# Strength increase below an old test embankment in Finland

M. D'Ignazio
*Tampere University of Technology, Finland, marco.dignazio@tut.fi*

T. Länsivaara
*Tampere University of Technology, Finland, marco.dignazio@tut.fi*

ABSTRACT
*An instrumented test embankment was built in Murro, Western Finland, in 1993. The purpose was to monitor the long term settlement of the embankment on a silty clay deposit, and use the information for the design of Highway 18 between the cities of Jyväskylä and Vaasa.*
*Field vane test was performed prior to construction and after 8 years of consolidation. Test results in 2001 showed increase of undrained shear strength up to a depth of 6-7 m under the centerline of the embankment, while below 7 m depth the strength decreased compared to the initial stage.*
*In 2013, 20 years after construction, CPTu and field vane test were performed by Tampere University of Technology at the test site. From CPTu measurements no strength decrease could be observed. The undrained shear strength below the embankment was higher than beside the embankment up to a depth of 11 m, after which the two coincided.*
*In this study, a critical comparison between the old soil data and the new tests results is made. The strength distribution below the test embankment is assessed and the strength increase evaluated. Existing correlations for undrained shear strength are compared with field measurements. Finally, the long term behavior and the impact of strength increase on the stability of Murro test embankment are studied through the Finite Element software PLAXIS 2D.*

**Keywords: soft soil; embankment; piezocone; consolidation; finite element.**

## 1 INTRODUCTION

In order to study the long term behavior of an embankment on a soft clay deposit, a highly instrumented test embankment was built in Murro, Western Finland, near the city of Seinäjoki, in 1993, commissioned by the Finnish Road Administration (Koskinen et al., 2002). The subsoil conditions consisted of a 23 m thick low organic silty clay deposit with presence of sulfur. The soft clay is overlain by a stiff, fissured crust layer less than 2 m thick. Settlement and pore pressures have been monitored for more than 20 years.
An important aspect to be evaluated in staged construction for both old and new embankments is how the undrained shear strength increases with time. The general assumption is that there will be an increase of strength if the preconsolidation pressure of the clay increases due to consolidation. To investigate this, field vane test was performed 8 years after construction below the embankment. Test results showed an increase in undrained strength up to 7 m depth. However, below 7 m the field vane test indicated that the strength would have decreased.
In contrast, CPTu tests performed in 2013 by Tampere University of Technology revealed strength increase up to 10-12 m depth, while no strength decrease could be observed.
In the present study, after a brief overview on the phenomenon of strength increase with time, the old soil investigation data from Murro site and the new test results are compared. The undrained shear strength distribution below the test embankment is assessed using both existing transformation models for undrained shear strength from piezocone and field vane test results. Finally,

an attempt to model the long term settlement of Murro test embankment and the strength increase is done using the Finite Element software PLAXIS 2D.

## 2 SHEAR STRENGTH INCREASE

Natural soft clay deposits generally show an over consolidation ratio (OCR = $\sigma'_p/\sigma'_{v0}$, ratio between vertical preconsolidation pressure and effective vertical stress) higher than 1 because of aging effect (Bjerrum, 1973; Hanzawa, 1995), without necessarily having previously experienced any release of overburden stress. Due to chemical bonding and secondary compression, $s_u$ of clays subjected to aging process possesses additional strength (Suzuki and Yashuara, 2007). As shown in Figure 1, the line A-B represents the process of aging where, under a constant overburden stress, the clay gains additional strength and becomes over consolidated with $s_u = s_{uf}$, effective vertical stress equal to $\sigma'_{v0}$ and preconsolidation pressure equal to $\sigma'_p$. For a given increase in consolidation stress ($\Delta\sigma_v$), $s_u$ will increase along C-D, provided that the total effective vertical stress is higher than $\sigma'_p$. On the contrary, below $\sigma'_p$, $s_u$ will be equal to $s_{uf}$.

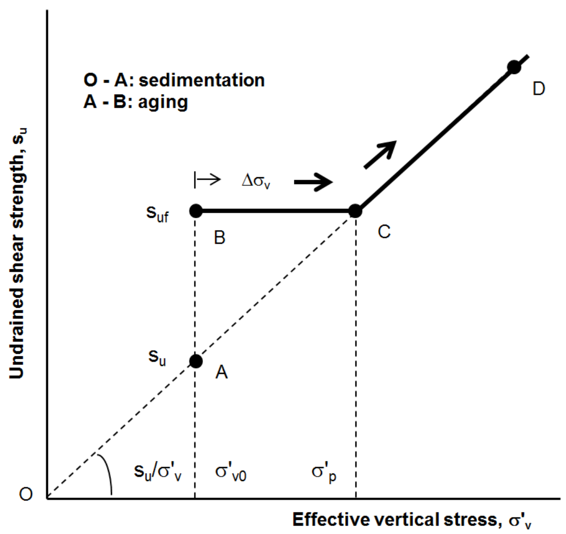


*Figure 1 Relationship between $s_u$, $\sigma'_v$ and $\sigma'_p$ (modified after Suzuki and Yashuara, 2007).*

The dependency of $s_u$ on consolidation stresses is a well-known phenomenon (e.g. Mesri, 1975; Jamiolkowski et al., 1985). The shear strength of a clayey soil increases under loading due to the consolidation process, as the dissipation of excess pore water pressure will lead to a change in the stress state beneath the load, with consequent increase of effective stress. Moreover, secondary consolidation (creep) will also contribute to the long term settlement (Craig, 2004).

For geotechnical structures built on clay deposits where the ground water table is located in the proximity of the ground surface, buoyancy effects will have a significant impact on the final stress distribution.

A correct assessment of the shear strength increase would seem beneficial when improving old embankments. Indeed, the improved capacity, which is often neglected, would reduce to a minimum or exclude possible countermeasures needed to achieve the new target safety level. For instance, when such phenomenon is taken into account, the calculated factor of safety of existing embankments will result higher than at the initial stage (e.g. Tavenas et al., 1978; Slunga, 1983).

Estimating the increase in undrained shear strength becomes also important when multi stage loading for construction on soft clay is performed (e.g. Tavenas et al., 1978).

Strain rate dependency of $s_u$ and $\sigma'_p$ (e.g. Leroueil et al., 1985; Länsivaara, 1999) must be also considered when assessing such properties. $s_u/\sigma'_v$ will result higher when the soil is sheared at faster strain rates than at low strain rates (Leroueil et al., 1985).

## 3 SITE INVESTIGATION AT MURRO

Murro test embankment is 2 m high and 30 m long (Figure 2). The top width of the embankment is 10 m. The body of the structure consists of crushed rock with a grain size of 0-65 mm.

The embankment was built on a 23 m deep normally to slightly over consolidated soft silty clay deposit, overlain by a dry crust layer 1.6 m thick. Displacements and pore water pressures have been monitored for over 20 years. The original purpose was to exploit the experimental observations for the design of Highway 18 between the cities of Jyväskylä and Vaasa.

Soil characteristics of Murro clay are shown in Figure 3. The water content (*w*) ranges from 65% to 100%, with sensitivity ($S_t$) varying between 2 and 10. The liquid limit ($W_L$) increases from 55% near the ground surface, up to 120% at 5 m depth. After that point, $W_L$ shows a negative trend with depth, decreasing up to about 60% below 15 m depth. Preconsolidation pressure ($\sigma'_p$) values in Figure 3 were obtained from CRS (constant rate of strain) oedometer tests.

A detailed description of the physical and mechanical characteristics of Murro clay can be found in Koskinen et al., (2002), Karstunen et al., (2005), Karstunen and Yin, (2010).

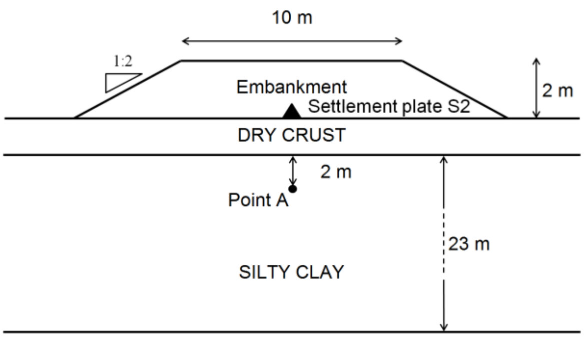


*Figure 2 Murro test embankment (section).*

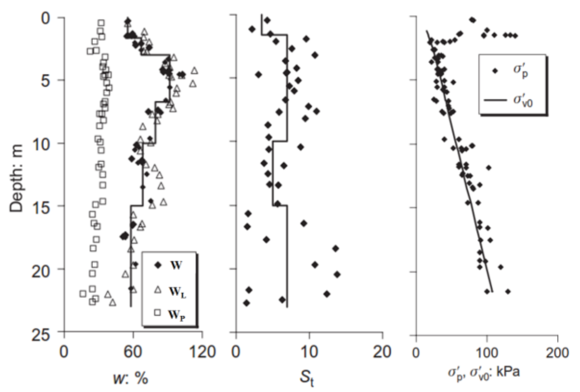


*Figure 3 Characteristics of Murro clay (modified after Karstunen and Yin, 2010).*

In order to assess the undrained shear strength due to consolidation of the subsoil, field vane test was performed in 2001, 8 years after construction. Corrected field vane test results showed increase of undrained shear strength up to a depth of 6-7 m below the centre-line of the embankment. However, below 7 m depth the strength seemed to have decreased compared to the initial stage (Figure 4). A possible explanation for the unexpected observed phenomenon could be the destructuration of the clay induced by the change in stress state (Karstunen et al., 2005; Karstunen and Yin, 2010).

In 2013, after 20 years of consolidation of the soft clay foundation, piezocone (CPTu) and field vane tests were performed by Tampere University of Technology at Murro test site. Field vane test was repeated also in 2015, as measurements from 2013 were available only up to 10 m depth.

Tampere University of Technology has recently bought 1) CPTu equipment with seismic and resistivity cone and, 2) a new field vane which allows for rotation and torque measurements right above the vane. The purpose is to increase the use of CPTu in Finland for the determination of soft soil properties and to improve the existing transformation models for strength and deformation characteristics of soft clays.

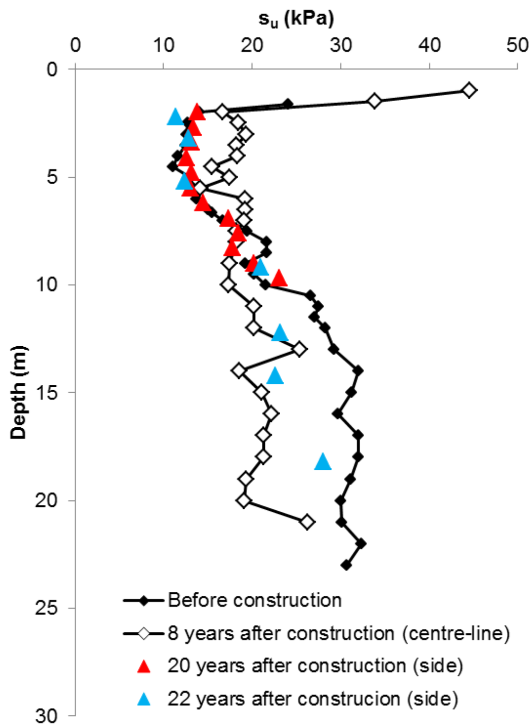


*Figure 4 Corrected field vane test results at Murro test site prior to construction, 8 years, 20 years and 22 years after the construction of the embankment.*

For the Murro test site, CPTu soundings are available from both the centre-line of the embankment and the East side (about 3 m off the slope). Field vane test was performed only at one side of the embankment, due to technical difficulties. Results from field vane test are presented in Figure 4. Measured $s_u$ agrees fairly well with $s_u$ measured at the initial stage. Fairly good correspondence can be also observed at greater depths, except for two points at 12.2 m and 14.2 m, where $s_u$ seems lower than at the initial stage. Furthermore, as shown in Figure 5, cone tip resistance differs significantly at the two measurement points, up to a depth of about 11 m. No strength decrease could be observed in the subsoil, thus contradicting the trend observed in Figure 4.

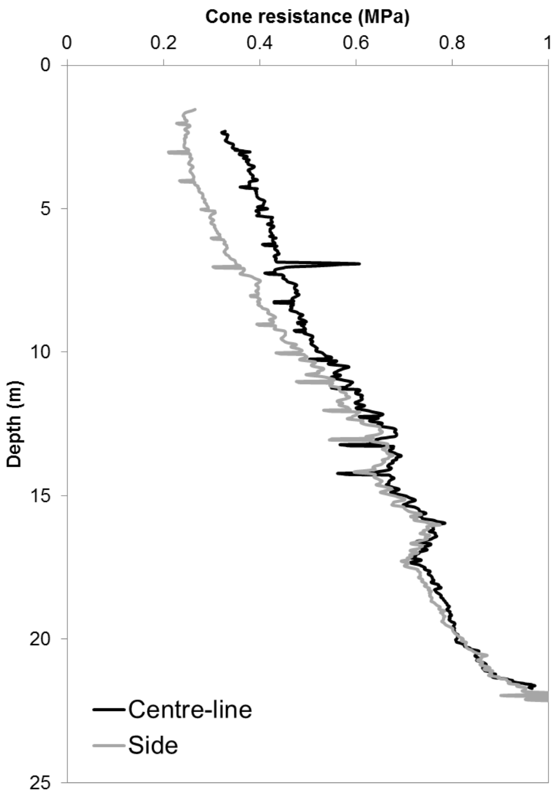


*Figure 5 Cone tip resistance at the test site under the centre-line and at the East side of the embankment.*

## 4 UNDRAINED SHEAR STRENGTH OF MURRO CLAY

Figure 6 shows the undrained shear strength profile at Murro test site. In-situ measurements obtained from piezocone and field vane test are compared. Field vane data points ($s_u^{FV}$) are corrected based on plasticity and converted to mobilized $s_u$ ($s_{u(mob)} = \mu \cdot s_u^{FV}$) according to Larsson and Åhnberg (2005) [eq. (1)]. The effective cone resistance ($q_t - \sigma_{v0}$) is converted into $s_u$ from the general transformation model of eq. (2).

$$\mu = \left(\frac{0.43}{W_L}\right)^{0.45} \tag{1}$$

$$s_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{2}$$

Where $W_L$ is the liquid limit of the clay, $q_t$ is the measured cone resistance corrected for pore pressure effects, $\sigma_{v0}$ the total overburden vertical stress and $N_{kt}$ is the cone factor. $N_{kt}$ is firstly evaluated according to Larsson and Mulabdic (1991) and secondly estimated from field vane test results.

According to Larsson and Mulabdic (1991), $N_{kt}$ increases linearly with increasing liquid limit, as $N_{kt}$=13.4+6.65$W_L$. Liquid limit profile with depth is shown in Figure 2 and both $\mu$ and $s_u$ are calculated accordingly.

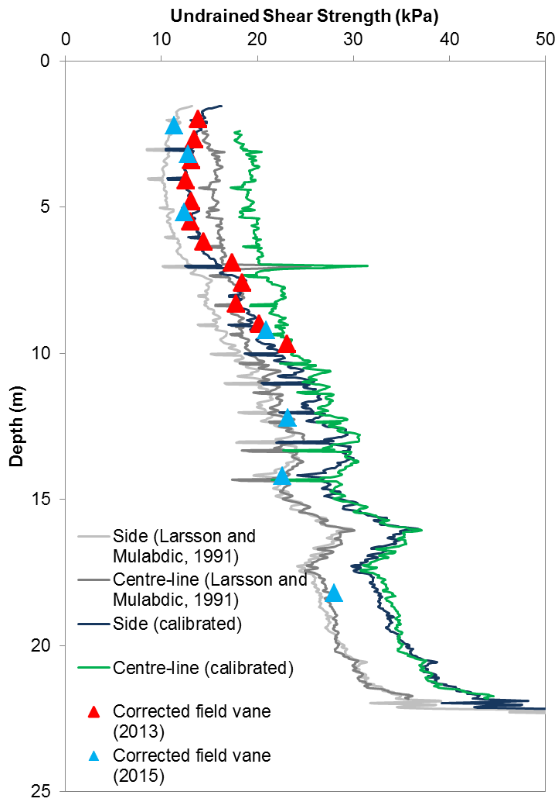


*Figure 6 Undrained shear strength from piezocone and field vane test at the test site.*

The stress increase below the embankment is estimated using the FE software PLAXIS 2D (see Section 5), in order to account for buoyancy effects. $W_L$ values are assumed to be the same before and after consolidation.

According to Figure 6, Larsson and Mulabdic's model ($N_{kt}$=18-19) predicts lower $s_u$ than the field vane. Therefore, such a model, which is used in Sweden, does not seem adaptable to the soil conditions at Murro test site. A calibrated average cone factor equal to $N_{kt}$=15 seems to provide a better description of the variation of undrained shear strength with depth. Assuming $N_{kt}$=15 also below the embankment, shear strength increase can be clearly observed up to a depth of 11 m. The maximum strength increase is observed at about 3.5 m depth, where $s_u$ results about 7 kPa higher than at the side of the embankment.

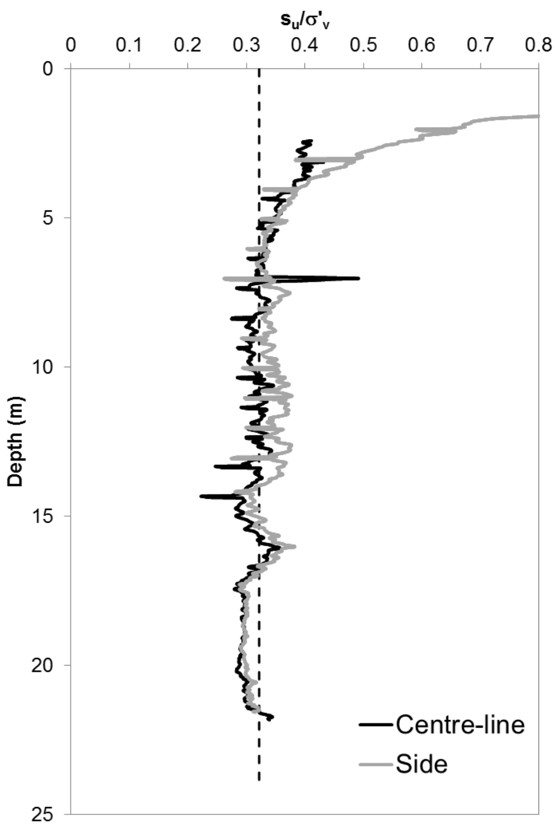


*Figure 7 Variation and change in $s_u/\sigma'_v$ with depth and consolidation.*

As shown in Figure 7, measured values of $s_u/\sigma'_v$ vary from 0.26 to 0.83 at the side of the embankment, with mean value of 0.36. Under the embankment, $s_u/\sigma'_v$ ranges from 0.22 to 0.49, with mean value equal to 0.32. The ratio $s_u/\sigma'_v$ is reduced due to consolidation, as also observed by Tavenas et al. (1978) from embankments built on Canadian clay deposits. Therefore, $s_u/\sigma'_v$ = 0.32 would give an indication of the undrained shear strength of Murro clay for normally consolidated state. $s_u/\sigma'_v$ = 0.32 would seem, in the authors' opinion, rather high for a soft normally consolidated clay. However, Murro clay is referred as silty clay or clayey silt (Karstunen et al., 2005). With respect to the latter definition, the silty component might play a key role. Laboratory tests (e.g. direct simple shear tests) are though needed in the future to verify the field observations.

## 5 FINITE ELEMENT ANALYSIS

### *5.1 Description of the soil models considered in this study*

An attempt to model Murro test embankment using finite element method is done. The Soft Soil (SS) model (Plaxis, 2012) is used and the analyses are performed through the finite element software PLAXIS 2D 2012 AE.

The Soft Soil (SS) model is an isotropic effective stress soil model based on the Modified Cam Clay (MCC) model. The model assumes a logarithmic relationship between volumetric strain and mean effective stress. The model does not include rate dependency of soft clays. A special feature of the SS model is the Mohr-Coulomb failure criterion, while for MCC model failure is defined by the critical state line. In the SS model, the *M* line (see section 5.3) just defines the shape of the yield surface, thus making the model more adaptable to the actual yielding behavior of soft Finnish clays (Mansikkamäki, 2015). The principal input parameters for the SS model are the modified compression index $\lambda$*, the modified swelling index $\kappa$*, the OCR, the effective cohesion ($c'$) and friction angle at critical state ($\phi'$), the unloading-reloading Poisson's ratio ($\nu_{ur}$), the lateral earth pressure coefficient for normally consolidated state $K_0^{nc}$ (used as yield parameter, discussed in Section 5.3) and the lateral earth pressure coefficient ($K_{0\,\mathrm{initial}}$).

The Soft Soil Creep model (Plaxis, 2012) is an extension of the Soft Soil model that takes secondary compression into account. SSC model is based on the isotache concept (Šuklje, 1957) and capable to model the time dependent creep behavior of clays. Input

parameters of the two soil models differ only for the modified creep index $\mu^*$ not included in the SS model.
However, as discussed by Mansikkamäki (2015), SSC model tends to severely overestimate the creep settlement of normally to slightly over consolidated soils. Especially for Finnish soft soil conditions, significantly reduced values of $\mu^*$ or increased OCR values have to be used to correctly simulate the creep settlement (Mansikkamäki, 2015). Mansikkamäki (2015) suggested that OCR should be at least 1.6-1.8 to achieve realistic creep strains. For this reason, SSC model does not seem suitable for modelling the creep behavior of Murro clay.

### *5.2 Finite element model and calculation results*

The embankment and the dry crust are modelled as Mohr-Coulomb drained materials. The soft clay is divided into 5 sublayers (see Figure 8), as suggested by Koskinen et al. (2002), and modelled using Soft Soil model. Soil parameters for Mohr-Coulomb and Soft Soil models are chosen according to Koskinen et al. (2002).
Furthermore, according to Koskinen et al. (2002) the ratio $\lambda^*/\kappa^*$ for Murro clay varies between 11 and 14 for Layers 1-4. For Layer 5, $\lambda^*/\kappa^*$=36.
The FE plane-strain model used for the analyses is shown in Figure 8. Due to the symmetry of the problem, only one half of the embankment is considered in the analyses. The lateral boundary is 36 m from the symmetry axis and the vertical boundary is at 23 m depth. A “very fine” type of mesh is used in order to ensure convergence of the results, consisting of 1094 15-noded triangular elements with average element size of 0.97 m. Full fixities are assigned at the base and roller conditions at the vertical sides. The ground water table is located at 0.8 m depth.
The initial stress state is generated assuming $K_0$ conditions and $K_{0\ \mathrm{initial}} = 1 - \sin\phi'$. A two-day loading phase is used to reproduce the construction of the embankment. Finally, a 20-year consolidation analysis is performed.

Two parallel analyses are conducted: firstly, using the standard calculation settings (small strain analysis), and secondly, using the updated pore water pressure option (large strain analysis). The advantage of using the large strain option is that buoyancy of the fill material and dry crust material is taken into account, as they become submerged during the consolidation process.

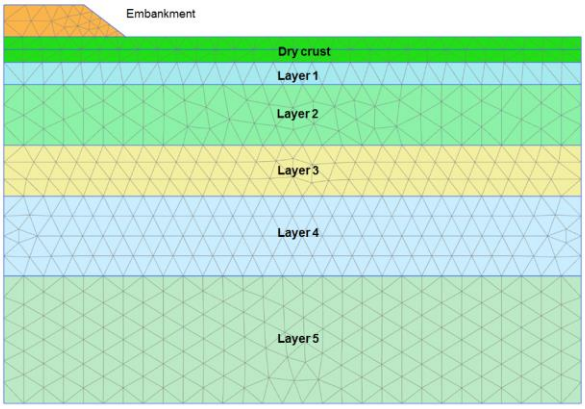


*Figure 8 FE plane-strain Mesh.*

Figure 9 shows a comparison between the measured settlement from Settlement Plate S2 (located at the centre-line of the embankment, as shown in Figure 2) and the results of the FE simulations. Figure 9 also illustrates the dissipation of excess pore pressures with time at Point A (see Figure 2), where the maximum strength increase was observed.

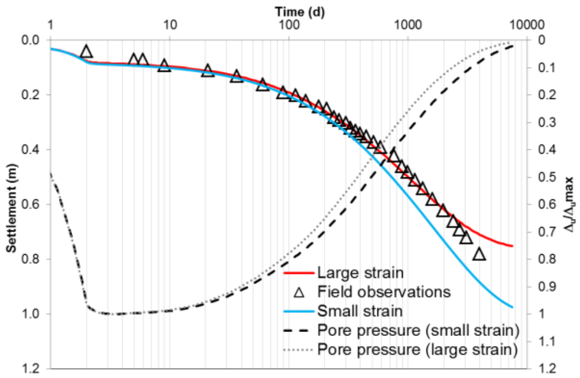


*Figure 9 Long term settlement behavior of Murro test embankment and decay of excess pore pressures with time.*

When using the large strain option, the settlement will result lower than in small strain analysis, as the stresses in the soil will be reduced due to buoyancy. As a

consequence, calculated pore pressures will be higher in small strain analysis. Overall, the settlement can be predicted reasonably well for the first 8 years after construction, before the calculation results deviate from measurements due to creep. Pore pressure dissipation, and therefore strength increase, mainly occurs during the first 6 years, as almost 90% of the maximum $\Delta u$ seems to have dissipated at t = 2000 d.
The calculated factor of safety of the embankment, evaluated through the "safety" calculation option in PLAXIS, increases from 2.02 to 3.14.

### 5.3 *Undrained shear strength in PLAXIS*

One method to determine $s_u$, corresponding roughly to the vane strength, is to conduct Direct Simple Shear (DSS) tests. DSS tests can be simulated through a Soil Test tool implemented into PLAXIS. In the Soft Soil model, the shape of the initial yield surface is defined by the *M* parameter. *M* is automatically defined based on $K_0^{nc}$ parameter, which can be set by the user. *M* can be approximated as in eq. (3) (from Plaxis user's manual):

$$M \approx 3.0 - 2.8K_0^{nc} \tag{3}$$

Being $K_0^{nc}$ calculated according to Jaky's $K_0^{nc}$ = 1-sinϕ', the resulting *M* will be much higher than *M* based on the friction angle of the material (Mansikkamäki, 2015), as shown in Figure 10.

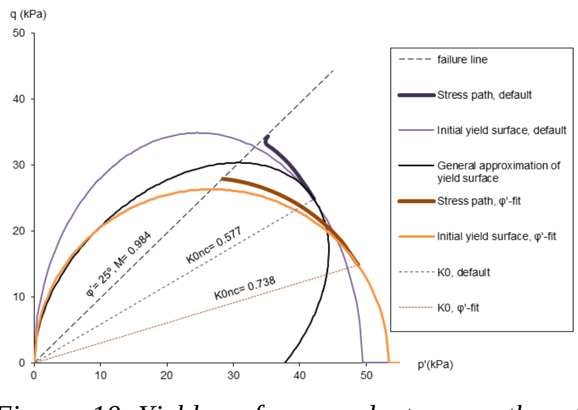


*Figure 10 Yield surfaces and stress paths of Perniö clay depending on $K_0^{nc}$- and M-values (Mansikkamäki, 2015).*

For $\phi'$=37°, the average default M value for Murro clay is 1.98. The "fitted" M value, based on the friction angle would be 1.51. The main benefit of this procedure is that the excess pore pressure potential before failure can be realistically estimated, thus leading to a maximum deviatoric stress level lower than in the default case. The fitted yield surface was found to reproduce the behavior of Finnish clays better than the default yield surface (Mansikkamäki and Länsivaara, 2010). Furthermore, rate effects are not considered in this study, as the Soft Soil model is a rate independent model.

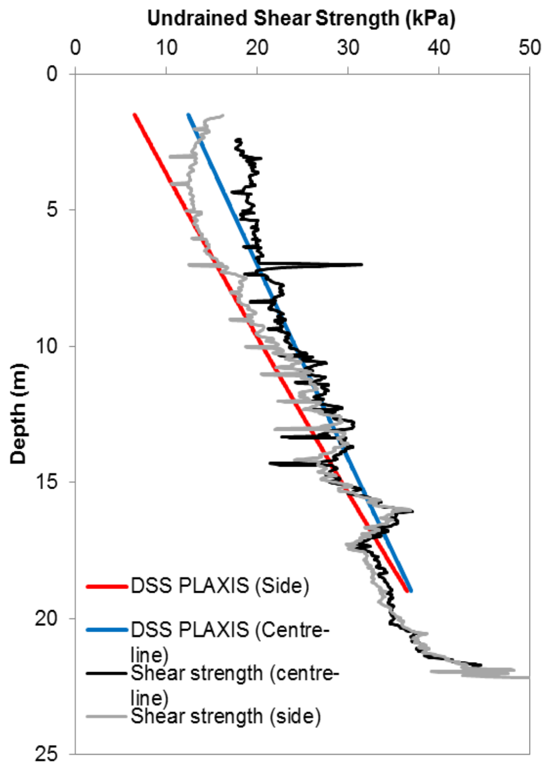


*Figure 11 Undrained shear strength of Murro clay in Soil Test DSS simulations in PLAXIS using Soft Soil model.*

Figure 11 shows the DSS strength modelled with PLAXIS compared to the undrained shear strength of Murro clay measured from piezocone. For OCR = 1, *M* = 1.51 and $K_{0\,initial}$=0.4, the predicted $s_u/\sigma'_v$ is equal to 0.32, which corresponds to the average measured strength in Section 4. The predicted $s_u$ agrees fairly well with the strength conditions at the test site up to 17 m depth. However, up to 5-7 m depth, measured $s_u$ appears to be higher than the calculated one. From 17 to 23 m depth, $s_u$ values are overestimated. Nevertheless, the soil strength is highly dependent on friction angle and $K_0^{nc}$ values assumed. For $K_0^{nc}$=0.4 (default value), $s_u/\sigma'_v$ is equal to 0.38.

## 6 DISCUSSION

According to Janbu (1985), undrained shear strength is theoretically related to preconsolidation pressure ($\sigma'_p$), effective friction angle ($\phi'$) and attraction ($a$) as shown by eq. (4):

$$s_u \approx \frac{1}{2}\left(\sigma'_p + a\right)\cdot \sin\phi' \tag{4}$$

Typically, $a$ is taken equal to zero for normally consolidated clays. Table 1 summarizes values of $s_u/\sigma'_v$ obtained using different friction angle and OCR values, based on eq. (4). By definition, $\sigma'_p$=OCR·$\sigma'_v$. For normally consolidated clays, eq. (4) for $\phi'$=37° gives $s_u/\sigma'_v$ equal to 0.30, against a value of 0.32 measured for Murro clay. $s_u/\sigma'_v$=0.32 can be obtained for $\phi'$=30° only if a light over consolidation exists (OCR~1.3), or perhaps if $a$ is greater than zero.

*Table 1 $s_u/\sigma'_v$ for different values of $\phi'$ and OCR assuming a=0 according to Janbu (1985).*

| ϕ' | OCR=1 | OCR=1.1 | OCR=1.3 |
|---|---|---|---|
| **25** | 0.21 | 0.23 | 0.27 |
| **30** | 0.25 | 0.28 | 0.33 |
| **33** | 0.27 | 0.30 | 0.35 |
| **37** | 0.30 | 0.33 | 0.39 |

According to the authors' experience, a friction angle of 37° (Koskinen et al., 2002) seems quite high for soft normally consolidated silty clay. In order to improve the knowledge of friction angle of Murro clay, triaxial compression and extension tests on block samples of Murro clay will be performed in the coming months at Tampere University of Technology.
Furthermore, the author's experience would suggest that OCR=1 is a too conservative assumption, as the aging effect would be neglected. Sample disturbance may have led to such a cautious hypothesis.

An attempt to evaluate the preconsolidation pressure ($\sigma'_p$) of Murro clay is done according to Larsson and Mulabdic (1991) using eq. (5).

$$\sigma'_p = \frac{q_t - \sigma_{v0}}{1.21 + 4.4\cdot W_L} \tag{5}$$

Where $N_{kt}(\sigma'_p)$=1.21+4.4$W_L$ is the cone factor for preconsolidation pressure. As shown in Section 4, the cone factor predicted by Larsson and Mulabdic for undrained shear strength ($N_{kt}$=18-19) is lower than the cone factor calibrated using the field vane test results ($N_{kt}$=15). Assuming that the same discrepancy exists for preconsolidation pressure, $N_{kt}(\sigma'_p)$ is then multiplied by 0.81.
The variation of OCR with depth is shown in Figure 12. The OCR under the embankment is calculated by dividing the initial $\sigma'_p$ values (at the side) with the effective stress values after consolidation at the same depth.

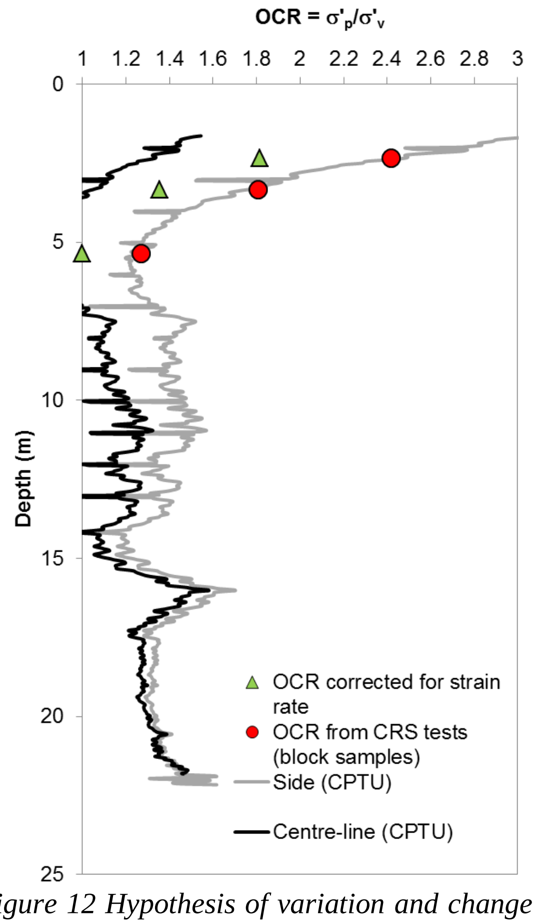


*Figure 12 Hypothesis of variation and change in OCR with depth and consolidation.*

According Figure 12, Murro clay seems to be slightly overconsolidated, with average OCR=1.4. OCR decreases from about 3 below the dry crust to 1.2 at about 6 m depth, while a marked fluctuation (OCR ranging from 1.1 to 1.6) is visible at depth of 14-18 m, where the soil is probably less homogeneous. This theoretical speculation is though consistent with the OCR trend with depth suggested by three OCR data points calculated from CRS oedometer tests on block samples of Murro clay. After 20 years, a reduction in the OCR is estimated (Figure 12). The normally consolidated state is only reached from 4 up to about 7 m depth. However, if $\sigma'_p$ is corrected for strain rate as suggested by Leroueil (1996), the clay would seem to be normally consolidated at 5-6 m depth.

## 7 CONCLUSIONS

In this paper, the increase in undrained shear strength below Murro test embankment was studied by means of in-situ tests and the finite element software PLAXIS 2D. A series of conclusions can be drawn from this study:

1. Piezocone test results from the centre-line and the side of the embankment proved that the strength after 20 years of consolidation is higher than at the initial stage, prior to construction. Field vane test results from 2001 are, hence, contradicted.
2. According to the more recent field vane test results, the transformation model proposed by Larsson and Mulabdic (1991) for Swedish clays seems to overestimate the cone factor and, therefore, underestimate the undrained shear strength of Murro clay.
3. Based on this study, the average normalized undrained shear strength ($s_u/\sigma'_v$) of Murro clay is equal to 0.32, which is, in the authors' opinion, a considerably high value for a normally consolidated silty clay. Generally, $s_u/\sigma'_p$ is assumed equal to 0.22 for inorganic clays (Mesri, 1975), roughly corresponding to DSS conditions. The observed elevated value might be due to the presence of silt particles and to the organic content of 2-3%. At the same time, the authors' experience would suggest that 0.22 represents a lower boundary strength value for Finnish clays.
4. Preconsolidation pressure from CRS oedometer tests (Karstunen and Yin, 2010) would suggest that Murro clay is normally consolidated. CRS tests on block samples of Murro clay done at Tampere University of Technology show OCR values higher than 1. OCR values greater than 2 are found below the dry crust. Moreover, the friction angle of 37° reported by Koskinen et al. (2002) seems excessively high according to the authors' experience with normally consolidated clays. In the near future, in order to further investigate this aspect, triaxial tests on block samples of Murro clay will be performed at Tampere University of Technology.
5. The long term settlement behavior of Murro test embankment can be satisfactorily modelled using the Soft Soil model in PLAXIS 2D. Being secondary compression not included, the observed deviation of the FE predictions from measurements was likely to be expected. According to the FE analyses, about 90% of the excess pore pressure seems to dissipate after the first 6 years.
6. For a realistic assessment of the change in stress state underneath the embankment, buoyancy should be taken into account. Neglecting the fact that the embankment and the dry crust become partly submerged during consolidation would lead to an overestimation of the effective stresses and, consequently, of the undrained shear strength.
7. The factor of safety of the embankment results magnified after consolidation (FOS=3.14 after 20 years, against FOS=2.02 after 2 days).
8. Undrained shear strength of Murro clay can be realistically modelled using the Soft Soil model in PLAXIS, provided that the initial yield surface is set based on the friction angle of the soil. When default parameters are used, $s_u$ is severely over predicted.

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