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Title:

Failure in anisotropic sensitive clays: a Finite Element study of the Perniö failure test

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Abstract

The railway network on coastal areas of Finland is predominantly located in soft clay areas. The undrained shear strength of such clays is generally low, highly anisotropic, rate dependent and it exhibits post-peak strain softening under undrained conditions. A full-scale failure test was performed by Tampere University of Technology in Perniö, Western Finland, in 2009. A shallow railway embankment built on a soft clay deposit was equipped with a loading structure and loaded to failure in about 30 hours. The embankment collapsed two hours after the last loading step.

In this study, data collected from the experiment is used, together with laboratory test results on high quality samples, to conduct advanced finite element analysis of the Perniö failure test.

The NGI-ADPSOFT model is used for this purpose, which is capable of simulating the strain-softening behavior of the clay.

Even though the observed rate effect is not taken into account in the analyses, the failure load can be predicted reasonably well. Good agreement is also observed for calculated displacements and failure mechanism with experimental observations.

Keywords

anisotropy; strain-softening; failure test; soft clays; FEM

INTRODUCTION

The design of embankments on soft soils remains nowadays still a challenge for geotechnical engineers. Soft clays are characterized by very low permeability and very low undrained shear strength (even less than 10 kPa near the ground surface). Undrained shear strength (s_u) is a rather complex parameter, although it is in practice often considered as a constant throughout a soil layer. However, it highly depends on direction (stress-induced anisotropy) and rate of loading, as well as the initial stress state. Furthermore, for some clays (e.g. sensitive clays), significant post peak strain-softening occurs.

In order to investigate how a very soft subsoil influences the stability of an embankment, a full-scale embankment failure test in Perniö, Western Finland, was commissioned in 2009 by the Finnish Railway Authorities and conducted by Tampere University of Technology. A shallow railway embankment was built on a 3.5-4.5 m thick deposit of soft sensitive clay overlain by a 0.8-0.9 m dry crust and an old sand fill layer, and brought to failure in less than two days. The embankment was equipped with loading structure and the load was increased by filling containers with sand. The embankment failed at a measured distributed vertical force of 217 kN/m, two hours after the last loading step (Lehtonen et al. 2015).

The extensive amount of data gathered from the experiment has been exploited to conduct advanced studies on the behavior of Finnish soft clays (Mansikkamäki et al. 2011; Länsivaara et al. 2011; Mansikkamäki and Länsivaara 2012; Länsivaara et al. 2014; D'Ignazio et al. 2014; D'Ignazio and Länsivaara 2015; Mansikkamäki 2015; D'Ignazio 2016).

The common practice in Finland is to perform total stress stability analyses using Limit Equilibrium Method (LEM), based on the isotropic s_u measured from the field vane (FV) test. However, features such as post-peak softening cannot be taken into account in LEM, even though it has a major impact on the actual soil resistance.

The aim of this paper is to back-analyze the Perniö full-scale test using the Finite Element Method (FEM). Data from triaxial compression (TxC), triaxial extension (TxE), FV and oedometer tests on Perniö sensitive clay is exploited to derive parameters for advanced FE analyses.

In order to model s_u anisotropy and strain-softening behavior of Perniö clay, the anisotropic total stress constitutive model NGI-ADPSOFT (Grimstad et al. 2010), developed by the Norwegian Geotechnical Institute (NGI) and implemented in PLAXIS 2D AE (PLAXIS 2012) finite element (FE) code, was adopted to model the soft clay layers.

The NGI-ADPSOFT model is based on the anisotropic “ADP” undrained shear strength concept (Bjerrum 1973). The s_u profiles for active (A), direct simple shear (D) and passive (P) loading (stress paths) are given as input data (Fig. 1).

First, in this study, background information on s_u anisotropy and strain-softening is given, followed by a description of the Perniö failure test and the test site. Then, the NGI-ADPSOFT model is presented in some detail. After that, the finite element model in PLAXIS and the material parameters are discussed. Finally, calculation results are compared with measurements from the full-scale experiment, with emphasis on the failure mechanism. Lastly, sensitivity analysis on the model parameters and effects of strain rate are considered.

BACKGROUND

Strain-softening

On coastal areas of Finland, the railway network is to a large extent located in soft clay areas. Finnish soft clays are mostly normally consolidated or slightly over consolidated and they can be often defined as “sensitive”. Sensitive clays are classified based on their sensitivity (S_t) value, defined as $S_t = s_u/s_u^{re}$, where s_u is the peak undrained shear strength and s_u^{re} is the remolded undrained shear strength.

According to SFS-EN ISO 14688-2 (2004), the sensitivity is low when $S_t < 8$, medium when $S_t = 8 - 30$ or high when $S_t > 30$.

Sensitive clays are characterized by an open soil structure with some bonds (e.g. Rosenqvist 1953, 1966). The natural water content of these clays can be considerably higher than the liquid limit (e.g. D'Ignazio et al. 2016). The maximum strength is maintained by the undisturbed soil until it is loaded undrained beyond the peak strength. Then, the structure collapses and the resistance

reduces dramatically. Such a phenomenon is called strain-softening: *“When subjected to a shear or a consolidation test, the clay shows a brittle behavior characterized by the fact that at a very small strain a critical stress level is reached, beyond which the magnitude and rate of deformations are large.”* (Bjerrum 1973).

Progressive failure mechanism and FE modeling

When a slope stability analysis is carried out without including anisotropy and/or strain-softening, one assumes that a given shear strength, independent of strain level and direction of loading, is mobilized along a potential failure surface. However, based on the findings of, e.g., Karlsrud et al. (1984), Bernander (2000) and Gylland (2012), this is an erroneous assumption for soft sensitive clays. Indeed, the mobilized shear strength along the potential failure surface is governed by the actual stress-strain curve of the soil. This would suggest the failure mechanism to be progressive in nature.

A simple but comprehensive description of the progressive failure mechanism is given by Skempton (1964): when in a clay mass the peak strength is passed at some particular point, the strength at that point will start decreasing and additional stress will be transmitted at some other point in the soil mass, causing the peak to be passed at that point too. As a direct consequence, a real chain reaction is established until a slip surface will develop further in the soil because of the strain-softening behavior. Eventually, when failure occurs, the shear strength will reach its residual value along the whole slip surface after sufficient deformation has occurred along the full length of the failure surface. Skempton (1964) also pointed out how failure may occur even before the residual strength is attained throughout the clay. However, once failure has started, the average s_u along the slip surface will fall towards its residual value.

In order to model and study a progressive type of failure it is necessary to take the strain-softening behavior into account. Accounting for strain-softening effects in stability analyses is a challenging issue. Over the last 15 years, FE analyses using advanced soil models capable of describing the undrained post-peak behavior of soft Norwegian clays at large strains have been proposed (e.g.,

Andresen et al. 2002; Jostad and Andresen 2004; Grimstad et al. 2010; Jostad and Grimstad 2011).

When peak strength is reached, local failure and strain localization will occur and shear bands will develop in the initially homogeneous soil. When the shear strains are localized, all the further deformation will tend to concentrate in these zones or bands (Desrues and Viggiani 2004).

Since the shear strain and thus the effect of undrained strain-softening increases with reducing shear band thickness, a regularization technique is needed so that both thickness and orientation of the shear band will not be dependent on the adopted mesh. In this way, the element size will not be acting as an internal length parameter and the thickness of the shear band will not reach its minimum size governed by the mesh (Bazant 1976; Pietruszczak and Mróz 1981). In addition, mesh refinement without a regularization technique will affect both peak capacity and global post-peak response (Gylland 2012). A detailed review of different regularization techniques is, for instance, given by De Borst et al. (1993).

Anisotropy

Another major factor, typical of soft sensitive clays, to be considered is the dependency of s_u on the stress path followed during undrained shearing, namely the stress-induced anisotropy. Sources of stress-strain-strength anisotropy can be found in stress-induced anisotropy as well as inherent (material) anisotropy (Bjerrum 1973; Hicher et al. 2000; Karstunen et al. 2005; Lämsivaara 1999; Koskinen 2014).

In undrained stability and bearing capacity problems, strength anisotropy is a factor which ought to be taken into account, since the distribution of s_u is known to vary along the slip surface according to the direction of the major principal stress (Bjerrum 1973).

For instance, in Finnish engineering practice, embankment stability analyses are commonly conducted as total stress analyses using the isotropic s_u determined from the FV test. However, s_u of Finnish clays is highly anisotropic (Karstunen et al. 2005; Koskinen 2014; Mansikkamäki 2015; Lehtonen 2015). In Norway and Sweden, s_u anisotropy is often accounted for in practice.

Undrained shear strength anisotropy can be reliably measured in the laboratory through triaxial (or plane strain) compression and extension tests together with direct simple shear (DSS) tests, as long as the specimens are consolidated anisotropically to the in situ stress level before shearing (Bjerrum and Landva 1966; Berre 1969; Bjerrum 1973).

When a soft clay specimen is sheared in compression (active, triaxial or plane strain) or in extension (passive, triaxial or plane strain), the behavior observed may be quite different, not only in terms of measured s_u (strength anisotropy), but also in terms of shear strains needed to mobilize peak strengths according to the major principal stress direction (strain anisotropy). In general, for normally consolidated to slightly overconsolidated soft clays, the shear strain needed to mobilize peak strength when the inclination of major principal stress is 0° from the vertical (active), is lower than the shear strain needed to mobilize peak strength when the inclination of major principal stress is 90° from the vertical (passive). The DSS peak strength is normally mobilized at a strain level between that of active and the passive, as suggested by the experimental observations reported by Karlsrud and Hernandez-Martinez (2013).

According to Bjerrum (1973), Ladd et al. (1977), Larsson (1980) and Jamiolkowski et al. (1985), the s_u anisotropy is highest when the clay has a low plasticity and it becomes more isotropic with increasing plasticity. This means that the anisotropy ratio s_u^P/s_u^A (see e.g. Grimstad et al. 2012), where s_u^P and s_u^A represent the peak s_u from triaxial or plane strain extension (superscript P for passive) and compression tests (superscript A for active), respectively, increases with increasing liquid limit (w_L) and plasticity index (PI) of the soil. This trend was also verified for the anisotropy ratio in DSS (s_u^{DSS}/s_u^A) (Bjerrum 1973; Ladd et al. 1977; Larsson 1980; Jamiolkowski et al. 1985).

However, such a trend might not be totally justified since, according to Mayne (1983) and Won (2013), it is based on limited test results and might have been strongly influenced by the definition of failure in extension tests (Won 2013). A definition of strength anisotropy dependent on PI should be carefully verified by thorough considerations on site characteristics, clay structure, mineralogy, soil history and spatial variability (Won 2013).

Thakur et al. (2014) collected data on anisotropic s_u of Norwegian clays. A positive trend between s_u^P/s_u^A and s_u^{DSS}/s_u^A and PI was observed.

Several clays from different parts of the world have been studied for research purposes and the anisotropy ratio has been characterized through numerous laboratory tests. In Table 1 values for s_u^{DSS}/s_u^A and s_u^P/s_u^A reported by some authors are summarized.

The dependency of s_u on the stress path followed during undrained loading is known to be a function of the shape and the inclination of the initial yield surface. The dependency on stress path is greater when the initial yield surface is rotated. A rotated yield surface indicates that an initial material anisotropy is present. According to Lämsivaara (1995, 1999), an approximation of the yield surface can be made if the critical state friction angle (ϕ') and the preconsolidation pressure (σ'_p) are known. The yield surface can be assumed to be ellipsoidal in shape; its size is determined by σ'_p .

PERNIÖ FAILURE TEST

Site description

The general stratigraphy of the test area is shown in the cross section provided in Fig. 2. As shown (from top down), an old 1.5 m thick fill made of sand and gravel lays on a 0.8-0.9 m thick weathered clay crust layer, followed by a 3.5 to 4.5 m thick soft clay layer. The underlying varved silty clay layer composed of thin clay, silt, and sandy silt layers extends for a thickness of approximately 1.5 m over a dense sand layer 2 to 8 m thick. The groundwater table was found at a depth of about 1.3 m below the ground surface.

The general characteristics of Perniö clay are presented in Fig. 3 and summarized in Table 2. Perniö clay is highly sensitive (average $S_t \approx 42$). From the Fall Cone (FC) test, s_u^{re} values lower than 0.5 kPa were observed. A clay content varying from 48 to 81% was measured in the soft clay. The natural water content (w) ranges from 48% to 109%, while the liquid limit (w_L) varies from 38 to 82%. Plastic limit (w_P) tests performed on samples collected slightly outside of the test area suggest that w_P of Perniö clay varies in the range 29-36%. Therefore, PI of the soft subsoil at the test site falls into the interval 36-49% (Lehtonen et al. 2015). The soft clay is slightly

overconsolidated, showing overconsolidation ratio (OCR) values generally lower than 1.5. The measured σ'_p values in Fig. 3 are based on constant rate of strain (CRS) oedometer tests. In the top portion of the clayey layer right below the dry crust (hereinafter called “stiff clay”), OCR reaches values higher than 2.

Field vane shear test shows s_u of 9 to 13 kPa near the top of the soft clay layer with an average strength increase with depth of 1.15 kPa/m. Therefore, according to Leroueil et al. (1990), the Perniö clay can be classified as a “very soft” clay ($s_u = 10\text{--}20$ kPa).

At the elevation below about 3 m the scatter of σ'_p is higher than in the top part (Fig. 3), where the measured points are grouped fairly well. According to Fig. 3, S_t in this layer may be higher than in the upper layer, even though this is suggested by only one data point.

A shallow railway embankment constituted of crushed ballast, 0.55 m thick, 3.2 m wide (top) and about 60 m long was built for this test on top of the existing sand fill layer. A ditch was excavated at one side in order to control the direction and the extent of failure.

The embankment was equipped with a loading structure and four containers were placed and filled with sand by means of a conveyor belt. The conveyor belt was placed at safe distance so that there would not have been any interaction with the containers in case of failure. The continuous filling process was designed so that the load was distributed homogeneously in all the containers. The sand fill was continuously delivered by the conveyor belt to containers one to four and vice versa.

The embankment behavior was expected to be mostly governed by the soft clay, which was highly instrumented and monitored during the whole duration of the test and where the majority of the available and exploitable measurements come from.

The extensive instrumentation (Lehtonen 2011; Lehtonen et al. 2015) consisted of: 37 pore pressure gauges positioned in four groups: under the embankment centerline, under the embankment toe, halfway between the embankment and the ditch and next to the ditch; 27 surveying prisms on the soil surface and the loading containers; 9 automatic two axis inclinometers, installed in three cross sections; three settlement tubes with a total of 53 pressure transducers; six total stress earth pressure gauges under the embankment; 76 slip surface

measurement tubes with surveying points on the surface; 32 strain gauges on the frameworks for weighing the containers. The instrumentation layout is shown in Fig. 4.

The loading process lasted about 30 hours, when the measured distributed vertical force reached about 217 kN/m for cars 2 and 3. Given that the loading area was 2.5 m wide, a bearing pressure of about 87 kPa was measured at the end of loading. The embankment failed two hours after the last loading step. For a detailed description of the experiment, see Lehtonen et al. (2015).

The Perniö failure test represents a case of a quickly loaded embankment if compared to other failure tests e.g. the Saint Alban test embankment (La Rochelle et al. 1974), which was brought to failure in about ten days.

Failure mechanism and observations

According to Lehtonen et al. (2015) the best estimate of the shape of the failure zone comes from the pre-failure pore pressure readings, coupled with inclinometer readings. In general, a distinct slip surface was difficult to detect. A shear zone seems to develop starting from behind the railway sleepers, reaching its lowest level in the soft clay layer under the embankment toe, before gently bending up and emerging at the bottom of the ditch. Based on the interpreted failure mechanism, shear seems to occur differently over three distinct zones along the shear band (Fig. 5): an active zone underneath the area of application of the load, a direct shear zone beneath the embankment toe and a passive zone moving upwards and outwards toward the excavated ditch.

Perniö full scale test was characterized by an undrained creep type of failure (Lehtonen et al. 2015), since rupture has occurred under the applied maximum load only after a certain time had elapsed from the last loading step. During that time (approximately two hours), both excess pore pressures and rate of excess pore pressures have increased until failure occurred (Fig. 6). Probably, the soil was initially subjected to a local failure below the embankment which developed further over time. Failure seemed to have occurred under undrained condition, as no dissipation of pore pressure could be observed (Fig. 6).

Studies carried out by Arulandan et al. (1971) and Holzer et al. (1973) have shown that a threshold level of deviator stress exists beyond which undrained creep will lead to an increase in rate of strain and pore pressure until rupture. Time to failure is inversely proportional to the ratio of stress level reached to that of the threshold deviator stress.

When a soft clay is subjected to a “fast” undrained loading, the stress-strain response will be stiffer than in a “slow” type of loading (Berre and Bjerrum 1973; Lefebvre and Leboeuf 1987; Graham et al. 1983a; Leroueil et al. 1985). This concept is shown in Fig. 7. The effective stress path A-B may ideally describe a “standard” or “slow” test, while the stress path A-C-E would be representative of a rapidly loaded sample. What probably happened in the field test, could be described by the stress path A-C-D: the soil was quickly loaded up to a deviator stress level beyond the threshold, which did not cause failure straight away, but only after a certain elapsed time because of undrained creep, under a given constant load. In addition, if the loading process had continued further, the soil may have failed at a higher load than 87 kPa (point E).

The evolution of shear strain versus time and applied load (q) is shown in Fig. 8. Measured horizontal displacements from inclinometers L2P1, L2P2 and L2P3 were used to derive the magnitude of shear strain during the test, according to eq. (1):

$$(1) \quad \gamma = \frac{\Delta u_x}{\Delta z}$$

where γ is the shear strain, Δu_x is the incremental horizontal displacement measured by the inclinometer between two consecutive measurement points and Δz is the vertical distance between the two points.

The evolution of shear strain with time at points A, B, C (located as shown in Fig. 5) from inclinometers L2P1, L2P2, L2P3, respectively, is shown in Fig. 8.

During the first day of loading, when the load was lower than 24 kPa, only very small deformations could be observed. A load of 24 kPa was maintained constant during the night, over a period of time of about 14 hours. However, when the load-to-failure process started after the night, both strains and rate of strain increased until the end of loading, and later increase until failure, within

the time-frame when the applied load was kept constant and equal to 87 kPa. The evolution of shear strain with time seems to change about 3-4 hours before failure (inclinometer L2P1), simultaneously with yielding of the soft clay. A similar yield point was identified by Lehtonen et al. (2015) from the pore pressure readings (Fig. 6). Fig. 8 would suggest that during the experiment, the shear strain rate increased by over a tenfold, thus justifying the rather high failure load achieved.

For a given time step, different strain levels mobilized at different locations where the inclinometers were placed, would suggest a progressive type of mechanism leading to failure (Fig. 8).

CONSTITUTIVE MODELS

The NGI-ADP soil model (Grimstad et al. 2012) is a total stress elasto-plastic model based on an anisotropic Tresca failure criterion. The anisotropic behavior follows the ADP framework by Bjerrum (1973), where the s_u profiles for active (A), direct simple shear (D) and passive (P) stress paths are given directly as input parameters. The non-linear hardening curves are described by input of the peak s_u and corresponding shear strains in the three directions of shearing represented by TXC, DSS and TXE, as shown in Fig. 9. A special version of the model, the NGI-ADPSoft (Grimstad et al. 2010), accounts for the anisotropic post-peak strain softening behavior, often observed during undrained tests on soft sensitive clays. The softening curves are described by input of the residual s_u and corresponding shear strains in the same three directions of shearing, as shown in Fig. 10. By interpolation between the three input curves, the model is able to model the anisotropic behavior of the clay for a general 3D stress state. The model chooses the stress-path to follow based on the orientation (θ) of the major principal stress from the vertical. For $\theta = 0^\circ$, the model assumes pure compression, while for $\theta = 90^\circ$, it assumes pure extension.

The undrained triaxial tests used to establish the input parameters, should be consolidated anisotropically to the assumed in situ effective stresses (CK₀UC and CK₀UE tests). The triaxial curves in Fig. 10 therefore start from the initial shear stress τ_0 . The initial inclination of the curves

comes from the initial shear modulus G_0 , which also defines the magnitude of the elastic strain components.

The model assumes that s_u varies linearly with depth within a soil layer. A constant $s_{u \text{ ref}}^A$ at a reference depth y_{ref} , together with a strength increase parameters $s_{u \text{ inc}}^A$, define the s_u profile in the soil layer. Furthermore, the reference depth y_{ref} may be given by a reference point x_{ref} and an inclination $\Delta y_{\text{ref}}/\Delta x$, for inclined clay layers.

In this study, for the sake of simplicity and because of lack of plane strain tests, the plane strain “A=active” behavior is assumed to be equal to the triaxial compression behavior, and correspondingly, the plane strain “P=passive” behavior is assumed to be equal to the triaxial extension behavior, as also suggested by e.g. Jostad et al. (2014).

The strain-softening response requires a regularization method so that numerical problems and mesh dependency are avoided. Therefore, the NGI-ADPSOFT model includes the “over non-local strain” formulation proposed by Brinkgreve (1994) that is based on the “non-local strain” approach formulated by Eringen (1981). This method introduces an internal length scale to overcome mesh dependency. The “over non-local” strain is calculated as an integrated average shear strain within a specified zone around the actual Gauss point, governed by the input of the internal length scale (l_{int}). Then, as long as l_{int} is selected sufficiently large compared to the element size, the shear band will be larger or equal to the element size, thus avoiding the shear band thickness to be affected by the orientation and size of the elements. The thickness of the shear band is controlled by l_{int} together with a parameter α . The thickness of the shear band (t_{sb}) is then calculated from eq. (2) (Brinkgreve 1994):

$$(2) \quad t_{\text{sb}} = \pi \cdot l_{\text{int}} \cdot \left[\ln \left(\frac{\alpha}{\alpha - 1} \right) \right]^{\frac{1}{2}}$$

According to Vermeer and Brinkgreve (1994) and Jostad and Grimstad (2011), $\alpha = 2.00$ represents a recommended value for the numerical integration of the non-local strain. The effective shear band thickness t_{sb} is then 2.62 times l_{int} .

The main advantage of using the over non-local regularization technique is that the implementation into a non-linear finite element program can be easily done, since no reformulation of the governing finite element equations is needed. This is generally required for the other types of regularization techniques. The regularization based on the non-local strain approach is fully solved at the material point level, with only additional information on the strains in the neighboring elements. Therefore, the regularization can be accounted for by a user-defined material model implemented in a commercial finite element program (Grimstad et al. 2010).

Formulation of NGI-ADPSOFT soil model

The formulation of the NGI-ADPSOFT soil model is based on an anisotropic approximated Tresca yield criterion (Grimstad et al. 2012), where the yield function F is defined by eq. (3):

$$(3) \quad F = \sqrt{H(\omega)\hat{J}_2} - \kappa_1(1 - \kappa_2) \frac{s_u^A + s_u^P}{2} - \kappa_2 \frac{s_{ur}^A + s_{ur}^P}{2} = 0$$

Where $\hat{J}_2$ is a modified second deviatoric invariant (Grimstad et al. 2012), the function $H(\omega)$ (eq. (4)) approximates the Tresca criterion and the hardening parameters κ_1 and κ_2 are computed from eq. (5) and eq. (6).

$$(4) \quad H(\omega) = \cos^2 \left[\frac{1}{6} \arccos(1 - 2a_1\omega) \right]$$

Where ω is a function of $\hat{J}_2$ and the modified third deviatoric invariant $\hat{J}_3$ (Grimstad et al. 2012), a_1 is the rounding ratio (Grimstad et al. 2012), which is set equal to 0.99 (default value).

$$(5) \quad \kappa_1 = 2 \frac{\sqrt{\gamma^p / \gamma_p^p}}{1 + \gamma^p / \gamma_p^p}, \quad \text{for } \gamma^p < \gamma_p^p, \kappa_1 = 1 \text{ for } \gamma^p \geq \gamma_p^p$$

$$(6) \quad \kappa_2 = \left(\frac{\gamma^p - \gamma_p^p}{\gamma_r^p - \gamma_p^p} \right)^{c_1} \left(2 - \frac{\gamma^p - \gamma_p^p}{\gamma_r^p - \gamma_p^p} \right)^{c_2} \quad \text{for } \gamma_p^p < \gamma^p < \gamma_r^p, \kappa_2 = 0 \text{ for } \gamma^p \leq \gamma_p^p, \text{ else } \kappa_2 = 1$$

The second deviatoric invariant $\hat{J}_2$ is based on a modified stress vector (Grimstad et al. 2010) that accounts for the anisotropy in strength and the initial stress conditions (τ_0). The strains γ^p , γ_p^p and γ_r^p are the plastic shear strain, plastic “peak” shear strain and plastic “residual” shear strain. The softening parameter κ_2 (eq. (6)), in this newest version of the model (NGI 2011), is controlled by two curve fitting parameters c_1 and c_2 . For $c_1 = 1$ and $c_2 = 0$, the softening curve will be linear from peak to residual shear strain (Fig. 11).

The parameters γ_p^p and γ_r^p are stress path dependent in a similar way as s_u . The interpolation function for γ_p^p and γ_r^p is described in detail in Grimstad et al. (2010) and in NGI (2011). The input parameters for the NGI-ADPSOFT model are summarized and briefly described in Table 4.

MODEL PARAMETERS FOR FINITE ELEMENT ANALYSES

Soft clay

The determination of the model parameters for NGI-ADPSOFT model was carried out by analyzing the results of anisotropically consolidated TXC and TXE tests on Perniö clay together with in situ tests. Measured data from 13 triaxial tests on 86 mm diameter tube samples and 32 FV tests were exploited to estimate the soil parameters to be used in the FE calculation. Tube samples for the triaxial tests were taken in 2013 from an area located about 50 m off the test area, with the purpose of evaluating the s_u anisotropy of Perniö clay and the determination of s_u at different OCR values (Lehtonen 2015).

Undrained DSS tests were not available for Perniö clay, since DSS tests are not used in Finland. According to Jamiolkowski et al. (1985) and Chandler (1988) the DSS strength is to some extent comparable with the FV shear strength. It is known how FV test is affected by uncertainty, especially in soft soil deposits when performed without casing (Ortigao and Collett 1988; Mansikkamäki 2015; Selänpää et al. 2017 In Press). Nevertheless, FV data from the initial soil investigation at Perniö site were exploited to verify the DSS strength of Perniö clay predicted by some existing correlations.

Characteristics of Perniö clay are summarized in Tab 2. 29 constant rate of strain (CRS) oedometer tests on tube samples were performed in order to evaluate σ'_p of Perniö clay. The preconsolidation pressure σ'_p is identified according to Kolisoja et al. (1989). The CRS tests were conducted at standard constant speed equal to 0.0015 mm/min (0.6%/h) and no correction was applied to σ'_p for strain rate. According to the oedometer test results, σ'_p in the soft clay (Fig. 3) decreases with depth for about 1 m below the dry crust before increasing with depth. The preconsolidation pressure σ'_p was found to be higher than the in situ stress, typically by about 18 kPa, on average. The variation of σ'_p with depth would suggest that also s_u is expected to decrease until about $z = 5$ m and then increase linearly with depth. The difference between σ'_p and σ'_{v0} is assumed not to be varying towards the ditch, as the thickness of the old fill layer was constant throughout the whole area before the ditch was excavated.

The measured peak s_u from TXC and TXE normalized with respect to the vertical effective stress (σ'_v) are compared to existing correlations for anisotropic s_u of Scandinavian clays (Larsson et al. 2007; Karlsrud and Hernandez-Martinez 2013).

The correlations studied are all based on the SHANSEP framework (Ladd and Foott 1974, eq. (7)) and constructed from the results of CK_0UC , CK_0UE and CRS oedometer tests.

$$(7) \quad \frac{s_u}{\sigma'_v} = S(OCR^m)$$

The parameter “S” represents the undrained strength ratio (s_u/σ'_v) for the normally consolidated state and “m” is a material dependent exponent.

According to Karlsrud and Hernandez-Martinez (2013), the SHANSEP parameters “S” and “m” correlate with the natural water content of the clay (w). The fitted average anisotropic s_u based on a large number of tests on block samples from different sites in Norway can be described as follows, from eq. (8), (9) and (10):

$$(8) \quad \frac{s_u}{\sigma'_v} = (0.27 + 0.1w)OCR^{(0.58+0.33w)}$$

$$(9) \quad \frac{s_u^{DSS}}{\sigma'_v} = (0.14 + 0.18w)OCR^{(0.35+0.77w)}$$

$$(10) \quad \frac{s_u^P}{\sigma'_v} = (0.26w)OCR^{(0.86+0.30w)}$$

where s_u^A , s_u^{DSS} and s_u^P are the peak shear strengths from TXC, DSS and TXE tests, respectively.

Larsson et al. (2007) suggested correlations for anisotropic s_u of soft Scandinavian clays as a function of w_L of the soil.

Based on eq. (7), Larsson et al. (2007) proposed expressions to derive “S” from curve fitting of test results on Scandinavian inorganic clays, while “m” is maintained constant and equal to 0.8, which is consistent with the studies done by Jamiolkowski et al. (1985) and Ladd (1991).

For triaxial compression, “S” seems independent of w_L , and it was observed to be equal to 0.33 (Larsson et al. 2007). Therefore, the anisotropic s_u can be expressed as in eq. (11), (12) and (13):

$$(11) \quad \frac{s_u^A}{\sigma'_v} = 0.33OCR^{0.8}$$

$$(12) \quad \frac{s_u^{DSS}}{\sigma'_v} = (0.125 + 0.205w_L / 1.17)OCR^{0.8}$$

$$(13) \quad \frac{s_u^P}{\sigma'_v} = (0.055 + 0.275w_L / 1.17)OCR^{0.8}$$

The calculated average w_L of the soft clay in Perniö is 57% (Lehtonen et al. 2015).

As clearly shown by the laboratory test results (Fig. 12), the variation of s_u/σ'_v as a function of the OCR can be fairly well described by the transformation model of eq. (8), using $w=90\%$, for TXC and by the transformation model of eq. (13) for TXE, using $w_L = 57\%$.

For $w = 90\%$, the correlations proposed by Karlsrud and Hernandez-Martinez (2013) can describe quite well s_u^A from Lehtonen (2015), while they clearly overestimate s_u^P from Lehtonen (2015).

The framework proposed by Larsson et al. (2007) (eq. (11), (12), (13)) seems to correlate relatively well with the measurements, although it underpredicts some of the TXC results.

The estimated anisotropy ratio s_u^P/s_u^A is about 0.5, based on eqs. (8), (10), (11) and (13).

The DSS strength was derived by comparing FV measurements with existing correlations (eq. (9), (12), (14), (15)). Jamiolkowski et al. (1985) proposed, for the in situ mobilized undrained shear strength [$s_u(\text{mob})$], roughly corresponding to DSS conditions (eq. (13)):

$$(14) \quad \frac{s_u(\text{mob})}{\sigma'_v} = 0.23OCR^{0.8}$$

Ching and Phoon (2012) proposed a framework for peak s_u of structured clays from FV and unconfined compression (UC) tests, where $s_u(\text{mob})$ is derived based on OCR and S_t (eq. (14)):

$$(15) \quad \frac{s_u(\text{mob})}{\sigma'_v} = 0.229OCR^{0.820}S_t^{0.121}$$

The measured uncorrected FV strength in Perniö varies between 9-13 kPa on top of the soft layer, increasing with depth about 1.15 kPa/m. It should be pointed out though that the FV test is performed rather quickly, at a rate of strain that is quite high ($\sim 60\%h^{-1}$, Ching et al. 2013) compared with conventional DSS tests ($\sim 1\%h^{-1}$). For this reason, measured values must be corrected in order to account for rate effect (e.g. Bjerrum 1972) based on PI of the clay. Field vane data points are corrected for plasticity as suggested by the Finnish guidelines for stability analysis (Ratahallintokeskus 2005). The average correction factor applied to the FV strength data points is equal to 0.95.

In Fig. 13 the normalized corrected FV strength for Perniö is plotted against the OCR. From Fig. 13 it is clear that the measured data disagree with the correlations proposed by Ching and Phoon (2012) and Karlsrud and Hernandez-Martinez (2013). Even though the measurements show evident scatter, the correlations proposed by Jamiolkowski et al. (1985) and Larsson et al. (2007) seem the most representative of the trend observable from the measured strength points. The trendlines suggested by eq. (12), for $w_L=57\%$, and eq. (14) seem to reasonably adapt to the data points for $OCR < 2$.

Finally, s_u^{DSS}/s_u^A is assessed to be approximately equal to 0.65, based on eq. (8) and eq. (12). The assumption of an average s_u^{DSS}/s_u^A of about 0.65 is consistent with that reported by Jamiolkowski et al. (1985), Karlsrud et al. (2005), Karlsrud and Hernandez-Martinez (2013) and Thakur et al. (2014).

Fig. 14 shows the fit of NGI-ADPSOFT model to CAUC and CAUE tests on Perniö clay reported by Lehtonen (2015). Shear strain at peak is assumed for s_u^{DSS} by selecting a value contained between the peak strain for compression and the peak strain for extension, according to the observations reported by Karlsrud and Hernandez-Martinez (2013) for Norwegian clays. Strain-softening is clearly visible for compression behavior and occurs after a peak shear strain of about 2% (brittle behavior), while for extension no clear peak is reached before 8-10% of strain. A residual strength level of about 50% of the peak in compression is reached at a shear strain level of 25%. The more sensitive layer is modelled simply assuming a lower strength at large strain (about 40% of the peak at 25% of shear strain), as shown in Fig. 15.

The NGI-ADPSOFT model is able to reproduce rather well both CAUC and CAUE tests. According to the authors' experience, for Finnish sensitive clays the 86 mm sampler used tends to assure a better sample quality than the 54 mm piston sampler. However, the tested specimens may have been subjected to some disturbance effects caused by sampling, transportation and trimming procedures. Sample disturbance can notably affect the stress-strain response of soft clays. According to Lunne et al. (1997), sample disturbance provokes a reduction in the peak strength, an increase in the shear strain at failure and a higher shear stress at large strain.

Input parameters for NGI-ADPSOFT model are shown in Table 4. Values for G_0/s_u^A are chosen according to Karlsrud and Hernandez-Martinez (2013). Reasonable soil parameters are chosen for the varved silty clay, based on the authors' experience, by assuming the same anisotropic behavior of the soft Perniö clay. For the clay layers below the embankment, y_{ref} varies according to a gradient ($\Delta y_{ref}/\Delta x$), in order to keep $s_u = s_{u,ref}^A$ at any point of the top part of the layers. The lateral stress coefficient (K_0) is calculated according to Kulhawy and Mayne (1990) [eq. (16)].

$$(16) \quad K_0 = (1 - \sin \phi') OCR^{\sin \phi'}$$

Dry crust

Clay deposits often contain a surface crust of weathered and fissured clay. Usually, the properties of these clays are affected because of phenomena of oxidation, desiccation, freezing-thawing or fluctuation of water table.

Part of the slip surface generally goes through the dry crust. Therefore, for an accurate prediction of the failure load it would seem important, in the authors' opinion, to reliably estimate the potential available strength in this layer. However, it is quite challenging to take undisturbed samples of the dry crust because of its non-homogeneous structure, often characterized by the presence of cracks (D'Ignazio 2016).

The FV test shows uncorrected s_u values well over 50 kPa (Lehtonen 2011, Fig. 3). It has been shown by some authors (Lo 1970, La Rochelle et al. 1974, Lefebvre et al. 1987, Khan 1993, D'Ignazio et al. 2015; D'Ignazio 2016) how the FV tends to overestimate s_u when the clay presents fissures. The presence of fissures in the layer will cause the strength measured on a small scale (field vane) to be larger than on a large scale (e.g. a failure surface) (La Rochelle et al. 1974).

14 isotropically consolidated triaxial compression (CIUC) unsaturated tests were done on 15 cm size block samples of Perniö dry crust, taken about 50 m off the test location. An extensive description of the tests performed to characterize the Perniö dry crust is presented by D'Ignazio (2016). The average measured TXC undrained strength at the top was 35 kPa decreasing with depth towards the soft clay, which is lower than the strength measured by the FV ($s_u > 50$ kPa).

As discussed by D'Ignazio et al. (2015) and D'Ignazio (2016), the dry crust layer in Perniö was originally about 1.5 m thick, while in the experiment and, therefore, in the FE model presented in this paper, it was roughly 1 m thick. A reason might be that the upper part of the clay crust was removed before the construction of the sand fill layer, several years before the experiment took place. Characteristics of Perniö dry crust are reported in Tab. 3. From the test results, the upper part of the layer (above 1.0 m) seems partially saturated, while the lower part is fully saturated. This might be the result of the seasonal fluctuations of the groundwater table. However, in this

study the clay crust is modelled with a total stress soil model (NGI-ADP); therefore, partial saturation will not constitute an issue, as suction is implicitly included in the s_u .

As discussed by D'Ignazio et al. (2015), the adoption of different soil models for the dry crust does not influence the global response of the FE analysis of Perniö failure test, as it seems that the ground response was mainly governed by the soft clay layers.

Input parameters for the clay units used in the model are reported in Tab. 4 and the undrained shear strength profile for Perniö is shown in Fig. 16.

Coarse layers

The embankment made of crushed ballast, the sand fill and the lower sandy layer are modelled as hardening isotropic materials using the Hardening Soil model (Schanz et al. 1999) implemented in PLAXIS.

Stiffness and strength parameters have been estimated according to the Finnish Road Design Manual (Tiehallinto 2001) and to Mansikkamäki (2015). $E_{\text{oed}}^{\text{ref}}$ was chosen equal to E_{50}^{ref} , while $E_{\text{ur}}^{\text{ref}} = 3 \cdot E_{50}^{\text{ref}}$, as suggested by the user's manual of Plaxis (Plaxis 2012). Effective friction angle (ϕ') and dilatancy angle (ψ) are chosen according to Mansikkamäki (2015). Input parameters for the Hardening Soil model are reported in Table 5.

FINITE ELEMENT ANALYSIS

The A-A (middle) cross section (Fig. 2) was chosen for the FE 2D analysis, being probably the most representative of plane-strain conditions.

The finite element mesh for the plane-strain analysis of the A-A cross section consisted of 3432 15-noded triangular elements with average element size of 0.32 m and it is presented in Fig. 17.

The global coarseness was set to "very fine", with cluster refinement in the soft clay layers.

The boundary conditions applied to the FE model are such that the bottom boundary was fully fixed, so that no movements were allowed both in the horizontal and the vertical direction. Roller

conditions were applied at the two vertical sides, meaning that movements in the horizontal direction were not permitted.

The adopted construction sequence was the same as that on site, assuming fully undrained conditions. Firstly, initial stress conditions were generated through a “ K_0 ” procedure, assuming the layers as horizontal, even though a gentle inclination is visible (results are not affected by the stress generation procedure). Secondly, the ditch was excavated and the embankment built, before the whole load-to-failure process.

The applied load was simulated using a rigid block (linear elastic material) one m high on top of the embankment with given unit weight, according to the load step phase. A rigid load would describe the loading over rails and the effect of the concrete sleepers, present in the experiment. In order to determine the failure load, an incremental multiplier procedure was adopted: a distributed load was placed on top of the rigid block with given unit weight of 1 kN/m^3 , and incremented by 1 kPa loading steps, until peak capacity occurs. In order to model the failure mechanism, the incremental multiplier analysis was further continued after peak until the vertical displacement of the embankment was approximately equal to 1 m (measured after the containers tipped over) at point A (Fig. 17).

The load-time-settlement behavior was modelled in three phases. In the first phase, a rigid load of 24 kPa was applied and, in a second phase, kept constant, to simulate the interruption of the loading process during night. In the third phase, a load equal to the failure load calculated from the incremental multipliers procedure was applied. The effect of the more sensitive clay layer on the failure mechanism was studied through two separate analyses. Firstly, the residual strength of the sensitive clay layer at large strain was set equal to the residual strength of the soft clay (50% of the peak compression strength), and secondly it was reduced by 20% (40% of the peak compression strength), as shown in Fig. 15.

Failure load

The computed peak failure load from the FE analysis using incremental multipliers (q_{soft}) is 80.8 kPa. According to the work done by Mansikkamäki (2015), the duration of loading process in the Perniö failure test has a significant impact on the bearing capacity, as a direct consequence of the rate dependency of s_u . If the embankment were loaded slowly enough, the failure load would have been lower than the measured 87 kPa (q_{test}) (Mansikkamäki et al. 2011; Mansikkamäki 2015).

3D analyses of Perniö failure test have shown that the 3D failure load might be 5-12 % higher than in plane strain (Mansikkamäki and Lämsivaara 2012; Mansikkamäki 2015). Therefore, the calculated value of $q_{\text{soft}} = 80.8$ kPa seems like a fairly good estimate considering (i) that rate as well as 3D effects are not taken into account and (ii) the uncertainties in the input parameters (e.g. the spatial distribution of the actual shear strengths).

The effect of strain-softening on the failure capacity is evaluated through an equivalent analysis where residual shear strengths are set equal to their respective peak strength in the three directions of loading (anisotropic elasto-plastic analysis). The resulting failure load in the equivalent analysis is 6.2% higher than when strain-softening is modelled.

Moreover, the impact of anisotropy on the ultimate capacity is examined by assuming an average isotropic s_u of the clay equal to $s_u^{\text{avg}} = (s_u^A + s_u^{\text{DSS}} + s_u^P)/3$. In this case, the failure load is 8.9% higher (88 kPa versus 80.8 kPa).

When using $s_u^{\text{avg}} = s_u^{\text{DSS}}$ the calculated capacity does not significantly differ from the ADP analysis with softening (0.4% less). However, for s_u^{avg} equal to the average measured FV strength (s_u^{FV} , corrected values), the capacity of the Perniö embankment is underestimated by 8.5%. The computed failure load for different scenarios and the input parameters that differ from those of Table 4 are summarized in Table 6. Input shear strains were maintained equal to their initial values reported in Table 4, as the focus was on the failure load, which is a function of the input s_u .

Failure mechanism

The failure mechanism was studied through two separate analyses, where the soft clay layer was firstly modelled as one single layer, and secondly divided into two layers with different post-peak behavior. When comparing the approximate failure surface from the failure test to the computed shear zone, they seem to share the same starting and ending points, even though the location differs slightly in the active side below the embankment. In Fig. 18 the modelled shear bands at failure predicted by the incremental deviatoric strain ($\Delta\gamma_s$) and the observed shear band (see Figure 5) are illustrated. The computed shear zones correspond to a strain level that is reached when the vertical displacement of the embankment is about 1 m (maximum displacement measured in the experiment after the containers fell on their side). The failure mechanism interpreted from the test seems deeper than the one modelled by the FE analysis, when the more sensitive clay layer is not modelled. When the deep sensitive layer is included in the calculation, the failure mechanism is forced down to a greater depth, which is in better agreement with field observations (Fig. 18). While the approximate failure surface appears to steeply reach the top of the stiffer silty clay layer, the modelled surface bends up more smoothly towards the ditch. The modelled shear zone reaches its maximum depth at a point which has approximately 5 m horizontal distance from the center line of the embankment. According to field data, the maximum depth was reached at point only 3.0 to 3.5 m from the centre-line (Fig. 5 and Fig. 18). The peak failure load is not affected by the two different model scenarios with different failure mechanisms.

Displacements

Fig. 19 shows the load-settlement-time curve for point A for the scenario described by the soil parameters of Table 4, which includes the sensitive soil layer with modified post-peak properties (see Fig. 15). Vertical displacements from the FE analysis are compared with settlement recorded by settlement transducer 53 (located as shown in Fig. 4). Failure occurred in the experiment right after a vertical displacement of about 0.11 m measured from the top of the embankment was reached.

Although creep is neglected, pre-failure settlement can be predicted reasonably well (Fig. 19), even though the results seem slightly overestimated after the onset of first yielding (about 3.5 hours before failure). On the other hand, vertical displacement right before failure is modelled quite accurately.

Horizontal movements during the loading process were recorded in the foundation soil by nine individual inclinometers located in three sections at the toe of the embankment, at the middle and edge of the ditch. Three tubes were placed in each section.

The inclinometer tubes are numbered one to three from the embankment towards the ditch. The solid lines in Fig. 20 show the predicted horizontal displacements (u_x) versus elevation at the toe of the embankment, at the middle and edge of the excavated ditch from inclinometers L2P1, L2P2 and L2P3, respectively.

Calculated horizontal displacements are compared (Fig. 20) with the measured displacements during loading and before failure, recorded at 21:25. Observed displacements correspond to given applied loads (q) of 68.5, 78.5 and 87 kPa (last loading step at 19:30). Given loads of 68.5 kPa and 78.5 kPa correspond to 78% and 90% of the measured failure load in the test ($q_{\text{test}} = 87$ kPa). As the computed failure load ($q_{\text{soft}} = 80.8$ kPa) is lower than q_{test} , field measurements are compared to displacements caused by loads corresponding to 78%, 90%, 99% and 100% of q_{soft} .

Measured horizontal movements are generally smaller than those predicted by NGI-ADPSOFT model, except for the last readings before failure from inclinometer L2P1 and L2P2, where the maximum horizontal displacement (u_x^{max}) is predicted with relatively low error. One possible reason could be that the peak shear strains used in the calculation are higher than those in situ, as a consequence of disturbed sampling.

However, it is quite challenging to accurately model the variation of horizontal movements with depth right before failure, especially in the lower layers at elevation of 2-3 m. One possible reason is that the computed shear band at peak occurs at higher elevation (as if the more sensitive layer was not modelled, see Fig. 18) than in the real experiment (Fig. 5). In contrast, at smaller strains the lateral displacement profiles can be reproduced fairly well, even though the computed values are higher than field measurements (Fig. 20).

Effect of strain softening and development of progressive failure

Fig 21 and Fig. 22 show the contours of hardening parameters κ_1 and κ_2 , respectively, corresponding to different embankment settlements. κ_2 becomes greater than zero when the mobilized stress overcomes the peak strength; while it is equal to one when the residual strength is fully mobilized. The pre-peak hardening control parameter κ_1 becomes equal to one when the peak strength is reached.

For a perfectly plastic material, failure occurs when all the material points along a critical failure surface have reached the maximum shear stress. On the contrary, with NGI-ADPSOFT model the unstable condition (failure) is locally achieved when the shear stress over some length along the shear band has passed the peak shear strength. When the peak failure capacity (q_{soft}) is reached for an embankment settlement of about 0.11 m, κ_1 is equal to one at some locations and lower than one in the surrounding soil (Fig. 21a), where the mobilized shear strength is still below the peak strength. At the same time, $\kappa_2 > 0$ indicates that peak strengths have been passed at some points in the subsoil (Fig. 22a). By increasing the deformation at this state further, the resistance in the post-peak (softening) zone will reduce more than the increase in the pre-peak (hardening) zone. The calculated global resistance will therefore be reduced after this state. For an embankment settlement of about 1 m (Fig. 21b and 22b), the peak state is reached (Fig. 21b) and passed (Fig. 22b) at some other locations in the subsoil. As further shown by Fig. 22b, the residual state is attained in the clay layers where $\kappa_2 = 1$.

Sensitivity analysis

Difficulties were encountered in the derivation of s_u for DSS type of loading from the comparison between FV test results and some existing correlations. When $s_u^{\text{DSS}}/s_u^{\text{A}}$ is set equal to 0.70, the resulting failure load is 5% higher than for $s_u^{\text{DSS}}/s_u^{\text{A}} = 0.65$; while when it is set equal to 0.75, the embankment can sustain a load of 88.10 kPa (9% higher), which seems to be a quite high load without accounting neither for rate effect nor for 3D geometry.

On the other hand, uncertainty in s_u^P is less important to the global behavior of the Perniö embankment, since the failure load becomes only 4% lower by reducing s_u^P/s_u^A by 20% to 0.4. The measured s_u^P/s_u^A may be, however, affected by sample disturbance. According to the study carried out by Karlsrud and Hernandez-Martinez (2013) on block samples of Norwegian soft clays, for undrained TXC tests the sample disturbance affects peak strengths by 10-50%, while in extension tests sample disturbance affects the peak strength by 0-10%.

By running a FE analysis dividing the peak shear strains in the three directions of loading by a factor of 2, nearly no influence on the peak capacity is observed (less than 2%); however, a higher impact is observed with regard to displacements, as the maximum computed horizontal displacement is reduced by 35% (Fig. 23).

When l_{int} is reduced from 0.4 m to 0.1 m, a failure load 5% lower is calculated; while when $l_{int} = 0.7$ m, a higher load (about 4%) is needed to reach failure.

The number of elements constituting the mesh has a small effect on the ultimate capacity, demonstrating how the non-local strain formulation implemented in NGI-ADPSOFT model truly avoids mesh dependency. By refining the mesh clusters in the soft clay layers and thus increasing the element number from 1203 to 1956 and 3432, the difference in the failure load is less than 3%. Both horizontal and vertical displacements have not shown any pertinent change with different element sizes.

The failure mechanism is also not linked to the number of elements of the FE discretization. Although l_{int} , and consequently t_{sb} , were varied, the calculated shear zone reaches the same elevation for all the analyzed configurations.

An increase of about 6% in the failure load is obtained by varying the softening parameters c_1 and c_2 ($c_1 = c_2$) from 1 (from fitting to CAUC tests) to 2. This is an expected result, since the post-peak response would provide “less” strain-softening immediately after the peak state, thus influencing the peak capacity.

The coarse layers do not significantly influence the stability of the Perniö embankment. Small variations in stiffness or in the friction angle do not lead to any remarkable differences. However, the s_u of the dry crust seems to notably affect the FE calculation results. When using a constant

input $s_u = 40$ kPa based on FV test results (according to Lehtonen 2011), the computed failure load is 94 kPa. This would confirm how FV test in weathered clays provides unreliable measurements, in agreement with the findings of Lo (1970), La Rochelle et al. (1974), Lefebvre et al. (1987), Khan (1993) and D'Ignazio et al. (2015).

DISCUSSION

The calculated failure load from the advanced FE analysis presented in this study (80.8 kPa) is 7.1% lower than the observed failure load in the experiment (87 kPa). The discrepancy in the results might be attributed to 3D effects or time/rate effects, which are not modelled in this study. As discussed by Mansikkamäki (2015), the FE 3D modelled failure load for simulations of the Perniö test can be 5 to 12% higher than in a 2D plane-strain analysis.

Partial drainage of excess pore pressure, which could also explain the high measured bearing pressure and, therefore, justify the erroneously predicted failure load in the FE calculation, may have occurred only over a very thin zone adjacent to one of the more permeable boundaries (Lehtonen et al. 2015). Measured excess pore water pressure before failure did not show any reduction (Fig. 6), thus reducing the credibility of the hypothesis of migration of the pore water. Failure must have occurred under nearly fully undrained conditions because of the short duration of the load-to-failure process and the low permeability of the soft clay layer (Lehtonen et al. 2015). Correlating strain rates occurring in the field with those from laboratory tests is quite challenging, as in the field the rate of strain comes from a load-controlled process, while conventional laboratory tests are performed at constant rate of strain.

Conventional triaxial undrained tests in Finland are performed at axial strain rate of about 1%/h, meaning that the soil sample will reach the failure state within a few hours. On the contrary, undrained failure in the field may also occur in several days, as discussed by e.g. La Rochelle et al. (1974). According to the study presented in this paper, s_u from TXC/TXE can fairly well represent a situation where the load is applied quickly, as in the Perniö failure test. Therefore, attention must be paid when triaxial strength is used for stability or bearing capacity calculation, as

it might lead to an overestimation of the safety level when the load is applied over a longer period of time than in the Perniö test. For instance, a critical case could be when the loading process is performed at a much lower strain rate (e.g. tenfold) than in a standard triaxial test (which is in most of the cases), from where the strength parameters are determined. In that case, s_u needs to be reduced because of rate effects, to be representative of the loading conditions and to provide a reasonable estimate of the failure load.

The effect of strain rate on the inferred σ'_p of soft Finnish clays, and therefore also on the s_u , is described by Lämsivaara (1995, 1999, 2012) and summarized by eq. (17):

$$(17) \quad \frac{\sigma'_{p2}}{\sigma'_{p1}} = \left(\frac{\dot{\epsilon}_2}{\dot{\epsilon}_1} \right)^B$$

Where σ'_{p2} and σ'_{p1} are the values of interpreted σ'_p from CRS oedometer tests performed at strain rates $\dot{\epsilon}_2$ and $\dot{\epsilon}_1$, respectively. Their dependency was found to be a function of the ratio between the creep index and the compression index of the soil ($B=C_\alpha/C_c$). For Finnish soft clays $B = 0.068-0.079$ (Lämsivaara 1995, 1999, 2012). For instance, for $B = 0.068-0.079$ and $\dot{\epsilon}_2/\dot{\epsilon}_1 = 0.1$, σ'_p is reduced by 17-20%. By applying the same reduction to s_u , the failure load in the Perniö failure test would be lower than 75 kPa (e.g. stress path A-B of Fig. 7).

CONCLUSIONS

This study has presented the finite element analysis of a shallow railway embankment built on a soft sensitive clay deposit in Perniö, Western Finland. The failure test analyzed was part of a joint research program between Tampere University of Technology and the Finnish Transport Agency.

A series of conclusions can be drawn from this study:

1. From the elaboration of inclinometers readings, it was demonstrated how the rate of shear strain in the full-scale test increased during loading, even while the load was kept constant before failure. This would suggest that failure in the Perniö failure experiment was caused

by undrained creep phenomena in the soft clay, thus supporting the conclusions drawn by Lehtonen et al. (2015) based on pore pressure measurements.

2. NGI-ADPSOFT soil model can appropriately reproduce the behavior of anisotropic saturated soft sensitive clays, which exhibit post peak strain-softening behavior, provided that model parameters are derived carefully and reasonable values for the non-local strain used to regularize the problem are chosen.
3. The predicted failure load of 80.8 kPa, based on the best estimate of soil properties, versus the 87 kPa measured, represents a very good result considering that rate and 3D effects are not taken into account.
4. Undrained shear strength from triaxial tests performed at standard strain rate seems to be representative of the “fast” loading conditions at the failure test. For slower loading or for longer load duration, it would seem necessary to reduce the undrained shear strength from triaxial tests with respect to strain rate.
5. The failure load is not found to be remarkably dependent on either mesh size or thickness of the shear band, since either varying mesh size or internal length, the variation falls into a narrow range of $\pm 5\%$.
6. The anisotropy ratio assumed for DSS (s_u^{DSS}/s_u^A) seems to have a pronounced impact on the predicted failure load, while the anisotropy ratio for extension (s_u^P/s_u^A) had a minor effect on the FE results. This may be due to the shape of the failure surface.
7. Pre-failure vertical settlement of the embankment was modelled quite accurately. The calculated pre-failure horizontal displacements followed the observed trends, even though the magnitude of calculated displacement did not exactly match the observations.
8. The interpreted failure mechanism in the full-scale test seems deeper than the shear zone predicted by the plane-strain analysis. Moreover, the shear band from the FE study reaches its maximum depth at a slightly different coordinate than in the failure test. Nevertheless, the failure zone is properly reproduced when a more sensitive layer is modelled on top of the stiff silty clay layer. By slightly changing the post-peak properties of this layer (i.e. 20% lower residual strength), more plastic strains will develop in the post

peak regime for a given strain level, thus dragging the failure mechanism deeper down.

The triggering peak failure load will remain substantially the same.

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Figures captions

Fig. 1: Varying stress paths along a slip surface (after Bjerrum 1973).

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Fig. 4: Instrumentation layout (after Lehtonen et al. 2015).

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Fig. 22: Hardening parameter κ_2 for an embankment settlement of (a) 0.11 m (at peak state) and (b) 1 m (at large strain).

Fig. 23: Effect of input peak shear strains on horizontal displacements.

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Tables

Table 1. Undrained shear strength anisotropy ratio of soft clays from triaxial and direct simple shear tests.

Author	s_u^{DSS}/s_u^A	s_u^P/s_u^A
Jamiolkowski et al. (1985)	0.53-0.79	0.51-0.61
Ladd (1991)	0.72-0.89	0.5-0.81
Karlsrud et al. (2005) (Norwegian clays)	0.6-0.8	0.3-0.55
Lunne and Andersen (2007)	0.78-0.82	0.59-0.65
Karlsrud and Hernandez-Martinez (2013), Norwegian low sensitive clays ($S_t < 15$)	0.57-0.82	0.3-0.52
Karlsrud and Hernandez-Martinez (2013), Norwegian sensitive clays ($S_t > 15$)	0.56-0.66	0.22-0.32
Thakur et al. (2014) Norwegian clays ($PI < 10\%$)	0.63	0.35
Thakur et al. (2014) Norwegian clays ($PI > 10\%$)	0.63-0.80	0.35-0.50

Table 2. Characteristics of Perniö clay.

Parameter	Unit	Value
Unit weight (γ_{tot})	kN/m ³	15.4
Water content (w)	%	48-109
Liquid limit (w_L)	%	38-82
s_u (FV)	kPa	9-13 on top of the layer, increasing 1.15 kPa/m
Remolded s_u (s_u^{re})	kPa	< 0.5
Sensitivity (S_t)	-	23-69
Over consolidation ratio (OCR) based on CRS tests	-	< 1.5 (soft sensitive clay); > 2 (stiff clay)
Clay content	%	48-81

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Table 3. General characteristics of Perniö dry crust.

Parameter	Unit	Depth 0 – 1 m	Depth 1 – 1.5 m
Unit weight (γ_{tot})	kN/m ³	17	15-16
Water content (w)	%	40-50	80-90
Liquid limit (w_L)	%	60-65	75-85
Plastic limit (w_P)	%	30-35	35-40
Plasticity index (PI)	%	30	40-45
Liquidity index (LI)	-	0.33-0.5	1.11-1.13
Clay content	%	51-58	48-64
Activity	-	0.38-0.52	0.70-0.83
Degree of saturation (S_r)	%	80-90	100

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Table 4. Input parameters for NGI-ADP and NGI-ADPSOFT models.

Parameter	Unit	Description	Dry crust (NGI-ADP)	Stiff clay	Soft clay	Sensitive clay	Silty clay
Material type	-	Material type used in the calculation	Drained	Drained	Drained	Drained	Drained
γ_{tot}	-	Total unit weight	16.5	15	15	15	17.8
G_0/S_u^A	-	Initial stiffness	700	700	700	700	700
$s_{u,ref}^A$	kPa	Reference active shear strength	35	22	17	19.4	22.5
$s_{u,inc}^A$	kPa/m	S_u increase with depth	-13	-7.1	1.8	1.8	3
y_{ref}	m	Reference depth	0	6.3 [†] /5.8 [‡]	5.6 [†] /5.1 [‡]	3.8	-8
x_{ref}	m	Reference x-coordinate	-7	-8 [†] /4.7 [‡]	-8 [†] /4.7 [‡]	4.7	3
$\Delta y_{ref}/\Delta x$	-	Reference depth gradient	-	-0.042 [†] /0 [‡]	-0.042 [†] /0 [‡]	0	-0.08
γ_p^C	%	Shear strain at peak in CAUC	2	2	2	2	3
γ_p^{DSS}	%	Shear strain at peak in DSS	2	4	4	4	3
γ_p^E	%	Shear strain at peak in CAUE	2	8	8	8	3
γ_r^C	%	Shear strain at residual state in CAUC	-	25	25	25	25
γ_r^{DSS}	%	Shear strain at residual state in DSS	-	25	25	25	25
γ_r^E	%	Shear strain at residual state in CAUE	-	25	25	25	25
τ_0/s_u^A	-	Initial mobilization	0	0.11	0.3	0.3	0.4
s_u^P/s_u^A	-	Normalized passive strength	0.88*	0.5	0.5	0.5	0.5
s_u^{DSS}/s_u^A	-	Normalized DSS strength	0.92*	0.65	0.65	0.65	0.65
s_{ur}^A/s_u^A	-	Normalized residual active strength	-	0.5	0.5	0.4	0.5
s_{ur}^{DSS}/s_u^A	-	Normalized residual DSS strength	-	0.5	0.5	0.4	0.5
s_{ur}^P/s_u^A	-	Normalized residual passive strength	-	0.5	0.5	0.4	0.5
ν'	-	Poisson's ratio	0.495	0.495	0.495	0.495	0.495
K_0	-	Lateral stress coefficient	1	0.77	0.65	0.65	0.65
c_1	-	Shape parameter	-	1	1	1	1
c_2	-	Shape parameter	-	1	1	1	1
α	-	Non-local strain parameter	-	2	2	2	2
l_{int}	m	Internal length	-	0.4	0.4	0.4	0.4

[†] For inclined layers below the embankment.

[‡] For horizontal layers below the ditch.

*Data from D'Ignazio (2016).

Table 5. Input parameters for the Hardening Soil model.

Parameter	Unit	Description	Embankment	Sand fill	Lower sand
Material type		Material type used in the calculation	Drained	Drained	Drained
γ_{tot}	kN/m ³	Total unit weight	21	19	19
E_{50}^{ref}	MPa	Secant stiffness for CD triaxial	100	30	30
$E_{\text{oed}}^{\text{ref}}$	MPa	Tangent oedometer stiffness	100	30	30
$E_{\text{ur}}^{\text{ref}}$	MPa	Unloading/Reloading stiffness	250*	90	90
m	-	Power for stress dependent stiffness	0.5	0.5	0.5
c'	kPa	Effective cohesion	1	1	1
p^{ref}	kPa	Reference stress for stiffness	100	100	100
POP**	kPa	Pre-overburden pressure	0	20	0
ϕ'	°	Friction angle	38	36	36
ψ	°	Dilatancy angle	8	6	6
ν'	-	Poisson's ratio	0.3	0.3	0.3

* $E_{\text{ur}}^{\text{ref}} = 2.5 E_{50}^{\text{ref}}$ according to Mansikkamäki (2015).

** $\text{POP} = \sigma'_p - \sigma'_{v0}$

Table 6. Calculated failure load in Perniö failure test for scenarios with different input s_u .

Input s_u	Anisotropy	Strain softening	Layer	$s_{u,ref}^A$ (kPa)	$s_{u,inc}^A$ (kPa/m)	Failure load (kPa)
s_u (peak values)*	Yes	Yes	-	-	-	80.8
s_u (peak values) [$s_{ur}^P/s_u^A = 0.5$, $s_{ur}^{DSS}/s_u^A = 0.65$, $s_{ur}^A/s_u^A = 1$]	Yes	No	Stiff clay	22	-7.1	85.8
			Soft clay	17	1.8	
			Sensitive clay	19.4	1.8	
			Silty clay	22.5	3	
$s_u^{avg} = (s_u^A + s_{ur}^{DSS} + s_{ur}^P)/3$ [$s_{ur}^P/s_u^A = s_{ur}^P/s_{ur}^A = 1$, $s_u^{DSS}/s_u^A = s_{ur}^{DSS}/s_{ur}^A = 1$, $s_{ur}^A/s_u^A = 1$]	No	No	Stiff clay	16.1	-5.4	88
			Soft clay	12.3	1.31	
			Sensitive clay	14	1.31	
			Silty clay	16.3	2.2	
s_u^{DSS} [$s_{ur}^P/s_u^A = s_{ur}^P/s_{ur}^A = 1$, $s_u^{DSS}/s_u^A = s_{ur}^{DSS}/s_{ur}^A = 1$, $s_{ur}^A/s_u^A = 1$]	No	No	Stiff clay	14.6	-4.9	80.5
			Soft clay	11.2	1.2	
			Sensitive clay	12.7	1.2	
			Silty clay	14.7	1.95	
s_u^{FV} (FV average, corrected values) [$s_{ur}^P/s_u^A = s_{ur}^P/s_{ur}^A = 1$, $s_u^{DSS}/s_u^A = s_{ur}^{DSS}/s_{ur}^A = 1$, $s_{ur}^A/s_u^A = 1$]	No	No	Stiff clay	13.7	-4.6	73.9
			Soft clay	10.5	1.15	
			Sensitive clay	12	1.15	
			Silty clay	14	1.85	

*Input parameters from Table 4.

Fig. 1: Varying stress paths along a slip surface (after Bjerrum 1973).
Fig. 1
38x17mm (600 x 600 DPI)

Fig. 2: Stratigraphy of the test site.

Fig. 2

125x85mm (600 x 600 DPI)

Fig. 3: General characteristics of Perniö clay, from samples taken over the test area.
Fig. 3
186x86mm (300 x 300 DPI)

Fig. 4: Instrumentation layout (after Lehtonen et al. 2015).

Fig. 4

77x32mm (300 x 300 DPI)

Fig. 5: Interpreted failure mechanism (modified after Lehtonen et al. 2015).

Fig. 5
87x41mm (300 x 300 DPI)

Fig. 6: Evolution of excess pore pressure and rate of excess pore pressure at different locations as a function of the applied load (adapted from Lehtonen et al. 2015)

Fig. 6

129x204mm (300 x 300 DPI)

Fig. 7: Idealized stress paths for Perniö clay.
Fig. 7
164x173mm (96 x 96 DPI)

Fig. 8: Shear strain versus time from inclinometers L2P1, L2P2, L2P3.

Fig. 8

126x88mm (300 x 300 DPI)

Fig. 9: (a) Typical effective stress paths and stress–strain curves for triaxial compression (TxC) and extension (TxE) tests (from Grimstad et al. 2012) and (b) stress–strain behavior in direct simple shear (DSS) tests (from Andresen et al. 2011) with definition of stress/strain quantities.

Fig. 9
 59x19mm (600 x 600 DPI)

Fig. 10: Input parameters for NGI-ADPSOFT model. The triaxial and DSS curves are presented in terms of deviatoric stress $((\sigma_y - \sigma_x)/2)$ and shear stress (τ_{xy}) , respectively, versus strain. The symbols x and y refer to horizontal and vertical directions.

Fig. 10

79x72mm (300 x 300 DPI)

Fig. 11: Impact of shape parameters c_1 and c_2 on the post peak behavior modelled by NGI-ADPSOFT soil model. In this example, $\gamma_D = 1\%$ and $\gamma_r = 10\%$.

Fig. 11
85x88mm (300 x 300 DPI)

Fig. 12: Peak values of s_u/σ'_v plotted against the overconsolidation ratio (OCR) and comparison with existing correlations for Scandinavian clays.

Fig. 12

64x48mm (300 x 300 DPI)

Fig. 13: Normalized s_u^{FV}/σ'_v of Perniö clay versus s_u^{DSS}/σ'_v and $s_u(mob)/\sigma'_v$ predicted by existing correlations for clays.

Fig. 13
60x45mm (300 x 300 DPI)

Fig.14: NGI-ADPSoft model curve fitting to TXC and TXE tests on Perniö clay.

Fig. 14

75x65mm (300 x 300 DPI)

Fig.15: Modelling of the sensitive clay layer.
Fig. 15
87x89mm (300 x 300 DPI)

Fig. 16: Anisotropic undrained shear strength profile for Perniö site.

Fig. 16

87x89mm (300 x 300 DPI)

Fig. 17: Finite Element mesh.
Fig. 17
85x40mm (600 x 600 DPI)

Fig. 18: Comparison between the observed failure mechanism and the modelled shear bands from the FE analyses.

Fig. 18

83x37mm (300 x 300 DPI)

Fig. 19: Modelled versus observed Load-Time-Settlement.
Fig. 19
85x88mm (300 x 300 DPI)

Fig. 20: Measured and modelled horizontal displacements for section A-A.

Fig. 20
105x60mm (600 x 600 DPI)

Fig. 21: Hardening parameter κ_1 for an embankment settlement of (a) 0.11 m (at peak state) and (b) 1 m (at large strain).

Fig. 21
198x215mm (300 x 300 DPI)

Fig. 22: Hardening parameter κ_2 for an embankment settlement of (a) 0.11 m (at peak state) and (b) 1 m (at large strain).

Fig. 22
198x211mm (300 x 300 DPI)

Fig. 23: Effect of input peak shear strains on horizontal displacements.

Fig. 23

119x173mm (300 x 300 DPI)