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A numerical study of time effects on the stability of a test embankment on sensitive soft clay

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ABSTRACT: The Ultimate Limit State (ULS) of embankments on soft and sensitive soils has long been a focus in geotechnical research, but primarily in the short-term. How the ULS and the bearing capacity changes with time have not much studied so far. In this paper, the Haarajoki test embankment was modelled using the Finite Element method and the advance constitutive model Creep-SCLAY1S. The model was able to match the settlements and the long term dissipation of the excess pore pressure. The bearing capacity was evaluated at different time steps by bring the embankment to failure with an additional line load on top. The calculated initial bearing capacity calculated by the model matched the bearing capacity calculated using the undrained shear strength profile of the soil. After one year of consolidation, the bearing capacity had not increased, after four years it had increased about 5% and after 104 years about 40%. During this time, the failure mechanism changed from Prandtl shaped initially to a punching failure after 104 years. The analyses also highlight the trigger levels for failure in terms of the excess pore pressure and horizontal displacements.

Keywords: Creep-SCLAY1S; Soft clays; Embankment; Numerical modelling

1 INTRODUCTION

The Ultimate Limit State (ULS) of embankments built on soft soils has long been a focus of research, e.g. La Rochelle et al., (1974), Zwanenburg et al., (2012) and Lethonen et al., (2015). Usually, however, the focus has been on the short-term behaviour, primarily during the construction and right after when the stability has been assumed to be the lowest. How the stability changes with time has not been a focus of the much of the research. An exception of this is Zdravković et al., (2019) who looked at how the long-term behaviour of the soil could affect the stability and demonstrated how it could be used to extend the life of linear infrastructure. The simulations, however, did not account for the degradation that is typical for sensitive clays.

The ULS of the embankment is usually just assessed by a stability analysis with the undrained shear strength (s_u) of the soil before construction. However, this is perhaps conservative, since it does not consider that s_u is an emerging property depending on several factors such as the type of loading, loading rate and stiffness. Several of these factors can change with time, due to time-dependent effects in the soil such as consolidation (dissipation of excess pore pressure) and creep (continuously developing strain under constant effective stress). In sensitive clays, there is also evidence of degradation of the undrained shear strength, Thus, with the forthcoming changes in the traffic loads and environmental loads, there is an emerging need to be able to quantify how the stability of embankment on sensitive clays has changed, to know if there is a need for any remedial measures, or if the embankments are fine as they are.

This paper is a continuation of the work by Hernvall et al., (2021) which focused on how s_u has developed with time under the centre line of the embankment. In contrast to the previous work that was looking at the strength development under one location, the focus of this paper is on how the overall stability of the embankment has developed with time.

2 HAARAJOKI TEST EMBANKMENT

For this paper, again the Haarajoki test embankment been chosen as the case to analyse. The embankment namely was built to increase the knowledge about evaluating long term settlements and changes in s_u by the Finnish Road Administration (Vepsäläinen, 2002). Afterwards, it has also been used to for several studies to benchmark new constitutive models, e.g. Yildiz et al., (2009) and Amavasai et al., (2017). However, the focus of these studies was the Serviceability Limit State (SLS) behaviour of the embankment. As Haarajoki embankment has not been brought to failure, there is no data on the failure to benchmark the numerical results against.

The Haarajoki test embankment is 100m long, 20m wide at the base, 8m wide at the crest and 2.9m high. The embankment is built on a 20m deep deposit of soft and sensitive clay, with a 2m thick dry crust on top of it. The clay has a sensitivity of 25 at the top of the deposit and increases to 50 towards the bottom. The s_u

varied from around 15 kPa in the top to around to 25 kPa at 15 m depth. A 0.4 m thick working platform was placed first and then left for 20 days, after that the embankment was raised to its final height in 15 days. The working platform extends 5m outside the embankments toe acting like a small support berm. Half of the embankment was built on natural soil, while the other half was built on prefabricated vertical drains.

3 METHODLOGY

The half of Haarajoki test embankment built on natural soil was analysed with a Finite Element model. To model the behaviour of the soft and sensitive clay deposit, the advanced constitutive model Creep-SCLAY1S was used. Creep-SCLAY1S can model features of soft clays behaviour such as anisotropy, destructuration and rate-dependency, for more information see Sivasithamparam et al., (2015) and Gras et al., (2017, 2018).

The parameters were first extracted from the relevant soil tests, e.g. drained and undrained triaxial tests and oedometer tests (both incrementally loaded and CRS tests). In the next step the extracted parameters were validated and calibrated by performing element level simulation of the laboratory tests. The results from the parameter calibration determine the layering in the model. The parameters used are displayed in Appendix, Table A1. They are the same as in Hernvall et al., (2021), and have been kept consistent through the entire calculation, with only state parameters been automatically updated during the analyses. Since the properties of the dry crust are uncertain, it was modelled using a Mohr-Coulomb (MC) model for simplicity. A high stiffness was chosen, resulting in only minor deformations to be predicted in the dry curst. The MC model was also used to model the embankment itself. The properties for both the dry crust and the embankment are displayed in Table A2 in the Appendix.

Next, the embankment was simulated as a boundary value problem. The results of the model were benchmarked against available field data like settlements, pore pressure dissipation and horizontal displacements. In the final step, the embankment was brought to failure at different time steps, to assess how the stability of the embankment has changed.

In the model the embankment was brought to failure with a line load applied on top of it. The line load was increased in stress increments of 10 kPa/day. If the embankment failed under a certain load, the last load increment was repeated with 5 kPa to see if the embankment still failed. The highest load was taken as the failure load. To assess if the embankment had failed the following criteria was used:

- A clear shear band (deviatoric strain)
- High excess pore pressure along the shear band
- The horizontal displacement of the embankment's toe increases indefinitely
- A clear failure mechanism from the displacement vectors

The analysis was performed in the FE code Plaxis 2D Version 22.02.00.1078 (Brinkgreve et al., (2016)). The model consists of five layers for the soft clay, one for the dry crust and one for the embankment itself. The mesh consists of 3994 elements, with a finer distribution of the elements under the embankment. The model goes from -100m to +100 in the horizontal direction and from -20m to 0m in the vertical direction, with the embankment centre is coordinate (0,0). A picture of the mesh is displayed in Figure 1. Since the overall stability of the embankment is of interest, it is not possible to take advantage symmetry to save computational time and cost when performing the calculation. Hence, the entire embankment was modelled.

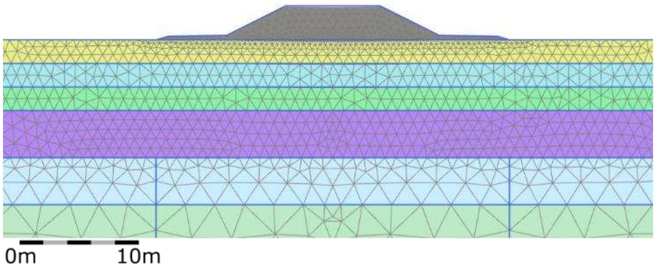

Figure 1. The mesh used for the calculation

4 RESULTS

The behaviour of the clay deposit was captured well by the model for the embankment as built. It was possible to predict the settlements under the centre of the embankment with good accuracy, as shown in Figure 2. The long-term prediction for dissipation of the excess pore pressure were also reasonable, as seen in Figure 3. The horizontal displacements at the embankments toe, shown in Figure 4, were however overestimated by the model. This might be due the lack of triaxial extension test available, which means that the anisotropy parameters of the model could not be calibrated properly. However, the position of the horizontal displacements in relation towards the depth matched the measurements well.

The bearing capacity of the embankment was evaluated at different stages. The time steps chosen for evaluation were directly after construction, one year, four years and 14 years after the construction, as well as after dissipation of 90% of the excess pore pressure which is equal to 104 years. Despite relatively large settlements, about 40 cm after four years, under the centre of the embankment, this did not translate to a large increase in overall bearing capacity. The soil was able to take the embankment load of about 60 kPa plus 35 kPa from line load directly after construction. This is in line with the bearing capacity calculated from the undrained shear strength (s_u) profile, which gives a bearing capacity about 87 kPa.

After one year of consolidation, the additional load did not increase the failure load and after three years it had only increased by 5 kPa to 40 kPa. This seem to be in line with Vepsäläinen et al. (2002), who only found minor changes in soil properties in the top of the deposit, when they investigated how the s_u had changed.

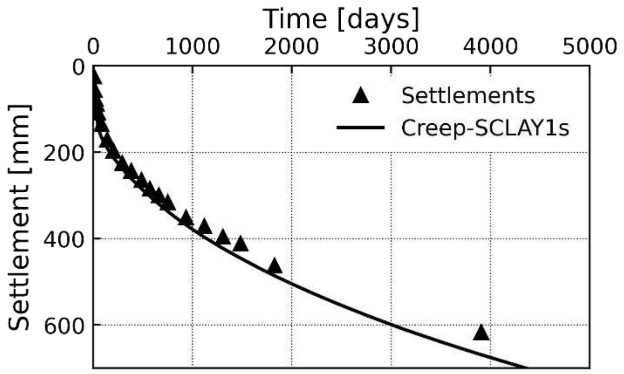

Figure 2. Settlement of under the centreline of Haarajoki embankment. Simulation and data from Hernvall et al., (2021)

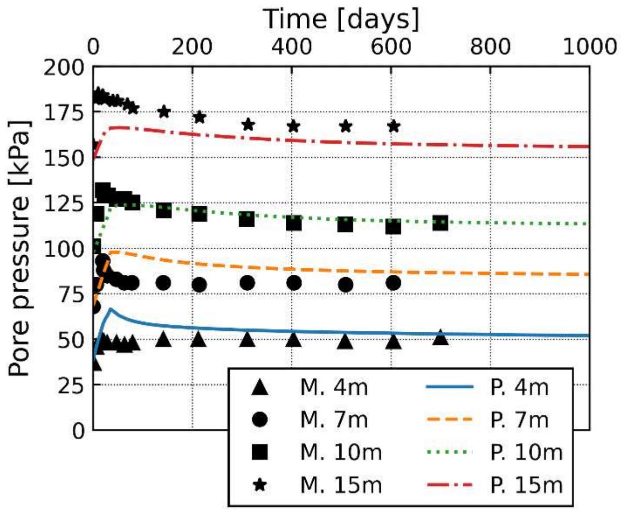

Figure 3. Dissipation of pore pressure at four depths under the embankment centre. M. is measurements, P. is prediction

After 14 years of consolidation the bearing capacity had only increased with 10 kPa to 45 kPa compared to the initial failure load. After 104 years the bearing capacity had increased to 70 kPa, but with that load the embankment was exposed significant functional damage, even if a clear failure surface had not yet been developed. The trend for the added load Q normalised by dividing with the initial failure load Q_{init} can be seen in Figure 5.

The failure mechanism for the different cases, shown with the total displacement vectors, can be seen from in Figure 6. For the failures: directly after construction, after one year and after four years there is a Prandtl shaped the failure mechanism, with an exaggeration towards one of the sides. Thus, the failure mechanism did not really change, most likely because only a minor increase in the bearing capacity was predicted. For the case after 14 years of consolidation, the failure mechanism was more of a global failure towards one side rather than a Prandtl failure mechanism, and the failure surface a bit deeper than in the previous cases. After 104 years, the embankment did not develop a similar global failure mechanism, but rather what seemed to be more of a punching failure in the soil.

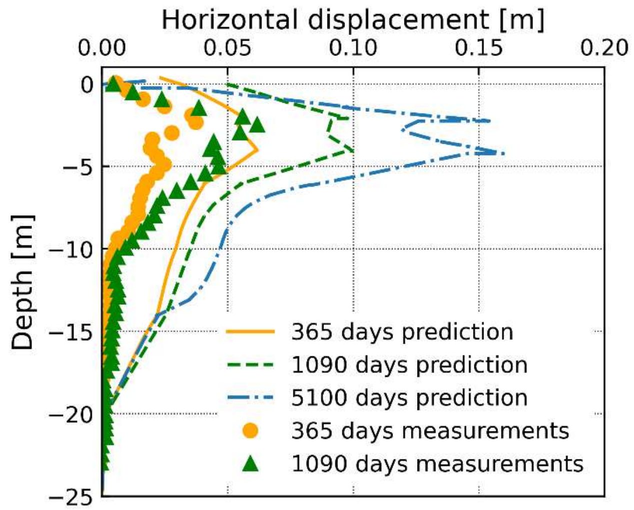

Figure 4. Horizontal displacements of the embankment toe

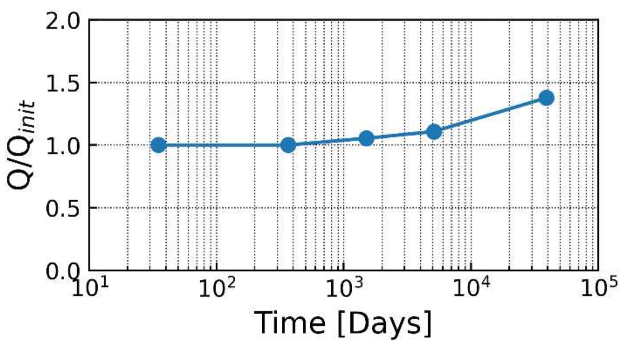

Figure 5. Development of the normalised load Q /Q_{init} over time

From the point of adding the last load increment of the line load until the failure occurred, it took several days. For the cases directly after construction, four years and 14 years, it took between three and seven days until the excess pore pressure had developed to a level to trigger the failure. This can be seen in Figure 7, which shows how the horizontal displacements at the embankment's toe develop versus time from when the line load is added on top of it. The case that is most noticeable here is for one year of consolidation, which took much longer to fail compared to the other analysed time steps. After applying the last load increment, it took almost 30 days more until the failure occurred. The horizontal displacements developed at a similar rate as the other cases initially, but after the last load increment was added the horizontal displacements continued to develop at a steady rate until the failure started. For all four cases the failure was triggered at similar level of the horizontal displacements, around 0.2 m this time.

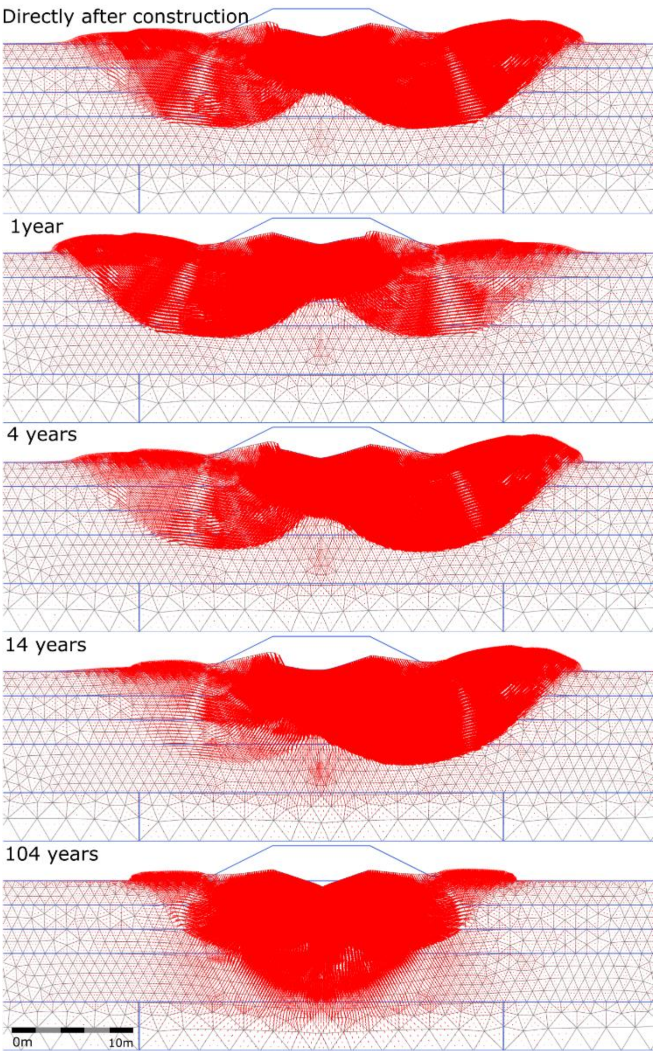

Figure 6. Total displacements vectors under the embankment at failure for the different time steps. Same scale for all time steps, scale in meter

During the calculation, the trigger level for failure from the excess pore pressure appeared to be about 50 kPa under the centre of the embankment. If the excess pore pressure continued to increase after that level, the embankment usually went to failure, if not it did remain stable for the current load. In the shear band that developed during the failure the excess pore pressure reaches over 100 kPa due to the collapse of the soil structure (destructuration), shown in Figure 8. In the case with 104 years of consolidation, the excess pore pressure reaches the same levels as the other cases, but without triggering a global failure. The higher level of excess pore pressure from six meters depth and downwards compared to the other cases might be because that the soil starts to collapse there. This could explain why this case developed a punching failure and not a general Prandtl type failure mechanism like the other cases.

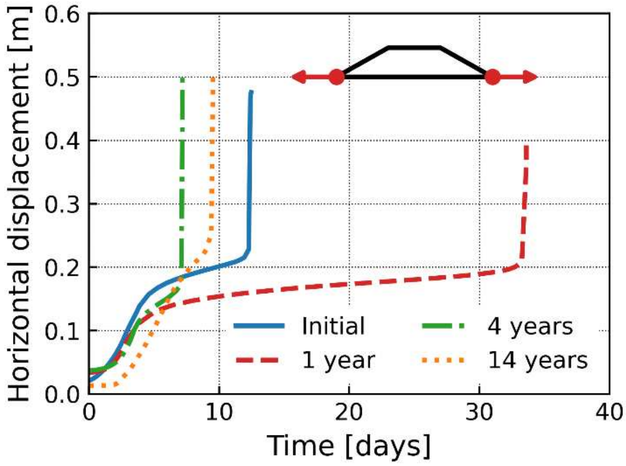

Figure 7. Horizontal displacement of the embankments toe at the different time steps

5 CONCLUSIONS

The Haarajoki test embankment stability has been analysed through numerical modelling using finite element and the advance constitutive model Creep-SCLAY1S. The model was able to predict the settlement and pore pressure dissipation well. With confidence that the model could capture the behaviour of the soft clay well, the bearing capacity of the soil developedover time was analysed.

The calculated initial bearing capacity from the model matched the bearing capacity calculated from the undrained shear strength (s_u) of the soil well. After one year of consolidation the bearing capacity of the soil remained the same, it was only after four years of consolidation that the model showed a modest increase of 5 kPa. After this the bearing capacity of the soil continue to increase, after 14 years it could take 10 kPa more and after 104 years it was 35 kPa more than the initial load of 95 kPa. This corresponds to an increase of 40%.

When the bearing capacity increases there is an evolution of the failure mechanism from a Prandtl failure mechanism to a more one side global failure and finally a punch like failure. This should be due to changes in strength profile under the embankment.

For the excess pore pressure and horizontal displacments there seem to be trigger levels for when the embankments are starting to apporach failure. Around 50 kPa for the excess pore pressure and between 0.15 to 0.2 meters for the horizontal displacements of the embankment's toe. This can be used to design counter measures if these levels are approached.

These findings should optimaly be benchmarked against data from a real failure of the embankment, which is unfortunely not avaible for this case. However, the results can be used to get an idea of what needs to be focused on when working with other embankments e.g., to extend their service life time.

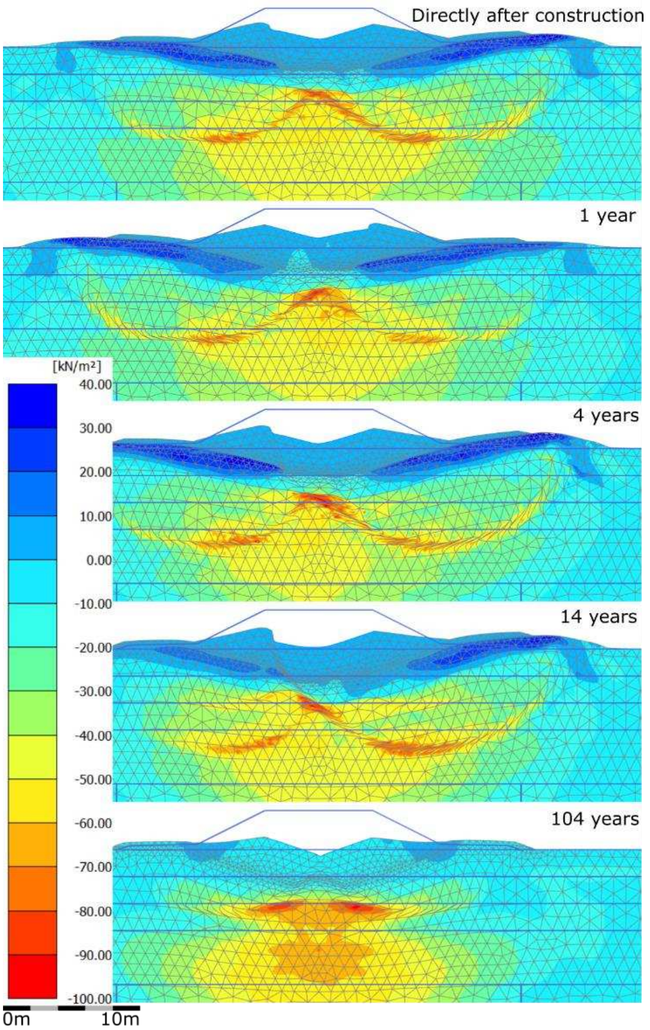

Figure 8. Excess pore pressure at failure for the different time steps

6 ACKNOWLEDGEMENTS

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8 APPENDIX

Table A1. Parameters used for the layers model with Creep-SCLAY1S

Parameters	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5
Depth (m)	2-4	4-6	6-10	10-14	14-20
Unit weight γ (kN/m^3)	13.8	14.2	15.0	15.0	16.0
Modified swelling index, κ*	0.0065	0.0088	0.0075	0.0100	0.0170
Poisson's ratio, v'	0.2	0.2	0.2	0.2	0.2
Intrinsic modified comp. index λ^*_i	0.108	0.100	0.084	0.084	0.087
Slope of CSL in compression, M_c	1.15	1.15	1.20	1.20	1.50
Slope of CSL in extension, M_e	0.83	0.83	0.86	0.86	1.00
Rate of rotational hardening, ω	24	35	20	20	20
Relative rate of rot. hardening, ω_d	0.144	0.622	0.600	0.600	0.600
Rate of destructuration, ξ	9	9	12	12	9
Relative rate of destructuration, ξ_d	0.20	0.22	0.25	0.25	0.25
Pre-overburden Pressure, *POP* (kPa)	50	40	35	35	25
Initial void ratio, e_0	3.30	3.00	2.40	2.40	1.96
Initial inclination of yield surface, α_0	0.43	0.45	0.46	0.46	0.46
Initial amount of bonding, χ_0	10	10	15	20	25
Reference time, τ (days)	1	1	1	1	1
Intrinsic modified creep index, μ^*_i	0.003	0.004	0.002	0.002	0.001
Earth pressure at rest at in normally consolidated range K_0^{NC}	0.51	0.51	0.51	0.51	0.76
Earth pressure at rest *in situ*, K_0	0.68	0.68	0.61	0.61	0.76
Permeability horizontal, k_h (m/day)	1.56E-04	1.56E-04	1.56E-04	1.30E-04	8.00E-04
Permeability vertical, k_v (m/day)	1.30E-04	1.30E-04	6.90E-05	6.50E-05	1.12E-04

Table A2. Parameters used for layers with Mohr-Coulomb

	Dry crust	Embank-ment
Depth (m)	0-2	-
Unit weight, γ (kN/m^2)	17	20
Young's modulus, E (kN/m^2)	4077	40000
Poisson's ratio ν	0.2	0.35
Cohesion intercept, c_{ref} (kN/m^2)	20	2
Friction angle, φ (°)	5	40
Earth pressure at rest *in situ*, K_0	0.9128	0.36
Permeability, *k* (m/day)	0.0013	1