DESIGN OF 15 METER HIGH RAILWAY EMBANKEMENT ON LOOSE SILT AND CLAY

Ibrahim Rashid, WSP/COWI Sweden, Fredrik Clifford, WSP Sweden

KEYWORDS

Ground Improvement methods, FEM modelling, transport infrastructure, railway embankment, observational method, pre-consolidation, preloading, vertical drains

ABSTRACT

To construct a 15-meter-high railway embankment through pre-consolidation using vertical drain with surcharge on settlement-prone silty clay is rare in Sweden. This poses significant challenges in terms of foundation design, assessment of preloading time, as well as production and environmental considerations.

A combination of analytical and FEM modelling in 2D and 3D has been carried out to assess strength growth in soil and predict settlement development. The FEM model mirrors the production sequence, as the railway embankment will be built incrementally with different laydown times for each stage.

1. INTRODUCTION

The Hallsberg-Stenkumla project is part of the Double Track Hallsberg-Degerön and consists of approximately 13 kilometers of rail, with about 12 kilometers in a new alignment. The project also includes construction of ten railway bridges, a road bridge and a 2.4-kilometer-long railway tunnel. Trafikverket (The Swedish transport administration) is developing this area to create grade separated intersections to increase capacity and enable more environmentally friendly transportation.

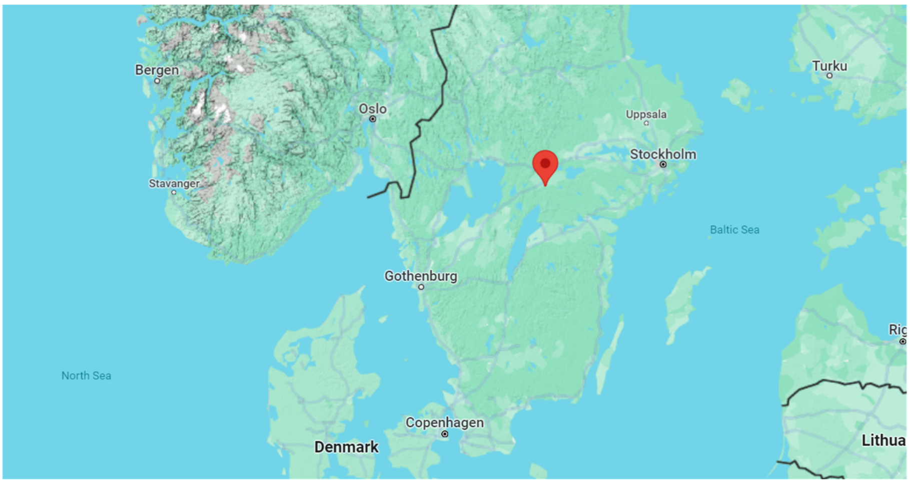

Figure 1: The project Hallsberg-Stenkumla is situated in the southern part of Sweden, approximately 200 km west from Stockholm.

This article aims to describe the design process of pre-consolidation using vertical drains with surcharge for the foundation of an approximately 15 meter high railway embankment on a 10 meter loosely layered sediments of silt and clay at Tälleslätten. The embankment is 150 m wide and includes support berms to improve the stability, see Figure 2.

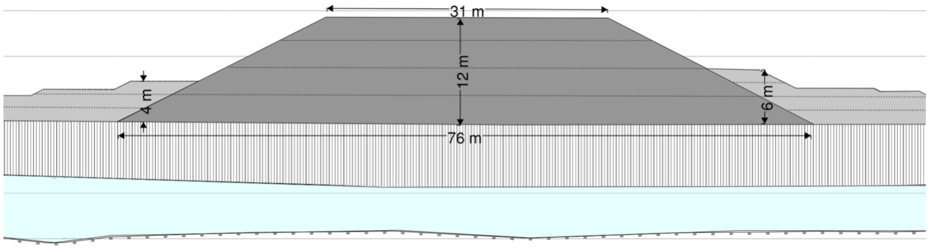

Figure 2: Typical cross section of the railway embankment over Tälleslätten. Hight of the embankment is without surcharge.

Three foundation methods have been investigated: Soil replacement, pile embankment and pre-consolidation using vertical drains with surcharge. Soil replacement and pile embankment were found to have approximately three times the climate impact as pre-consolidation with vertical drains.

A combination of analytical and Finite Element Modelling (FEM) modelling in 2D and 3D has been carried out to assess strength growth and predict settlement development of the embankment. The FEM model mirrors the production sequence, as the railway embankment will be built in stages with different consolidation time for each stage. These models will be iteratively updated during the construction with field data from monitoring and follow up of the soil behavior to adjust the prognosis in accordance with the observation method.

2. GEOTECHNICAL CONDITIONS

The geology of the area consists of an upper layer of sand and silty clay underlaid by silt. The sediments overlay a layer of sand.

The clay is silty with layers of silt and sand. The thickness of the clay ranges from approximately 1 - 10 meters and is loose to very loose. The water content varies between approximately 22 - 40 percent, and the liquid limit ranges from approximately 25 - 40 percent. The clay is normally consolidated with an OCR of 1.1 – 1.2. The undrained shear strength is about 15 kPa and increases with depth. Figure 3 shows the layer sequence on the left and the undrained shear strength for the silty clay on the right.

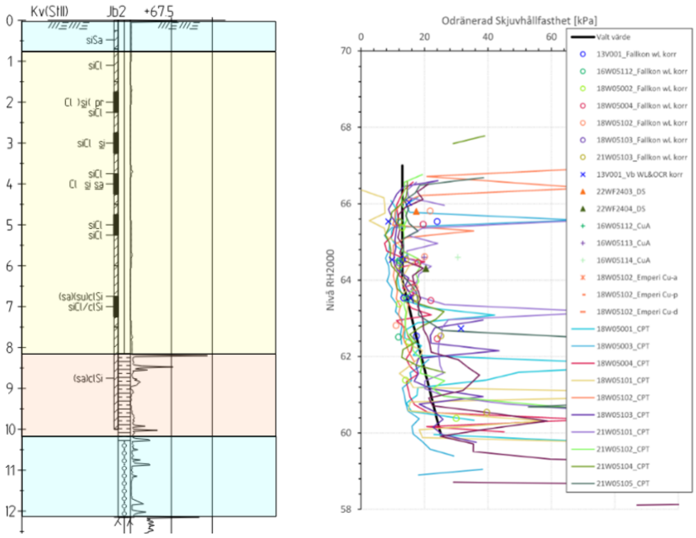

Figure 3: Layer sequence and the undrained shear strength of the area.

The high silt content in the clay made it challenging to obtain undisturbed samples and has led to uncertainty in the interpretation of laboratory results. Only undisturbed samples of high quality have been considered to evaluate the parameters of clay. To capture the deformation behavior of the silty clay, CRS oedometer-test and incremental oedometer tests have been conducted. The value of the undrained shear strength is based on CPT-data, shear vane test, direct shear test and triaxial test.

3. DESIGN BASIS

The chosen reinforcement method consists of pre-consolidation with surcharge and vertical drains and support berms. The design has been an iterative process to optimize the consolidation degree and time to ensure that the embankment is stable throughout the process.

The design has been carried out according to the document TRVINFRA-00230 which is the design requirement of the Swedish Transport Administration. The design includes stability checks, short and long-term settlement prediction as well as the design of vertical drains.

The construction time is approximately 48 months, of which the construction of the embankment accounts for 24 months. The remaining 24 months are required to consolidate the clay. The first step is the installation of the vertical drains. The embankment can start to be constructed at the earliest 3 months after their installation. For the construction phase, safety class 2 (SK2) is used, while safety class 3 (SK3) is used for the operational phase.

The spacing between the drains has primarily been determined based on the requirement for strength increase in the clay and the time available for consolidation.

The embankment is constructed through sequential filling in 6 stages. Between each stage, the clay is allowed to consolidate to enable sufficient increase in shear strength to ensure the embankment is stable before the next stage. The additional stress and strength increase in the clay are calculated according to the equation shown below, but also considering that full consolidation is not achieved for each load stage:

$$\Delta c_u = a \cdot [\sigma'_{v0} + \sum_{n=1}^{i}(\Delta\sigma_{tot.n} \cdot U_n) - \sigma'_{c0}] \qquad [1]$$

Where σ'_{v0} is the in-situ effective stress, $\Delta\sigma_{tot.n}$ is the increase in vertical total stress induce by the embankment fill. U_n is the degree of consolidation and σ'_{c0} is the in-situ pre-consolidation pressure. The parameter 'a' is a constant that depends on the type of soil and the anisotropic behavior. This constant has been set to 0.1 TRVINFRA-00230 limits it to that if the design is based solely on calculations. This is a conservative assumption. Empirical relationships give an ‘a’ factor of 0.2 for the specific soil in direct shear, see Figure 4. The design will be optimized during construction with the data collected in the field during construction.

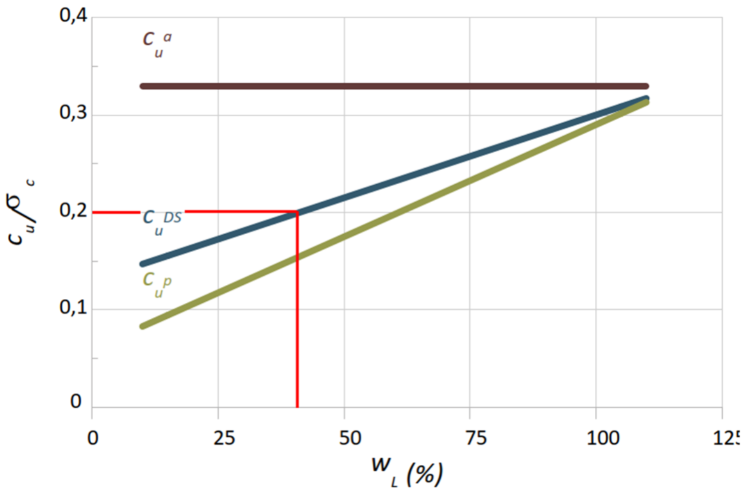

Figure 4. Empirical relationship between undrained shear strength (Cu) for different values of liquid limit (W_L), (SGI Vägledning 8).

The requirement for stability during the construction stages has driven the size and consequentially the number of load steps. Stability calculations have been performed using Slope/W software with Morgenstern/Price method for both the construction phase and the operational phase. This software was used as the 'a' parameter can be chosen manually while in PLAXIS it is not possible. In the construction phase, a traffic load of 15 kPa is applied, and in the operational phase, a train load equivalent to 32 kPa is applied.

Consolidation and settlements are calculated using FEM in PLAXIS-2D. The soil model "Soft soil creep" has been used for the clay, which is an advanced stress-dependent model that considers creep behavior. The model can be verified against the soil tests performed in the laboratory. It can also be validated and optimized during the staged construction. The Mohr-Coulomb soil model has been used for the remaining soils. The design of the surcharge is performed to ensure compliance with the settlement requirements.

The settlement model has been built with different calculation stages and consolidation time between each stage to reflect reality. Figure 5 below shows several calculation steps from the PLAXIS-2D model.

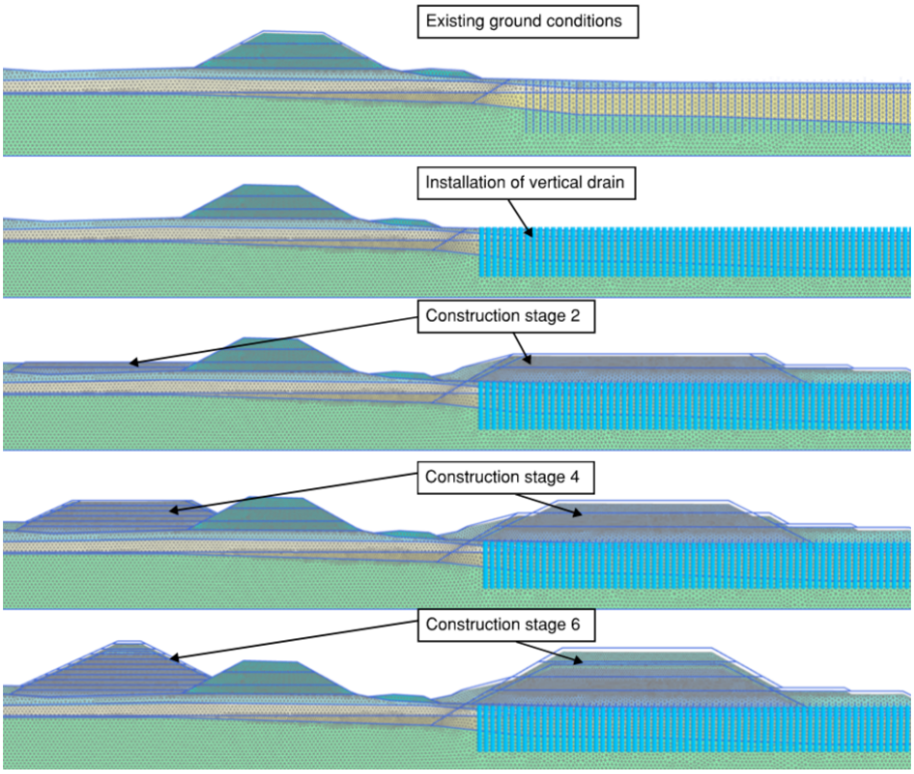

Figure 5: Calculation steps from the PLAXIS-2D model.

4. ANALYSIS

The requirements in the serviceability limit state mean that total, differential and transverse settlements, shall be fulfilled during the operational period (40 years). To fulfill the above stated criteria, there should not be any significant primary consolidation remaining in the clay once the surcharge is removed and the risk of secondary creep settlements should be low.

Settlement calculations has been carried out in several sections as shown in the figure below.

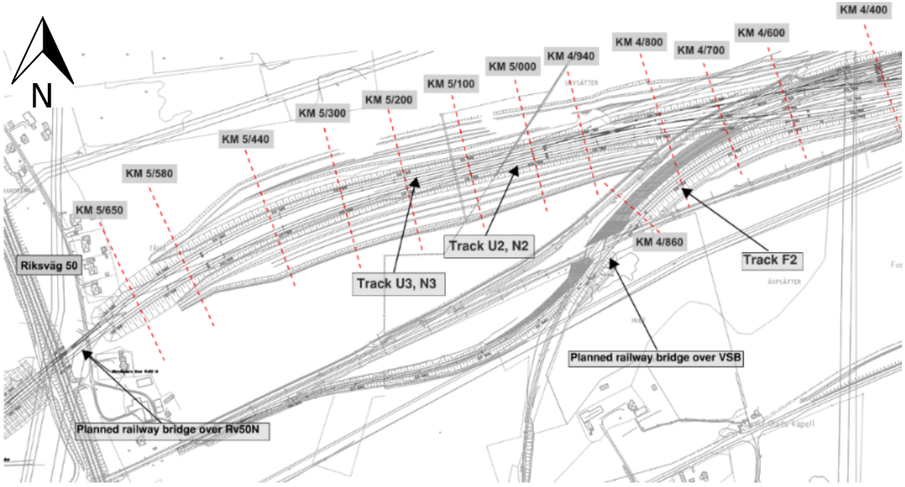

Figure 6: Planview with the calculation sections.

The settlement model has not been able to be verified against a trial bank or settlement data from nearby structures. Therefore, it has been decided to carry out advanced laboratory test to capture the deformation properties of the clay. The soil behavior in PLAXIS has been verified against the performed laboratory tests.

The main parameters in the used soil model for clay are the following:

- λ^* (modified compression index)
- κ^* (modified swelling index)
- μ^* (modified creep index).

The parameters λ^* and κ^* describe the stiffness and unloading properties, while μ^* describes the creep properties.

The parameters λ^* and κ^* have been evaluated through with simulation of CRS oedometer-test and incremental oedometer tests in PLAXIS. This was done by curve fitting against laboratory results. The creep parameter (μ^*) has only been determined through the simulation of incremental oedometer tests in PLAXIS. Empirical relationships have been chosen as input values for the selection of parameters (λ^*, κ^*, and μ^*). The chosen empirical relationships are based on the document SGI Info 13 (Swedish geotechnical institution, information 13). The equations are shown below.

$$\kappa^* \approx \frac{2 * \sigma'_v}{M} \qquad [2]$$

$$\lambda^* \approx \frac{2 * \sigma'_{cv}}{M_L} \qquad [3]$$

$$\mu^* \approx \frac{w_n^{1,5}}{75} \qquad [4]$$

The figure below illustrates an example of curve fitting for CRS oedometer-test and incremental oedometer tests.

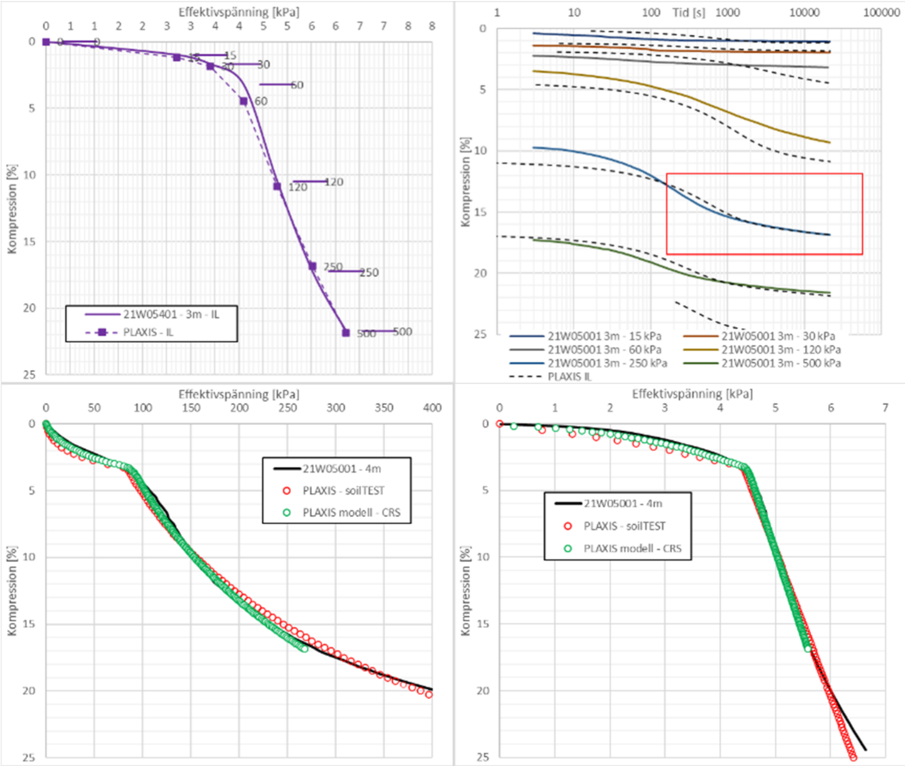

Figure 7: Example of curve fitting against laboratory tests for CRS oedometer-test and incremental oedometer test.

The figure below presents a summary of the parameters obtained through the soil test in PLAXIS.

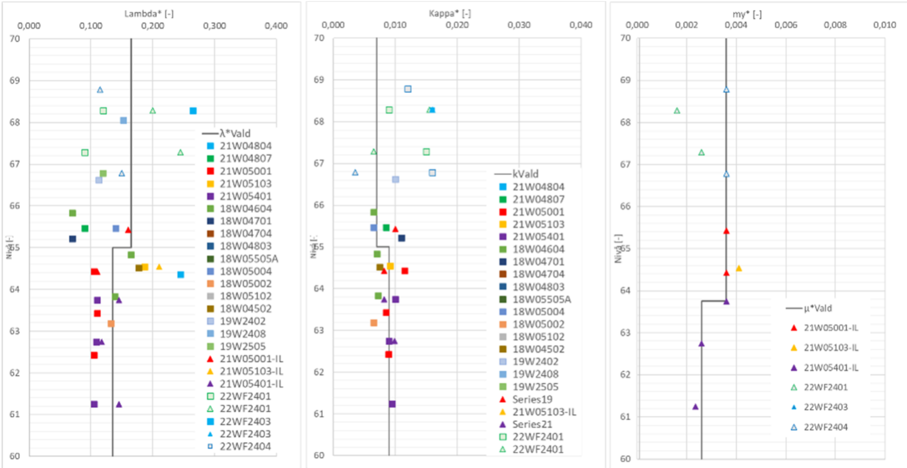

Figure 8: Summary of the parameters obtained through the soil test simulation in PLAXIS (CRS test (squares), Stepwise oedometer test (triangle)).

The permeability of the clay has been assumed from CRS oedometer-test based on the pre-consolidation pressure. Changes in permeability with respect to the loading conditions are adjusted in PLAXIS by using the parameter C_k. C_k can be determined using the equation below.

$$C_k = \frac{1}{\beta_k} * e_0 \qquad [5]$$

5. MONITORING

A monitoring program has been developed to describe the control and follow-up measurements that needs to be performed during the construction period to ensure that the geotechnical requirements in the serviceability limit state and the failure limit state are met.

Serviceability limit state

The controls in the serviceability limit state require that the allowable total, transverse and longitudinal settlements are not exceeded in the design period. The total settlement must not exceed 20 cm. The maximum tilt on each side of the track is 1%. In the longitudinal direction, the differential settlement must not exceed 5 cm over a distance of 10 m.

To verify that the settlement requirements are met, the following measuring equipment is used:

Measurement of reference markers and settlement tube monitoring are used to track the settlement progress at individual points as well as in sections. The purpose of these measurements is to compare actual settlements with the calculated settlement profiles, thus verifying the calculation model that formed the basis for the design. The measurements also provide an indication of the development of consolidation settlements and whether there is a need to revise the time between stages.

Pore pressure measurements are carried out to monitor the consolidation in the clay. These measurements provide the basis for calculating the increase in effective stress in the clay, as well as the increase in pre-consolidation pressure and degree of consolidation. However, during the compression of the clay and subsequent settlement, there is a risk that the pore pressure probes may follow the downward movement of the soil. This means that the original measurement levels will change, and the excess pore pressures may not converge to zero but towards a slightly higher value. This is something that needs to be considered when evaluating the measurement results. To get an understanding of the actual settlement towards depth, settlement plate measurements can be used as a complement to the other prescribed measurements.

When the excess pore pressures have dissipated, the consolidation process is considered to be completed. By comparing the increase in pre-consolidation pressure with the effective stress in the clay after the removal of excess load, the degree of overconsolidation (OCR) can be assessed. The design is based on an OCR value $\geqslant 1.1$ in the middle of the clay layer, to reduce the risk against potential creep settlements that may occur during the operational period.

Settlement monitoring during the construction period is compared with the predicted settlements, allowing an evaluation of the assumptions that formed the basis for the calculation model. If the measured settlements deviate from the predicted settlement range, the calculation model may need to be adjusted, and a new forecast established. If the excess pore pressures have dissipated completely, consolidation settlements should be completed. The measurements also provide an overall understanding of the settlement process and is an indication of ongoing creep settlements.

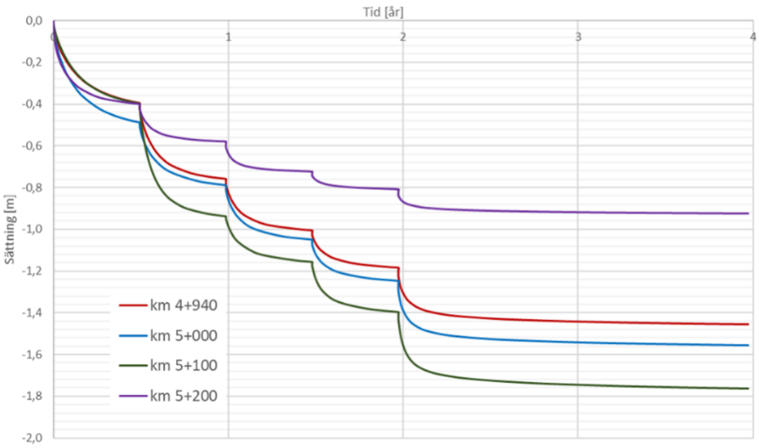

Figure 9: Settlements as a function of time for different sections during a period of four years.

Settlements and consolidation processes have been checked with PLAXIS. Sensitivity calculations have been performed by varying the coefficient of variation by 10% from its base value for parameters λ*, μ* and k_{Init}. This is done to understand how these factors affect consolidation and settlements. Figure 10 shows the results of the sensitivity analysis for parameter λ* (modified compression index). This parameter is governing the settlement process during the construction phase. In the operational phase, settlements are mainly comprised of creep settlement, which is controlled by the parameter μ*.

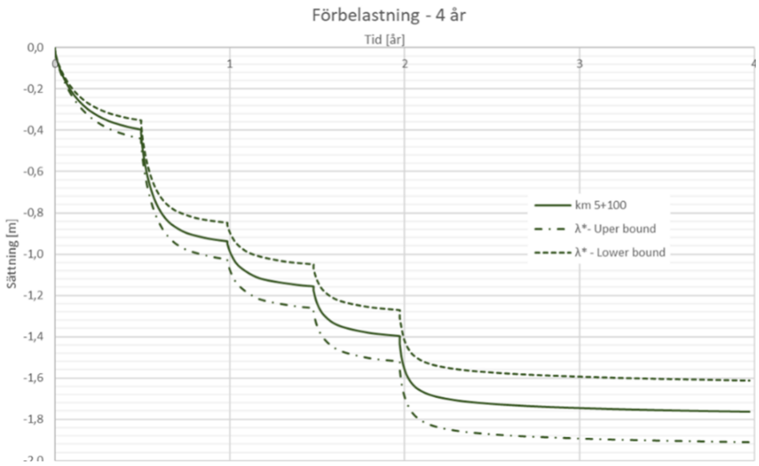

Figure 10: Sensitivity of the parameter λ during four years of preloading.*

Ultimate limit state

The requirements in the ultimate limit state are placed on the minimum acceptable safety factor against stability failure. When designing with partial factors (design values of material parameters), the safety factor F_{EN} should exceed 1.0 during the construction phase (safety class 2) and 1.1 during the operational period (safety class 3).

To meet the safety requirements during the staged loading of the embankment, it is necessary for the undrained shear strength of the clay, c_u, to increase along with consolidation. The measured increase in undrained shear strength forms the basis for decisions regarding when the next load step can be applied or if the duration of each stage needs to be adjusted. To evaluate the above, pore pressure measurements and the increase of the undrained shear strength is calculated using the data from CPTu (Cone Penetration Testing) probes.

CPTu probes are used to measure the net cone pressure at different points in time. By comparing this with the net cone pressure from the control probing (baseline measurement) in the undisturbed, naturally deposited clay, the increase in strength of the clay can be calculated with the equation below.

$$c_u = c_{u0} \cdot \frac{(q_t - \sigma_v)}{(q_{t0} - \sigma_{v0})} \qquad [6]$$

where q_t and σ_v are the current total cone resistance and overburden pressure evaluated during the CPTu probing. The parameters q_{t0} and σ_{v0} and corresponding values from the in situ measurement. The expected strength increase can also be determined from vertical effective stress calculated from pore pressure measurements using the relationship below where a can be assumed to be in the order of 0,2 according to empirical relationship, see Figure 4.

$$c_u = a \cdot \sigma_c' \qquad [7]$$

6. CONCLUSIONS

The designed foundation method is cost-effective with low environmental impact. However, one condition is that the construction schedule allow for the consolidation of the clay and that the fill material should be available nearby to minimize transportation distance. This design method requires rigorous geotechnical investigation and laboratory testing. During the construction stage, extensive monitoring is required as well as optimizing the design during the construction phase.

Some conclusions from the project:

To obtain undisturbed samples, CRS tests and evaluation of permeability and consolidation coefficient have been performed on samples with the highest clay content. In reality, the settlement-prone soil profile consists of silty clay with draining layers of sand. Therefore, the samples and results are not representative of the soil layer sequence. The assumptions made are conservative and result in larger estimated consolidation time.

The consolidation time is highly dependent on the horizontal consolidation coefficient, c_{vh}. The Swedish Transport Administration [6] advises that c_{vh} can be set to 2 times the vertical consolidation coefficient c_{vv}, for clay. To avoid an excessively conservative approach, this parameter could have been determined from in-situ measurements such as pore pressure dissipation time in a piezocone CPTu.

The determination of shear strength increase has been considered with a factor "a" corresponding to 10%, which is a conservative assumption. If direct shear tests (DSS) from various depths and consolidation stresses had been conducted, they would likely have supported the design with a higher rate of strength increase, potentially in the range of 20%, in line with empirical relationships such as those presented in Figure 4.

The design of the embankment is just a preliminary design. The laydown time for each stage and thickness of each fill layer will be updated during construction with respect to the data collected in the field using the observational method.

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