Back analysis of Kivikko test embankment

Analyse du test de la Berge Kivikko

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ABSTRACT: Kivikko test embankment is one of the test sites of the European EuroSoilStab project, aimed to improve the design and construction methods for stabilization work. The embankment is constructed on mass and column stabilization. In this article some results of the back calculations of the Kivikko embankment are presented. Calculations with the finite element method were done using both axisymmetric and plane strain models. Calculations with the axisymmetric model corresponded well with observed behavior on the center line of the embankment. The shape of the cross sectional settlement profiles obtained from the plane strain model corresponded also well with the measurements. However, the time settlement prediction from the plane strain model was poorer than for the axisymmetric model.

RÉSUMÉ: Le test de la berge Kivikko est l´un des sites tests du projet européen EuroSoilStab, dont l'objectif est d'améliorer les méthodes de design et de construction des travaux de stabilisation. Dans cet article est présenté quelques résultats des calculs du test de la berge Kivikko. Les calculs réalisés, avec la méthode de l'élément fini, ont été effectués en utilisant à la fois des modèles asymétriques et des modèles de déformation de niveau. Les calculs avec les modèles asymétriques correspondent bien au comportement observé dans la ligne centrale de la berge. La forme de la coupe transversale des profils, obtenue du modèle de déformation de niveau, correspond également bien avec les mesures. Cependant, la prédiction du temps obtenue par le modèle de déformation de forme de niveau était un peu plus pauvre que celle obtenue avec le modèle l'asymétrique.

1 INTRODUCTION

Kivikko test embankment is one of the full-scale test sites of the EC financed EuroSoilStab project that started in 1997. This project has gathered members from six European countries to improve the design and construction methods for mass and column stabilization. The test embankment in Kivikko consists of three different sections where different kind of stabilization design are studied and evaluated based on quality control tests and monitoring results. In the article some results from back calculations of the test embankment are presented.

2 SOIL CONDITIONS

The test area at Kivikko is situated in wooded swamp area with an approximately 2 m thick peat layer on top. The water content of peat at depth 1...2 m is around 750 %. Under the peat there is a muddy clay layer with a water content of around 150 % under which a layer of silt can be found. Below the silt there are layers of muddy clay and clay. In a depth of 7-8 meters there is an about 1,5 meters thick very stiff silt layer where the columns were designed to be ended. Under this silt layer there is an other soft clay layer of about 4 m thickness. A more detailed description of the soil properties can be found in Ilander et al. (1999a).

3 TEST STRUCTURES

The test embankment consists of 3 different test sections, each of them 20 m long, and each of them constructed as a combination of mass and column stabilization, Figure 2. Mass stabilization has been executed with mass stabilizing equipment. The principle of the used mass stabilization method is presented in Jelisic & Leppänen (2000). The whole stabilized depth was about 7 m equaling mass stabilization to depth 2.0-2.5 m and column stabilization at depth 2.0-2.5...7.0 m. Different columns spacing (c/c) and design strength for the stabilization were studied at the different sections. In section 1 a column spacing of 2.1 m and a design shear strength of 240 kPa were used.

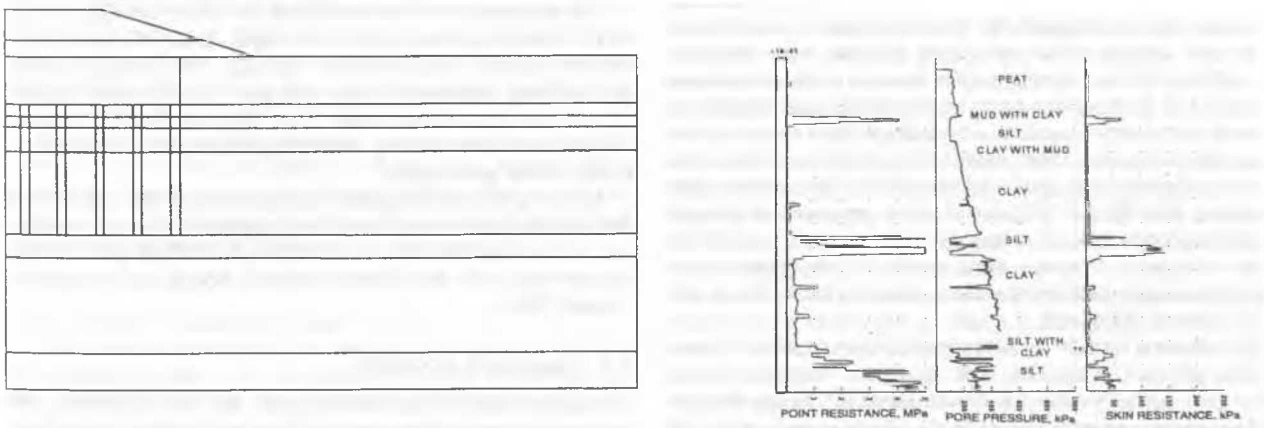

Figure 1. Principle cross section for the plane strain model and CPTU results from Kivikko.

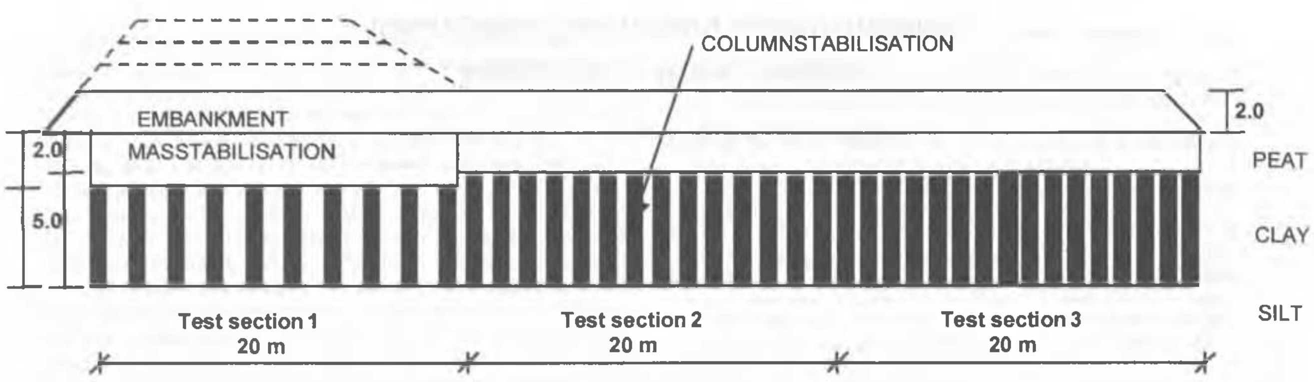

Figure 2. Longitudinal profile of the test structures at Kivikko.

In sections 2 and 3 the values were 1.45 m and 120 kPa contra 1.25 m and 90 kPa. The design strengths of the mass stabilization work were 80...100, 60 and 40 kPa corresponding to sections 1, 2 and 3.

A comprehensive quality control were conducted consisting of both field and laboratory tests. It revealed, that there were some inhomogenities in the stabilization work. A more detailed discussion on the quality control tests can be found in Ilander et al. (1999b). Design strength values were mostly achieved or exceeded except for section 1. Due to this the embankment height in section 1 was only raised to 3 m instead of the intended 5 m.

4 BACK ANALYSIS OF THE TEST STRUCTURES

4.1 *Geometry model*

An embankment founded on column stabilized soil is a truly three-dimensional problem. In the calculations made the geometry was modeled both in axisymmetric and plain strain conditions. An axisymmetric model represents fairly the situation on the centerline of the embankment. However, the boundary conditions given by constraints in horizontal directions are not representative when the factor of safety is low. This have an effect also to the stress distribution to lower layers, and the model does not apply for the outermost columns. With a plane strain model one may evaluate the total behavior of the embankment and estimate for example the horizontal displacements. However, as the columns are modeled as strips the arching effect and distributions of stresses in the subsoil may differ from reality. Therefore careful assessment have to be made when interpreting some of the results. It can though be considered that a fairly representative picture of the behavior can be modeled when applying both types of geometry models.

All sections of the test embankment were modeled with an axisymmetric model. In addition, Section 2 was also modeled in plane strain conditions. The transformation rules from actual geometry to the calculation model were based on keeping the improvement ratio unchanged. In the axisymmetric one column model true column radius were used and the outer boundary were adjusted to keep the right ratio between stabilized and unstabilized soil. In the plane strain model, where the columns are modeled as strips, the spacing of the strips were chosen as the actual column spacing. The width of the strips was adjusted to keep the improvement ratio unchanged. The calculations were performed with Plaxis 7.1 finite element program. In the axisymmetric models 15-node triangular elements were used, while 6-node triangular elements were applied in the plane strain model. The super mesh for the plane strain model of cross section 2 is shown in Figure 1.

The left-hand boundaries (symmetry line) were always closed for pore pressure dissipation. The right hand boundaries were also closed except for the silt layers. However, in the axisymmetric models an open drainage on the silt layer may cause too high drainage conditions in the middle of the embankment. To account for this the permeability of the silt layers were reduced in the axisymmetric models.

4.2 *Material modeling*

The subsoil consist of peat and very soft clay with some organic material. For this kind of soils creep plays a significant role in the long time settlement behavior. To account for this in the back-analysis the Soft Soil Creep model in Plaxis were applied for the soft clay layers (Plaxis 1998). For the columns the Hardening-Soil model was used. This model includes for example an initially high stiffness which decreases in deviatoric loading resulting in plastic strains. The embankment, mass stabilization and the silty soil were modeled with a linearly elastic perfectly plastic Mohr-Coulomb model.

The parameters for the models were evaluated based on both laboratory tests and soundings. Material parameters for the subsoil were determined based on CPT-soundings, oedometer tests, triaxial tests and classification tests. Some empirical relationships were also used for the validation of the parameters. According to Janbu (1998) the modulus number for a linearly stress dependent oedometer modulus can be evaluated from equation:

$$m = \frac{700\%}{w_n} \quad \pm 30\% \tag{1}$$

where w_n = natural water content. An empirical formula was also used to evaluate the creep properties of the clay layers. According to Länsivaara (1999) the relationship between the modulus number m and the creep number r_s, or the secondary compression index C_α and the compression index C_c is equal to the rate parameter B:

$$B = \frac{m}{r_s} = \frac{C_\alpha}{C_C} \tag{2}$$

In the evaluations values of B between B = 0.05...0.09 were used so that higher values were applied for softer soils with higher organic content.

The parameters for the stabilized soil were evaluated based on CPT soundings and triaxial test results on samples taken from the test site after a curing time of 180 days. The drained strength and stiffness parameters were evaluated directly from drained triaxial tests. In addition results from the extensive CPT sounding program were used by calibrating the results in comparison to the triaxial test results.

Based on the investigations performed both the subsoil and the stabilized soil were divided in subsequent layers in order to account for the variability in natural and stabilized soil. The parameter values for the different material models are presented in Viatek (2000).

4.3 *Calculation procedure*

The settlements of the embankment are still continuing. As shown in Figure 1 there is a soft clay layer below the silt layer where the columns end. This layer will continue to settle still for several years. High pore pressure has also still been measured in the clay between the columns. Because of the above a drained analysis do not tell much about the current situation in the embankment. It is also difficult to verify the results from such an analysis. It was therefore decided to perform consolidation analysis. The principle of the calculation procedure were the following:

0. Calculation of initial stresses using K_0-procedure.
1. Staged construction in undrained conditions for the stabilization work and preload embankment.
2. Consolidation analysis.
3. Staged construction in undrained conditions; increasing the height of the fill.
4. Consolidation analysis.

Steps 3 and 4 again if necessary.

Performing a consolidation analysis is always a difficult task. No matter how accurately the deformation properties may have been determined, the outcome of the time-settlement analysis is very much governed by the used permeability and boundary conditions. This is the largest uncertainty, as the permeability is seldom well known especially for stabilized soil.

In the SSC-model creep properties is given by a single parameter. However, it is well evidenced that soils creep much less in the overconsolidated range than in the normally consolidated range. Therefore a lower value for the creep parameter was used in the calculation stages 1 and 2, and it was increased to its assumed value in the normally consolidated range in stage 3.

4.4 *Calculation results*

In the following the results from the back analysis for the axisymmetric model for section 1 and plane strain model for section 2 are discussed.

4.4.1 *Section 1, axisymmetric model*

The measured and calculated time settlement behavior at ground surface and at the level where the columns end is presented in Figure 3 a). As can be seen the calculations provide a fairly reasonable estimation of the settlement behavior. It should be noted that measurement of the settlement from the ground surface was started not until June 1999. In Figure 3 only an estimation of the settlements before this time is presented based on measurements from the uppermost settlement plates in the subsoil. Some settlement have occurred even before these settlement plates were installed as a work embankment for the stabilization work was loading the subsoil several moths (from August 1998) before the installation of the settlement plates. This settlement/preload is not included in the measured settlement values, which are thus somewhat too low.

The calculated settlements are initially too high compared to the measured. This is at least partly due to the preload described above. The calculated settlement rates correspond though very well with the measured after the initial settlement has occurred and until the embankment height was increased to 3 meters. After this the measured settlements increase with a much higher rate than the calculated. This is probably due to horizontal displacements caused by a low factor of safety, which cannot be accounted for in an axisymmetric model.

Measured and calculated excess pore pressures at 21.9.99 between the columns are shown in Figure 3 b). Again a fairly reasonable correlation can be found. Probably the lower clay layer is estimated to be too thick, as the calculated excess pore pressures are there much higher than the measured.

4.4.2 *Section 2, plane strain model*

For the plane strain model the deformation behavior for the whole cross section could be studied including horizontal displacements at the toe of the embankment. Measured and calculated settlement profiles for the cross section are presented in Figure 4. The shape of the calculated profiles corresponds well

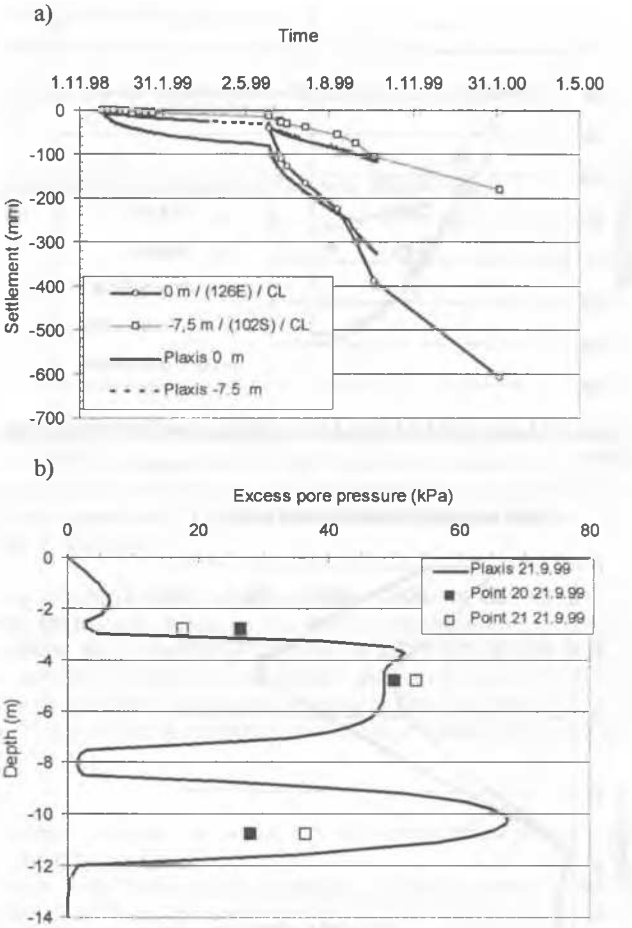

Figure 3. Measured and calculated time settlement behavior a) and excess pore pressures between the columns in Section 1 at 21.09.1999 b).

with the measured. However, the time settlement prediction is not that good.

The difference in time-settlement behavior between the two geometry models are probably due to different stress distribution and boundary conditions for pore pressure dissipation near the center line of the embankment. In the axisymmetric model the silt layers function more as drains as the right hand boundary is open for drainage. In the plane strain model the drainage effect of the silt layers were much lesser even though the used permeability's were higher. However, there is a horizontal drainage effect in the plane strain model also for the clay layers. It also has to be remembered, that the measured settlements do not contain the initial settlements from the preload embankment.

Total horizontal displacements for one cross section for the same time intervals are also shown in Figure 5. No inclinometers were installed into Section 2. The calculated horizontal displacements are probably too excessive. In Section 1 the maximum measured horizontal displacements until 20.9.1999 were less than 50 mm. The inclinometers were installed in 13.01.1999 so they do not include all developed displacements. However the horizontal displacements until that date for the 0.7 m high embankment are quite small. The maximum calculated horizontal displacements occurs at the soft muddy clay layer at a depth of 3...4 m. The maximum measured horizontal displacements in Section 1 occurred in a depth of about 4 m.

The development of arching at the top of the columns with time could well be evidenced by studying the principal stress directions at different time intervals, and by looking at the calculated effective vertical stresses at different locations in the columns According to the calculations the columns take more load from the soil as time proceeds. This is quite natural as the soft soil deforms with time and the stiff columns take more load. Similar

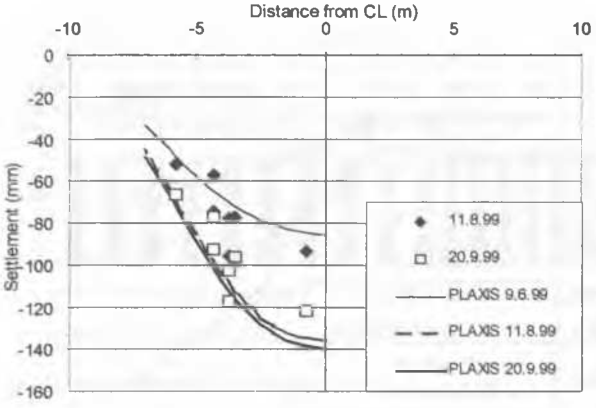

Figure 3. Measured and calculated time settlement profiles for cross section 2.

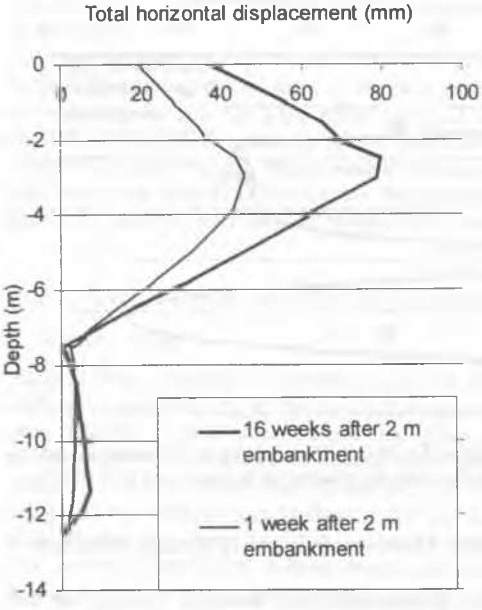

Figure 4. Calculated horizontal displacements at the toe of the embankment.

kind of behavior could be obtained from the earth pressure cells at test section 3. For the other test sections the earth pressure cells did not work properly.

5 CONCLUSIONS

In this article some of the results from back calculations of the Kivikko test embankment have been presented. Calculations were made in axisymmetric conditions for all three sections and in plane strain conditions for section 2. Consolidation analyses were performed to better be able to compare calculated and measured values. The outcome of consolidation analysis is much governed by the used permeability's and boundary conditions.

Calculations made with the axisymmetric models correspond reasonably well with measured time-settlement and excess pore pressure behavior on the centerline of the embankment. The shape of the cross sectional settlement profiles obtained from the plane strain calculations corresponded also well with the measured. However, the time settlement prediction of the plane strain model was poorer than what the axisymmetric model provided. This difference between the two geometry models is mostly caused by differences in excess pore pressure dissipation due to different boundary conditions. The horizontal displacements obtained from the plane strain calculation were probably too large. Maximum calculated horizontal displacement occurred in the same layer than what was observed in Section 1.

The follow up of the embankment will still be continued. This will give further confidence about the assumptions made in the modeling.

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