	69,9	1,95	0,5	1,20	0,203	0,019	0,007
28_B_11	11,65	74,5	1,90	0,5	1,03	0,198	0,025	0,006
28_B_12	11,65	74,5	1,90	0,5	1,14	0,212	0,020	0,005

$\varphi' = 24°$, $c'_{ref} = 5$ kPa , $\psi' = 0°$, $\upsilon_{ur} = 0,15$, $K_0' = 0,6$

The SSC model is not able to capture the non-linearity of real clay. The input data for soil compressibility used in PLAXIS is of this reason too stiff immediately after σ_c' and too soft after the unloading reloading loop. Interpreted parameters for permeability are summarized in Table 10. The relationship between horizontal and vertical permeability is assumed to be 1.5.

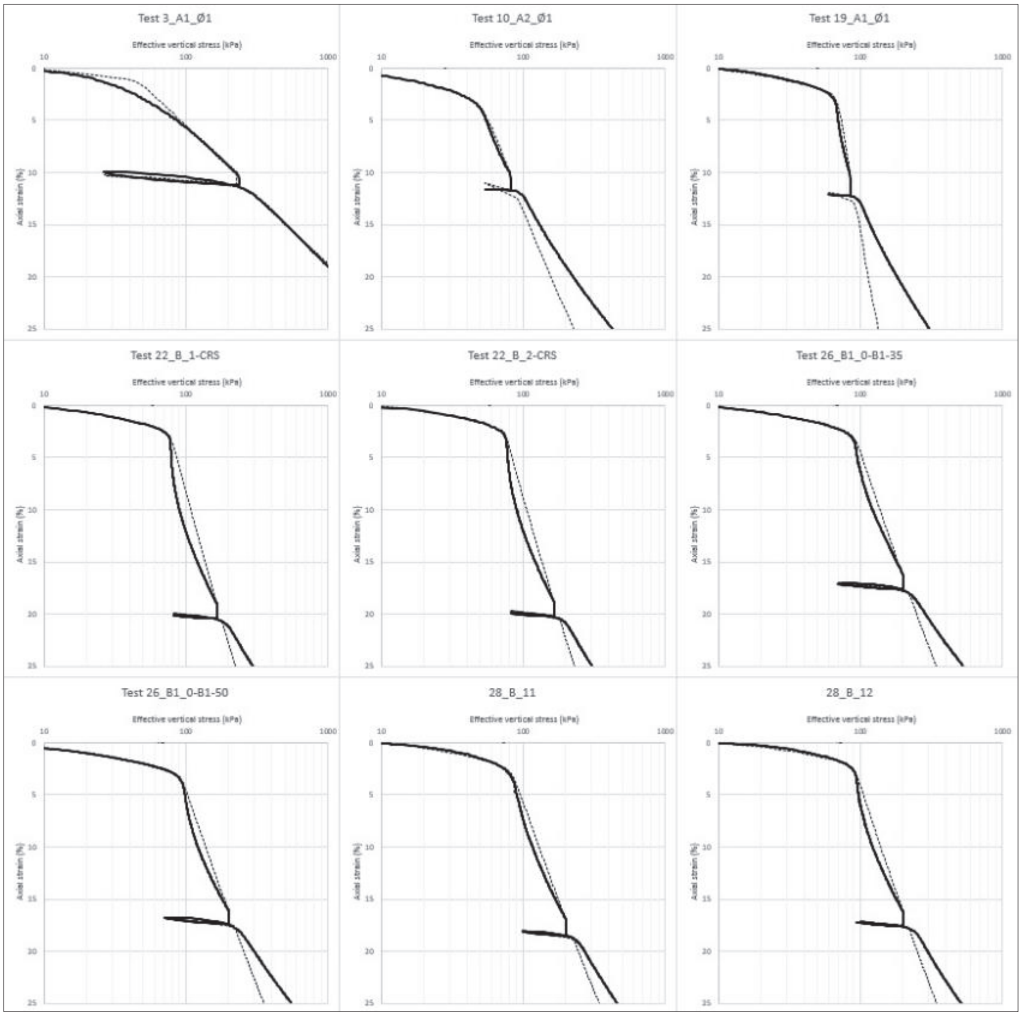

Figure 18: Back-calculation of oedometer tests. Black line is the measurements and dotted line is the simulation in PLAXIS.

Table 10: Permeability parameters interpreted from oedometer tests

Test	Depth [m]	σ_{v0}' [kPa]	k_y [$*E^{-4}$ m/day]	k_x [$*E^{-4}$ m/day]	c_k
3_A1_Ø1	1,01	10,1	1,01	1,52	0,53
10_A2_Ø1	3,87	28,6	3,07	4,60	0,58
19_A1_Ø1	7,45	50,6	3,46	5,18	0,73
22_B_1-CRS	8,86	58,9	0,86	1,30	2,41
22_B_2-CRS	8,91	59,2	1,19	1,79	1,23
26_B1_0_B1_35	10,82	69,9	0,93	1,39	0,98
26_B1_0_B1_50	10,82	69,9	0,86	1,30	1,13
28_B_11	11,65	74,5	1,24	1,86	0,97
28_B_12	11,65	74,5	1,32	1,98	0,84

4.2.4 Soil data

The finite element model consists of five layers from the terrain down to 25 meters. The first layer from depth of 0 to 0.6 meters is dry crust modelled with the soft soil material model. The embankment consists of sand modelled with the Mohr-Coulomb model. The soil behavior for the four lower clay layers are modelled with SSC and the parameters are based on the back calculated oedometer tests.

Table 11: Input data for material layers modelled with SSC

Layer	e_i	λ*	κ*	μ*	k_y [*E^{-4} m/day]	k_x [*E^{-4} m/day]	c_k	OCR	POP [kPa]
0.6 – 2.0 m	1,3	0,07	0,015	0,004	4,0	6,0	0,6	1,0	25
2.0 – 5.0 m	1,7	0,19	0,040	0,007	4,0	6,0	0,8	1,0	25
5.0 – 10.0 m	2,0	0,25	0,025	0,008	1,5	2,0	1,0	1,0	25
10.0 – 25.0 m	1,9	0,20	0,020	0,006	1,5	2,0	1,0	1,0	25
	$\varphi' = 28°$, $c'_{ref} = 3$ kPa , $\psi' = 0°$, $\upsilon_{ur} = 0,15$, $K_0^{NC} = 0,6$, $K_{0,x} = 0,6$, $\gamma = 16,3$ kN/m^3								

Table 12: Input data for material layer modelled with SS

Layer	e_i	λ*	κ*	μ*	k_y [*E^{-4} m/day]	k_x [*E^{-4} m/day]	c_k	OCR	POP [kPa]
Dry Crust, 0 – 0.6 m	1,2	0,065	0,005	-	100,0	100,0	1	1,0	170
	$\varphi' = 28°$, $c'_{ref} = 3$ kPa , $\psi' = 0°$, $\upsilon_{ur} = 0,15$, $K_0^{NC} = 0,55$, $K_{0,x}$ = Auto , $\gamma = 17,8$ kN/m^3								

Table 13: Input data for material layer modelled with MC

Layer	e_i	γ [kN/m^3]	c'_{ref} [kPa]	φ [°]	k_y [*E^{-4} m/day]	k_x [*E^{-4} m/day]	c_k	E' [kPa]	υ'
Sand fill	0,5	20,3	0,01	39,0	100 000	100 000	10E^{15}	20 000	0,3

4.2.5 Result and analysis for prediction

Figure 19 shows the calculated settlements for a point just under the centerline of the fill. For this calculation, there is one line without updated mesh and updated pore pressure (UM) and one line with this option turned on. When comparing these two lines with the measured settlements it is clear that the calculation with UM is more parallel to the measured results, but the line without UM fits more of the measured curve. The rate of settlement after 1000 days is more correct when using UM. The final settlement after 1120 days is off by approx. 6.3% for the calculation with UM, which is a good result for predicting the total settlement.

Updated mesh is an option in PLAXIS, which should be used when large deformations occur, for example in soft soils. The function is based on large deformation theory and uses the co-rotational rate of Kirchhoff stress and Updated Lagrangian formulation (PLAXIS, 2012). When parts of an embankment settle below the ground water, the buoyancy effect will reduce the load on the original terrain. This effect is captured in PLAXIS when using updated mesh together with updated water pressure. This option is used for all calculation presented in this report, except for the example (dotted line) in Figure 19.

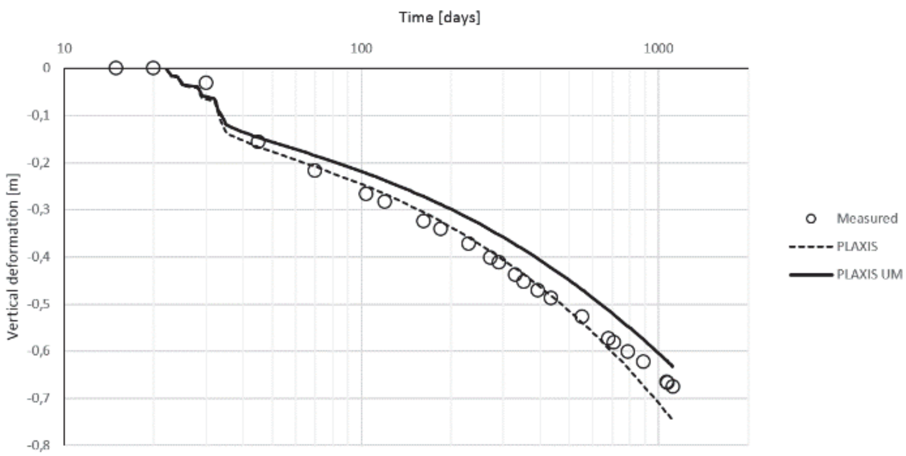

Figure 19: Settlement just under the fill with SSC (prediction)

Figure 20 and Figure 21 show the excess pore pressure under the centerline of the fill and five meters out from the centerline. Day 36 is the last step when building up the embankment. The results are taken from the updated mesh calculation. The figure shows that the pore pressure after 36 days is over predicted by approx. 25 %. After 1120 days, the calculated pore pressure shows a good agreement with the measurements for the upper 12 meters. At the depth of 20 meters, the pore pressure is over predicted by 20 to 100 %.

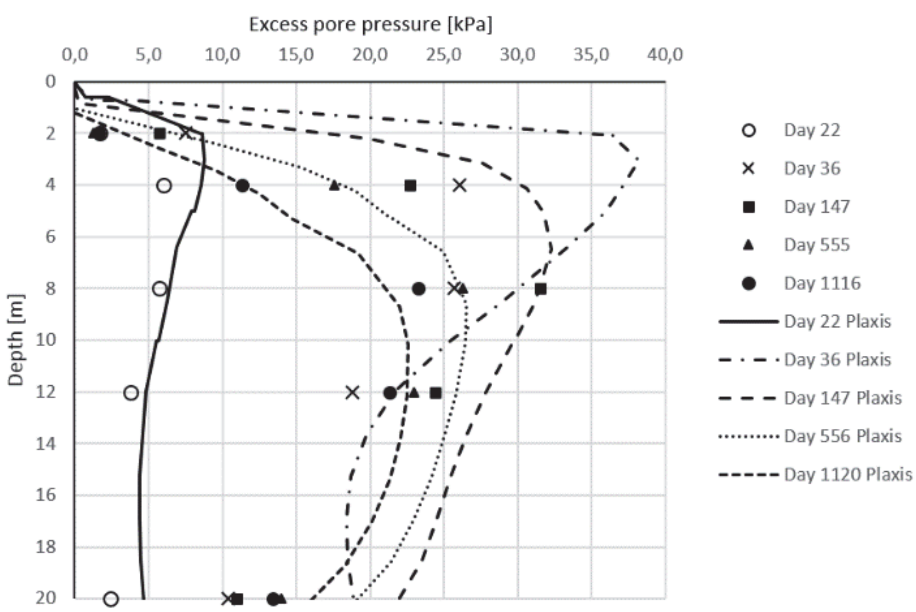

Figure 20: Excess pore pressure under centerline of fill with SSC (prediction)

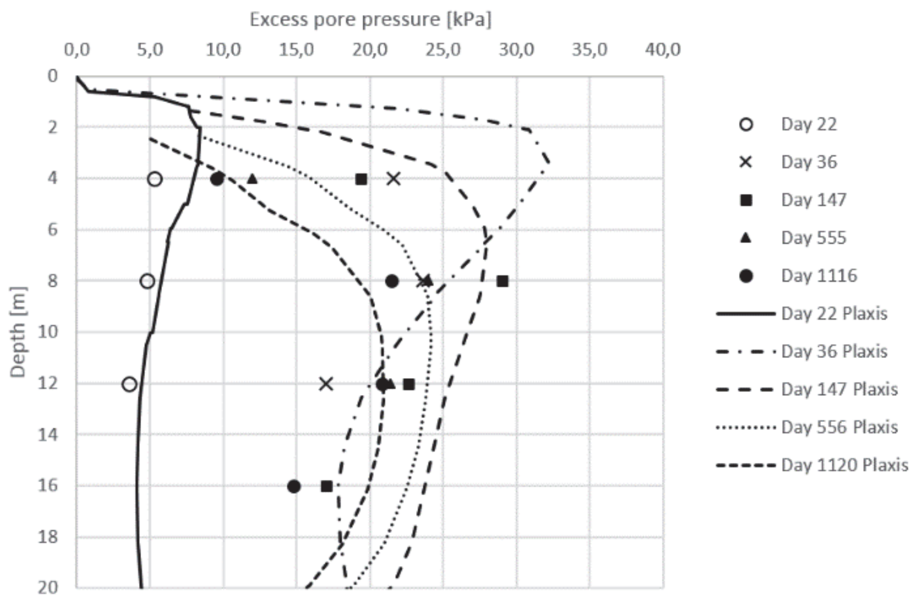

Figure 21: Excess pore pressure 5 m from centerline of fill with SSC (prediction)

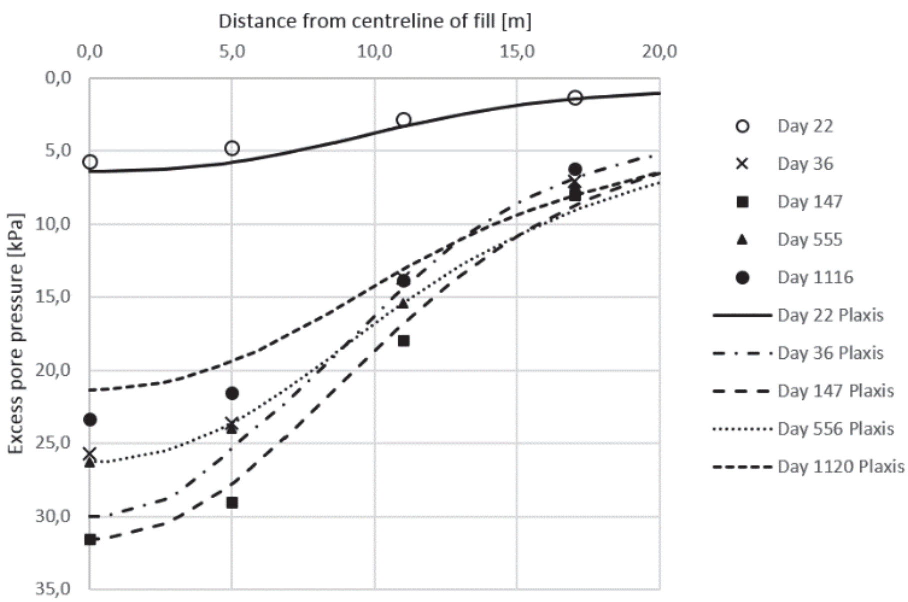

Figure 22: Excess pore pressure 8 m below ground surface with SSC (prediction)

The pore pressure predicted at 8 meter depth agrees well with measured data, especially for day 22, 147, 555 and 1120. For day 36, the pore pressure is too high just under the embankment. Usually it is expected that the excess pore pressure is close to the size of the load when dealing with an undrained material, but the measurements shows a ratio between excess pore pressure and load of around 0.7.

Figure 23 shows the horizontal displacement measured 5 meters west and east of the centerline after 36, 147 and 1116 days. The calculation shows a tendency to over predict the undrained displacement at day 36, and under predict the long-term horizontal displacement for the depth interval of 0 to 8 meters. This implies that the undrained shear stiffness is too low, and the M-parameter is too high to get the right amount of shear creep over time. The SSC model does not take in to account the highly non-linear shear stiffness starting with an initial high (small strain) stiffness, which is found in most soil materials, and this explains why the model is unable to predict realistic horizontal deformations under undrained condition. By decreasing K_0^{NC} it is possible to decrease the horizontal displacement, but they would probable still be too high.

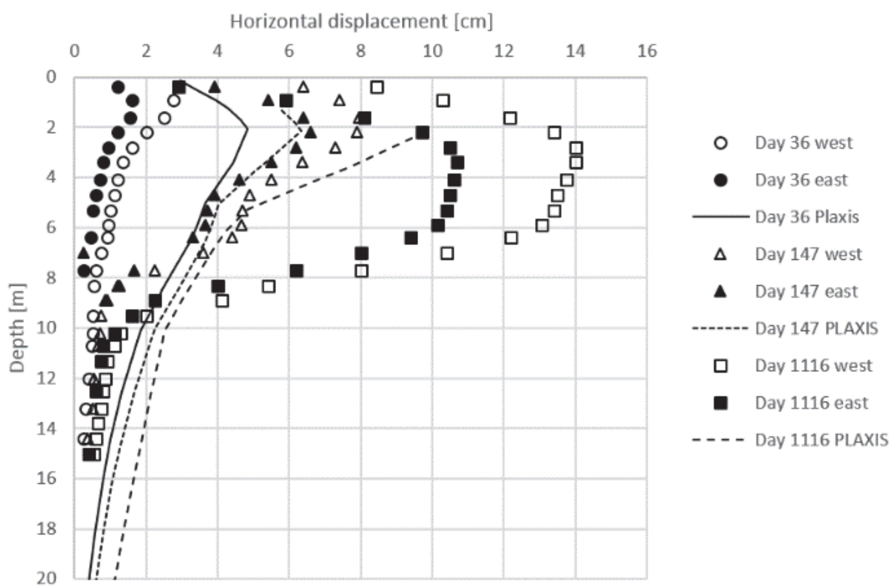

Figure 23: Horizontal displacement 5 m at each side of the centreline with SSC (prediction)

Figure 25 shows the vertical displacement both beneath and outside the embankment. If the calculation had been run for 1120 days without the fill, the settlements would be the same as seen at each side of Figure 25. The soil creeps without any change in effective stress and this can explain the high excess pore pressure at 20 meters depth in Figure 20, the large horizontal displacement at the depth from 9 to 20 m and the large vertical strains in the same depth interval. In Figure 24 there is expected some over prediction of the vertical strains because the reality is 3-dimensional and not plain strain. The load distribution is therefore a bit different in the real case.

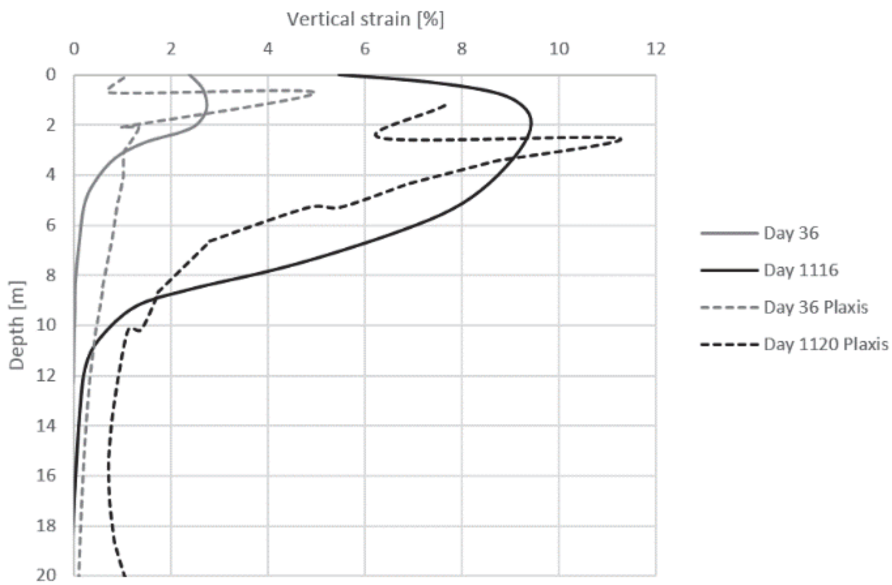

Figure 24: Vertical strains under centreline of fill with SSC (prediction)

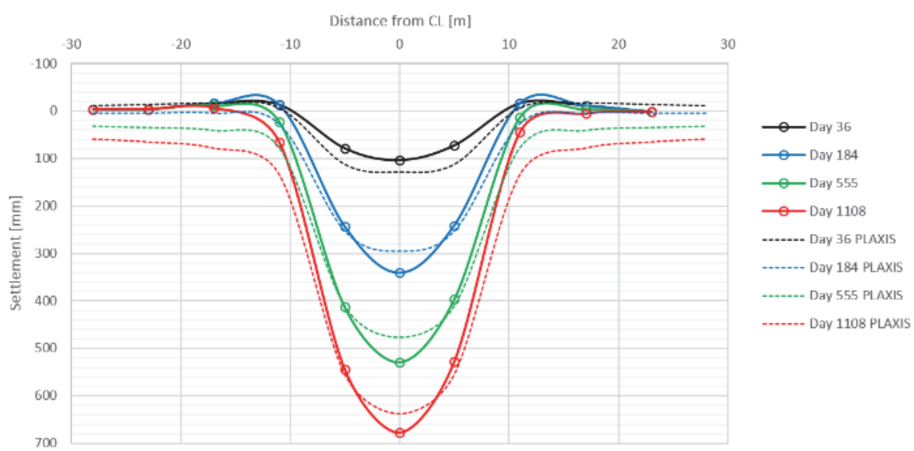

Figure 25: Surface settlement with SSC (prediction)

4.2.6 Creep strain rate

The prediction is good at estimating the total settlements, but the vertical strain distribution, surface settlement outside the embankment, horizontal displacement and pore pressure versus depth are wrong. The main reason for this is the large creep strain rate with the selected input parameters. The creep strain rate is controlled by the following equation:

$$\dot{\varepsilon}_c = \dot{\varepsilon}_{c0} \cdot OCR^{-\left(\frac{\lambda^* - \kappa^*}{\mu^*}\right)}$$

Where the modified creep index controls the initial creep strain rate,

$$\dot{\varepsilon}_{c0} = \frac{\mu^*}{\tau_0}$$

I this case the initial creep rate for the layer at depth of 10 to 25 m would approx. be:

$$\dot{\varepsilon}_c = \frac{0{,}006}{1\ day} \cdot 1{,}25^{-\left(\frac{0{,}2-0{,}02}{0{,}006}\right)} \cdot 365\,\frac{days}{year} = 0{,}27\ \frac{\%}{year}$$

This gives a settlement in the area of 12 cm after 1120 days without any loading if the creep rate had been constant. In reality, the creep rate will decrease with time because OCR is increasing with time in the SSC model. Table 14 illustrates how the creep rate changes depending on OCR and the relationship between the compressibility parameters.

Table 14: Creep rate depending on OCR and compressibility

$\frac{\lambda^*-\kappa^*}{\mu^*}$ / OCR	30 [% / year]	25 [% / year]	20 [% / year]	15 [% / year]	10 [% / year]
1,1	21	34	54	87	140
1,2	1,5	3,8	9,5	24	59
1,4	0,02	0,08	0,4	2,3	13
1,6	0,0003	0,003	0,03	0,3	3,3
1,8	0,00001	0,0002	0,003	0,05	1,0

$\mu^* = 0{,}01$ and $\tau_0 = 1$ day

The red box in Table 14 shows creep rates that can be considered as negligible. A creep rate of approx. 0.01 % / year or less is acceptable for an undisturbed clay that is more than thousand years old.

4.2.7 Optimized soil data for best fit

When optimizing the soil parameters to obtain a best fit for the measured data, focus is first on increasing OCR to reduce the creep strain rate. This will in fact reduce the total settlements, therefore other parameters has to be altered as well. Depending on how the vertical stresses after loading are compared to the initial pre-consolidation stress, increasing λ* can lead to a decrease of settlement. This is because an increase in λ* will decrease the creep strain rate as shown in Table 14. To account for the lack of small strain stiffness κ* is made small in the bottom layer. To increase the vertical strain between 0 to 10 m depth the creep number μ* and the permeability is increased. The new soil input parameters are shown in table 15.

Table 15: Input data for material layers modelled with SSC

Layer	e_i	λ^*	κ^*	μ^*	c'	φ'	K_0^{NC}	k_y [*E^{-4} m/day]	k_x [*E^{-4} m/day]	c_k	OCR	POP [kPa]
0.6 – 2.0 m	1,3	0,07	0,020	0,006	3	25	0,577	15,0	20,0	0,5	1,0	25
2.0 – 5.0 m	1,7	0,25	0,020	0,008	2	24	0,593	15,0	20,0	0,5	1,0	25
5.0 – 10.0 m	2,0	0,25	0,020	0,010	2	24	0,593	1,5	2,0	1,0	1,0	25
10.0 – 25.0 m	1,9	0,25	0,012	0,006	3	25	0,577	0,3	0,5	1,0	1,3	0
	$\psi' = 0°$, $\upsilon_{ur} = 0,15$, $K_{0,x} = 0,6$, $\gamma = 16,3$ kN/m^3											

The friction angle is lowered to fit the long-term horizontal deformations. K_0^{NC} is based on the new friction angle. As can be seen in Table 16, the initial creep strain rate is not very different in the top three layers, but it is decreased significantly in the bottom layer.

Table 16: Estimated average creep strain rates at the initial state

Layer	$\dot{\varepsilon}_{c0}$ [1 / year]	OCR (average)	$\dfrac{\lambda^* - \kappa^*}{\mu^*}$		$\dot{\varepsilon}_c$ [% / year]
0.6 – 2.0 m	2,19 (1,5)	3,84	8,3 (13,8)	⇒	0,0029 (1,9 E^{-6})
2.0 – 5.0 m	2,92 (2,6)	2,10	28,8 (21,4)	⇒	1,6 E^{-7} (3,3 E^{-5})
5.0 – 10.0 m	3,65 (2,9)	1,53	23,0 (28,1)	⇒	0,0206 (0,002)
10.0 – 25.0 m	2,19 (2,2)	1,30 (1,23)	39,7 (30,0)	⇒	0,0066 (0,44)
Numbers in parentheses are from the first calculation					

Another way of increasing the settlements is to adjust the permeability parameters. In the first calculation the permeability was based on the block samples in (Berre, 2010). A figure produced by (Lunne *et al.*, 2003) shows the permeability with depth with several lab and field tests. This figure indicates that the permeability can be lower in the bottom layer and higher from 0.6 to 5 meter depth.

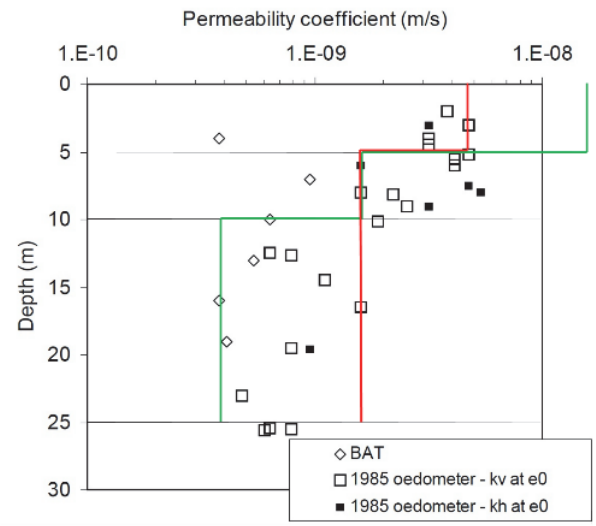

Figure 26: Permeability with depth at Onsøy(Lunne et al., 2003). Red line is the SSC-prediction. Green line is from SSC-"best fit".

4.2.8 Results from best fit calculation

Figure 27, 28 and 29 shows effective vertical stress, total vertical stress, preconsolidation pressure and pore water pressure before loading, after 36 days and 1120 days.

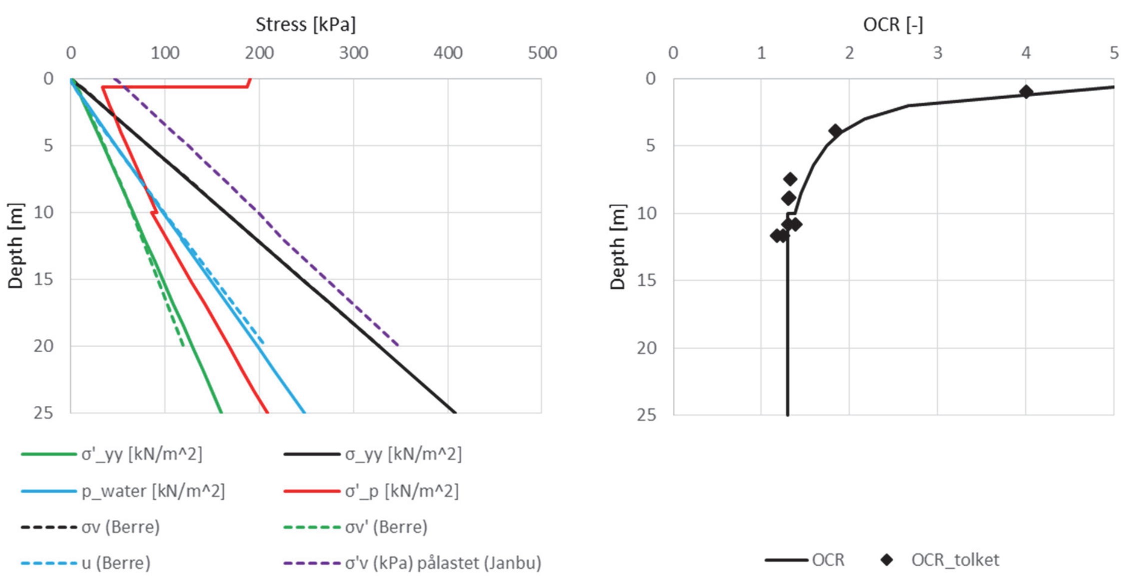

Figure 27: Effective stress, total stress and OCR before loading under centerline.

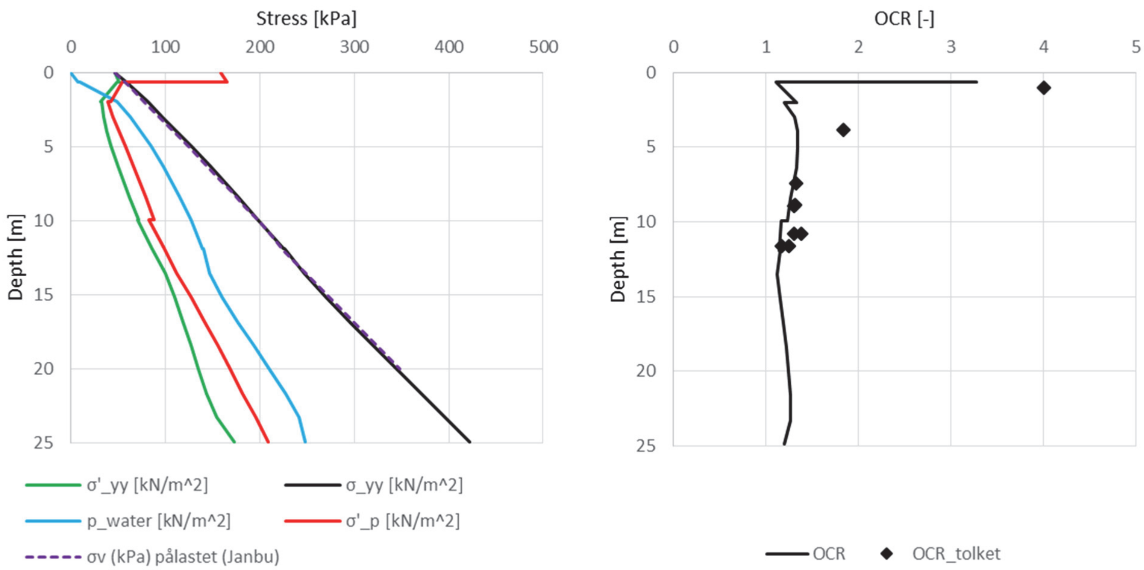

Figure 28: Effective stress, total stress and OCR at day 36 under centerline.

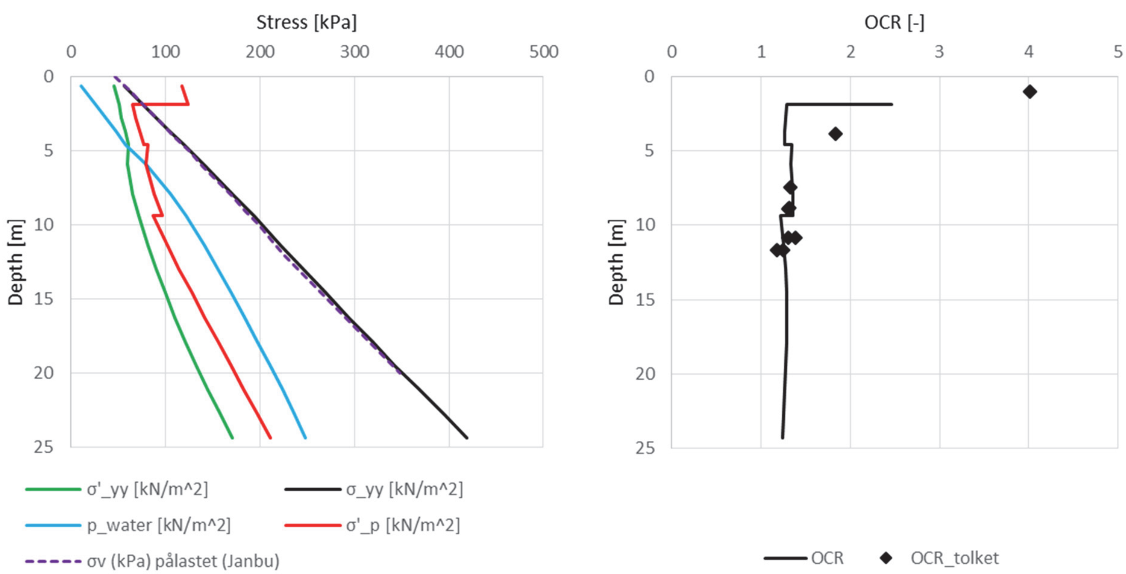

Figure 29: Effective stress, total stress and OCR at day 1120 under centerline.

Figure 30 shows an almost perfect fit for the total settlements just under the fill. This is achieved with only small changes in the soil parameters. One could argue that the estimated settlements should be a bit larger due to a perfect plain strain condition in PLAXIS and a more 3-dimensional effect in reality, especially in deeper layers. The fact that SSC over predicts the horizontal deformations also imply that the vertical deformation should be larger the first 100 to 200 days. In this case, it is chosen to fit the whole curve, and this will be a reference for the rest of the calculations.

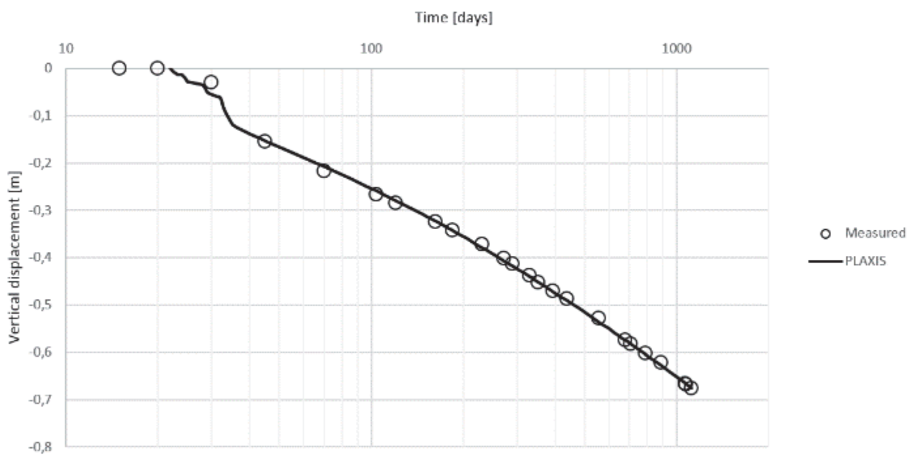

Figure 30: Settlement just under the fill with SSC (best fit)

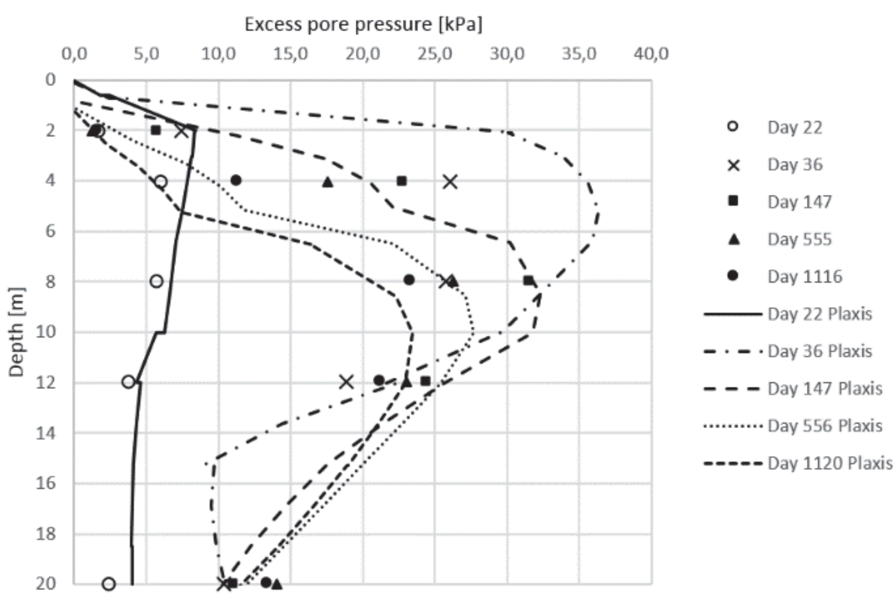

Figure 31: Excess pore pressure under centreline with SSC (best fit)

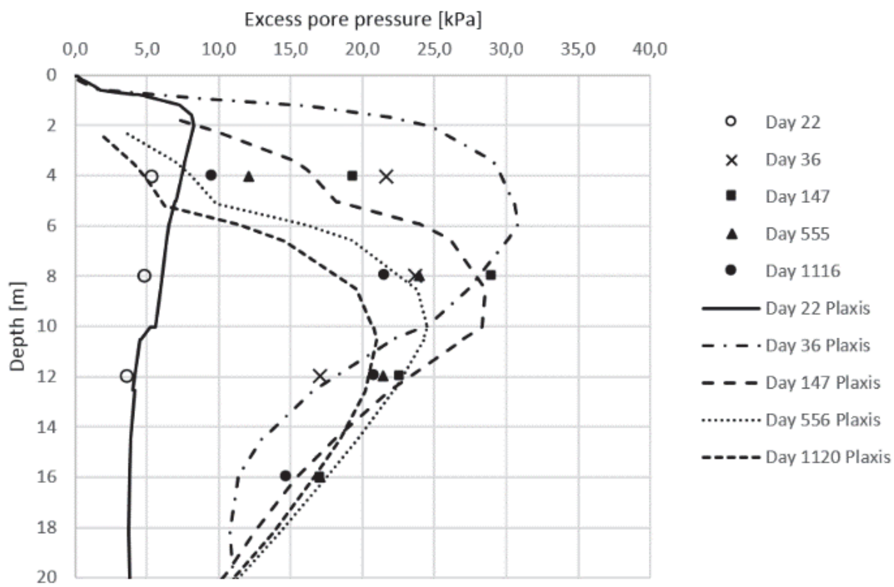

Figure 32: Excess pore pressure 5 m from centreline with SSC (best fit)

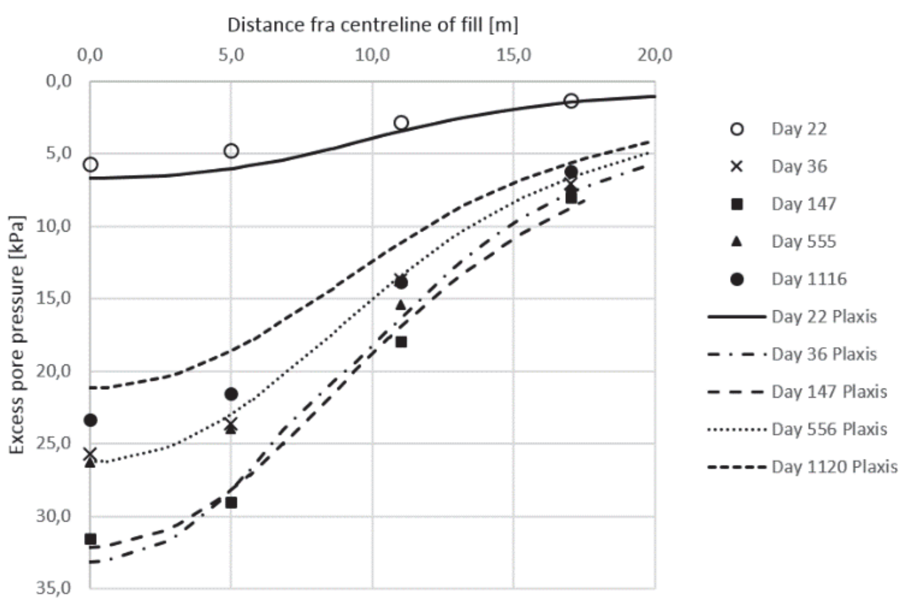

Figure 33: Excess pore pressure 8 m below ground surface with SSC (best fit)

The calculated pore pressure shows a good agreement with the measurements. For the depth interval from 10 to 20 meter the new parameters gives a better result than the previous parameter set. Still the excess pore pressure at day 36 is too high compared to the measurements. Figure 34 show how the pore pressure develops over time at 4 and 12 meters depth. The pore pressure is increasing more with each load step in the model than in reality. The average ratio between the excess pore pressure response and applied load is 0.7 for the measurements and 0.83 for the calculation at 4 meters depth.

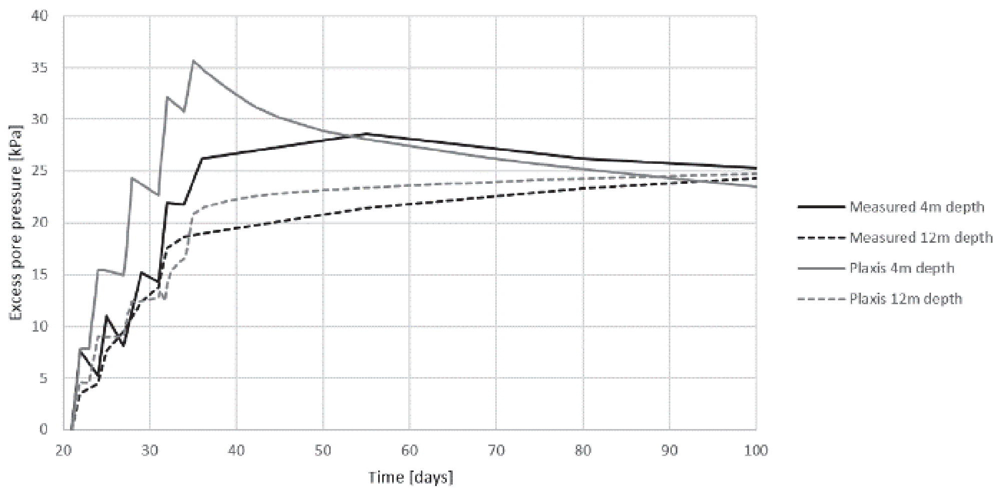

Figure 34: Excess pore pressure versus time with SSC (best fit)

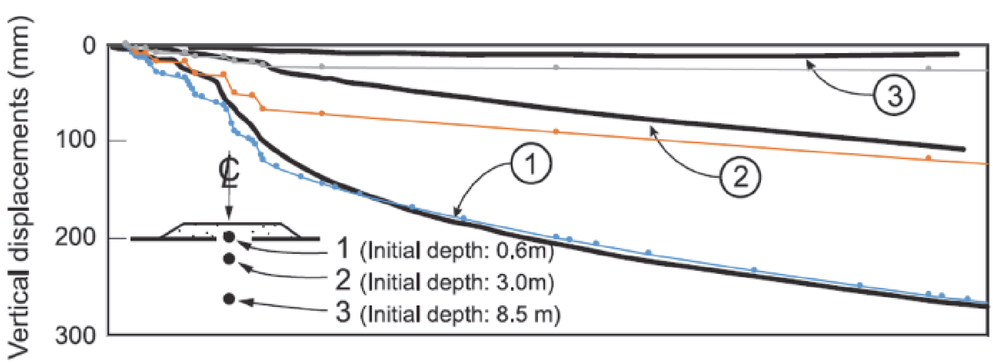

Figure 35: Settlement versus time. Blue, orange and grey line from PLAXIS. SSC (best fit)

For the upper 8 meters, the horizontal deformations are almost the same as in the prediction for a cross-section 5 meter out from the centerline. Because of the reduced friction angle, the deformations are a bit larger after 1120 days. SSC is still not able to give a good estimate of the short-term undrained shear deformations.

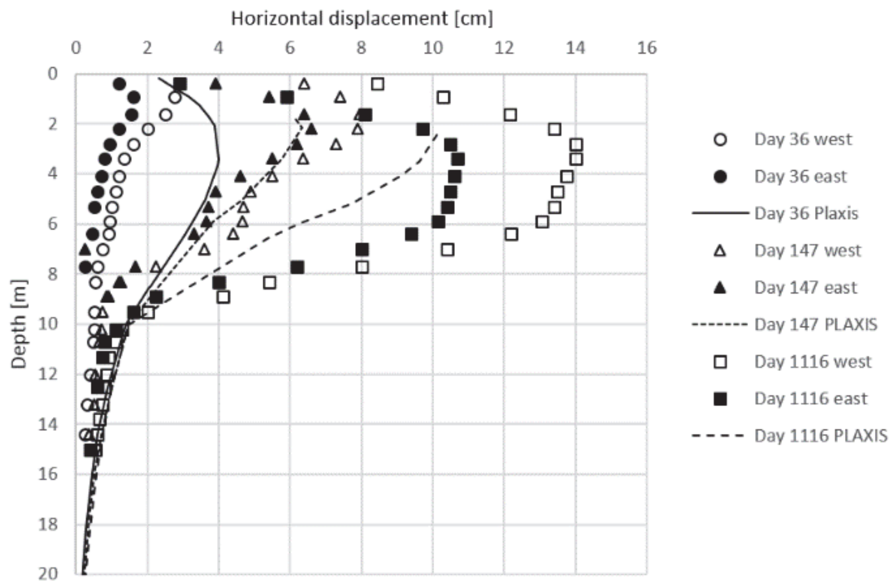

Figure 36: Horizontal displacement 5 m from each side of centerline with SSC (best fit)

The vertical strains with depth at the centerline and the surface settlements are captured with good accuracy. The problem with large settlements outside the fill is now almost negligible. However, the vertical strains predicted after 36 days are too large from depth of 3 to 20 meters. This is probable due to the low shear stiffness in the SSC.

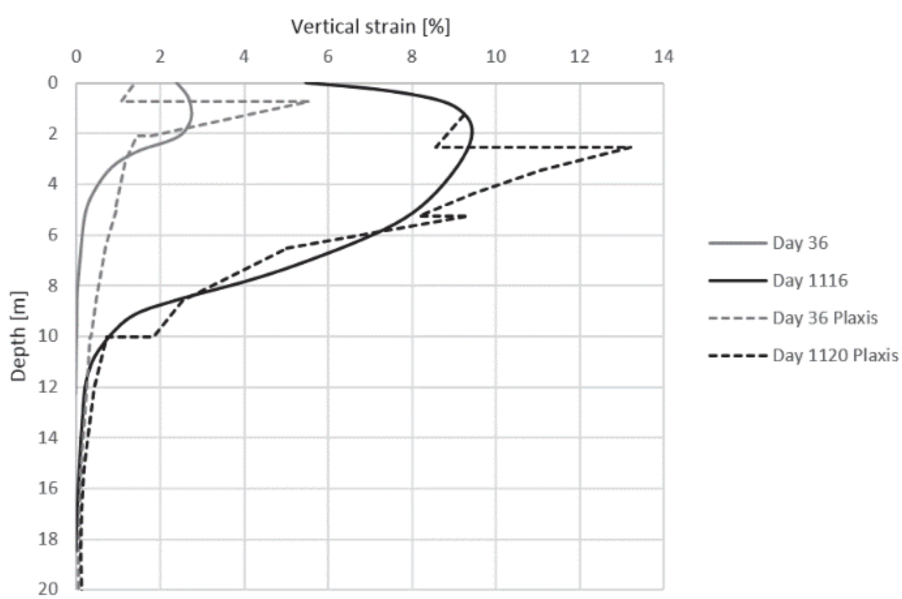

Figure 37: Vertical strain with depth under centerline with SSC (best fit)

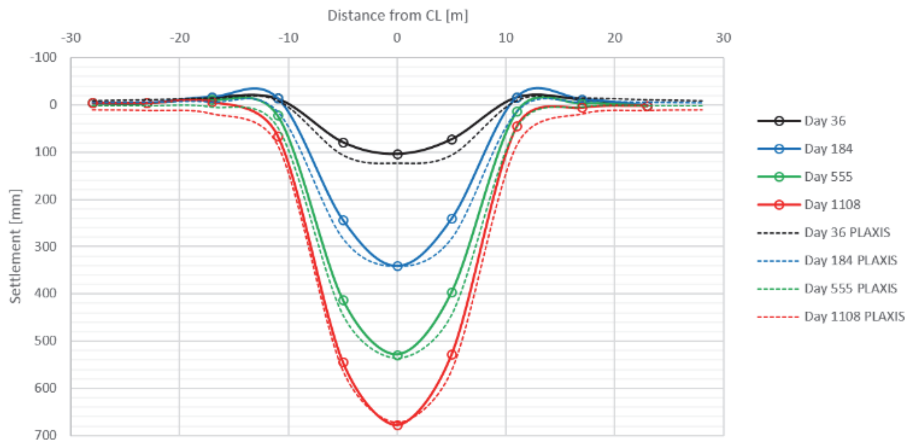

Figure 38: Surface settlement with SSC (best fit)

Overall, the SSC model gives a good prediction of vertical deformation and pore pressure for the Onsøy test fill, with reasonable soil parameters. The main issue with the model is predicting horizontal deformation under undrained loading. Resolving this issue will also improve the vertical strains under this type of loading.

4.3 Calculations with Critical State Soft Soil Creep (G)

4.3.1 Soil data

This model has a small strain G-modulus as input. Besides the G-modulus input, the model is more or less the same as SSC. The increased shear stiffness should give a better fit for the horizontal displacement of the clay beneath the embankment. Since the problem with large horizontal deformations is located to the three lowest layers, only those are modelled with CS-SSCG. The layer from 0.6 to 2 meters has the same parameters as the calculations presented in Chapter 4.2.

Table 17: Input data for material layers modelled with CSSSCG

Layer	ζ	η	K_0^{NC}	M_c	τ [day]	P_{ref} [kPa]	y_{ref} [m]	G_{ref} [kPa]	G_{inc} [kPa/m]
2.0 – 5.0 m	0,9	0,55	0,593	1,41	1	100	25	6500	1250
5.0 – 10.0 m	0,9	0,55	0,593	1,41	1	100	25	6500	1250
10.0 – 25.0 m	0,6	0,55	0,577	1,45	1	100	15	19 000	1450
Input data for soil compressibility, permeability, initial stress condition and soil strength on interfaces are the same as in SSC "best-fit"									

Figure 39 (Lunne *et al.*, 2003) shows how the selected shear stiffness profile compares to measured data from Onsøy.

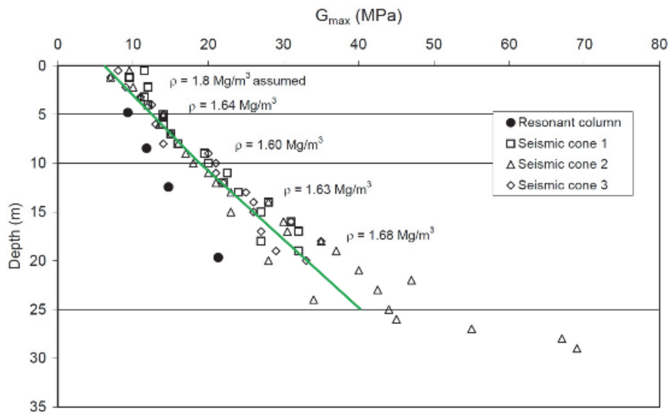

Figure 39: G_{max} versus depth (Lunne et al., 2003). Green line is input.

4.3.2 Results

The total settlements is captured in a satisfactory manner also with this model, but it shows slightly less settlement the first 200 days and slightly more the last 900 days, see Figure 40.

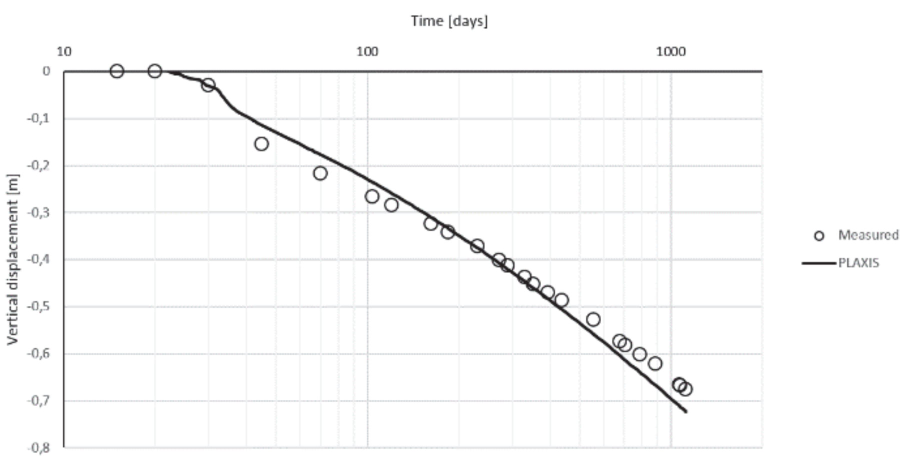

Figure 40: Settlement just under centreline with CS-SSCG

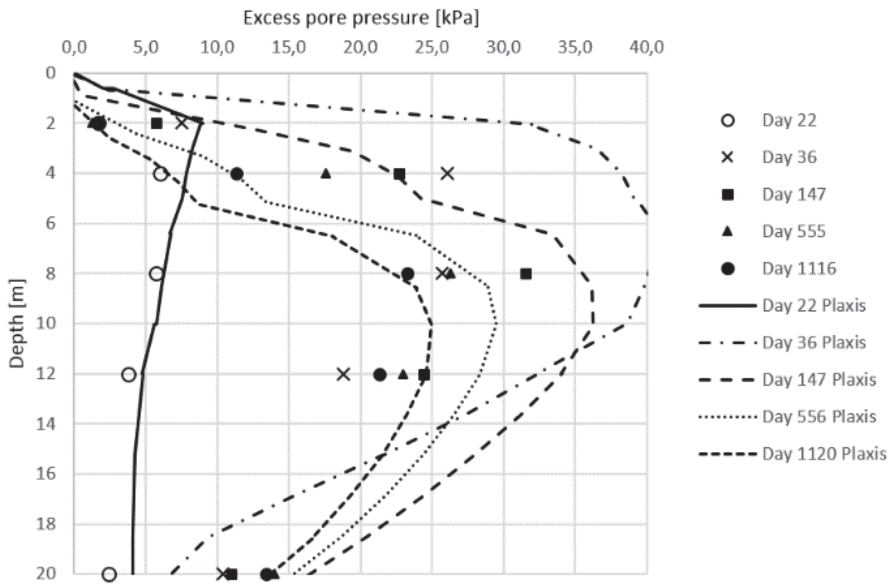

Figure 41: Excess pore pressure under centreline with CS-SSCG

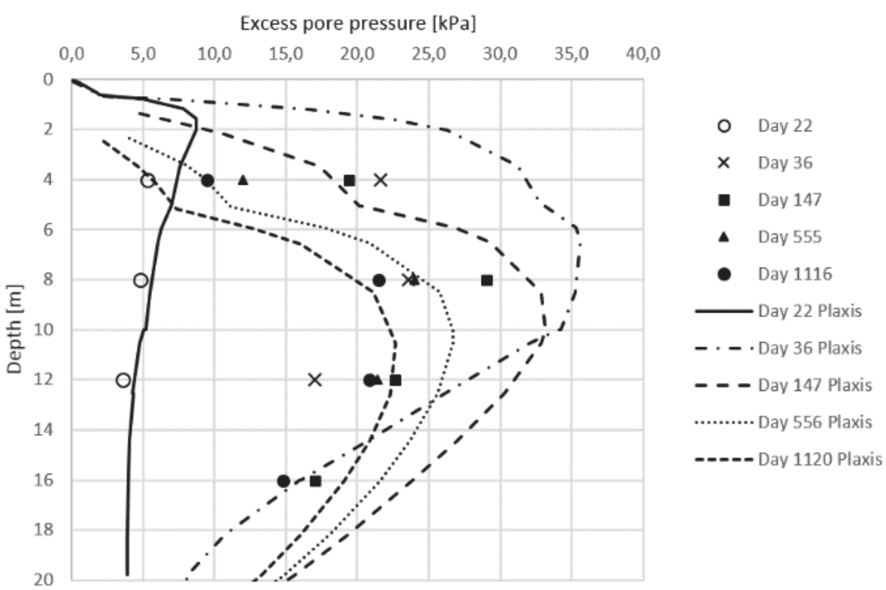

Figure 42: Excess pore pressure 5 m from centreline with CS-SSCG

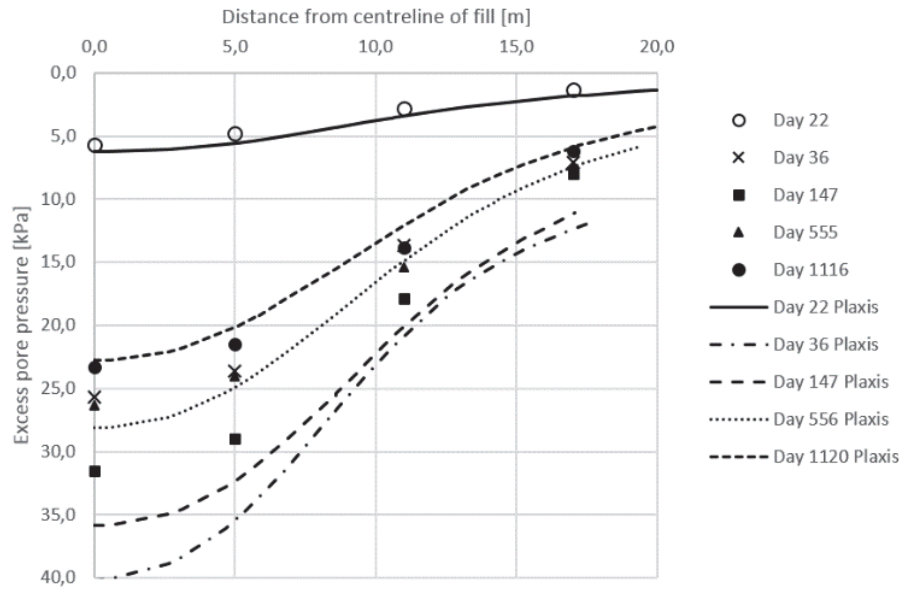

Figure 43: Excess pore pressure 8 m below centreline with CS-SSCG

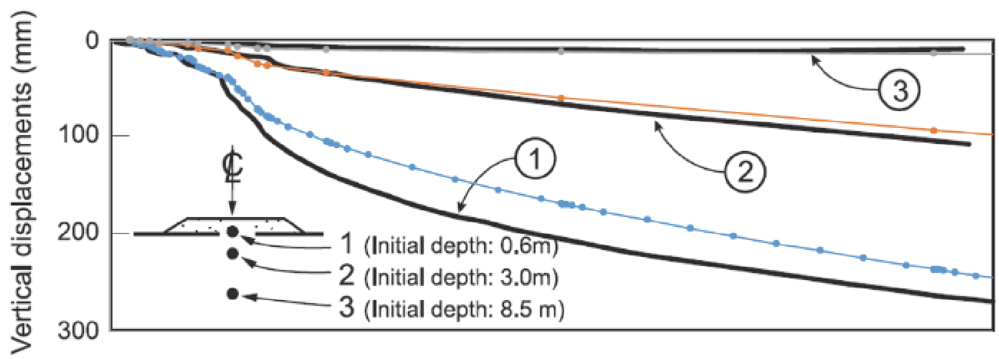

Figure 44: Settlement versus time. Blue, orange and grey line is PLAXIS. CS-SSCG

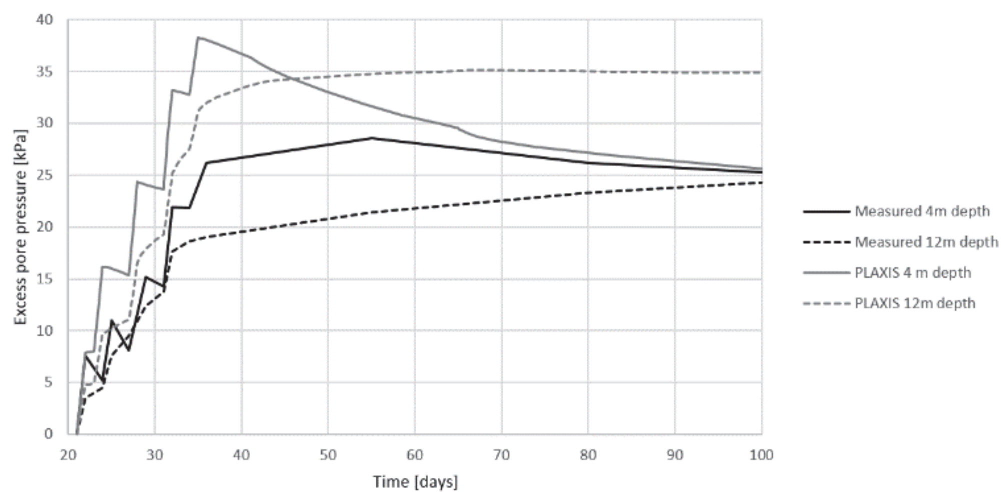

Figure 45: Excess pore pressure versus time with CS-SSCG

Figure 41 to Figure 43 shows that the excess pore pressure is over predicted for day 36, 147 and 555. Figure 45 indicates that the average ratio between the excess pore pressure response and applied load is 0.87 for this calculation, which is significantly higher than measured.

The horizontal deformations are displayed in Figure 46. The model gives an almost perfect prediction after 36 days. This is also reflected by the vertical strains in Figure 47. The reason for different vertical strain profile after 1120 days between the SSC and CSSSCG is probable due to the differences in implementation of the models, e.g. creep related to the plastic multiplier and lode angle dependency.

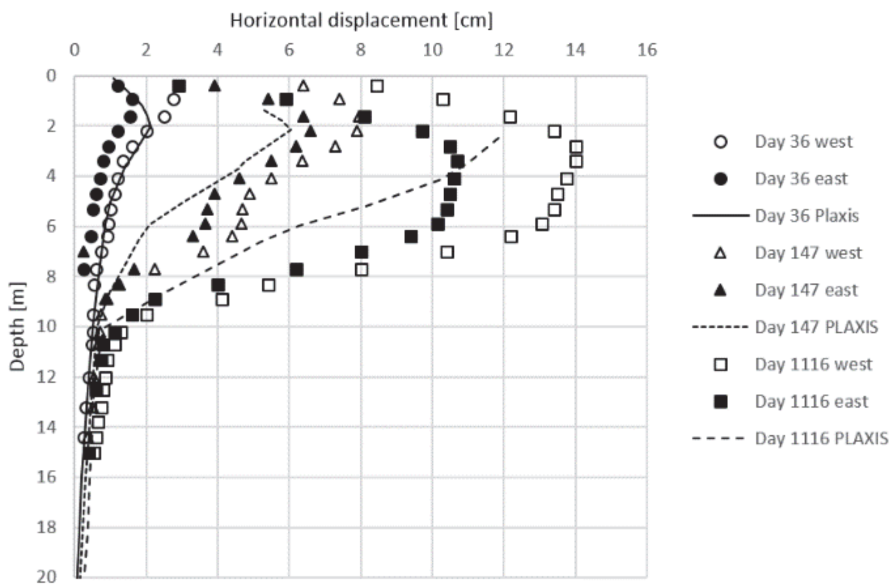

Figure 46: Horizontal displacement 5 m from centreline with CS-SSCG

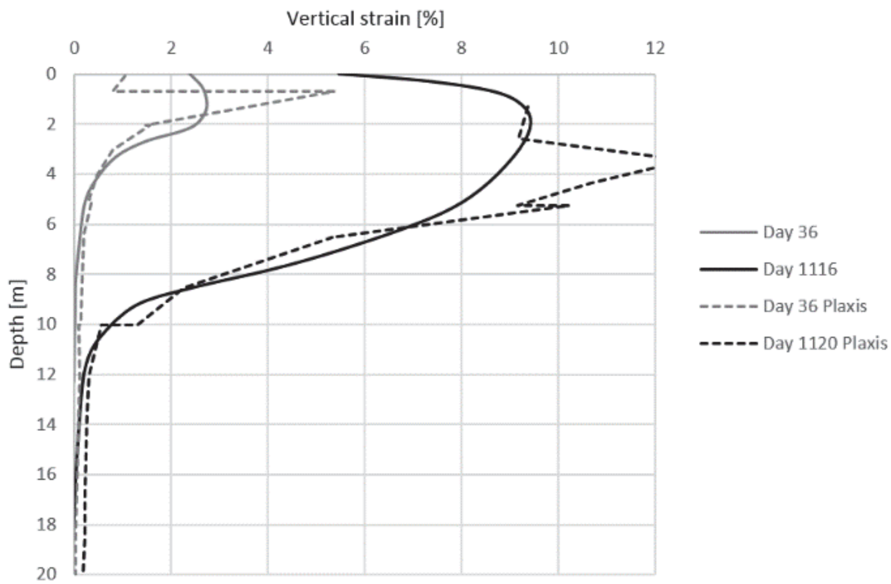

Figure 47: Vertical strain in centreline with CS-SSCG

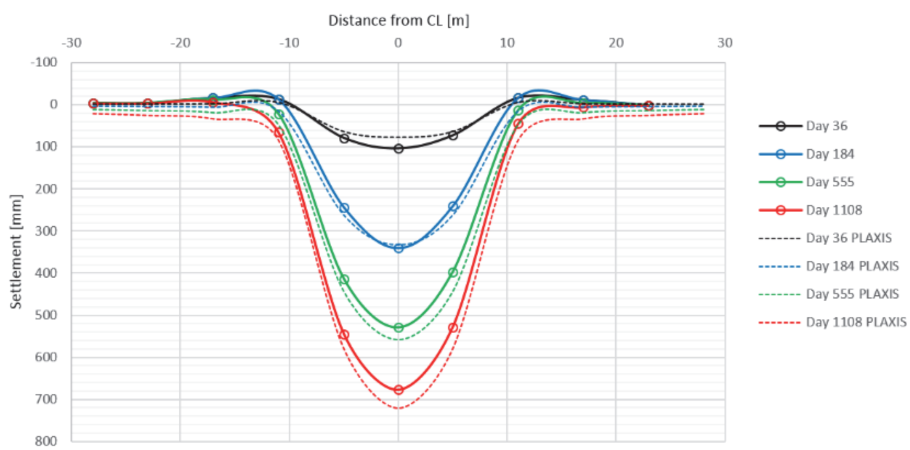

Figure 48: Surface settlement with CS-SSCG

4.4 Calculations with Sekiguchi – Ohta

This soil model did not give the right initial conditions in the first calculation phase, and it was therefore necessary to use the SSC model to create the initial stress condition. When the model finally ran, it was difficult to obtain good results with a reasonable $\dot{v}_0$-value. Because of the fact that $\dot{v}_0$ is a relationship between α^* and t_c, which is the time for end of primary consolidation, there are uncertainties regarding interpretation of this parameter. The results are therefore not shown in this report.

4.5 Calculations with n-SAC

4.5.1 Soil data

The soil parameters required as input in the n-SAC model is slightly different from the SSC model. To determine the input data, three different laboratory tests are necessary: CRS and IL oedometer tests and one undrained triaxial test (CAUC). After the parameters are interpreted, all three laboratory tests should be back-calculated. A short description on how to determine the most important parameters is given below.

Stiffness parameters, E_{ref} and E_{oed}^{ref}

The reference E-modulus, E_{ref}, is controlling the stiffness in the over consolidated stress state. This parameter is interpreted from a standard CRS oedometer test with an unloading-reloading(UR)-loop. The slope of the UR-loop in a log(σ')-ε-plot gives $C_{r\varepsilon}$.

$$C_{r\varepsilon} = \frac{C_r}{(1+e_0)} = \frac{\ln 10}{2} \cdot \kappa^*$$

$$m_{OC} = \frac{1}{C_{r\varepsilon}}$$

$$M_{OC}^{ref} = m_{OC} \cdot p'_{ref} = m_{OC} \cdot 100\ kPa$$

$$E_{ref} = M_{OC}^{ref} \cdot \frac{(1-2\upsilon)(1+\upsilon)}{(1-\upsilon)}$$

When calculating the relationship between the Young's modulus and the oedometer modulus, a low Poisson ratio, e.g. $\upsilon = 0.15$, should be used. Below is an example with a low κ* from Chapter 4.2.

$$E_{ref} = \frac{1}{\left(\frac{\ln 10}{2}\kappa^*\right)} \cdot 100 \cdot \frac{(1-2\upsilon)(1+\upsilon)}{(1-\upsilon)}$$

$$E_{ref} = \frac{1}{\left(\frac{\ln 10}{2}0{,}004\right)} \cdot 100 \cdot \frac{(1-2\cdot 0{,}15)(1+0{,}15)}{(1-0{,}15)} = 21\,700 \cdot 0{,}947$$

$$E_{ref} \approx 20\,500\ kPa$$

The reference oedometer stiffness, E_{oed}^{ref}, is one of the parameters controlling the stiffness in the normally consolidated stress state. This parameter is also interpreted form a standard CRS oedometer tests. E_{oed}^{ref} is a secant stiffness defined by a straight line in an M-σ'-plot. The oedometer test should be loaded to a high stress level. The reference oedometer stiffness can also be obtained from a log(σ')-ε where a straight line is drawn to fit the last part of the stress-strain-curve where it becomes straight at high stress levels as shown in Figure 49 and Figure 50. This line is also called the intrinsic normal compression line for a reconstituted sample. The slope is given by the modified intrinsic compression index, λ_i*.

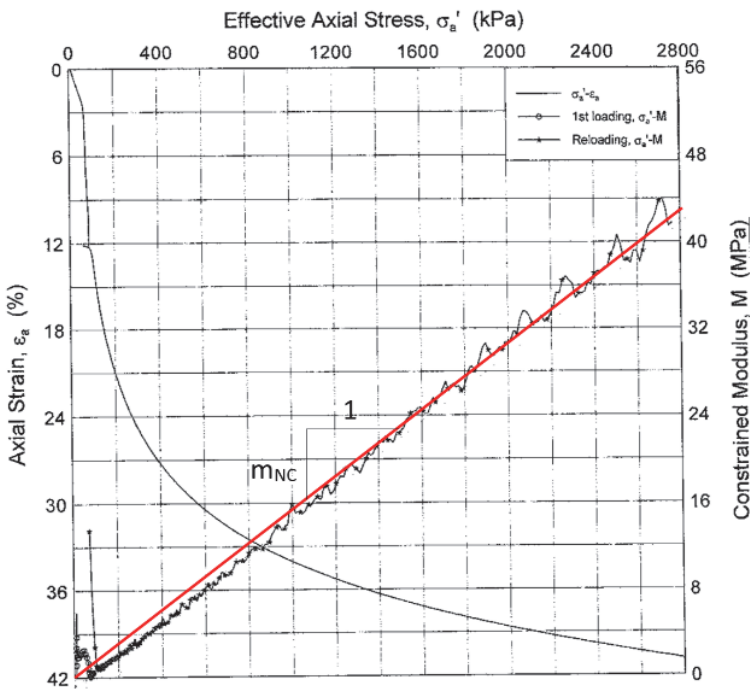

Figure 49: Interpretation of m_{NC}

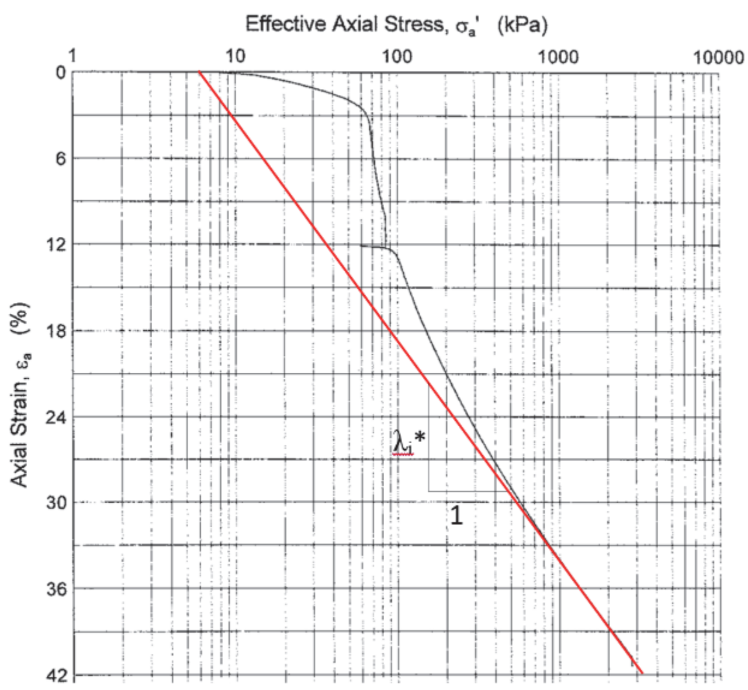

*Figure 50: Interpretation of λ_i**

The lines from Figure 49 and Figure 50 gives:

$$m_{NC} = \frac{\Delta M}{\Delta \sigma} = \frac{43\ 000}{2\ 800} = 15{,}5$$

$$\lambda_i{}^* = \frac{\Delta \varepsilon}{\ln\frac{\sigma_2}{\sigma_1}} = \frac{0{,}42}{\ln\frac{3100}{5}} = 0{,}067$$

$$m_{NC} = \frac{1}{\lambda_i{}^*} = \frac{1}{0{,}067} = 14{,}9$$

$$E_{oed}^{ref} = m_{NC} \cdot p'_{ref} = 15{,}0 \cdot 100\ kPa = 1500\ kPa$$

Creep parameters, $r_{s,min}$ and $r_{s,i}$

The creep number, $r_{s,min}$, is obtained from an incrementally loaded oedometer test. For each phase with constant load one determines a creep number. The r_s-values are then plotted against effective vertical stress. The lowest value observed in the plot is the $r_{s,min}$. This value usually occurs around the pre-consolidation pressure. For higher stresses the creep number will gradually increase and it seems to approaches an asymptotical value. This value is the intrinsic creep number, $r_{s,i}$. An IL oedometer test on a remolded sample will typically give a constant value for r_s independent of the effective vertical stress. This constant r_s is the intrinsic creep number. Figure 51 illustrates the procedure.

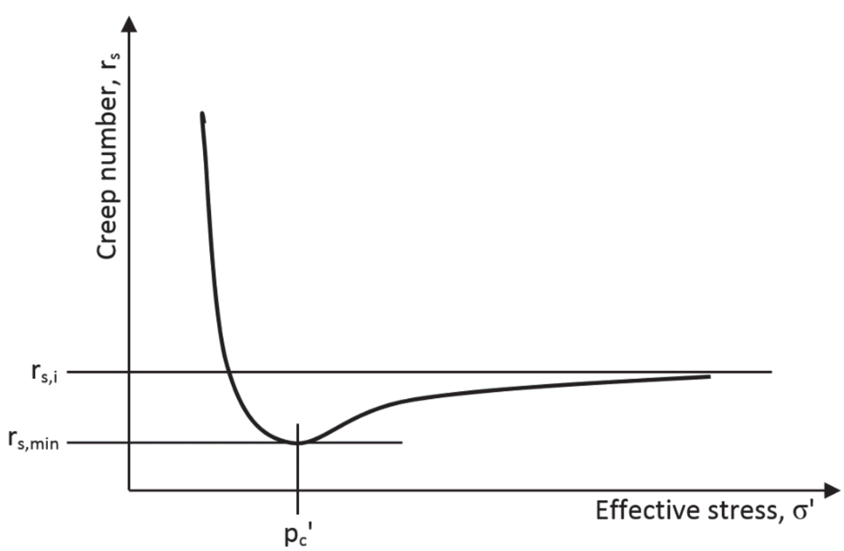

Figure 51: Minimum and intrinsic creep number

Strength parameters, φ_p, φ and ω

From an undrained CAUC triaxial test on a contractive material φ_p represents a line through the origin and the peak strength. φ is a critical state parameter and represents the line the stress path approaches at higher strains in the same test. This is illustrated in Figure 52.

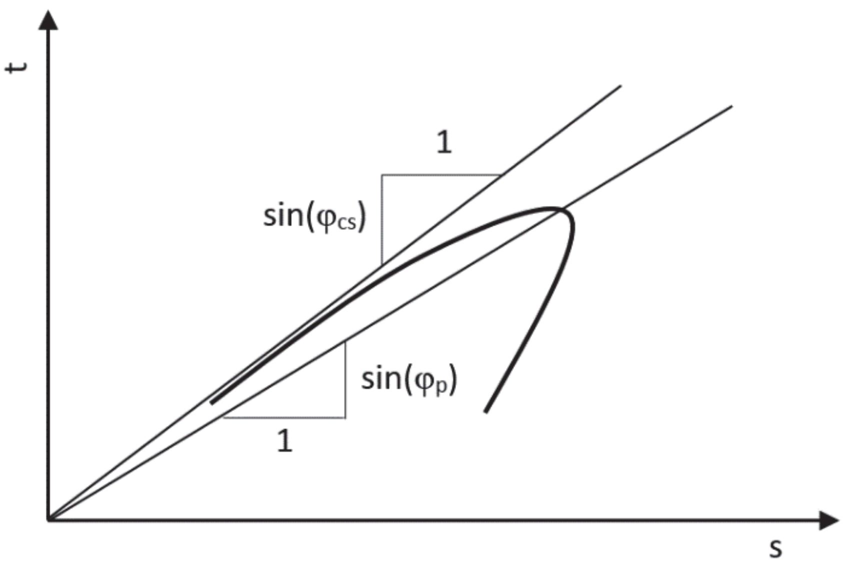

Figure 52: Critical state and peak friction angle from CAUC triaxial test in s-t-plot

The parameter ω controls the slope of the stress-strain-curve after the peak strength is reached in a CAUC test, i.e. it controls the softening or the contractive behavior in the material. The appropriate value for ω is obtained by running for example soil test in PLAXIS and fitting the test results.

The parameters for this calculation are based on triaxial tests presented in NGI (2010). The parameters are summarized in Table 18. Note that the parameters form NGI (2010) are from a preliminary study. For a better prediction all laboratory tests should be back-calculated with the model and improved parameters should be determined.

Table 18: Input data for material layers modelled with n-SAC

Layer	E_{ref} [kPa]	E_{oed}^{ref} [kPa]	$r_{s,min}$	$r_{s,i}$	φ_p	φ	ω	OCR_τ	POP_τ [kPa]
0.6 – 2.0 m	20 000	2000	110	660	28°	35°	0	1,0	25
2.0 – 5.0 m	20 000	1650	90	550	28°	35°	0	1,0	25
5.0 – 10.0 m	20 000	1500	80	500	28°	35°	0	1,0	25
10.0 – 25.0 m	20 000	19000	150	900	28°	35°	0	1,3	0

$\upsilon = 0{,}15$, $K_0^{NC} = 0{,}53$, t_{max}= 300 000 days , $p_{ref} = 100$ kPa , $\tau = 1$ day , $\upsilon_u = 0{,}495$,
Other material parameters are the same as in the "best fit"-calculation.

4.5.2 *Results*

As can be seen from figure 53 - 59, the n-SAC model gives a very good fit to the vertical deformations and the deformations outside the fill is almost zero. When it comes to the horizontal deformations, the low shear stiffness gives too large undrained horizontal displacement after 36 days. After 147 and 1116 days it gives a very good fit. The pore pressure response is similar to the CSSSCG-model. Further discussion is given in chapter 5.

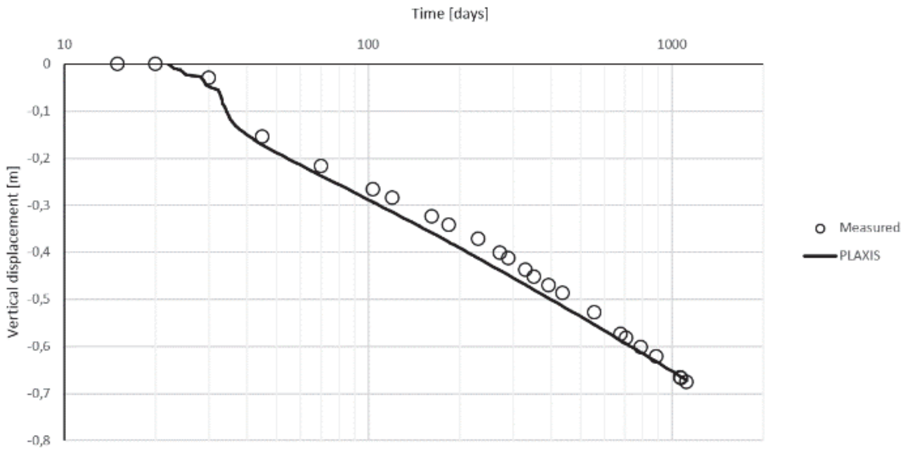

Figure 53: Settlement just under centreline with n-SAC

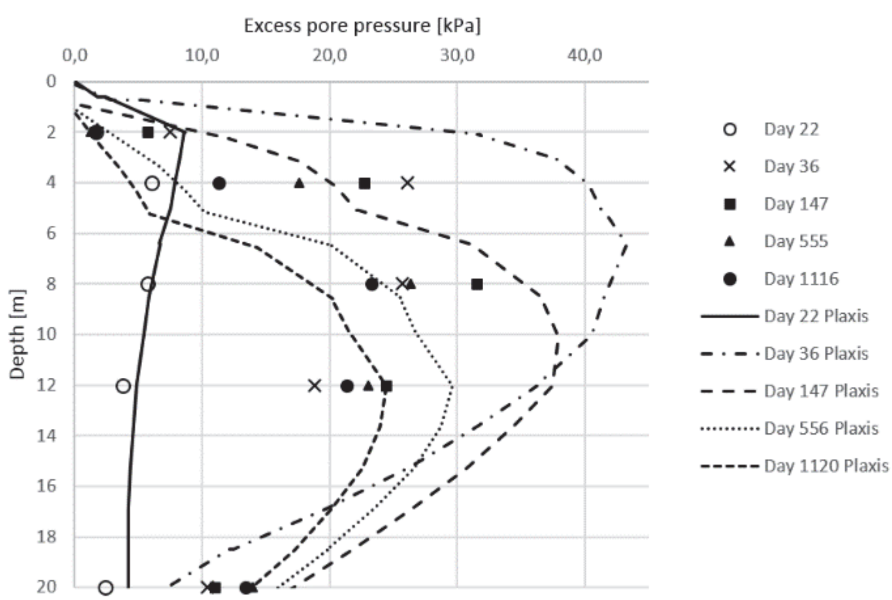

Figure 54: Excess pore pressure under centreline with n-SAC

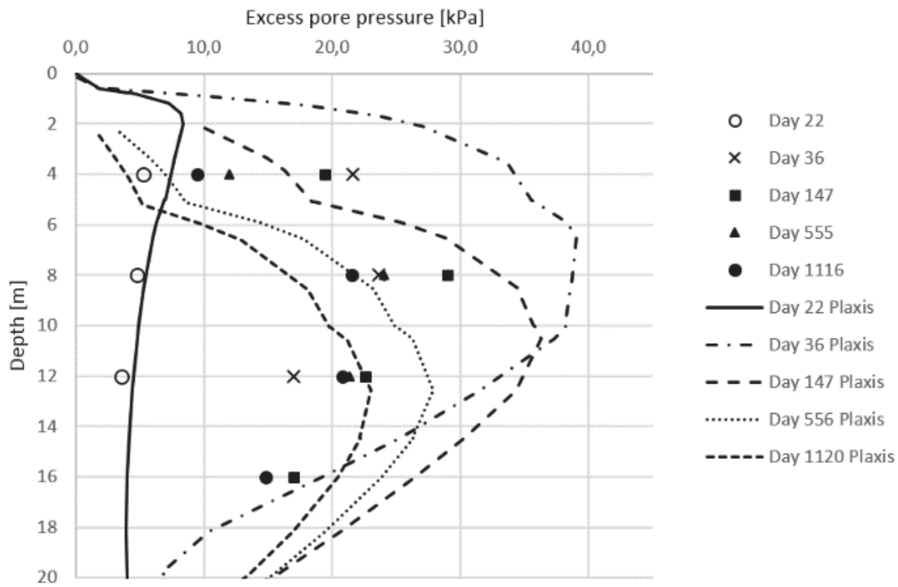

Figure 55: Excess pore pressure 5 m from centreline with n-SAC

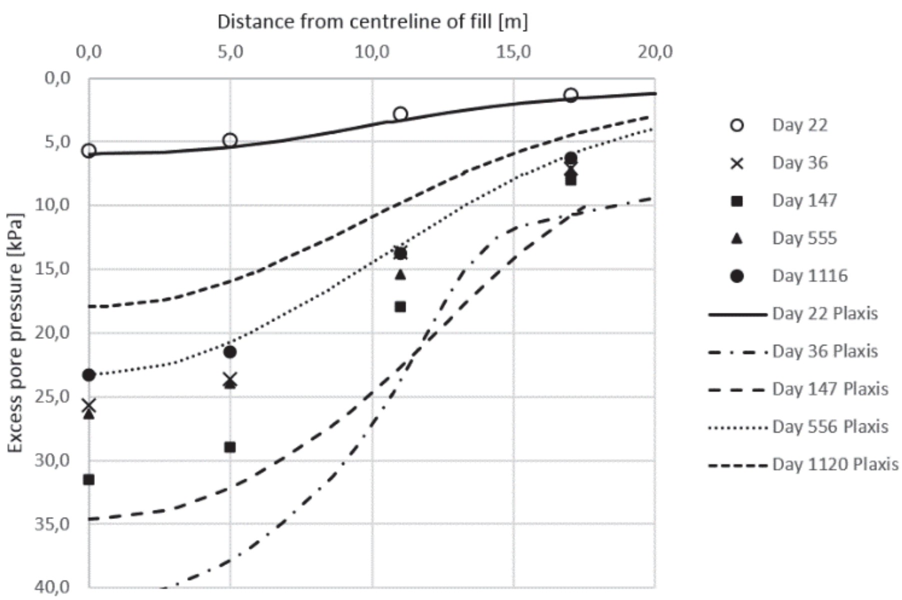

Figure 56: Excess pore pressure 8 m below ground surface with n-SAC

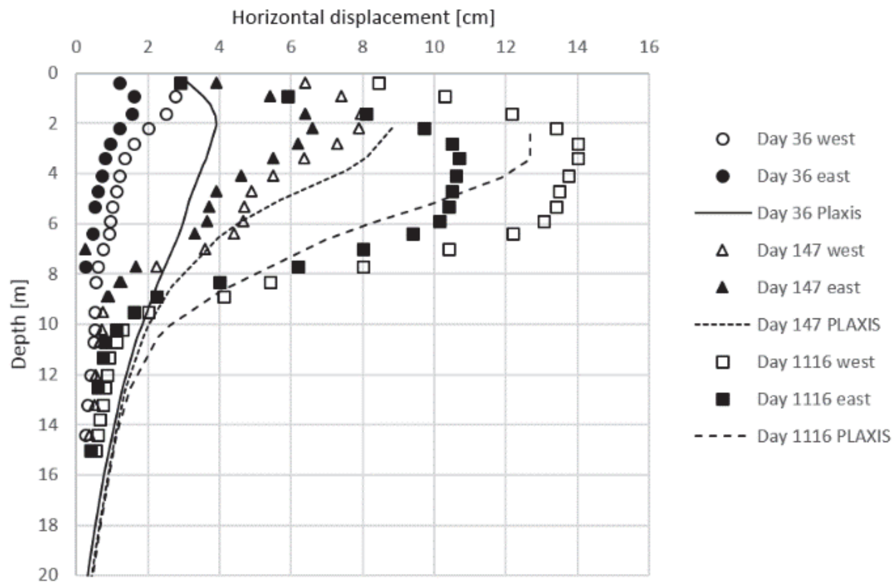

Figure 57: Horizontal displacement 5 m to each side of centreline with n-SAC

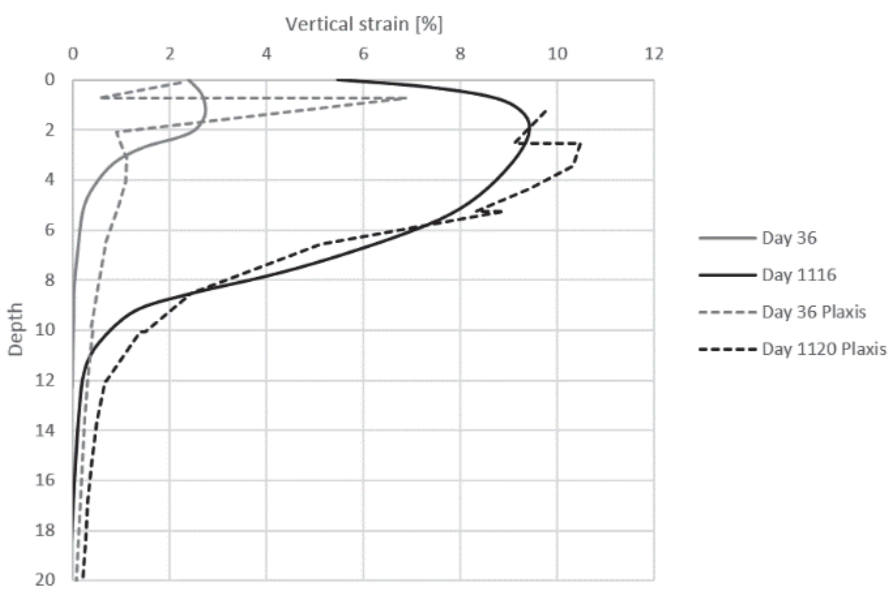

Figure 58: Vertical strain under centreline with n-SAC

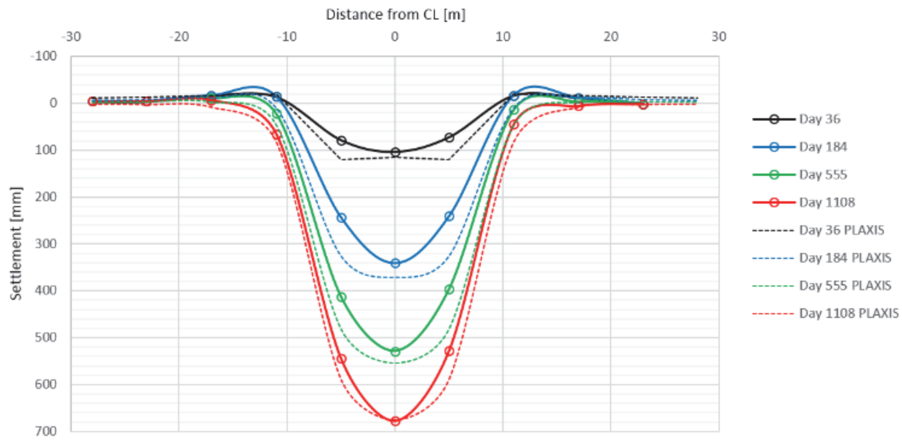

Figure 59: Surface settlement with n-SAC

4.6 Calculation with Soft Soil

4.6.1 Soil data

For comparison, a calculation with a model similar to SSC, but without creep, is performed. This is the Soft Soil model, which is based on the Modified Cam Clay model. Input data for the calculation is the same as in the "best fit"-calculation, and is given in Table 19.

Table 19: Input data for material layers modelled with SS

Layer	e_i	λ*	κ*	c' [kPa]	φ'	K_0^{NC}	υ_{ur}	γ [kN/m³]	OCR	POP [kPa]
0.6 – 2.0 m	1,3	0,07	0,020	3	25°	0,577	0,15	16,3	1,0	25
2.0 – 5.0 m	1,7	0,25	0,020	2	24°	0,593	0,15	16,3	1,0	25
5.0 – 10.0 m	2,0	0,25	0,020	2	24°	0,593	0,15	16,3	1,0	25
10.0 – 25.0 m	1,9	0,25	0,012	3	25°	0,577	0,15	16,3	1,3	0
Input data for permeability and initial stress condition are the same as in SSC "best-fit"										

4.6.2 Results

The SS model greatly underestimates the settlements after 3 years and this shows how much of the settlement that is due to creep in the other models. When it comes to excess pore pressure, the model gives a very good prediction after 36 days, but the excess pore pressure after 1120 days is underestimated. This gives a clear indication of what role creep play when it comes to delaying the dissipation of excess pore water.

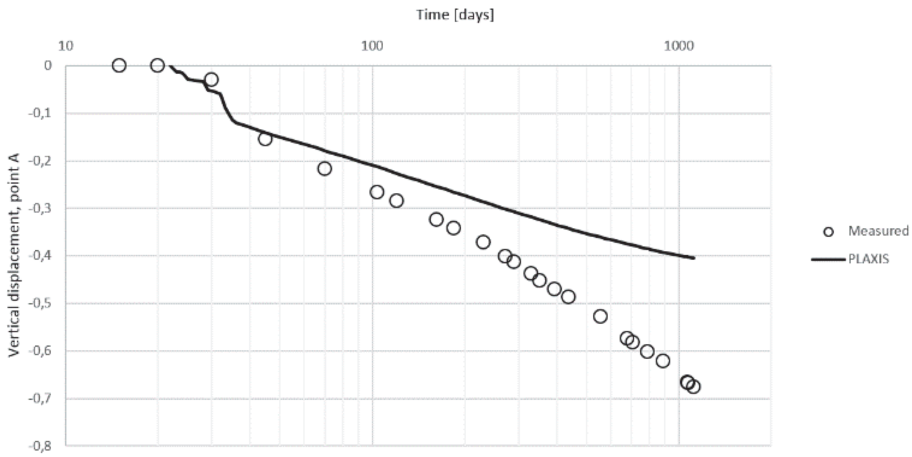

Figure 60: Settlement just under centreline with SS

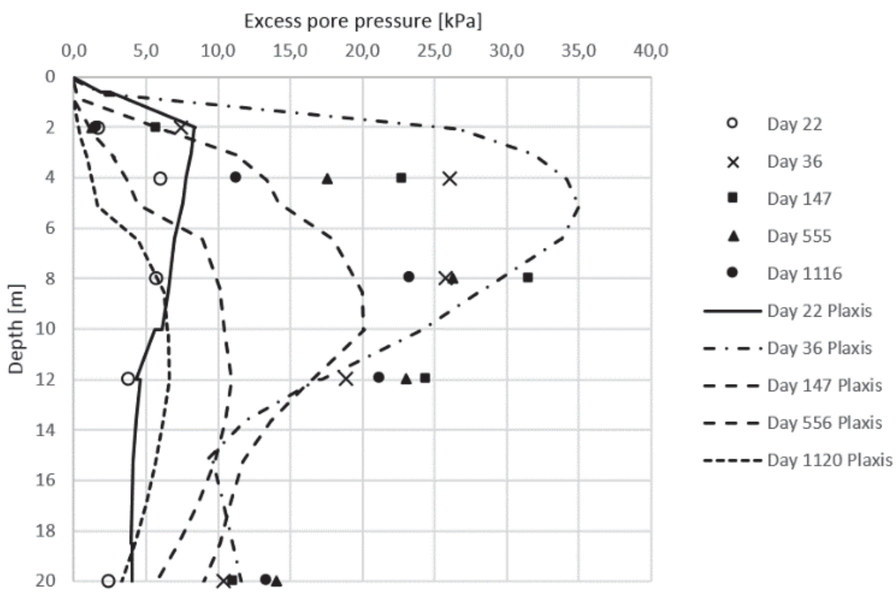

Figure 61: Excess pore pressure under centreline with SS

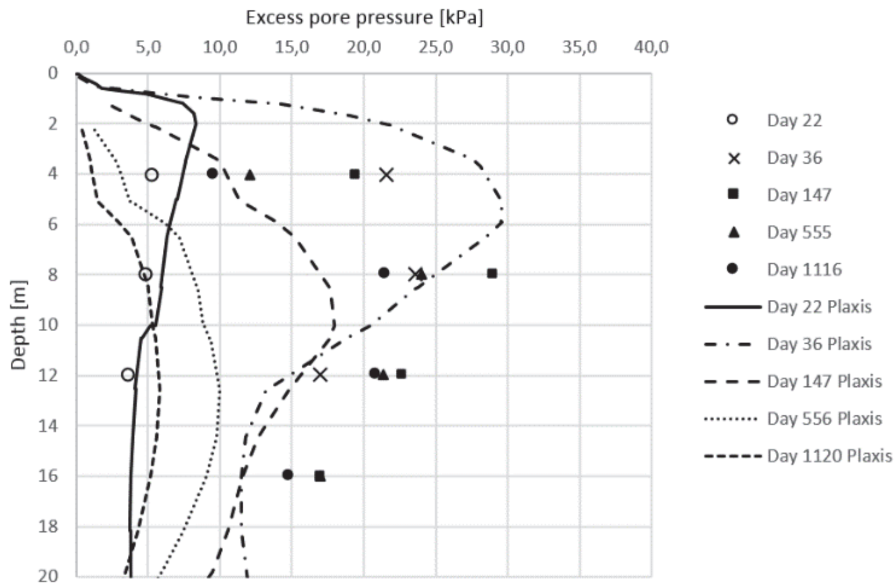

Figure 62: Excess pore pressure 5 m from centreline with SS

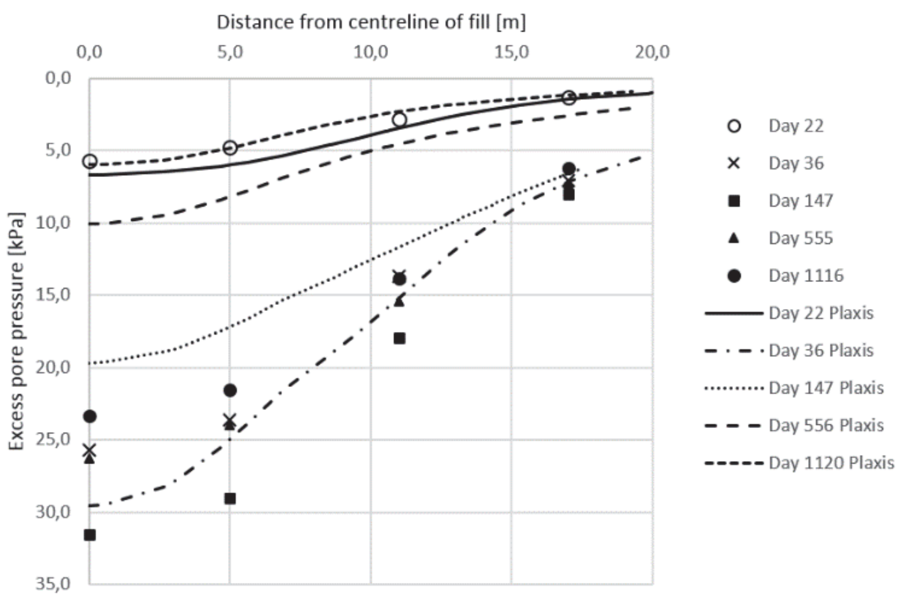

Figure 63: Excess pore pressure 8 m below ground surface with SS

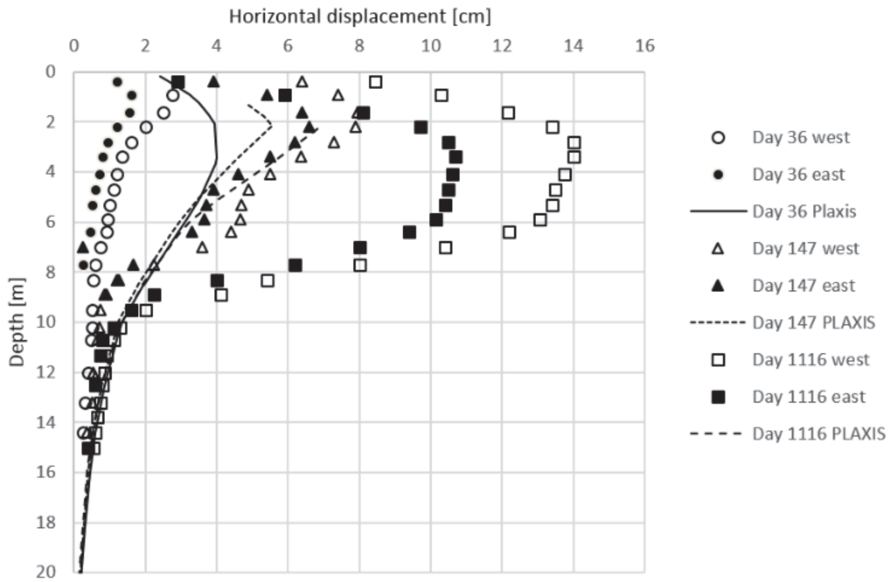

Figure 64: Horizontal displacement 5 m to each side of centreline with SS

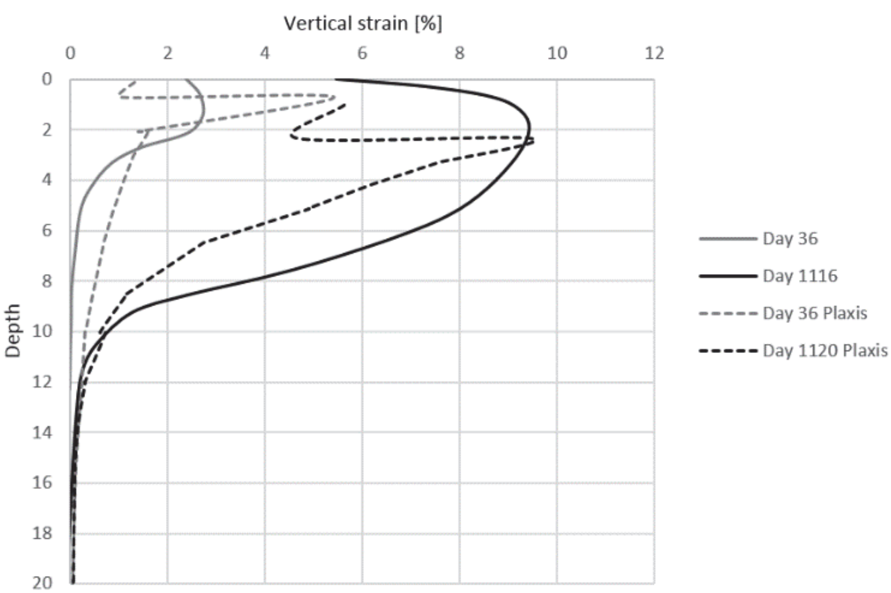

Figure 65: Vertical strain under centreline with SS

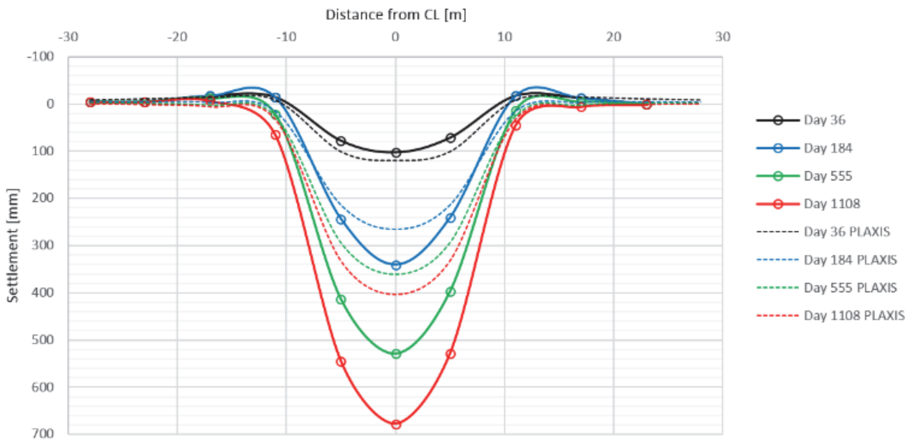

Figure 66: Surface settlement with SS

4.7 Calculations with KRYKON

The KRYKON calculations were performed using the Geosuite settlement program (www.geosuite.se). The input parameters used in the analysis are given in Table 20. Two different parameter sets were used. KRYKON_1 uses a low R_c value in combination with a relatively high p_c', while KRYKON_2 uses a high R_c value and a relatively low p_c'. The reason for selecting these two approaches is that the KRYKON_2 model should with this set of parameters become very similar to the SSC model for a 1D case. The equations used for converting the parameters are given below.

Converting parameters from SSC to KRYKON:

The Oedometer modulus in OC state is found from:

$$M_{OC} = \frac{p_c' - \sigma_0'}{\ln\left(\frac{p_c'}{\sigma_0'}\right)}$$

Where σ_0' is the vertical affective stress prior to filling.

The modulus number for the NC stress region is found from:

$$m_{sec} = \frac{1}{\lambda^*} = m \cdot \frac{\ln\left(\frac{\sigma_1'}{p_c'}\right)}{\ln\left(\frac{\sigma_1'}{p_c'} - \frac{p_r'}{p_c'}\right) - \ln\left(1 - \frac{p_r'}{p_c'}\right)}$$

Where σ_1' is the vertical affective stress after filling and consolidation is almost completed.

The time resistance number is found from:

$$r_s = \frac{1}{\mu^*}$$

The OCR is adjusted such that a "new" p_c' would fit to that of a value for R_c not equal to $R_{ref} = \tau / \mu^*$. In KRYKON_2 the R_c is selected such that the OCR is just above 1.0 for most layers or 1000 times R_{ref}.

$$OCR = \left(\frac{R_{ref}}{R_c}\right)^{\frac{m_{sec}}{\left(1-\frac{m_{sec}}{m_{OC}}\right)\cdot r_s}} \cdot OCR_\tau \approx \left(\frac{R_{ref}}{R_c}\right)^{\frac{m_{sec}}{r_s}} \cdot OCR_{ref}$$

Table 20: Input data for material layers modelled with KRYKON/Janbu (KRYKON_1 and KRYKON_2 parameters are separated with "/")

Lag	γ [kN/m^3]	M_{OC} [kPa] (top-bunn)	m	R_c (top-bunn)	r_0	R_{pc}	e_i	k [m/år]	c_k	p_c' [kPa] (topp-bunn)
0.0 – 0.6 m	17.8	6225-9934	14,3	-	-	-	1,2	3,65	1,0	170-170 / 170-170
0.6 – 2.0 m	16.3	1121-1955	14,3	0,457 / 4570-411	2000 / 167	167	1,3	0,55	0,5	35,6-44,5 / 11,9-19,6
2.0 – 5.0 m	16.3	2537-4211	4	0,342 / 3420	1500 / 125	125	1,7	0,55	0,5	44,5-63,4 / 32,3-46,0
5.0 – 10.0 m	16.3	4041-6995	4	0,274 / 2740-274	1200 / 100	100	2,0	0,06	1,0	63,4-94,9 / 42,5-70,0
10.0 – 25.0 m	16.3	11829-27820	4	0,457 / 4570	2000 / 167	167	1,9	0,02	1,0	90,9-205,5 72,1-169,5
		p_r' = 0 kPa,	m_r = 0							

4.7.1 Results

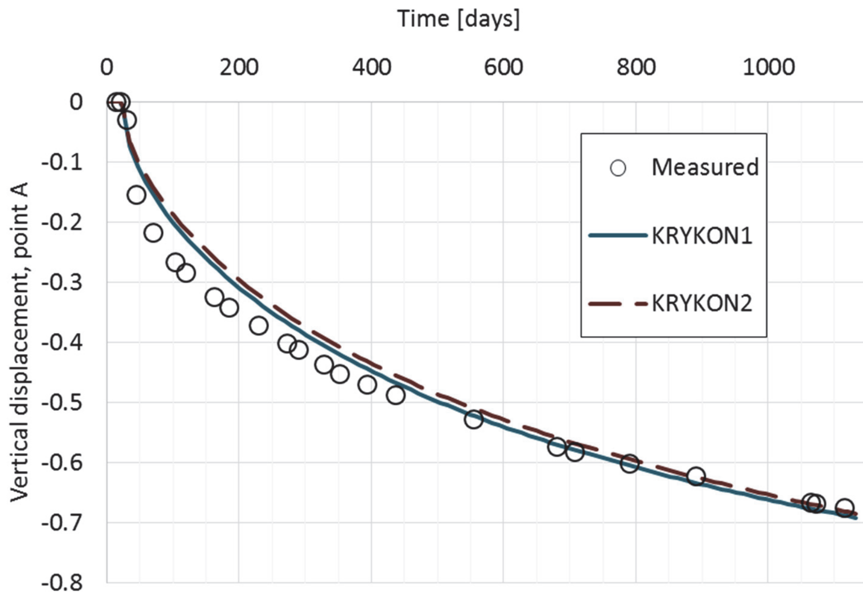

Figure 67: Settlement just under centreline with KRYKON

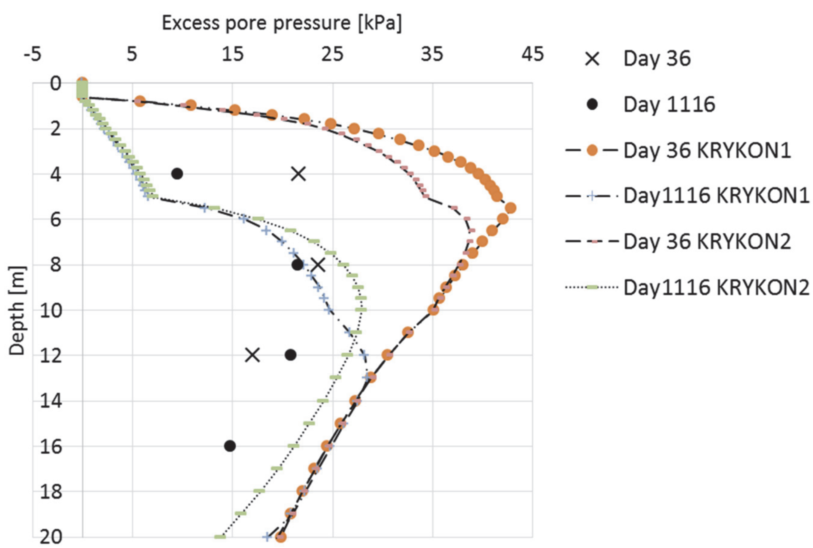

Figure 68: Excess pore pressure under centreline with KRYKON

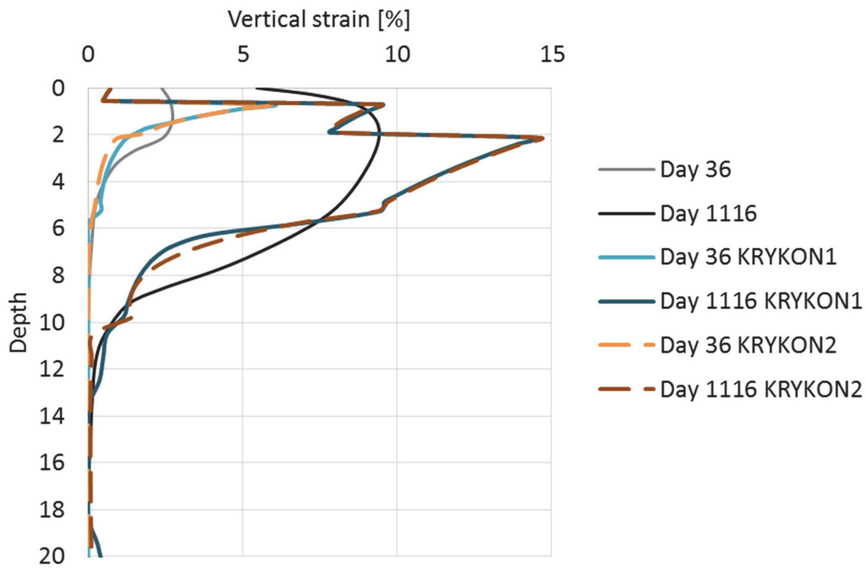

Figure 69: Vertical strain under centreline with KRYKON

As can be seen from the figures, the KRYKON model (1 or 2) gives a very good fit to the vertical strain profile. For the pore pressure the KRYKON_1 model does not fit to the measurement at all, since there is almost no reduction after 3 years below 13 m depth. For KRYKON_2 the model over-predicts pore pressure. However, since e.g. radial drainage is not included this is expected behavior. The trend of KRYKON_2 is closer to the measured. Notice that, because of the 1D assumption, the vertical deformation just after loading is under-predicted.

5 Discussion of results

5.1 Displacements and strains

All the calculations with the considered creep models give a good prediction of the total settlements beneath the embankment. Even the first prediction with SSC gives a reasonable result. The main difference between the models is observed in the distribution of the vertical strains with depth. For the SSC-prediction, too much vertical strain develops in the bottom layer of the model. This is mainly due to the low OCR giving an unrealistically large creep rate in this layer. The OCR is interpreted from oedometer tests with some degree of sample disturbance. The sample disturbance leads to an initial loss of the material fabric, which normally gives a softer response in the overconsolidated region and a lower preconsolidation pressure. The sample disturbance is often more pronounced in deeper samples. The degree of disturbance is difficult or impossible to quantify, but the OCR should always be set to a minimum value depending on the assumed age of the deposit and the relationship between the compressibility parameters when the OCR is induced by aging. This could typically be an OCR-value from 1.3 to 1.8.

The effect discussed above is also seen in the plots of surface settlements. The first prediction gives large settlements outside the embankment, but the other three calculations give much less settlement in this area. The n-SAC model gives almost no surface settlement outside the embankment. This is due to the t_{max} parameter in n-SAC that slows down creep rate more than linearly when approaching an “age” of t_{max}.

The reference calculation with SS shows approximately how much of the settlement is due to creep. After 1120 days, the difference between measurements of surface settlement and SS is 27 cm, i.e. 40% of the total settlement. And the rate of settlement is also quite different, so the percentage error obtained by using SS is increasing with time.

Deformations in the first 36 days are mostly undrained (except in the upper 2-3 m of the soil profile) where the pore pressure has little time to dissipate and it is therefore almost no volume change in the clay. Therefore, the vertical displacement profile after the first 36 days is dependent on the shear stiffness of the material. The plots of vertical strain with depth show that SSC and n-SAC over predicts the vertical strain from 3 to 20 meter of depth. This is due to the low shear stiffness derived from κ^* or E_{ref}. The CS-SSCG model has a small strain shear stiffness with mobilization dependent degradation. This model gives a very good match with the measured vertical strain profile and the horizontal deformation just after loading. This shows that one has to include some form of small strain stiffness formulation in order to model the undrained deformation pattern of the clay realistically.

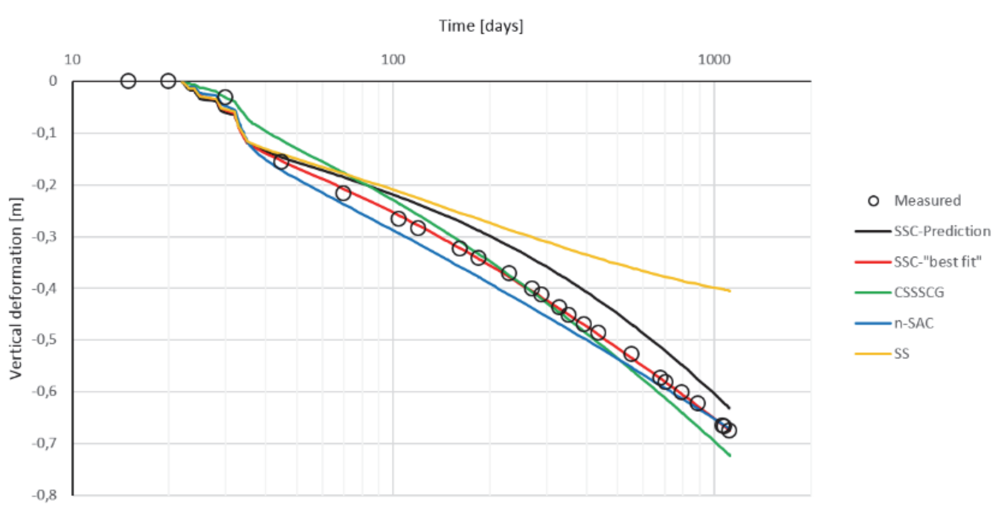

Figure 70: Settlement just under the fill

n-SAC and SSC over predict the horizontal displacements after 36 days by more than 100% due to the low shear stiffness as discussed in the previous section. CS-SSCG gives an almost perfect match with input parameters directly from measured field and laboratory tests. All three models predicts the horizontal displacement after 1120 days in a satisfactory way, but n-SAC produces the best fit. This is obtained with the input of a friction angle related to compression strength. The anisotropy included in the model reduces the strength for direct and passive shear, which in turn gives larger horizontal displacement. To achieve this in SSC the friction angle has to be reduced; hence the compression strength is also reduced. The Lode angle dependency also increases the horizontal displacement. When M is decreasing, because stress state is between compression and extension, more of the creep strains are due to shear creep and the horizontal deformation will increase. CS-SSCG without Lode angle dependency would give almost the same as SSC.

5.2 Excess pore pressure

Figure 71 shows plots of calculated distribution of excess pore pressure from all of the PLAXIS calculations compared with field measurements. The calculations show that the fairly simple, SSC model is able to estimate the excess pore pressure over time with good accuracy. The pore pressure measurements also show the importance of including creep in a soil model. An example: By using input data from disturbed samples, with a soft response in the OC region and a low pre consolidation pressure, in a rate independent model like SS, it is possible to back-calculate the settlements from the Onsøy test fill with good accuracy. However, the calculated pore pressure after 3 years would generally be far from the measurements. The creep deformations are creating the extra excess pore pressure with time, and it is a major difference after just 3 years. This is important if one want to know the safety against failure for an embankment after some time of settling or if there is a new stage construction after some time.

The calculations with the SS model and the SSC model, show similar results for the excess pore pressure after 36 days. The two user-defined soil models (n-SAC and CSSSCG) give similar results, but they show a higher excess pore pressure response than the standard soil models. After 1120 days both the standard and the user-defined models show similar results (the SS model is disregarded). The main reason for this is that the stress distribution with depth is different. Both n-SAC and CS-SSCG gives a stiffer undrained response and the soil column straight under the embankment therefore takes almost all the load. SSC and SS are softer with the given soil parameters, and distributes the load more out to the sides. n-SAC and CS-SSCG also show slightly more contractive behavior than SSC and SS, which also adds to the excess pore pressure.

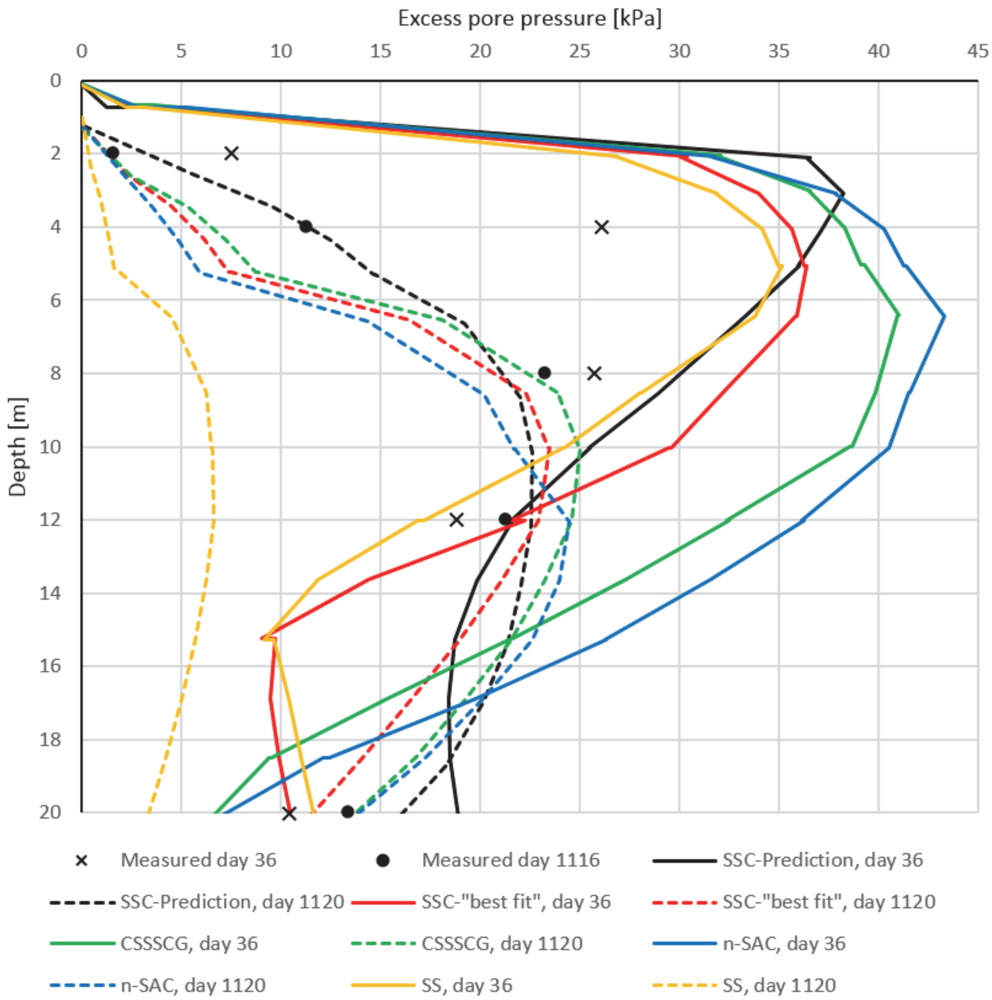

Figure 71: Excess pore pressure under centreline

Under rapid loading, the fact that water is almost incompressible implies that almost all of the load should be taken by increase in pore pressure. The load is then transferred to the soil skeleton during consolidation. The measurements at Onsøy show that the pore water takes only about 70% of the load. Part of the reason for not measuring pore pressure equal to the load can be due to dissipation during building of each layer, the response of the hydraulic piezometers, the exact time for reading of the piezometers, partly saturated soil / air bubbles in the soil and stiffness anisotropy in the soil. To account for these effects in the model one can reduce the stiffness of water, but this is not recommended. One could also use a higher permeability during loading and switch to a material with low permeability during consolidation, but to capture the true material behaviour one should perhaps instead introduce stiffness anisotropy in the soil model.

Due to the sedimentation process clay has been subjected to and anisotropic nature of most clays, it is not surprising that stiffness also is an anisotropic parameter(Pennington *et al.*, 1997). In p'-q-space this elastic anisotropy can be simplified by the following equation:

$$\begin{bmatrix} \Delta p' \\ \Delta q \end{bmatrix} = \begin{bmatrix} K & J \\ J & 3G \end{bmatrix} = \begin{bmatrix} \Delta \varepsilon_v^e \\ \Delta \varepsilon_q^e \end{bmatrix}$$

In an isotropic elastic material J would be equal to zero and in an undrained condition with no volume change this would give no change in the effective mean stress. If J>0, which would correspond to a material with larger vertical stiffness than horizontal, Δp' will be a positive number even with no volume change, because elastic shear strains is always positive in compression. In a p'-q-plot a dilative behaviour would be observed which gives less excess pore pressure. The anisotropic elasticity can perhaps explain the reason for less measured excess pore pressure than predicted.

KRYKON with use of a low R_c value and a high r_0 is the model that does "worst" in predicting the development of excess pore pressure. The constant R_c for the laboratory input curve should not be used for describing the initial creep rate. The way it is done in the 3D models (e.g. SSC/n-SAC) seems more appropriate, i.e. initial creep rate is determined from OCR, reference time, creep ratio and creep index (time resistance number).

5.3 Long term settlements

The duration of measurements at Onsøy was 3 year. The calculations show that creep has a significant influence on settlements and pore pressures after the load is applied, and 3 years is actually enough to study some aspects of creep. The question is what happens after 3 years? Figure 72 shows how the creep models would predict settlements for the next 17 years and the total settlements vary from 1.03 to 1.22 meters. The slope of the measurements in the figure indicate that n-SAC or SSC gives the best estimate, but what happens in the soil after 20, 40 or 100 years. Is the creep number still constant, or is r_s increasing because of e.g. structuring or chemical bonding in the clay?

It is not possible to answer this as long as we don't have laboratory test for such a timespan. In respect to this, n-SAC has the ability to increase the creep number with time controlled by the parameter t_{max}. Adjusting this parameter can give a maximum reached OCR value due to aging.

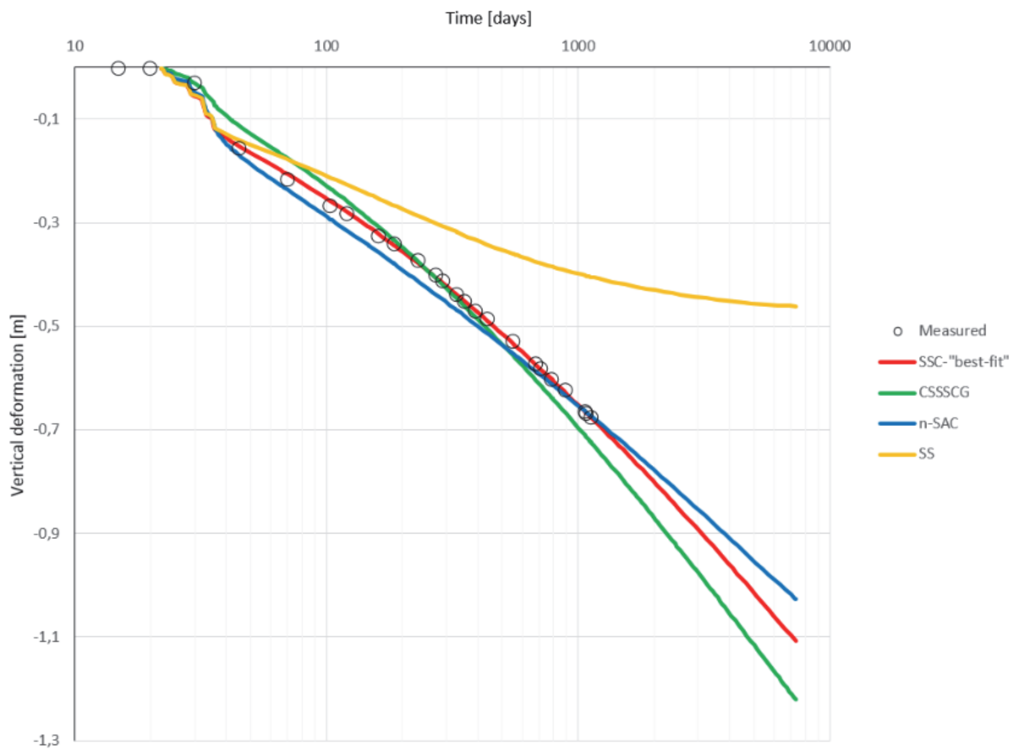

Figure 72: Settlements just under embankment after 20 years

5.4 CRS with different models

Figure 73 shows how SSC and n-SAC performs when loaded in an oedometer state. For the SSC calculation the parameters from the 5-10 m depth material in the best-fit calculations is used. The input data for the n-SAC calculation is optimized for this oedometer. The figure shows that n-SAC is able to fit the whole oedometer curve.

Figure 74 illustrates how n-SAC and SSC performs when back calculating a CAUC triaxial test. The input parameters are adjusted to fit the triaxial test. n-SAC is able to fit the stress-strain curve before and after the peak. The steepness of the post-peak curve is controlled by ω and the residual strength is controlled by the relationship between the minimum and the intrinsic creep number. Because of softening, n-SAC also gives higher pore pressure than SSC. Up to the peak undrained shear stress the two models behave similar and predicts almost the same undrained strength. However, for a CAUE triaxial test n-SAC gives more realistic undrained shear strength because of anisotropy.

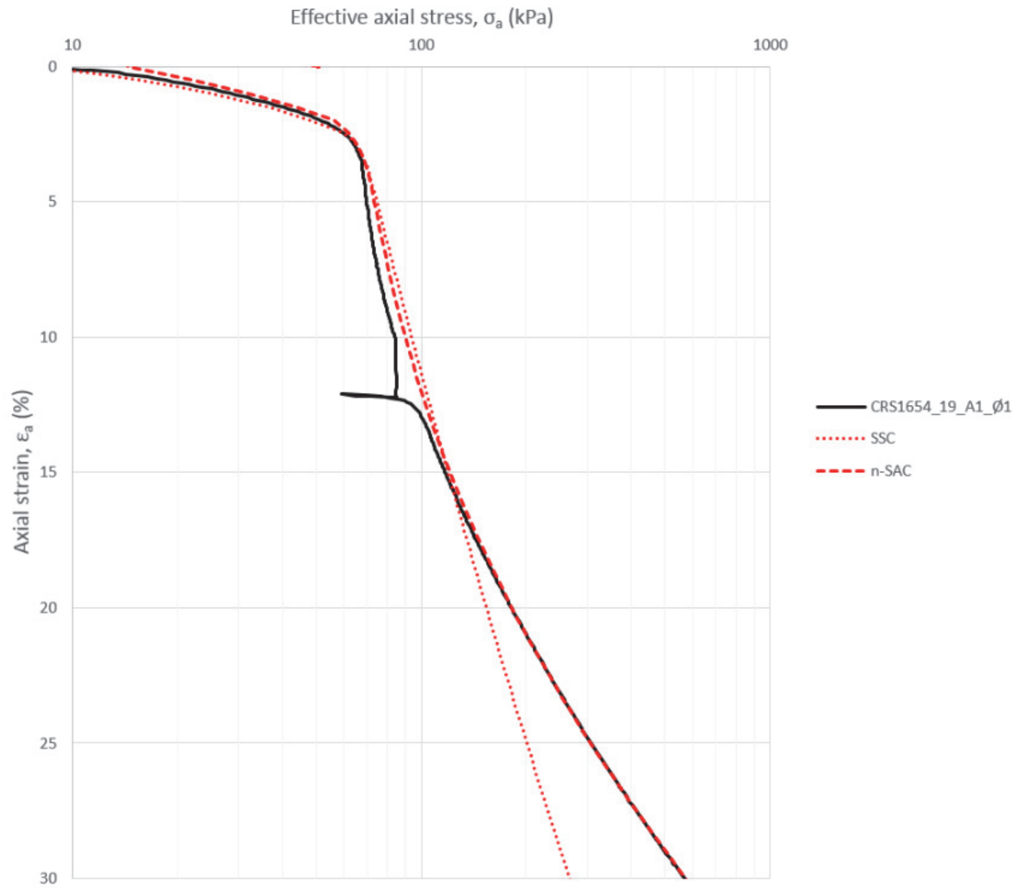

Figure 73: Back-calculated CRS oedometer test at 7.45 m depth with SSC and n-SAC

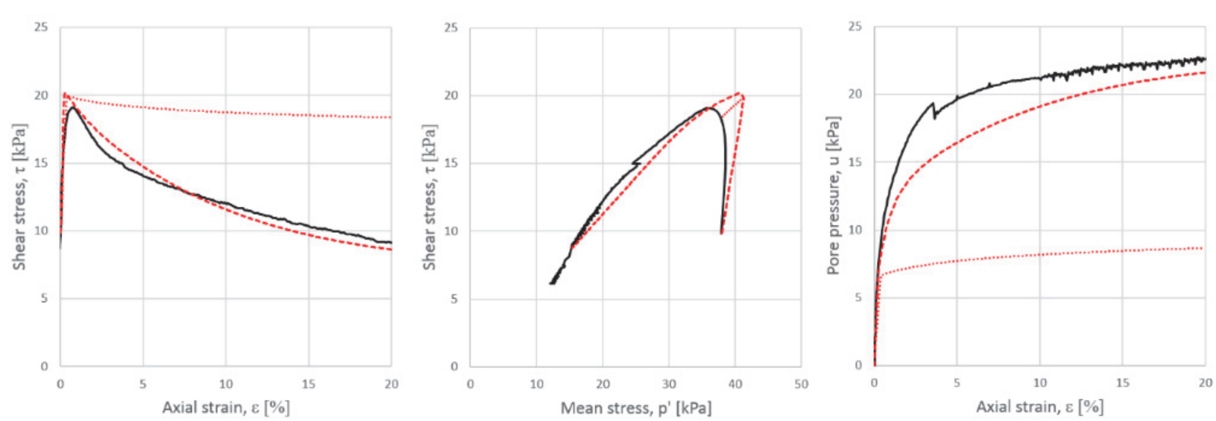

Figure 74: Back-calculated CAUC triaxial test at 7.0 m depth with SSC and n-SAC. Black line is the lab test. Red rectangular dot is n-SAC and red round dot is SSC.

6 Recommendations

Based on the calculations documented in this report together with the discussion, some recommendations are given for use of advanced soil models with creep formulation.

- OCR may be the most important parameter in creep calculations. Together with the compressibility parameters, it defines the initial creep rate. Input data for these parameters should be collected from high quality laboratory tests and the degree of sample disturbance must be evaluated. Assuming no previous loading of the clay, the assumed age of the clay and relationship between the compressibility parameters will give a minimum OCR to be used in the calculation. It is recommended to do a calculation without any load to check whether the initial creep rate is reasonable.

- Permeability is also a very important parameter for settlement predictions. In PLAXIS, it is possible to control the reduction of permeability with increasing volume change with the parameter c_k. Values for this parameter is obtainable from standard CRS oedometers tests and it is recommended to use real values instead of the default value in PLAXIS.

- A perfect prediction is never realistic and it is therefore recommended to use several parameter sets for a settlement problem (min/max). This will give a range of potential settlement over time.

- When evaluating the undrained / short term deformation pattern, creep is not that important. A total stress based model can be a good alternative where the non-linear behaviour of G is given as input. The shear modulus should be evaluated from G_{max} with degradation correlated with mobilization or shear strain. The CS-SSCG is a good alternative, but not commercial. SSC and n-SAC normally over predict horizontal deformations.

- Models like n-SAC is somewhat more demanding when it comes to input data, but when the data is obtained it is able to better fit the stress-strain-curve from an oedometer test. Care should be taken when including strain-softening because it will give mesh dependent results at failure.

- Always back-calculate oedometer and triaxial test after interpreting input data. Undrained triaxial test can be modelled with soil test in PLAXIS, but oedometer tests should be modelled as a boundary value problem to include consolidation. It is always important to fit the stress range relevant for the problem at hand. Be aware of sample disturbance.

- If high quality field and laboratory test are not available in the project, one should be careful about including creep. Estimates can be done by taking into account the sample disturbance and evaluating similar clay tested on high quality samples.

- Always do hand-calculations. One should have an opinion about the magnitude of settlement expected before a FEM calculation. This can be done by using the 1D principals behind the SSC-model or by using principles introduced by Janbu (1969).

- The following laboratory tests should be performed to interpret creep parameters: One CRS oedometer test with unloading-reloading-loop and one IL oedometer test with 24-hours load steps for several depths. Alternatively, three CRS oedometers tests with different strain rates for several depths. All CRS oedometers tests should include a creep phase at a stress state above the pre-consolidation pressure.

- It is recommended to use the function update mesh and update pore water when the deformations are expected to be large. This will automatically take into account the increased buoyancy for soil being pushed down under the ground water table.

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8 Recommended reading

Below is a list of some recommended articles about creep in addition to the references in chapter 7.

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