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Title of paper: Prediction of embankment settlement on Swedish peat using the Soft Soil Creep model

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ABSTRACT

Despite the prevalence of peat in northern areas of Europe and elsewhere there are comparatively few case histories of carefully monitored embankments constructed directly on peat. In addition the use of relatively advanced and commercially available numerical models for prediction of the settlement in peat have only infrequently been documented. Practicing geotechnical engineers lack advice on the use of such models and in particular on the selection of the input parameters. This paper demonstrates that a combination of routine oedometer tests, careful use of published correlations and simulations of independent data can provide reliable input parameters. The work demonstrates a good match between field measurements and simulations but shows that it is difficult to model all stages of loading accurately with one set of input parameters. The user may need to focus on what specific output is critical and a number of analyses may be needed to obtain accurate prediction for all cases. Although the focus of the model used was on creep prediction, the work demonstrates that, other benefits of using a finite element analysis over hand calculations, such as modelling layers, buoyancy, variation in parameters such as permeability with strain and time are perhaps more important than creep.

KEY WORDS: peat; settlement; creep; numerical model

INTRODUCTION

The geotechnical characteristics of fibrous Swedish peat are relatively well understood (Carlsten, 1988; Carlsten, 2000). The well-known von Post method (von Post and Granlund, 1926) for classifying peat was developed in Sweden in the early years of the 20th century for the purposes of making an inventory of Swedish peat resources. Like in many other places the peat is characterised by its very high water content and compressibility, low density and shear strength as well as its highly heterogenous nature. The material therefore presents significant challenges to geotechnical engineers working with these soils. In Sweden many roads and railways have been built and are planned to be built on peat and as a result there has been several very useful publications on this topic (Carlsten, 1993; Carlsten, 1995; Vesterberg, 2017). In addition due to the very positive carbon storage effects of peatland there is further pressure to prevent excavation and replacement of peat for infrastructure developments. For example present day Norwegian authorities have adopted new regulations prohibiting the cultivation of peat areas (Norsk-Lovtidend, 2020). In the near future these authorities may implement stronger rules to avoid developments which involve the excavation and removal of peat.

Despite this work there are relatively few published case histories of the performance of road or railway embankments on Swedish peat. Preloading was used to construct an 850 m long section of Väg 73 at Dalarövägen just south of Stockholm (Carlsten, 1985; Carlsten, 1988). Similar techniques were used to construct a section of Rv 50 Mjölby – Motala as described by Johansson et al. (2012) and Vesterberg et al. (2016) also for a road widening / rebuilding project at Väg 296 Kårböle - Z länsgränsen, see Ånäs (1999) or Vesterberg et al. (2016).

Long and Boylan (2013) reviewed the prediction of the settlement of embankments on peat in Ireland and Norway. They found that the primary compression of the peat layer is often well predicted, especially in cases where the applied stress was well above the preconsolidation / yield stress. However secondary compression or creep is often underestimated. In addition the rate of primary compression is difficult to predict as the permeability of peat reduces significantly with increasing stress. Therefore it is difficult to make reasonably accurate predictions with hand calculation methods.

In addition to documenting an additional case history of the measurement of settlement of a road embankment constructed directly on peat at Färgelanda in south-western Sweden, the purpose of this study is to demonstrate the applicability of a commercially available engineering tool for settlement analyses in peat. Here the chosen tool is the Soft Soil Creep

(SSC) constitutive model which is available in the commercial software PLAXIS (PLAXIS, 2020). It is acknowledged that more advanced models exist within the research community, see for example Grimstad et al. (2017), and that more simple models, such as SSC, cannot account for factors such as anisotropy and / or fabric, which are important in peat. However, in the authors opinion there is a gap, both in the application and the terminology used, between engineers working in practice and researchers. This discourages an advance from simple hand calculation estimations to what, from an engineering perspective, is sometimes viewed as advanced analyses, i.e. using the finite element method. The purpose of this paper is thus to help in addressing these issues.

Guidance will be provided for the selection of the input properties for future similar work in peat, especially for the key parameters such as the preconsolidation / yield stress and the permeability.

THE SITE

Location and history

The site is located some 5 km north of Färgelanda in south-western Sweden (and about 25 km north of Uddevalla) and the work comprised the upgrading / raising of Väg 2081. Over the years the road had suffered from some settlement related damage. The existing road was about 5 m wide and passed through an area of forestry and farmland. Some pictures of the road section studied are shown on Figure 1. These comprises (a) the area prior to upgrading July 2011, (b) during the ground investigation works by Skanska in Summer 2015, (c) during the upgrading works in Spring 2016 and (d) after completion of the works in August 2018. The relatively poor condition of the road can be observed in the cracking and the several repaired sections on Figures 1a and 1b. The existing road construction was some 0.5 m thick.

Ground investigation

The results of two ground investigations are available. The investigation for the original road upgrading was undertaken by Skanska Sverige AB (Skanska, 2015) and included a boring, some limited lab testing and a piezocone test (CPTU) at the point under study; ST4 on cross section 8040 m. ST4 was drilled through the edge of the existing road pavement having first hand dug to 1 m to check for services and provide access for the equipment.

An additional investigation was undertaken by University College Dublin (UCD) in October 2018 at the same location with the focus of providing information on the peat. The testing location (11.97417°E, 58.66427°N) was about 40 m north-west of ST4 (with 35 m offset from the road) in a clearing within the forested area.

A Russian or Jowsey auger (Jowsey, 1966) was used to obtain a 45 mm diameter “half core” of the peat over the full depth profile. The peat was logged on site using the extended version of the von Post and Granlund peat classification (Hobbs, 1986). Samples of the peat were taken at 0.1 m intervals for water content measurements. These water content measurements would later be used in conjunction with the field shear wave velocity measurements which were taken at the same depth intervals and frequency and down the same borehole as that used for logging the peat.

Block samples of the peat were obtained for laboratory testing by hand carving the material using a knife with a serrated cutting edge from the bottom of a hand excavated hole at between 0.5 m and 0.7 m depth. The samples were sealed using plastic film and aluminium foil and stored in airtight storage boxes prior to testing. Most of the samples were taken to the laboratories of Lund University (LTH). Some of the samples were transported to Ireland by private car.

Shear wave velocity (V_s) measurements were made using the portable downhole sonde which was developed for the purposes of taking V_s readings through a vertical peat column by Trafford and Long (2020). A broad bandwidth geophone (10 Hz resonant frequency) is encompassed within a high-density polyethylene housing to form the sonde. The sonde is initially pushed to the bottom of the peat profile, down the same hole which was used for peat logging and water content sampling. Measurements are made as the sonde is retrieved at intervals of 0.1 m to create a continuous seismic profile through the full peat column. The transmitted shear wave is produced at the surface by striking a hammer against an instrumented block.

Laboratory testing

Water content (oven dried at 80°C for 24 hours) and density measurements were carried out to the requirements of BS1377 (BSI, 1990). Constant rate of strain (CRS) oedometer tests were carried out at the laboratories at Lund University using the procedures outlined by Magnusson et al. (1989). The specimens were 77 mm in diameter and 15 mm high (H_0) and were prepared by pushing the oedometer ring into a 100 mm square block of peat while gradually trimming the peat using a knife with a serrated edge. In order to investigate the influence of the rate of strain, various rates were trialled and ultimately a rate of 3.75%/hour was chosen. Long and Boylan (2013) assessed the effects of the test rate in CRS tests by carrying out tests at four rates between 1.5%/hour and 10%/hour on peat from 4 sites and found only minor differences between the results.

Summary of ground conditions

A summary of the ground conditions is shown on Figure 2. Index and hydraulic parameters for the materials are summarised on Table 1. Approximately 5.8 m of peat overlies organic clay, which is known locally as gyttja (Larsson, 1990) to about 10 m depth over very soft clay to 21 m depth. The base of the peat can be seen clearly in the CPTU response with the change in the profile and the reduction in the values of the corrected cone resistance (q_t) and sleeve friction (f_s). The gyttja / soft clay interface is most clearly seen on the f_s profile, where f_s reaches a minimum value and then remains more or less constant with depth, but there is also a change in the slope of the q_t profile at this point. Friction ratio ($R_f = 100*f_s/q_t$) values are typically 6.0 in the peat dropping to about 0.5 at the top of the soft clay. Pore pressure (u_2) values are close to hydrostatic (u_0) in the peat and are well in excess of u_0 in the gyttja and soft clay.

The inset into Figure 2a shows that the water content (w_i) in the gyttja decreases from about 300% to 55% with an average of some 150%. The soft clay has unit weight (γ) of about 16 kN/m^3 and average water content of some 68%. Swedish fall cone tests indicate that this material has an average undrained shear strength of about 23 kPa, remoulded undrained shear strength of 0.3 kPa and sensitivity of 77.

Characterisation of peat

Details of the peat layer at the study site are shown on Figure 3a. It is characterised by some 2.2 m of a low degree of decomposition (von Post H = 3) orange/brown sphagnum peat overlying about 3.6 m of dark brown fibrous fen type peat with timber fragments. The lower fen peat has a pastier texture with increased degree of decomposition (H4 to H6) compared to the upper sphagnum peat. Below about 4.0 m there were some layers of more clay rich material (organic clay) within the fen peat.

The water content decreases from between 1000% and 1500% (average 1200%) in the upper sphagnum peat to between 600% and 900% (average 750%) in the lower fen peat. The increase in clay content within the peat between 4.0 m and 5.2 m depth corresponded with a decrease in water content as would be expected. The lowest peat layer showed a significant increase in w_i (up to about 1000%) immediately above the transition to the underlying clays. The spike in the readings at 3.1 m is due to a large timber inclusion.

The V_s value was found to vary between 14 m/s and 34 m/s throughout the peat profile. The upper sphagnum peat shows as relatively constant profile with V_s about 20 m/s gradually increasing to a velocity of about 30 m/s for the underlying fen peat. The V_s profile is complicated by the presence of more clay rich bands and variable timber content below about

4.2 m depth. Overall these variations are broadly consistent with an inverse relationship to the water content.

Unfortunately, in order to safely obtain hand dug block samples of the lower fen peat, it was necessary to investigate a another site where the upper sphagnum peat had been removed to provide fuel for domestic purposes. Details of the peat at this site are shown on Figure 3b and it can be seen that it is very similar in nature to that of the lower fen peat at the study site with average w_i of 820%, V_s of 22 m/s and H = 5.2.

SOFT SOIL CREEP MODEL (SSC)

General

The SSC model (Grimstad and Nordal, 2018; Neher et al., 2001; Stolle et al., 1999; The et al., 1998; Vermeer and Neher, 1999; Vermeer et al., 1998) combines the isotache principle of viscous compression with the CamClay model of shear strength, and is designed to deal well with creep, ageing (creep hardening), stress relaxation and critical state shear deformations. A full description of the SSC model is beyond the scope of this paper. The model, as well as the background isotache theory, has been thoroughly described in the articles mentioned above and in the PLAXIS manuals (PLAXIS, 2020).

Input parameters for SSC

Waterman and Broere (2005) give some guidance on selection of parameters for SSC and briefly discuss the importance of initiating the model with reasonable initial strain rate. For initial conditions prior to loading, the creep strain rate should be reasonably small. For low OCR (overconsolidation ratio), the model employs relatively large creep strain rates and will therefore calculate large settlements. This is further discussed by Grimstad et al. (2016) for the case of clay but the findings are generally just as applicable to peat. PLAXIS (2020) recommends setting OCR (for 1 day as reference) in the order of 1.2 to 1.4 for clays to avoid unrealistic initial creep rates. For peat a significantly higher OCR, than recommended by

PLAXIS (2020), may be needed to give realistic initial creep rates. This is particularly important for low levels of loading or for no loading (e.g. well outside of an embankment footprint). Due to the low unit weight of peat, and a yield stress of different nature to that in clay, the use of OCR to initialise the pre-consolidation stress may be difficult in peat. OCR will normally vary significantly through the deposit from very high values at the surface reducing towards 2 to 3 at deeper levels. Therefore it may be more convenient to use the pre-overburden pressure (POP) to initialise the model, even though no such pre-overburden has ever existed. This will be discussed in more detail later.

The stiffness parameters also need to be determined carefully. In particular the reloading stiffness needs to be chosen from an actual unload – reload load step in the oedometer test, rather than from the initial loading which is often softer.

A direct application of parameter ranges, experienced in clay for example to peat, using an advance model, would be wrong. It is clearly necessary to calibrate any numerical model to the material found on site. Most projects do not have the combination of tests necessary to calibrate such models and at present there is no detailed study correlating basic peat properties (such as water content, von Post H value or organic content) to the material parameters essential to advanced models. However the parameters of the SSC model are very much in line with classical engineering parameters and such correlations could easily be obtained.

Previous publications on the use of SSC in peat

van der Meij (2007) used the SSC model to back-calculate the deformation of two embankments on peaty soils in the Netherlands. The cases were the “No-recess case” at Hoeksche waard (van Duijvenbode et al., 1999) and the Betuweroute railway embankments. It was shown that the model was well able to capture the vertical time-settlement curve as well as the horizontal deformations at the toe of the embankments. Horizontal deformations away from the embankments were underestimated and pore pressures due to loading were overestimated. The critical input parameters were found to be OCR, the strength parameter (or slope of the critical state line) M, the coefficient of lateral earth pressure in the normally consolidated condition (K_0^{NC}), the initial permeability (k_{v0}), the rate of change of permeability with increasing stress (C_k or β) and the effective cohesion (c').

Tan (2008) and Tan and Paikowsky (2008) report on the performance of a relatively complex sheet piled wall embankment over a peat bog. SSC was used as the constitutive model for the peat and the finite element modelling yielded predictions which were in good agreement with the field measurements.

Osorio-Salas (2012) investigated a 10 m x 10 m vacuum consolidation trial in peat deposits at Ballydermot bog in Ireland. He found that the SSC model correctly modelled the stress – strain – time behaviour of the peat in the oedometer test. The model also well captured the compression strain – time behaviour during the field trial but overestimated heave. The predicted pore water pressures were not in agreement with those measured in the field trial.

Den Haan and Feddema (2013) report on the modelling of the deformation and strength of two embankments on soft Dutch soil. These were the Betuwelijn railway embankment, near Sliedrecht and the IJkdijk trial embankment at Booneschans. A specific feature of this work was the use of K_0-CRS testing to provide all the input parameters. In both cases the deformation and pore pressure behaviour of the embankments was well captured and in addition the failure of the IJkdijk trial embankment was also successfully modelled.

Tyurin and Nevzorov (2017) used SSC to predict the settlement of road embankments in Arkhangelsk, Russia. Some discrepancies were found between the SSC parameters obtained from oedometer tests and those which were determined from the back-analysis of measured settlements. The field settlement data included some which was collected over a period of 23 years.

DERIVATION OF PEAT AND SOIL PROPERTIES FROM IN SITU AND LABORATORY TESTING

General

In the following sections the derivation of the peat parameters for input into the SSC modelling will be described. The approach taken is to analyse the test results and obtain the relevant parameter in classical soil mechanics format, so the resulting values can be compared to published correlations. The derivation of the specific SSC properties will then be outlined. A summary of the index properties of the material are given on Table 1 and the model specific properties are outlined on Table 2.

Results of CRS oedometer tests

The results of the CRS tests are shown on Figure 4. The results are taken from the Master thesis of Markström (2018). The data are presented in (a) $\log_{10}$ stress (σ_v') versus strain ($\varepsilon = \Delta H/H_0$) format and (b) constrained modulus M_t ($=\Delta\sigma_v'/\Delta\varepsilon$) versus σ_v'. It proved very difficult to prepare samples of the sphagnum peat due to its loose, fibrous and honeycombed nature and unfortunately only part of one test was successful. The most notable feature of the results is the highly non-linear nature of the stress – strain response, especially around the yield / preconsolidation stress.

CRS testing has several advantages, for example the test is rapid and is not labour intensive, pore pressures are directly measured, and it also yields a continuous stress – strain curve making it easier to analyse the result. However it is not possible to simply determine creep properties from the test. Ideally it would be useful to perform both CRS and incrementally loaded oedometer tests but the latter equipment was not available at Lund University.

Yield stress (p_{vy}')

Use of the term “preconsolidation stress” is considered inappropriate for peat as the process of material formation was not by normal sedimentation. Here the term “yield stress” (p_{vy}') is used to nominally divide elastic and plastic behaviour (Chandler et al., 2004; Vaughan et al., 1988). The reasons why peat shows a yield stress are complex. Lefebvre et al. (1984) attribute snow loading, drainage, water table fluctuations and the creep characteristics as the cause of this. Hobbs (1986) also suggested that the structure of the plants and the decaying process that takes place in the upper layers of peat contributes to this “critical pressure”.

There is no well accepted method for determining p_{vy}' in peat. It had been hoped to use the Janbu (Janbu, 1963; Janbu, 1969) technique, which involves an assessment of the variation in M with σ_v', but no clear yield can be observed in these plots (Figure 4b). As a result two methods have been used namely the classical Casagrande (1936) as well as the “work method” of Becker et al. (1987). Both methods gave similar output with a range of p_{vy}', for the fen peat of between 13 kPa and 17 kPa (average 16 kPa) and these values are in good agreement with the correlation proposed by Ajlouni (2000) that p_{vy}' (kPa) = 150/e_0, where e_0 is the initial void ratio. Here e_0 was obtained from the equation:

$$e_0 = w_i G_s \tag{1}$$

where G_s is the specific gravity obtained from the correlation with the measured loss on ignition value (Den Haan and Kruse, 2007).

In PLAXIS the initial stress state (or initial size of the reference surface) can be specified using the POP as an alternative to the OCR. As explained earlier the use of POP gives better control of the initial state than use of OCR. POP is defined as:

$$POP = p_{vy}` - \sigma_{v0}` \tag{2}$$

where σ_{v0}' is the initial vertical effective stress. In this case, as the ground water table is close to the ground surface the σ_{v0}' in peat and gyttja is very small, and a POP value in the range 8 kPa to 10 kPa has been used.

For the clay an OCR of 1.6 has been assumed. The output is quite insensitive to the choice of this parameter, as it is well above 1.4 (see discussion earlier) and due to the limited effective stress increase in the clay layer.

Compression index (C_c) and modulus number (m)

As can be seen from the data presented in Figure 4a, the value of the compression index (C_c) has to be chosen with caution as the e (or ε) versus $\log_{10} \sigma_v'$ plot is non-linear. For example the compression coefficient ($C_c/1+e_0$) varies between 0.1 at stresses less than p_{vy}', to 0.35 around p_{vy}' and is about 0.55 for stresses just above p_{vy}'. For stresses ≈ p_{vy}' + 20 kPa, which is often the range of loading of most interest to practicing engineers, C_c values range between 6.1 to 7.6 ($C_c/1+e_0$ varies between 0.5 and 0.6). These data agree well with the correlation proposed by Hobbs (1986) for fen peat who suggested $C_c = 0.008\ w_i$.

In Scandinavian practice settlement in the normally consolidated zone is often determined using the modulus number m, which is the slope of the M_t versus σ_v' curve after p_{vy}', see Figure 4b. Values of m (again for stresses ≈ p_{vy}' + 20 kPa) are between 6 and 7.5 and these values agree very well with the empirical relationship between m and dry density (ρ_d) proposed by Carlsten (2000) for Swedish peat.

The SSC model's relevant input parameter is the modified compression index (λ*) where:

$$\lambda^* = \frac{C_c}{2.3(1+e)} \tag{3}$$

The λ* values have been chosen from the data on Figure 4, where appropriate or otherwise from the correlation with w_i.

Swelling index (C_s)

No specific unloading stage was included in the CRS tests and the value of the swelling index C_s was obtained from the relationship C_s/C_c = 0.1 to 0.12 (Mesri and Ajlouni, 2007).

In the SSC the modified swelling index (κ*) is used where:

$$\kappa^* \approx \frac{2C_S}{2.3(1+e)} \quad (4)$$

Waterman and Broere (2005) suggest the ratio λ^*/κ^* should normally be between 2 and 10 and here the ratio of the values used (3.9 to 10) falls within this range.

Creep coefficient (C_α)

The creep coefficient C_α (=$\Delta e/\Delta \log t$) was determined from the correlation between C_α and C_c, first introduced by Mesri and Godlewski (1977). This proposes that values of C_α/C_c is in the range 0.01 to 0.07 for all geotechnical materials. For peat, Mesri and Ajlouni (2007) suggest C_α/C_c= 0.06 ±0.01. Mesri et al. (1994) have pointed out that similar to C_c, C_α need to be chosen cautiously due to the non-linearity of the stress-strain response. In SSC use is made of the modified creep index:

$$\mu^* = \frac{C_\alpha}{2.3(1+e)} \quad (5)$$

and here it is assumed μ^*/λ^* = 0.06 for the peat, reducing to 0.4 for the gyttja and 0.3 for the soft clay (Mesri and Godlewski, 1977).

Waterman and Broere (2005) recommend that when using the SSC model, the user should consider the creep ratio: $(\lambda^* - \kappa^*) / \mu^*$. This ratio can have a wide range of values, normally between 5 and 25. For most practical cases the ratio falls within the range of 10 to 20 for peat, about 20 for gyttja and 20 to 30 for clays.

Coefficient of permeability (k_v)

Peat is well known to display large values of permeability at in situ effective stress (k_{v0}) due to the open fabric and large initial void ratio. The great variability of the particle types as well as the arrangement of the particles results in a wide range of permeability values (Ajlouni, 2000). In addition it is well accepted that the k_v value subsequently decreases rapidly with increasing stress. In this project k_{v0} values (i.e. at about 5kPa) were determined from the interpretation of the pore pressure data from the CRS tests by following the procedure outlined by Tavenas et al. (1983). The values ranged between 10^{-7} m/s and 5×10^{-6} m/s which are in agreement with the empirical trendlines (between e_0 and k_{v0}) proposed by Mesri and Ajlouni (2007) and that (between w_i and k_{v0}) proposed Carlsten (2000).

In Scandinavian practice the slope of the strain to log coefficient of permeability relationship is denoted by β and is defined:

$$\beta = \frac{\Delta log k}{\Delta \varepsilon} \tag{6}$$

and β values of between 5 and 6 were obtained from the CRS tests, and these are consistent with the findings of Carlsten (2000).

Otherwise reference is often made to C_k, which is the permeability change index defined as:

$$C_k = \frac{\Delta e}{\Delta log k} \tag{7}$$

Mesri and Ajlouni (2007) suggested that C_k/e_0 for peat is about 0.25 as compared to 0.5 for soft clay and silt material. The values obtained here from the CRS tests are consistent with the Mesri and Ajlouni correlation.

Strength parameters

The strength of the material in the SSC model is limited to the lesser of the boundary of the cap surface (M parameter determined based on K_0^{NC}) and what is limited by the Mohr-Coulomb (MC) surface (defined by the friction angle, φ, and the dilatancy angle, ψ). These parameters need to be consistent with one another. M is defined as:

$$M = 3\sqrt{\frac{\left(1-K_0^{nc}\right)^2}{\left(1+2K_0^{nc}\right)^2} + \frac{\left(1-K_0^{nc}\right)(1-2\upsilon)\left(\frac{\lambda^*}{\kappa^*}-1\right)}{\frac{\lambda^*}{\kappa^*}\left(1+2K_0^{nc}\right)(1-2\nu)-\left(1-K_0^{nc}\right)(1+\nu)}} \tag{8}$$

where ν = Poisson's ratio (elasticity constant).

K_0^{NC} values in peat are low due to the restraining effect of the fibers. Furthermore $K_0 = K_0^{NC}$ is used as an inherent feature of the SSC model and it is assumed that K_0 is constant under 1D creep (Den Haan, 2001).

Here K_0^{NC} values have been determined from empirical correlations published in the literature (Ajlouni, 2000; Den Haan and Feddema, 2013). The friction (and dilation angle) for the peat layers are obtained from the M value, where:

$$Sin\varphi = \frac{3M}{6+M} \tag{9}$$

It is acknowledged that the chosen friction angle for peat is unusually high. Similar high values have been reported for peat from many other countries. Mesri and Ajlouni (2007) give a useful summary of international work on this topic. Den Haan and Kruse (2007) and others explain that the reinforcing effects of the mostly horizontally orientated fibers contributes to this effect and point out that the high frictional angles are mostly associated with low effective stress levels.

Furthermore a tension capacity for the peat ($\sigma_t = c - cot\varphi$) can be applied based on cohesion (c) and friction angle. Here σ_t values of 2 kPa to 3 kPa have been used depending on the degree of decomposition (H) of the peat (Dykes 2008). These are assumed to be isotropic due to the limitations of the model. For the gyttja and clay σ_t is assumed to be zero.

Poisson's ratio

The effective Poisson's ratio during unloading / reloading (ν) can theoretically range between -1.0 and +0.5. However, the lowest value allowed in PLAXIS is 0.0 and this is adopted for all the materials in this work. This is done to ensure the highest possible elastic shear stiffness. Grimstad et al. (2016) experienced that this maximum shear modulus (G) along with a relative short distance to the vertical boundaries (20 m from centreline) ensures more elastic horizontal deformations under load application. A low shear stiffness, combined with a long distance to the boundary, would give unrealistic large horizontal deformations for undrained loading of a soil at low effective stress operating within the elastic range. PLAXIS (2020) determines G from the expression:

$$G = \frac{p^{'} + cCot\varphi}{1.5\kappa^{*}\frac{(1-2\upsilon)}{(1+\upsilon)}} \qquad (10)$$

where p′ = mean effective stress. For the conditions and parameters assumed here, outside of the embankment, with ν = 0, G is of the order of 100 kPa. If ν = 0.2 then G is approximately 50 kPa. The small strain shear modulus (G_{max}) interpreted from the shear wave velocity measurements given in Figure 3a is in the order of 200 kPa to 400 kPa. This is consistent with the G value corresponding to ∨ = 0 and indirectly justifies the value chosen for the Poisson's ratio.

CONSTRUCTION DETAILS

A plan layout of the study area and a cross section through the road construction are shown on Figures 5a and 5b respectively. A geotextile was initially laid on the existing surface and the road was built up in three stages, each about 0.5 m thick. Imported gravel fill was used for this purpose. In the initial design an option of a fourth surcharge stage was considered but was not ultimately implemented. At Section 8040 m the following construction sequence was followed:

Day 0 – Day 17: load step 1a,

Day 17 – day 82: load step 1a+1b,

Day 82 – onwards: load step 1a+1b+2.

In situ settlement measurements

Settlements were measured by levelling the top of vertical steel rods attached to steel plates which were placed on the original ground surface. Measurements were made at approximately 40 m intervals along the length of the road. Three measurements were made at each cross section on the east, west and center of the road. Conventional geodetic surveying was used, and the expected accuracy was ±1 mm. Maximum centreline settlements varied between zero where there was no peat present to about 1690 mm at Section 8120 m, just north of the section under study (Figure 5a). The measured settlements at Section 8040 are shown on Figure 6.

There was a significant increase in settlement at each stage of loading and the settlement at the center of the section was approximately the average of that on the east and west sides of the road. The maximum measured centreline settlement was 1320 mm on Day 202 when settlement measurements ceased.

An estimate of the likely total settlement using the simple empirical method of (Carlsten, 1989) is also shown on Figure 6. The estimate of about 1330 mm is very close to the actual measured settlement.

NUMERICAL MODELLING

General

The full width of the embankment was modelled resting on a layer 40 m wide and 22 m deep (base at stiff layer, see Figure 2 and Figure 8). Groundwater pressures were assumed to be hydrostatic from ground level. The boundaries were assumed closed on the sides and bottom, but the top was kept open. Altogether 1286, 15 noded triangular elements were used (e.g. the sides of the elements were approximately 1 m long). Due to the large deformations involved the influence of the geometry change of the mesh in equilibrium conditions is accounted for by using the Lagrangian mesh updating function in PLAXIS. Water pressures are also updated. These functions are particularly essential in cases such as this when the effective weight of the fill will gradually reduce due to buoyancy forces.

Construction sequence

The modelled construction sequence was as follows:

1. Initial creep phase. This includes the historical road construction of 10 days in addition to unchanged conditions for 20,000 days. After this, displacements are reset to zero. The main purpose of this stage is to generate the appropriate initial geometry and account for the correct OCR due to the previous road. In general, a calculation step with a long time span before the remaining analyses are carried out will help start the model

at a reasonable initial strain rate, or identify if the initial rate is too high (for example if unrealistic pore pressures are generated).

2. Load step 1a: 0.5 m, construction period 7 days, consolidation 10 days.
3. Load step 1b: 0.5m, construction period 7 days, consolidation 58 days.
4. Load step 2: 0.5 m, construction period 7 days, consolidation 113 days

SSC input parameters for peat, gyttja and clay

Analyses using the SSC model were performed using three parameter sets for the different materials. These parameters are summarised in Table 2. In addition some analyses were carried out using the PLAXIS soft soil model (SS), which is essentially identical to SSC except that it disregards creep. The reference time in the SSC simulations is set to 1 day in all analyses.

The background for the various parameter sets are as follows:

- Set I is an engineering guess, using established rules of thumb, and ensuring reasonable initial deformation rates in combination with the initial estimated pre-consolidation stress. The creep ratio ($\lambda^* - \kappa^*$) / μ^* is selected as 15 for the sphagnum peat, 18 for the fen peat, 22.5 for the gyttja and 30 for the clay.
- Set II gives parameters fitted around the pre-consolidation stress based on simulation of the CRS tests (see next section). The set also reflects a more careful assessment of the pre-consolidation stress (i.e. POP). Therefore analysis results using Set II are expected to give the best results for the moderate loading (Load steps 1a and 1b). However it is expected to over-estimate settlements for significant loading well above the pre-consolidation stress (Load step 2).
- Set III uses the index properties (e.g. water content) to identify model parameters for large stress / strain interval using correlations from the literature as described earlier. These estimates are done without any concern for the relevant stress /strain interval for this particular problem. POP is set to 10 kPa for peat and gyttja as discussed above.

- In Set SS the parameters of Set I is used but the model is changed to SS. This means that Set SS is the same as Set I except for the creep parameter, μ^*. This demonstrated the relative importance of creep in peat when compared to analyses that disregard creep.

Embankment and geotextile parameters

For the embankment material a classical Mohr-Coulomb constitutive model was employed with: γ_{dry} = 17 kN/m^3, γ = 20 kN/m^3, Young's modulus (E) = 40 MPa, ν = 0.2. φ = 55°, ψ = 25°, k_v and k_h = 10 m/day. The relatively high friction angle for the embankment fill was chosen such that the analysis is not to a large extent influenced by plastic behaviour of this material, so as to allow the geotextile to be correctly modelled, and also to reflect the behaviour of this material at low effective stress.

For the N4 Geotextile it was assumed EA (A is the cross-sectional area) = 12 kN/m and yield strength (N_p) = 11 kN/m

Simulation of CRS oedometer test

The model and the input parameters were first calibrated by simulating the CRS tests shown on Figure 4. These simulations are all one-point analyses, so pore pressure dissipation is not considered. The results are shown on Figure 7 and compared to the fen peat tests from depths of 500 mm and 700 mm – test 2. These latter tests represented the range of the CRS measurements. In general the simulations capture the measured behaviour very well. All three simulations are stiffer than the measurements before yield. This reflects some sample disturbance effects which are known to reduce the initial stiffness and the yield stress. Simulations for the Set I and Set II parameters are similar except for the behaviour around and just beyond the yield stress. Set III simulations underestimate the post yield laboratory deformations.

Simulation of embankment monitored for settlements

The settlements predicted by the full embankment model are shown on Figure 6, together with the measurements. It is worthwhile to mention that no attempt has been made to alter the model input parameters so that they give “best fit” to field measurements. The parameters established based on the laboratory test results, empirical correlations and other experience as summarised above have been used directly without alteration. Overall it can be observed that the results of the model correctly captures the pattern of measured settlements.

Set I underestimates the initial settlements but matches the final settlements reasonably well. As expected the Set II parameters capture the initial load steps (1a and 1b) very well but tends to overestimate the settlements due to the higher loads. Set III underestimates all the measured settlement, consistent with the findings from the CRS test modelling, shown on Figure 7. As expected the SS simulation initially follows the SSC Set I predictions until the creep movements begin to play an increasingly important role, in the later stage of applied load.

Figure 8 shows the vertical strain contours, superimposed on the deformed shape, for the Set II and Set III simulations after Day 202. In Set II the strain in the upper sphagnum peat layer is significantly larger than for Set III, while there is a slightly higher strain level in the gyttja layer for the Set III compared to Set II. In both cases the strains in the clay layer are negligible.

An analysis was also carried out where the PLAXIS mesh and water pressure updating routine was not used and this resulted in the prediction of unrealistically large settlements.

Many recent contracts are constructed under a form of contract in which the contractor has also long term responsibility for the performance of the built construction. In these cases prediction of long-term settlements is very important so the contractor can estimate the likely maintenance costs. This consideration may not be clear for limited calculation duration as shown in Figure 6 where the importance of creep is not apparent. In order to appreciate this aspect a prediction of long-term settlement, using Set III parameters with and without creep is shown on Figure 9. The importance of accounting for creep in the calculations can now clearly be seen. In the prediction with the SS model, after about 5 years, all of the excess pore pressures have dissipated resulting in a settlement of 1400 mm that remains constant with time. However the SSC model predicts substantially greater settlement due to the creep effects. The SSC model gives an increasing settlement with time but at a decreasing rate of increase. This is considered to be a realistic long-term behaviour.

CONCLUSIONS AND RECOMMENDATION FOR FUTURE USE OF THE MODEL

The main objectives of this work were to present the results of some careful monitoring of the construction of a road embankment in peat in Sweden. Such construction is likely to become more important in future years due to the extensive peatland areas in Sweden and other northern countries, increased development of these areas and also to for carbon retention purposes. The paper also sought to attempt to close the gap between researchers and engineers working in practice on the application of a reasonably advanced but commercially available numerical model for the prediction of settlements in peat. Some conclusions from the work are as follows:

- The work presented demonstrates a relatively good match between field measurements and simulations, using the SSC model with input parameters obtained from a combination of measurements, published correlations and model calibration based on laboratory tests.
- However, it is difficult to model all stages of loading accurately with one set of input parameters in SSC. The user needs to identify and focus on critical output of the analyses (e.g. short-term / long-term / situation at the end of particular loading steps etc.). Several analyses may be needed to obtain accurate prediction for all cases.
- It is important to initially calibrate the model by first simulating some independent data, for example in this case the results of oedometer tests.

- Although the focus of the model used was on creep prediction, the added benefit of using a finite element analysis over hand calculations is that various layers can be easily modelled and buoyancy can be accounted for. In addition, parameters such as permeability can be updated with time / deformation and time dependency can also be evaluated. Perhaps these fundamental parameters are more important than creep for peats.
- Use of the SSC model requires careful choice of input parameters especially initial stress state (OCR) and permeability (where the correlations in the literature can suggest widely varying values) and stiffness. Creep properties are especially important if long-term behavior is of interest. All of these parameters were available from routine oedometer tests, careful consideration of published correlations and validation by laboratory test simulations.
- For peat, where the unit weight is close to that of water, determination of the ground water table and the piezometric regime is very important. Perhaps more efforts are required during ground investigations to clearly define these conditions
- From a practical point of view, both the modelling and the measurements show that placing a geotextile on the peat surface prior to construction will be of significant benefit.

ACKNOWLEDGMENTS

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TABLES

See document attached

Summary of figures / figure captions

Fig. No.	Caption	File Ref
1	Pictures of study area, (a) prior to upgrading July 2011 (from Google maps), (b) during the ground investigation works by Skanska in Summer 2015, (c) during the upgrading works in Spring 2016 and (d) after completion of the works in August 2018. Photographs from pmsv3.trafikverket.se.	DELL/MiscRes/Skanska Färgelanda Väg 2081 – photos in Grapher file.grf
2	Study site stratigraphy and parameters (a) water content, (b) CPTU q_t, (c) CPTU f_s and (d) CPTU u_2 and u_0	DELL/MiscRes/Skanska Färgelanda Väg 2081 - water content and CPTU.grf
3	Peat properties on (a) study site (von Post H, water content and V_s) and (b) on other site used for block sampling of lower peat	DELL/MiscRes/SwedishPeat Färgelanda.grf Ageröds Mosse
4	CRS oedometer tests after Markström (2018) (a) $\log_{10} \sigma_v'$ versus ε and (b) σ_v' versus M_t	DELL/MiscRes/SwedishPeat Oscars best tests
5	(a) Plan layout of study area and (b) cross section through road construction	DELL/MiscRes/Skanska Färgelanda Väg 2081 – study area and cross section.grf
6	Measured and predicted settlements at cross section 8040m	DELL/MiscRes/Skanska Färgelanda Väg 2081 – settlements.grf
7	Simulation of CRS oedometer tests by SSC	DELL/MiscRes/SwedishPeat CRSsimulation.grf
8	Vertical strain at Day 202 for parameter Sets II and III superimposed on deformed mesh	DELL/MiscRes/Skanska Färgelanda Väg/PLAXIS/ Verticalstrains.grf
9	Prediction of long-term settlements	DELL/MiscRes/Skanska Färgelanda Väg 2081 – Long term settlements.grf

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Table 1. Index and hydraulic properties of the sol materials

	Layer	Depth	w	e_0	γ	k_v and k_h	C_k
		m	%		kN/m^3	m/day	
Set I	Sphagnum peat	0 – 2.2	1200	18	10.3	86.4E-3	6
	Fen peat	2.2 – 6	800	12	10.4	8.64E-3	6
	Gytja	6 – 9	150	3.0	12.5	432E-6	0.8
	Clay	9 – 21	53	1.43	17.0	86.4E-6	1.1
Set II steep	Sphagnum peat	0 – 2.2	1170	17.5	10.3	0.864	3.4
	Fen peat	2.2 – 6	740	11.1	10.4	0.011	3.4
	Gytja	6 – 9	190	3.8	12.1	0.432E-3	1.2
	Clay	9 – 21	70	1.9	15.9	86.4E-6	1.0
Set III index	Sphagnum peat	0 – 2.2	1170	17.5	10.3	0.864	3.4
	Fen peat	2.2 – 6	740	11.1	10.4	8.64E-3	2.2
	Gytja	6 – 9	190	3.8	12.1	864E-6	1.2
	Clay	9 – 21	70	1.9	15.9	86.4E-6	1.0

Table 2. Model specific parameters for SSC model

	Layer	Depth	λ^*	κ^*	μ^*	ν	*POP*	*OCR*	$K_0^{(NC)}$	*M*	φ	ψ	σ_t (and *c*)
		m					kPa				Deg.	Deg.	kPa
Set I	Sphagnum peat	0 – 2.2	0.5	0.05	0.03	0.0	10	n/a	0.164	3.0	~90	~90	3.0 (~inf)
	Fen peat	2.2 – 6	0.4	0.04	0.02	0.0	10	n/a	0.276	2.435	60	60	2.0 (3.46)
	Gytja	6 – 9	0.2	0.02	0.008	0.0	10	n/a	0.429	1.85	45	0	0 (0)
	Clay	9 – 21	0.1	0.01	0.003	0.0	n/a	1.6	0.572	1.418	35	0	0 (0)
Set II steep	Sphagnum peat	0 – 2.2	0.56	0.065	0.0330	0.0	8	n/a	0.163	3.0	~90	~90	3.0 (~inf)
	Fen peat	2.2 – 6	0.45	0.054	0.0220	0.0	9	n/a	0.273	2.435	60	60	2.0 (3.46)
	Gytja	6 – 9	0.22	0.022	0.0088	0.0	10	n/a	0.429	1.85	45	0	0 (0)
	Clay	9 – 21	0.10	0.010	0.0030	0.0	n/a	1.6	0.572	1.418	35	0	0 (0)
Set III index	Sphagnum peat	0 – 2.2	0.27	0.070	0.0165	0.0	10	n/a	0.153	3.0	~90	~90	3.0 (~inf)
	Fen peat	2.2 – 6	0.27	0.060	0.0160	0.0	10	n/a	0.261	2.435	60	60	2.0 (3.46)
	Gytja	6 – 9	0.16	0.040	0.0082	0.0	10	n/a	0.402	1.85	45	0	0 (0)
	Clay	9 – 21	0.090	0.020	0.0027	0.0	n/a	1.6	0.547	1.418	30	0	0 (0)

Figures for paper by Long et al. on use of SCC in Swedish peat

FIGURE 1. Pictures of study area, (a) prior to upgrading July 2011 (from Google maps), (b) during the ground investigation works by Skanska in Summer 2015, (c) during the upgrading works in Spring 2016 and (d) after completion of the works in August 2018. Photographs from pmsv3.trafikverket.se

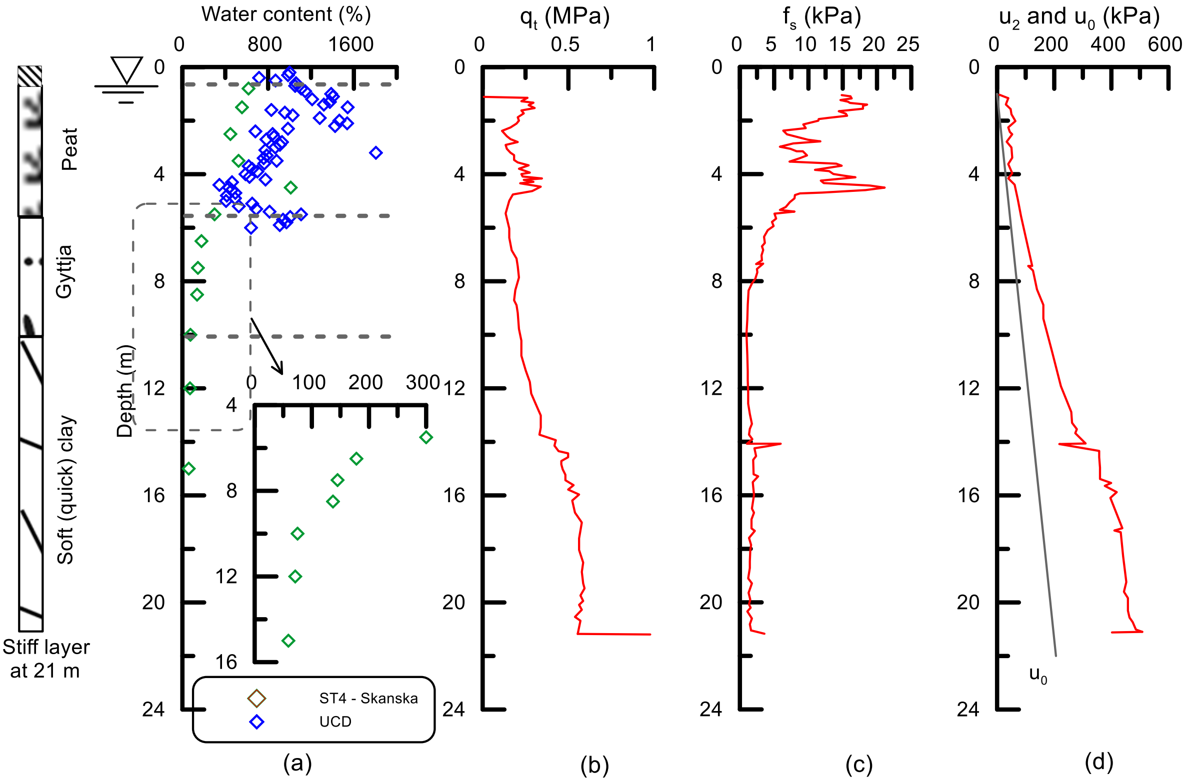
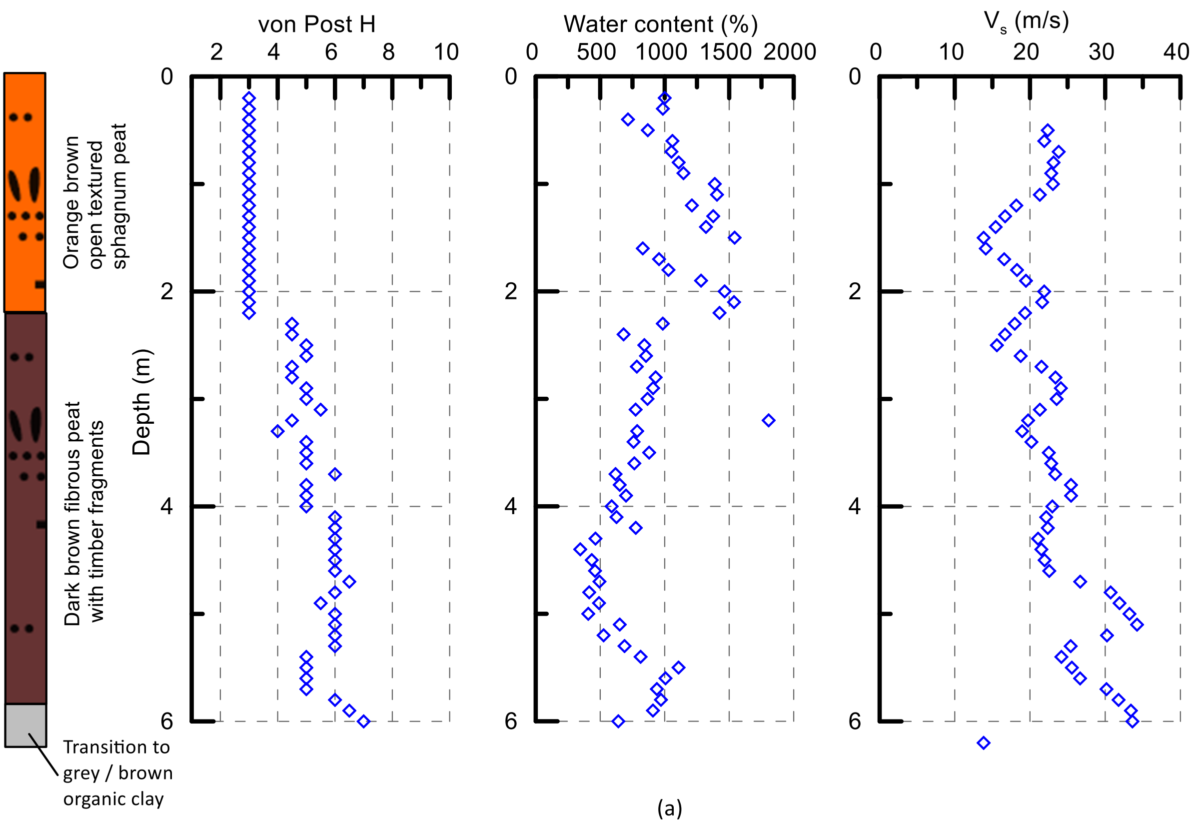


FIGURE 2. Study site stratigraphy and parameters (a) water content, (b) CPTU q_t, (c) CPTU f_s and (d) CPTU u_2 and u_0



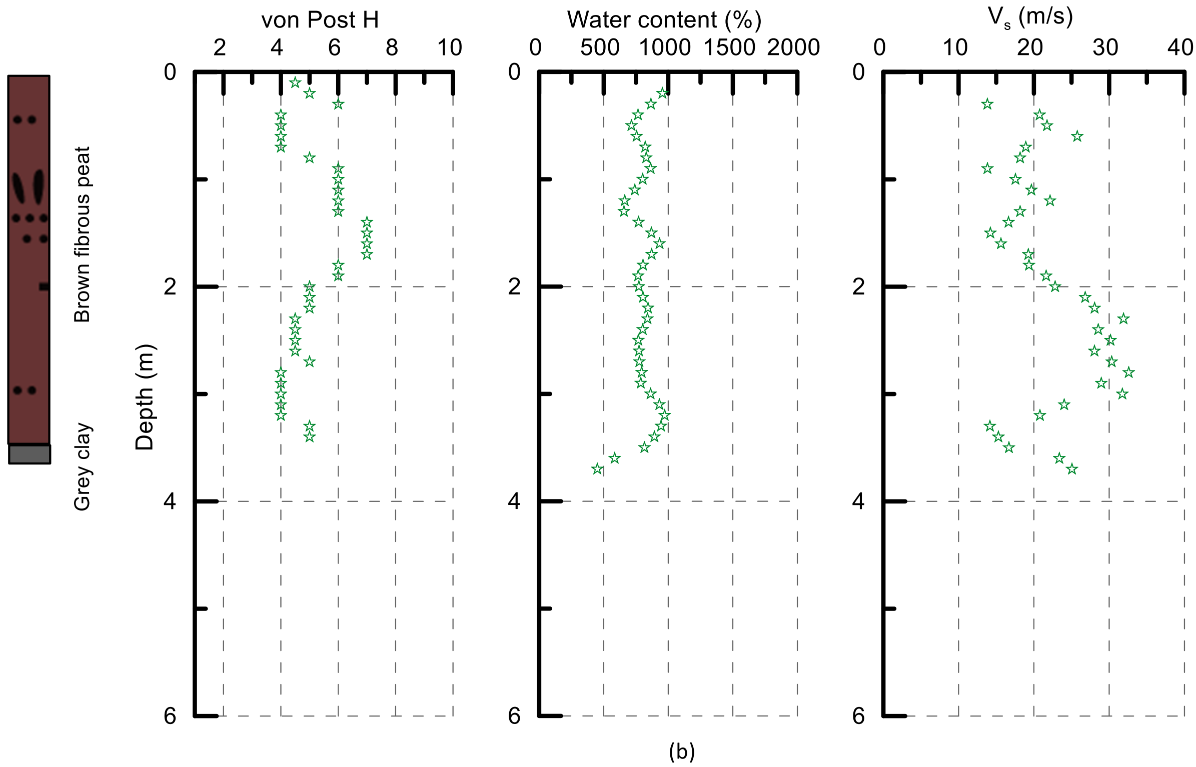


FIGURE 3. Peat properties on (a) study site (von Post H, water content and V_s) and (b) on other site used for block sampling of lower peat

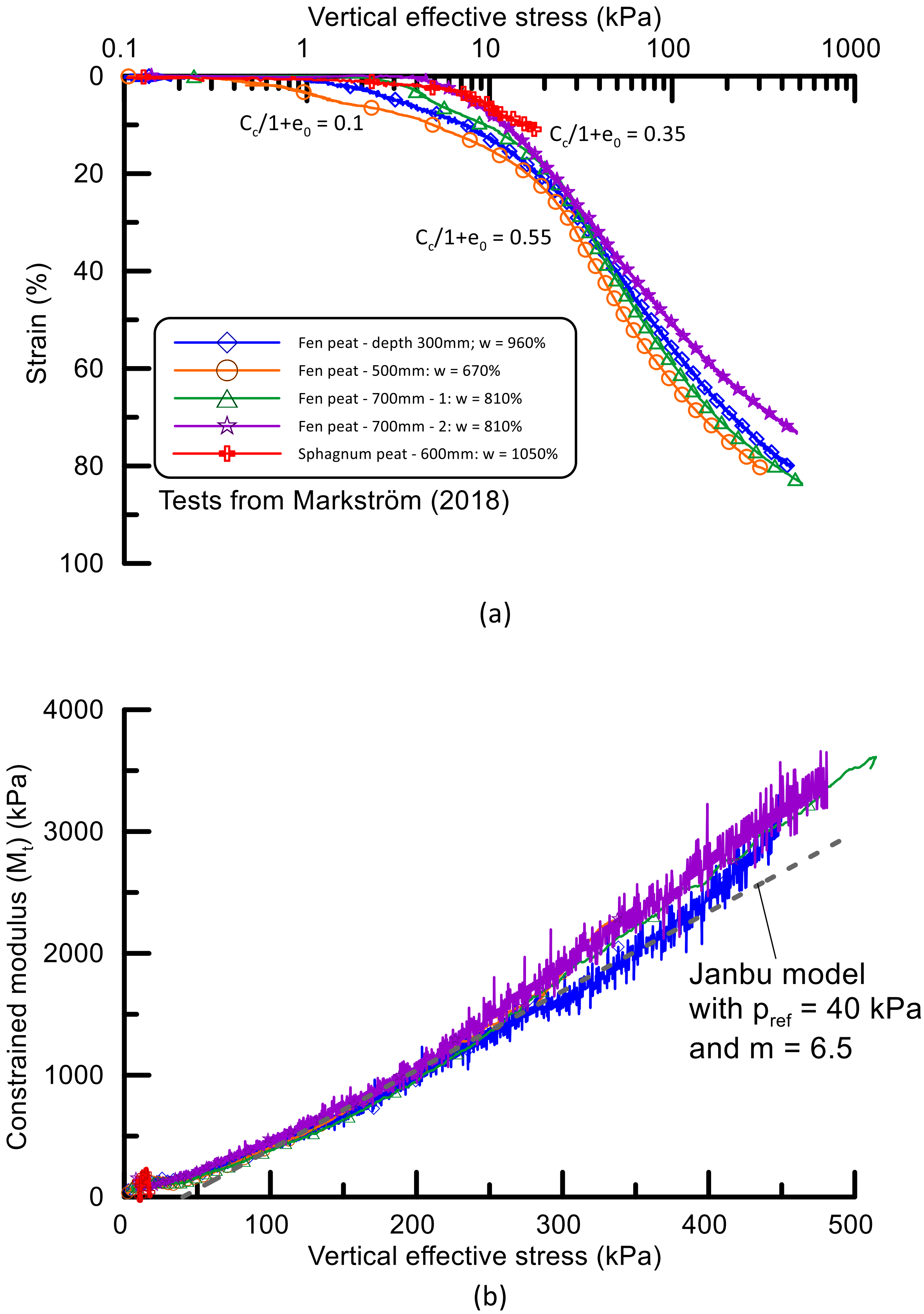
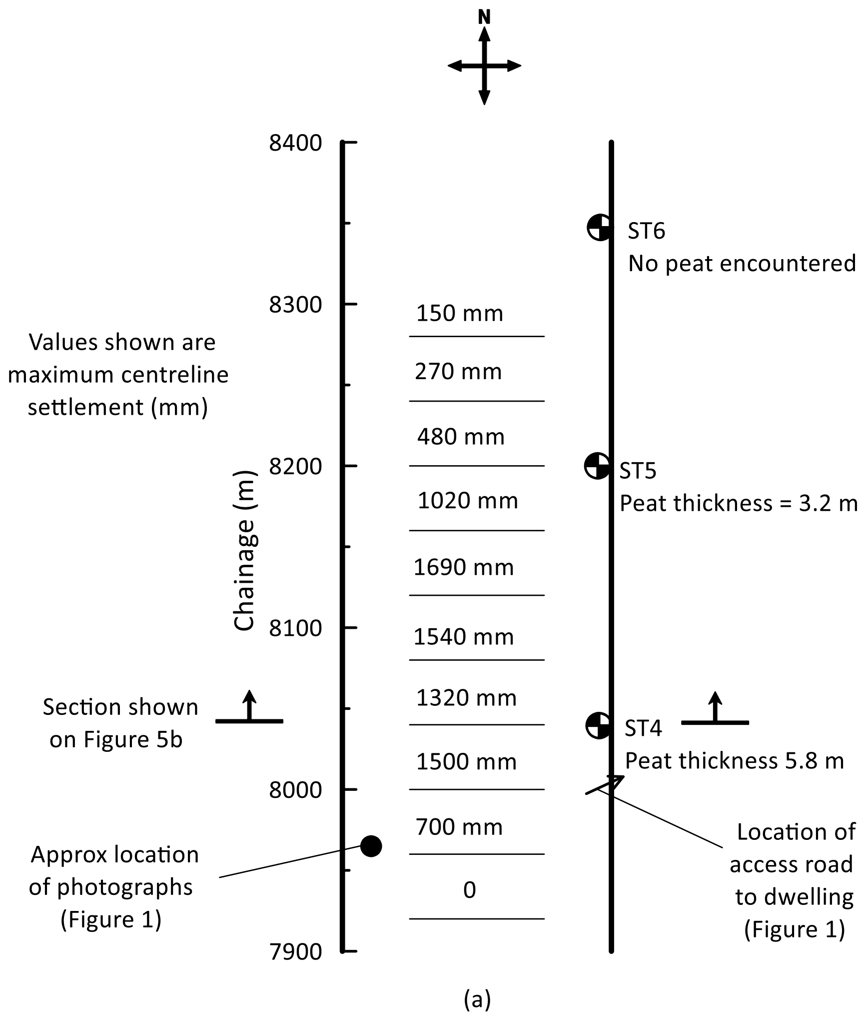


FIGURE 4. CRS oedometer tests after Markström (2018) (a) $\log_{10} \sigma_v'$ versus ε and (b) σ_v' versus M_t



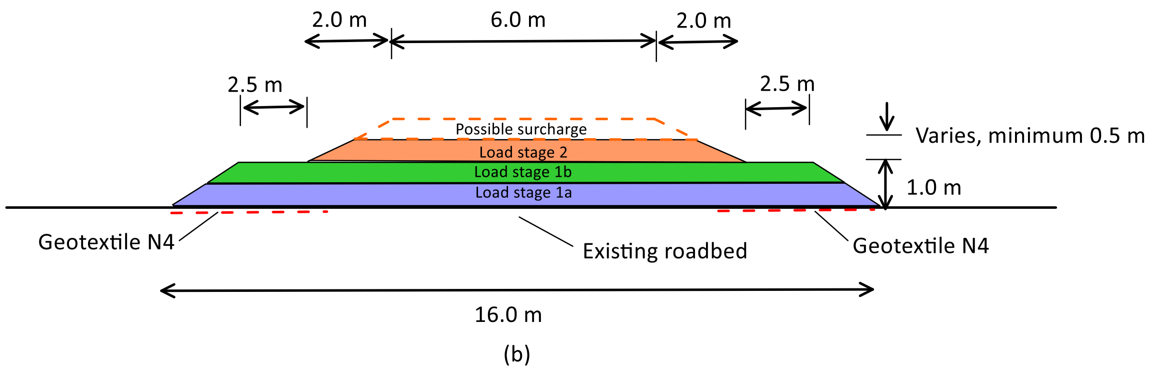


FIGURE 5. (a) Plan layout of study area and (b) cross section through road construction

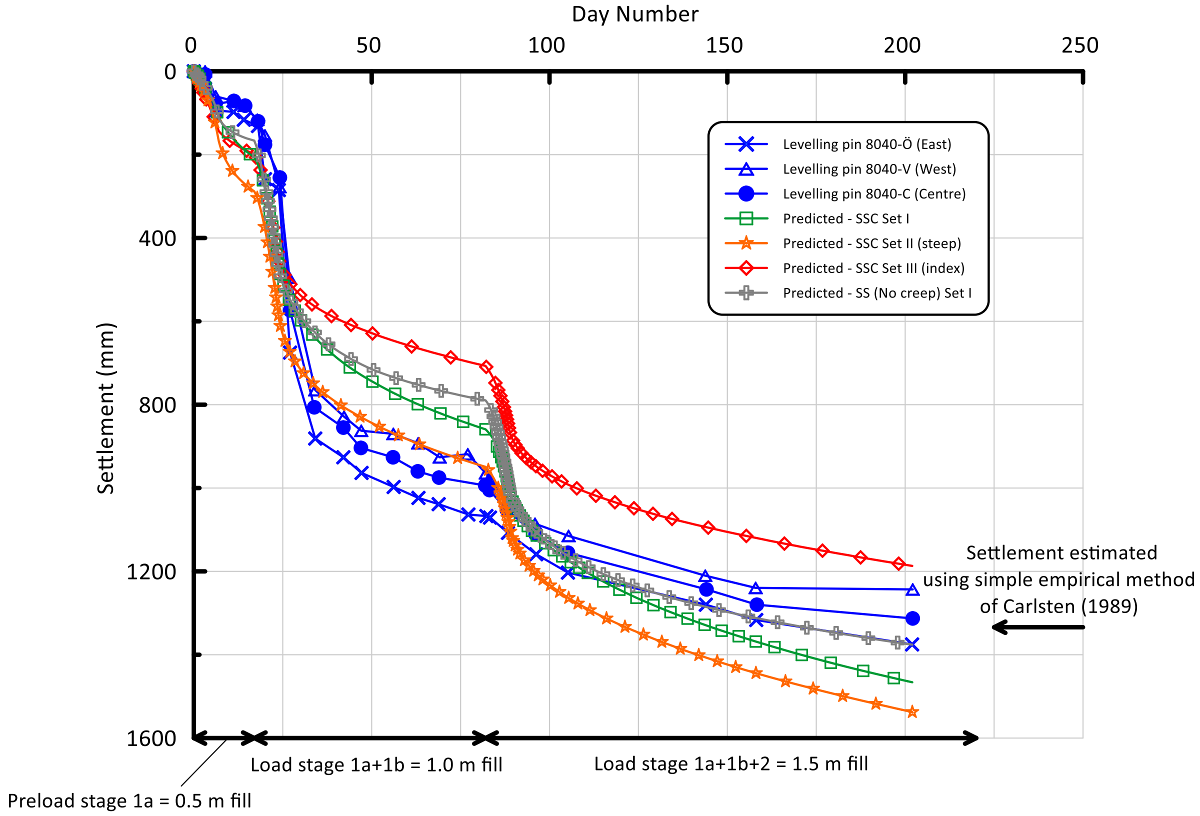


FIGURE 6. Measured and predicted settlements at cross section 8040m

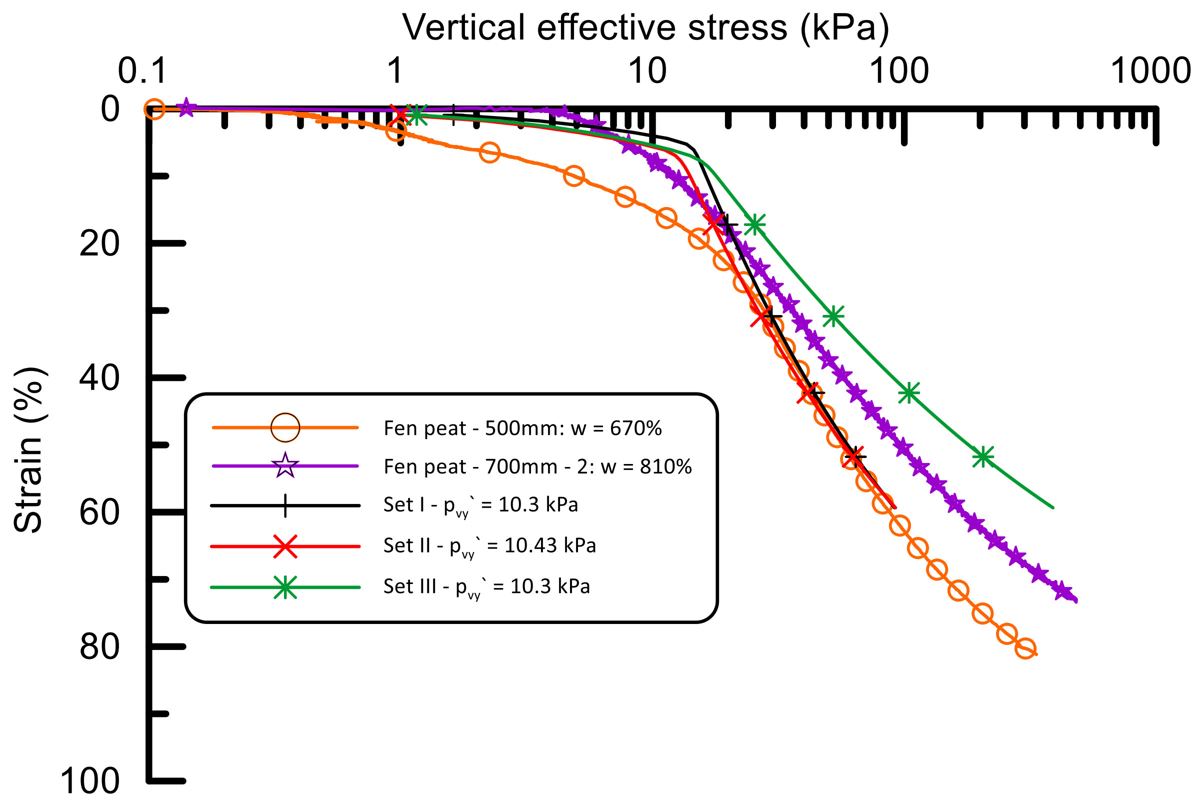


FIGURE 7. Simulation of CRS tests by SSC

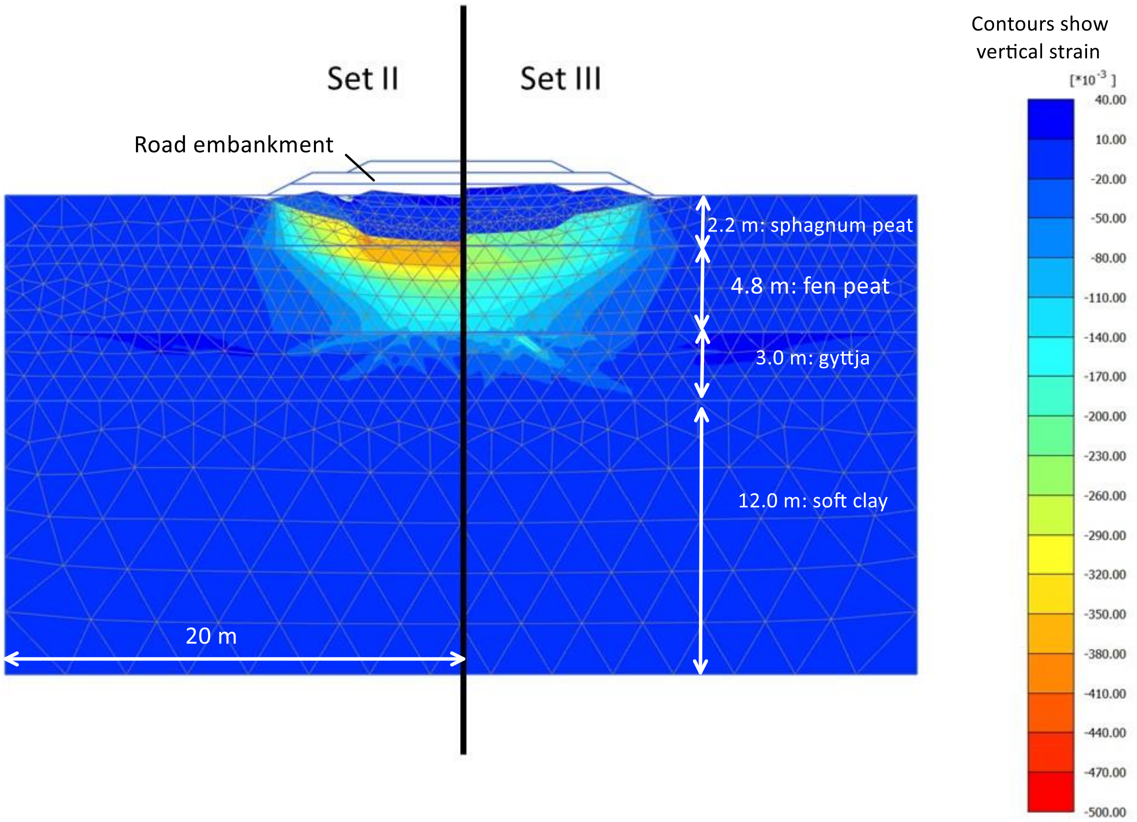


FIGURE 8. Vertical strain at Day 202 for parameter Sets II and III superimposed on deformed mesh

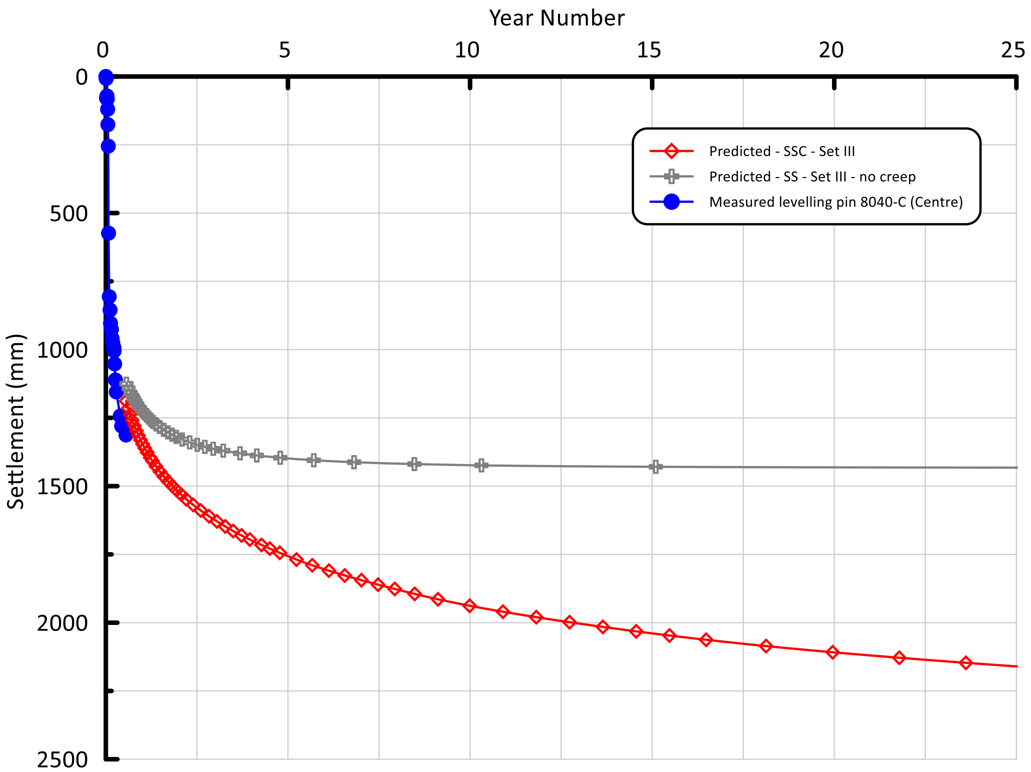


FIGURE 9. Prediction of long-term settlements

oOo

List of symbols and abbreviations

Term / symbol	Units	Meaning
CPTU		Piezocone or cone penetration test with pore pressure measurements
CRS		Constant rate of strain oedometer test
LTH		Lund University, Sweden
OCR		Overconsolidation ratio
POP	kPa	Pre-overburden pressure $POP = p_{vy}` - \sigma_{v0}`$
SSC		Soft soil creep constitutive model
C_c		Compression index
C_k		Rate of change of permeability with increasing stress $C_k = \frac{\Delta e}{\Delta logk}$
C_s		Swelling index
C_α		Creep coefficient C_α (=Δe/Δlogt)
G	kPa	Shear modulus (G_{max} = small strain shear modulus)
G_s		Specific gravity
H		Degree of humification in von Post scale
H_0	mm	Original sample height before test
K_0^{NC}		Coefficient of lateral earth pressure in the normally consolidated condition
M		Model strength parameter (Critical State Soil Mechanics strength parameter)
M_t	kPa	Constrained modulus $=\Delta\sigma_v'/\Delta\varepsilon$
Np		
R_f	%	Friction ratio ($R_f = 100*f_s/q_t$) in CPTU test
V_s	m/s	Shear wave velocity
c	kPa	Cohesion
e_0		Initial void ratio
f_s	kPa	Sleeve friction in CPTU test
k	m/s	Soil permeability
m		Modulus number
p'	kPa	Mean effective stress
p_{vy}'	kPa	Yield stress
q_t	kPa	Corrected cone end resistance in CPTU test
u_0	kPa	In situ or ambient pore water pressure
u_2	kPa	Pore water pressure measured just behind cone in CPTU test
w_i	%	Initial water content in soil sample
β		Rate of change of permeability with increasing stress $\beta = \frac{\Delta logk}{\Delta\varepsilon}$
ψ	Degrees	Dilation angle
ε		Strain = $\Delta H/H_0$
ϕ	Degrees	Friction angle

γ	kN/m^3	Unit weight of soil sample
κ^*		Modified swelling index in SSC
λ^*		Modified compression index in SSC model
μ^*		Modified creep index
ρ_d	Mg/m^3	Dry density
σ_t	kPa	Tension capacity of peat
σ_v'	kPa	Vertical effective stress
ν		Poisson's ratio (elastic).