**Proceedings of the XVII ECSMGE-2019**
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3
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doi: 10.32075/17ECSMGE-2019-0149

XVII European Conference
ECSMGE 2019
Reykjavik
on Soil Mechanics
and Geotechnical Engineering

# Soil property changes below existing embankments

## Changement de la propriété du sol sous les remblais existant

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**ABSTRACT:** The properties of cohesive soils below embankments change with time after construction. The changes depend mainly on initial properties and structure of the soil layers, the size and distribution of the embankment load and time since the embankment was built. In order to analyse changes in soil properties, 80 embankments in southern Sweden built 100-150 years ago have been studied. Properties determined based on tests beside and below the embankments have been compared with empirically determined values. The results show that for embankments where the load has caused stresses above the preconsolidation pressure, the undrained shear strength ($c_u$), the density and the natural water content have all changed within the same soil volume below the embankment. The sizes of changes in $c_u$ were mostly in the order of 20-25 % of the increase in effective stresses above the preconsolidation pressure. Additional increase in $c_u$ due to creep should also be accounted for. A methodology for estimating the increase in $c_u$, has been developed. The methodology consists of geotechnical investigations outside existing embankments, settlement calculations, a specially proposed laboratory procedure and empirical relationships. Knowledge of the original overconsolidation ratio and shear strengths, the total load applied and perception of changes in $K_0$ due to an embankment, are required to determine the property changes.

**RÉSUMÉ:** Dans les remblais, les propriétés des sols cohésifs changent avec le temps après la construction. Les changements dépendent principalement des propriétés initiales et de la structure des couches de sol, de la taille et de la répartition de la charge du remblai, ainsi que du temps écoulé depuis la construction de ce dernier. Afin d'analyser l'évolution des propriétés des sols, 80 remblais construits il y a 100-150 ans dans le sud de la Suède ont été étudiés. Les propriétés déterminées sur base d'essais à côté et en dessous des remblais ont été comparées à des valeurs déterminées empiriquement. Pour les remblais où la charge a provoqué des contraintes supérieures à la pression de préconsolidation, la résistance au cisaillement non drainée ($c_u$), la densité et la teneur naturelle en eau ont toutes changé dans le même volume de sol sous le remblai. L'ampleur des variations en cu était principalement de l'ordre de 20 à 25 % de l'augmentation des contraintes effectives au-dessus de la pression de préconsolidation. Une augmentation supplémentaire de la résistance au cisaillement due au fluage doit également être prise en considération. Il a été mis au point une méthode d'estimation de l'augmentation en cu. Elle consiste en des études géotechniques à l'extérieur des remblais existants, des calculs de tassement, des procédures de laboratoire spécialement proposées et des relations empiriques. Afin de déterminer les changements de propriété, il est nécessaire de connaître le rapport de surconsolidation et les résistances au cisaillement d'origine, la charge totale appliquée ainsi que la perception des variations de Ko consécutives à un remblaiement.

**Keywords:** undrained shear strength, cohesive soils, embankment, property change determination

## 1 INTRODUCTION

Many railway embankments in Sweden were originally built from mid to late 19th century. There is an ongoing demand for broadening (more tracks) and higher axial loads, why knowledge of the soil properties below the embankments is necessary. Investigations below embankments in operation are expensive and requires extensive planning, why there is a need for other methods to determine the properties.

If a normally consolidated clay is subjected to uniform loading, the clay starts to consolidate.

The consolidation causes settlements and changes in shear strength ($c_u$), void ratio and deformation properties.

Bjerrum (1967) showed that there is a direct relationship between the effective vertical pressure and $c_u$, and that the quota between them depends on the soil plasticity. Therefore, during sedimentation, not only the vertical pressure increases but also $c_u$. After sedimentation, the vertical pressure does not increase any longer but the soil creeps (settlements under constant effective stress) resulting in a decreasing void ratio. A decreasing void ratio leads to an increasing $c_u$ and an increasing preconsolidation pressure. Many researches have reported increasing $c_u$ below embankments, i.e. Tavenas et al (1978), Larsson and Mattsson (2003), D'Ignazio and Länsivaara (2016).

The strengths and stresses in a soil may be illustrated in a two-dimensional diagram as shown in Figure 1 (see also Sällfors et al, 2017). The changes in stresses and strength during sedimentation are shown as a stress path going from A to B, along the $K_0$-line. The active shear strength, $c_u^a$, for a stress situation as point B, is found on the y-axis. A geometrical construction and assuming c'=0, yields that:

$$\frac{c_u^a}{\sigma'_{cv}} = \left(\frac{sin\theta'}{1+sin\theta'}\right) \qquad (1)$$

where $c_u^a$ (kPa) is active, undrained shear strength, $\sigma'_{cv}$ (kPa) is preconsolidation pressure and $\phi'$ (degrees) is frictional angle. Equation 1, with $\phi'$=30°, reduces to:

$$c_u^a = 0{,}33 \cdot \sigma'_{cv} \qquad (2)$$

Creep, after a vertical loading or sedimentation, leads to an expansion of the yield envelope, resulting in an overconsolidation and increasing $c_u$. According to Mesri and Hayat (1993), $K_0$ increases as a result of secondary compression. An expansion of the yield envelope with an increasing $K_0$, is given in Figure 2.

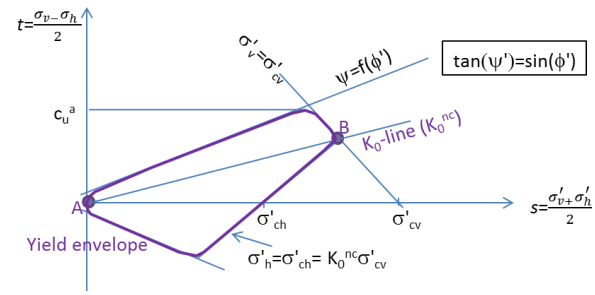


*Figure 1. Stresses showed in a yield envelope.*

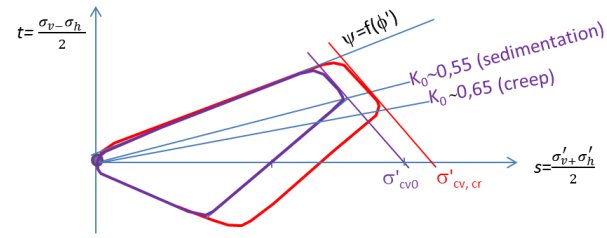


*Figure 2. Expansion of the yield envelope during creep ($K_0$ increased).*

## 2 HYPOTHESIS AND METHODOLOGY

Simplified methods to estimate the increase in $c_u$ due to consolidation from a vertical load without investigations below the embankments have been presented by e.g. Larsson et al (2007), Andersson-Berling et al (2011), Ye and Guo (2010). The methods all use the estimated increase in preconsolidation pressure to calculate $c_u$. Mostly, the increase of the preconsolidation pressure only includes the primary consolidation, i.e:

$$\Delta\sigma'_{cv} = \Delta\sigma'_v - \sigma'_{cv0} \qquad (3)$$

Also, the natural water content and density change due to consolidation. Measurements of the changes have been reported by many researchers but no method to determine the changes without tests, have been found.

The hypothesis is that it is possible to calculate the increase in $c_u$ due to both primary and secondary consolidation by knowing the following parameters:

- shear strength in situ
- overconsolidation ratio in situ
- overburden load (weight of embankment)
- stresses below embankment (compensation for remaining excess pore pressure)
- settlements, including creep

It is believed that these parameters may be determined by studying the properties of the soil beside the embankment, making rigorous settlement calculations (including creep) together with a newly proposed laboratory procedure. The hypothesis is further that changes in natural water content and density may be determined using the proposed laboratory procedure.

## *2.1* Field- and laboratory tests

The soil beside the embankment should be investigated in the field and the laboratory. Field tests include CPT, pore pressure gauges and piston sampler. Laboratory tests include oedometer tests, water content, liquid limit, density, direct shear tests and active, undrained triaxial tests.

A special laboratory procedure has been developed to measure properties of consolidated samples. Firstly, the sample is consolidated in a triaxial cell for the calculated additional vertical stresses from the embankment, including creep. During the test the drained water is captured and weighted. Thereafter a piece of the sample is cut and trimmed into a direct shear ring. The ring is weighted and the density determined. A shear test is performed and the shear strength determined. The natural water content and liquid limit of another part of the consolidated sample are measured.

## *2.2* Calculations

The vertical stresses in the soil below the embankment are calculated using for instance Boussinesq's equation. The stresses may be derived from settlement calculations.

During the many years that have passed since the embankments were built, settlements have occurred. In order to maintain the same level of the railway tracks, compensation by additional filling has been performed. The amount of filling has though seldom been documented which means that the total load is usually unknown. To determine the total weight of the embankment, including compensation for settlements, settlement calculations are performed. The calculations include primary and secondary consolidation. Settlement calculations are also used to determine the stresses in different layers necessary to give the creep settlements calculated. These stresses minus the preconsolidation pressure in each layer are used to calculate the increase in $c_u$ due to creep. They are also used as loads in the proposed laboratory procedure.

The increase in $c_u^a$ (where $c_u^a$ = undrained shear strength in active shear direction) due to consolidation, has been calculated based on the empirical relationship presented in equation 2. The increase in $c_u^{DS}$ after consolidation, $\Delta c_u^{DS}$, may be assumed to be related to the strength in active $c_u^a$ and passive $c_u^p$ directions as:

$$\Delta c_u^{DS} = \Delta 0{,}75 c_u^p + \Delta 0{,}25 c_u^a \qquad (4)$$

Using the following equations, and an assumption that ϕ'=30° (see also equation 2):

$$\Delta c_u^a = 0{,}33 \Delta \sigma_{cv}' \qquad (5)$$

$$\Delta c_u^p = 0{,}33 \Delta \sigma_{ch}' \qquad (6)$$

$$\frac{\sigma_{ch}'}{\sigma_{cv}'} = K_0^{nc} \qquad (7)$$

equation (4) reduces to:

$$\Delta c_u^{DS} = 0{,}33 \Delta \sigma_{cv}' (0{,}25 + 0{,}75 K_0^{nc}) \qquad (8)$$

With $K_0^{nc}$=0,6 one gets that:

$$\Delta c_u^{DS} = 0{,}7 \Delta c_u^a = 0{,}231 \Delta \sigma_{cv}' \qquad (9)$$

The increase in $\Delta \sigma_{cv}'$ has been calculated in two ways; with and without respect to creep consolidation. Without creep:

$$\Delta \sigma_{cv}' = \Delta \sigma_v' \qquad (10)$$

With creep the increase has been calculated using the calculated increase in vertical stress needed to obtain the creep consolidation. This vertical stress is a fictitious stress, not found in the soil. The increase is calculated as:

$$\Delta\sigma'_{cv} = \Delta\sigma'_{creep} \quad (11)$$

The shear strength below the embankment is equal to:

$$c^{DS}_{u,below} = c^{DS}_{u,in\ situ} + 0{,}23\Delta\sigma'_{cv} \quad (12)$$

Based on the investigations performed so far it may be concluded that an increase in shear strength has occurred below all embankments studied where the preconsolidation pressure has been exceeded. The increase in effective stresses and shear strengths are dependent on the remaining excess pore pressure, which has to be accounted for and studied for instance with settlement calculations.

# 3 TEST SITE FELLINGSBRO

## *3.1* Field- and laboratory tests

The test site Fellingsbro is located along the railway line between Örebro and Arboga 150 km west of Stockholm. The railway line was built in 1857. The railway embankment has a height above the surrounding ground surface of 1,5 m, a width at the track level of 5 m and is surrounded by flat farmland.

The natural soil at Fellingsbro consists of 0,5 m dry crust clay on postglacial gyttja-bearing clay and clay followed by varved glacial clay, down to about 8 m of depth, followed by sand and moraine.

Geotechnical field and laboratory investigations were performed at the test site during 2005. During 2016 and 2017 the Swedish Geotechnical Institute performed complementary geotechnical field investigations outside the railway embankment and below the existing embankment including cone penetration piezocone tests (CPTU), field vane tests, undisturbed piston sampling, disturbed soil sampling and installation of pore pressure piezometers with measurements once a day.

The geotechnical laboratory investigations have included soil classification, determination of water content, liquid limit and density, oedometer tests, creep tests, fall cone tests, direct shear tests and active, undrained triaxial tests. On undisturbed samples from outside the embankment the special laboratory test has been used.

## *3.2* Measured geotechnical soil parameters outside and below the embankment

The groundwater level outside the embankment, measured with pore pressure gauges at two levels in the clay, varies over a one-year cycle between 0-0,6 m below the ground level.

The gyttja-bearing clay and clay layer outside the embankment is overconsolidated at the top. The overconsolidation ratio decreases towards depth to slightly overconsolidated. Below the existing railway embankment, the preconsolidation pressures have increased between 15–20 kPa, compared to values outside the embankment.

The gyttja-bearing clay and clay layer outside the embankment have a $c_u^{DS}$, determined by direct shear tests, varying between 10 kPa at 2 m depth increasing to 12 kPa at the bottom of the clay layer. The $c_u^a$, determined by triaxial tests, varies between 13 and 15 kPa. Below the existing railway embankment, the $c_u^{DS}$, has increased between 5–7 kPa, and $c_u^a$, between 7–9 kPa. Undrained shear strength has also been determined by fall cone test and field vane test.

Undrained shear strength determined by direct shear test on samples taken outside the embankment are 10 to 30 % higher than values determined by fall cone and field vane test. Below the embankment undrained shear strength determined by direct shear test are 0 to 95 % higher than values determined by fall cone and field vane test. Undrained shear strength

determined by active triaxial test, direct shear test and field vane test is shown in Figure 3.

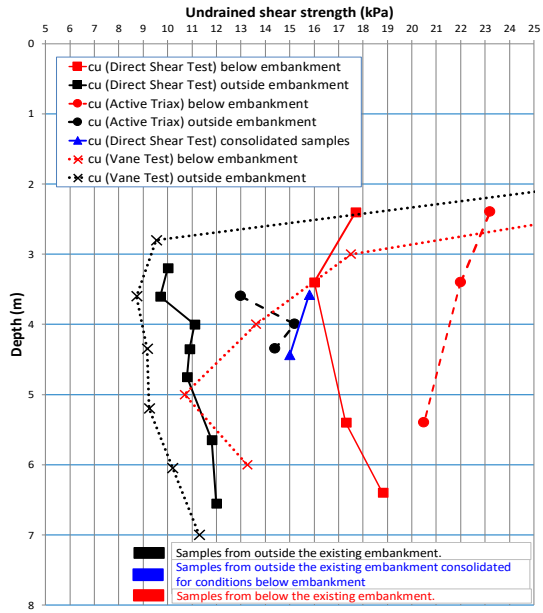


*Figure 3. $c_u$ determined by active, undrained triaxial test, direct shear test and field vane test (depth for samples outside the embankment are corrected due to settlement).*

Outside the embankment, the natural water content is varying between 100 and 125 % in the gyttja-bearing clay, below the dry crust, and between 60 and 85 % in the underlying clay. Below the railway embankment, the natural water content has decreased between 20 and 30 % in the upper layer and between 10 and 20 % in the lower layer.

Outside the embankment, the density is varying between 14 and 15 kN/m$^3$ in the gyttja-bearing clay, below the dry crust, and between 15 and 16 kN/m$^3$ in the underlying clay. Below the railway embankment, the density has increased between 1 and 1,5 kN/m$^3$ in the upper layer and between 0 and 0,5 kN/m$^3$ in the lower layer.

Liquid limit of the gyttja-bearing clay, 30 m outside the embankment varies between 80 and 105 %. Towards the embankment the liquid limit decreases, to about 10-20 % lower values just outside the foot of the embankment. The liquid limit then decreases towards the depth and varies in the underlying clay between 55 and 70 %. The liquid limit values below the embankment correspond to the values from the foot of the embankment.

## 3.3 Calculated stresses and settlements below the embankment

The embankment in Fellingsbro was originally built in 1857, and settlements should therefore be almost fully developed. Calculation of settlements has been carried out with Embankco (Larsson et al, 1997), Trimble Novapoint GS Settlement and Plaxis BV 2D for a total load time of 150 years. In Novapoint and Plaxis the creep model Creep-SCLAY1S was used (see Sivasithamparam et al, 2015). In the calculations, assumptions have been made that compensation for settlements (i.e., filling of the track) has been performed after 30, 70 and 100 years.

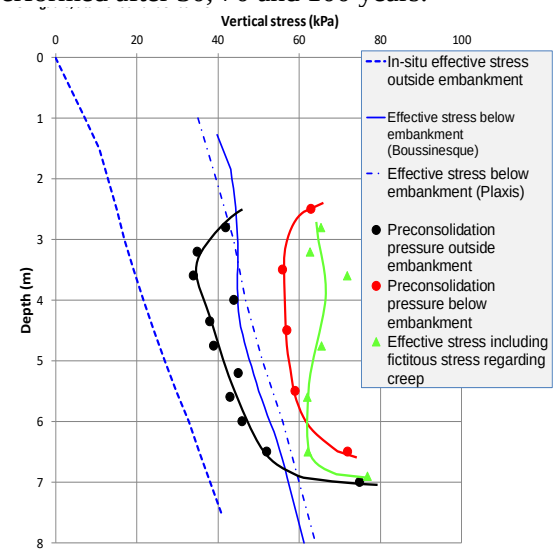


*Figure 4. Measured preconsolidation pressure and calculated stresses below the embankment (depth for values outside the embankment have been corrected for settlements).*

As shown in Figure 4, the embankment has caused stresses above the preconsolidation pressures below the dry crust. Calculation of settlements, including creep settlements and compensation for adjustment of settlements, shows that the embankment has settled approximately 0,8 m, which is consistent with measured

total thickness of the embankment. Full primary and secondary consolidation has been obtained (excess pore pressures less than 2 kPa). Effective stresses below the embankment 150 years after the construction, calculated with Embankco, and Plaxis are shown in Figure 4.

To also include the effect of the increase in strength due to creep settlement, a fictional creep stress was calculated, see section 2.2. The effective stresses for this load are shown in Figure 4 and used as the consolidation load for the proposed laboratory procedure. These are somewhat higher than the measured preconsolidation pressures below the embankment.

### *3.4* Calculated increase of shear strength

The shear strength below the embankment has been calculated according to equation 12 (both with and without creep). The values are compared with measured values in Figure 5. In the diagram also results from the proposed laboratory procedure are given. It may be concluded that $c_u$ calculated only based on the effective stresses below the embankment give too low values.

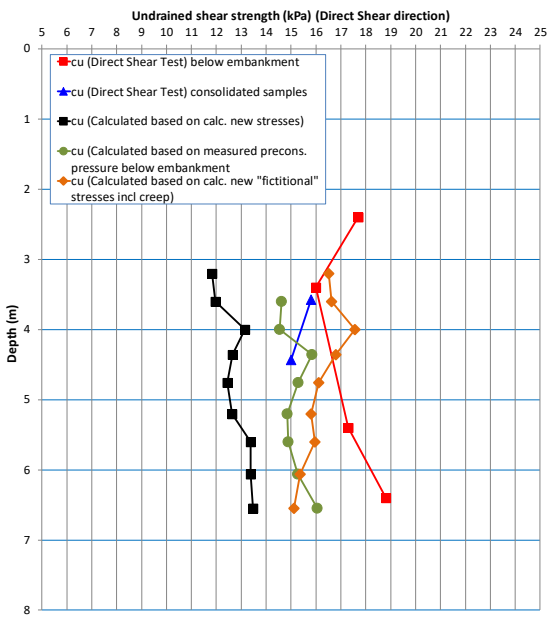


*Figure 5. Measured and calculated $c_u^{DS}$.*

## 4 OTHER TEST SITES

In addition to the test site Fellingsbro, four further sections have been studied. Strength increase has also been studied briefly for 80 sections where geotechnical investigations had been performed below and outside of the track within design for axial load increases. In addition, also the overconsolidation ratio for naturally unloaded clay has been analysed for 75 test sites based on geotechnical investigations carried out within new construction projects for railway. The purpose of the compilations and comparisons is to study the differences between values measured in the field, theoretical analyzed and laboratory tested. Some examples of results from the inventory are presented below.

### *4.1* Overconsolidation ratio

The overconsolidation ratio outside the embankment versus depth has a great impact on possible increase of shear strength below the embankments. Therefore, oedometer tests (CRS) have been performed outside embankments at five test sites on each meter of the clay layer down to the bottom of the layer. Long-term pore pressure measurements have been performed to determine the variations during at least one year.

The five test sites have been complemented with results from new railway construction projects, where oedometer tests (CRS) and short-term pore pressure measurements or measurements of groundwater pipes have been performed. The complementing sites are located in the vicinity of the five test sites. Figure 6 shows the overconsolidation ratio (OCR) for all the invented sites. The results have been grouped based on the predominant soil type; clay/varved clay (green dots), gyttja-bearing clay/somewhat sulphide-bearing clay overlaying clay/varved clay (red dots).

At the analyzed sites the clay layers are overconsolidated down to depths varying between approximately 2 and 10 m and below these depths, clay layers are slightly overconsolidated. The OCR profiles determined for the five test

sites are well matched with the invented results from the additional test sites.

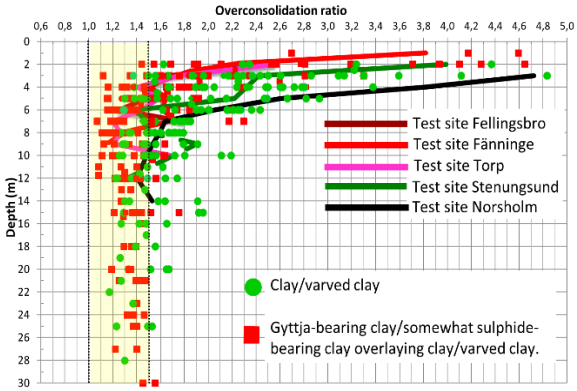


*Figure 6. Overconsolidation ratio for the five test sites (solid lines) together with additional test sites (dots).*

## *4.2* Measured increase of shear strength below existing embankments

During the 1990's and early 2000's, extensive stability investigations were carried out in connection with upgrading of railways to 25 tons of axle load. Field vane test and/or piston sampling and fall cone test were made under and outside existing embankments, to determine the $c_u$ of the clay.

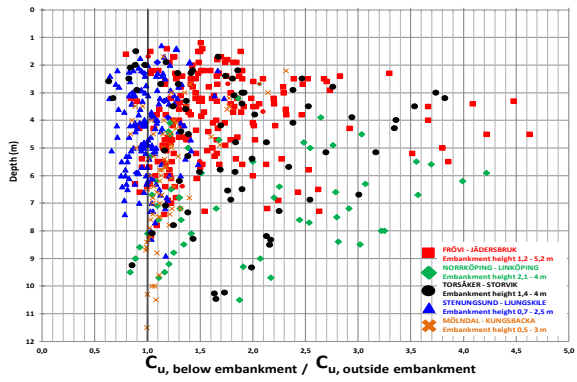


*Figure 7. Measured increase of $c_u^{cone,vane}$ below existing railway embankments.*

Figure 7 shows the quota between measured $c_u^{cone,vane}$ below and outside the embankments for approximately 80 test sites. The depths below the embankments have been corrected to account for the settlements during 100-150 years, in order to compare clay from the same layers (outside and below the embankment). The comparison shows that strength increase has been obtained up to 4.5 times the original shear strength. However, it is doubtful whether the highest values (above 3) are reasonable. One reason for the high values may be uncertainty of the results from the field vane tests. Some values in Figure 7 are lower than zero, which is believed to be due to the natural strength variation in the clay and the accuracy of the methods used.

# 5 DISCUSSION

Increase of shear strengths below embankments, have been measured. In many of the test sites studied only field vane tests have been performed below the embankments. At the Fellingsbro test site direct shear tests have been performed, which show higher values of $c_u$ compared to field vane tests. The differences are high enough to have important influences on the calculated slope stability, why the direct shear test is recommended.

To calculate the increase in shear strength under embankments it is important to have detailed knowledge of the shear strengths, effective stresses and preconsolidation pressures in the natural soil outside the embankments. Therefore, detailed geotechnical investigations, including pore pressure measurements, are required. Advanced laboratory methods should be used. Calculations of settlements should be performed using a sophisticated soil creep model. Important input into these calculations are applied load (embankment height) including compensations for settlements. Calculations of increase in shear strengths should take into account the increase due to creep, but a cautious value should be chosen.

There is no method available for calculation of changes in other soil parameters but comparison of changes in natural water content and density done by the special proposed laboratory procedure and values measured on undisturbed samples below the embankment, have shown that around the same values are obtained.

## 6 CONCLUSION

The changes in soil properties due to consolidation below existing embankments built on soft clay, have been studied and a methodology for calculations and laboratory investigations of the changes, have been developed.

Based on the investigations performed so far, it may be concluded that an increase in shear strengths have occurred below all embankments studied in soil layers where the preconsolidation pressures have been exceeded. Knowledge of the precon-solidation pressure in situ is therefore basic information needed. The field vane test has shown to give too low values beneath embankments compared to direct shear tests and triaxial tests (also when the effect of shear direction is taken into account).

The total thickness of the embankment applied is important input to settlement calculations. Often this is unknown and therefore, for instance, geo-radar along the track line may be performed. Another important factor to study carefully is the load spreading, both with depth and horizontally along the section.

Determination of the increase in $c_u$ is possible with detailed knowledge of the in situ conditions and the total thickness of the embankment, including compensation for settlements. The increase due to creep has also to be accounted for.

More studies will be performed of the changes in soil parameters below existing embankments, and presented in 2019.

## 7 ACKNOWLEDGEMENT

The Swedish Transport Administration and the Swedish Geotechnical Institute have founded the investigation.

## 8 REFERENCES

Andersson-Berlin, K. Länsivaara, T., Käkelä, E. Silvast, M. 2011. A method for classification of the Finnish railroad network in accordance with forthcoming EU regulation. *International symposium on railway geotechnical engineering, Proceedings*, **1, 167-176**. Paris.

Bjerrum, L. 1967. Engineering geology of Norwegian normally consolidated marine clays as related to settlements of buildings (Seventh Rankine Lecture), *Geotechnique* **17**, 81-118.

D'Ignazio, M.D. Länsivaara, T. 2016. Strength increase below an old test embankment in Finland. *Proceeding of the 17th Nodic Geotechnical Meeting*, 357-366. Reykjavik.

Larsson, R. Bengtsson, P.E. Eriksson, L. 1997. Prediction of settlements of embankments on soft, fine-grained soils. Calculation of settlements and their course with time. Information 13E, Swedish Geotechnical Institute, Linköping.

Larsson, R. Mattsson, H. 2003. Settlements and shear strength increase below embankments - long-term observations and measurement of shear strength increase by seismic cross-hole tomography. Report 63. Swedish Geotechnical Insitute, Linköping.

Larsson, R. Sällfors, G. Bengtsson, P.E. Alén, C. Bergdahl, U. Eriksson L. 2007. Skjuvhållfasthet, utvärdering i kohesionsjord. Information 3. Swedish Geotechnical Institute, Linköping.

Mesri, G. Hayat, T.M. 1993. The coefficient of earth pressure at rest. *Canadian Geotechnical Journal* **30**, 647-660.

Tavenas, F. Jean, P. LeBlond, P. Leroueil, S. 1983. Permeability of natural soft clays, 2: Permeability characteristics. *Canadian Geotechnical Journal* **20**, 283-305.

Sivasithamparam, N. Karstuen M. Bonnier, P. 2015. Modelling creep behaviour of anisotropic soft soil. *Computers and Geotechnics*, **69**, 46-57.

Sällfors, G. Larsson, R. Bengtsson, P.E. 2017. Bestämning av den ordränerade skjuvhållfastheten med specialiserade metoder i praktiska tillämpningar. Rep 4. TRV, Borlänge.

Ye, G. Guo, Y. 2010. Tests and analyses on shear strength increment of soft soil under embankment fill. *Electronic Journal of Geotechnical Engineering* **15**, 1-9.