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Use of vertical drains and lime-cement stabilization to improve stability and reduce settlement of a high embankment on very sensitive, soft soil

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Abstract. This article presents a case-study of the design and construction of a 14 m high embankment for a highway construction project in mid-Norway. The ground consists of up to 30 m soft clay, with high sensitivity over bedrock. The embankment increased stresses in the clay beyond the preconsolidation stress, causing settlements of 1-1.5 meters. To satisfy requirements related to stability and settlements, lime-cement columns and prefabricated vertical drains (PVD) were installed. The aim in the design phase was to limit the extent of lime-cement stabilization and thereby the carbon footprint. The PVDs were designed both to maintain stability and settlement requirements. The increase in shear strength in the clay from consolidation during the fill works, was utilized to document stability both permanent and during the construction phase. During the construction phase, monitoring the pore pressure was crucial to control the filling rate to maintain the assumptions from the design. To verify the design strength, CPTU soundings were performed to investigate strength properties after consolidation. Settlement plates were installed and monitored during construction. By applying a probabilistic prediction fitted to the measured settlements (Asaoka's method), the future settlements were predicted and shown to be within the given requirements. The measured increase in strength of the clay after consolidation fit well with the design values. The measured settlement during construction were a bit higher than expected. The practical methodology presented in this article can be applied to other embankments constructed on soft soils, especially where stresses under the embankment exceed the preconsolidation stress.

1 Introduction

This article describes the design and the construction of an embankment for a new highway in mid-Norway. The road project, which is 21 km long in total, consists of tunnels with a combined length of 13 km. The high proportion of tunnels generates a significant surplus of rock. The most economical and environmentally friendly solution is to reuse the excavated material in the roadway so that as little as possible must be deposited. This reduces both transport costs and the need for landfill areas. Road projects require large resources for mass transport, and when the demand for fill is large, there are substantial gains to be made by limiting transport. It is also advantageous to transport the materials only once and get them quickly to their final placement.

In this project, the surplus excavated material was utilized by constructing extensive embankments for the highway. At the same time, the aim was to reduce the need for intermediate storage and avoid multiple re-handlings by placing the tunnel material directly in the embankments. The design was adapted to the rapid delivery of material from the tunnel, and this was taken into account in the planning.

The embankment in Langsteindalen Valley, is up to 14 m high. The ground conditions are mainly soft, sensitive clay of varying thickness over bedrock, up to 30 m. Without any ground improvement measures, the time for primary consolidation would have been several decades and not compatible with the project's completion requirements. To the west, the area is bounded by a river, which forms the limit of the embankment. Ground improvement measures were therefore also necessary to obtain sufficient stability of the fill.

In the design phase, the aim was to limit climate-impacting ground improvement measures. Prefabricated vertical drains in combination with preloading are considered to be the most economical and eco-friendly technique, compared lime-cement stabilization of soft soil [1]. In the design phase, the concerns regarding settlements and stability were addressed. The solution was to combine vertical drains with preloading. Lime–cement columns were installed along the toe of the embankment for stability. The extent of lime–cement stabilization was reduced by exploiting the gradual increase in strength in the clay during the embankment works through pore pressure dissipation.

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2 Ground conditions

Lindgård et al. [2] thoroughly describes the ground conditions in Langsteindalen. A summary is presented in this article. The thick marine deposits in Langsteindalen were deposited in sea water when the glaciers retreated approximately 12,000-13,000 years ago [3]. Following the unloading of the weight of the glaciers, the land has risen approximately 175 meters relative to present-day sea level. The bottom of Langsteindalen is situated at ca. 100 MASL, well below the marine limit. As the land emerged from the sea, erosion has formed the landscape in the region. Ground investigations at the bottom of the valley suggest that previous terrain elevation has been up to ca. 10 meters above the current terrain level.

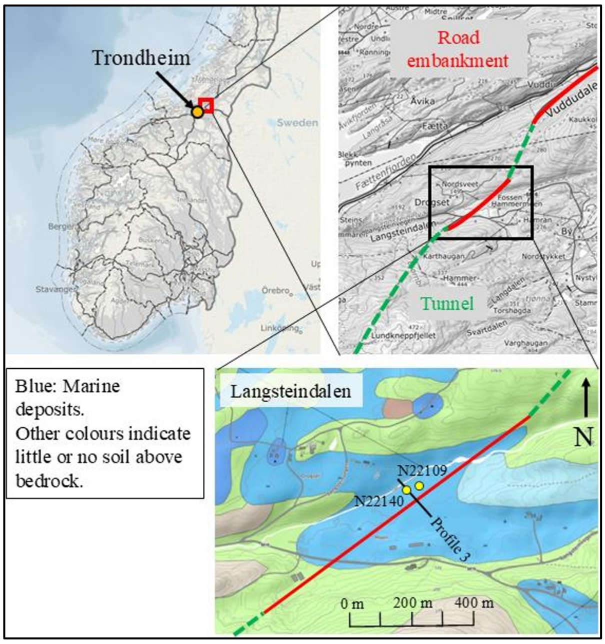

Fig. 1. Top left: Site location. Top right: Location of road embankments and tunnels. Bottom: Quaternary geological map. Maps from [4], [5].

Fresh water percolating through the marine clay deposits causes leaching of the salt that strengthens the bond between the clay minerals. The result of this process is the formation of brittle clay, called quick clay, which has a very low remoulded shear strength. Deposits of quick clay have been involved in several major landslides in Norway, for instance the slides in Kattmarka [6] and Rissa [7].

Fluvial and beach deposits of various thickness cover the marine deposits in the region. In Langsteindalen, a top layer of silt, sand and gravel increases up to ca. 10 meters thickness towards northeast in the valley. Soft to medium clay is found below the top layer, mainly quick clay. In situ strength of the clay is generally in the range 20-80 kPa. The clay has low plasticity.

Stiffness parameters for the clay have been estimated based on oedometer tests. In the overconsolidated range, the constrained modulus M_0 is about 2-12 MPa, increasing with higher preconsolidation stress. In the relatively flat areas at the bottom of the valley, a hydrostatic ground water condition from terrain level is observed. In the clay slopes towards the south, below hydrostatic pore pressure is mainly observed. There are also some observations of pore pressure above hydrostatic in the slopes, which is assumed to be a result of seepage from the hills surrounding the valley.

To illustrate the soil stratigraphy, data from laboratory tests from borehole location N22109 and N22140 is shown in Fig. 2. Samples were collected using the Sherbrooke mini-block (160 mm) sampler. The boreholes are located in the flat area at the bottom of the valley (see location in Fig. 1) and provides a good representation of the soil in this area.

The SHANSEP factor S, relating effective stress to undrained shear strength in a normally consolidated state, was investigated by consolidated undrained triaxial compressive strength tests with consolidation stresses above p_c'. Based on the results, a factor of 0.29 was used in the design.

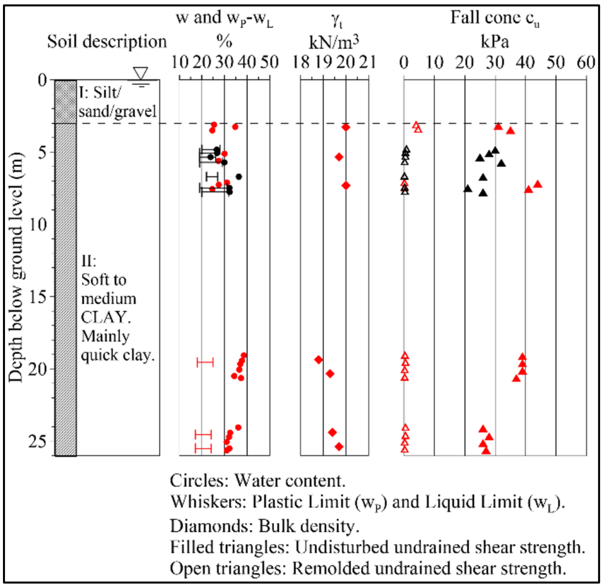

Fig. 2. Soil layering and engineering properties at locations N22140 (black) and N22109 (red). Figure adapted from [2].

The creep resistance factor r_s is interpreted from 12 incremental loading (IL) oedometer tests, and shown in Fig. 3, with the colours representing different sample qualities.

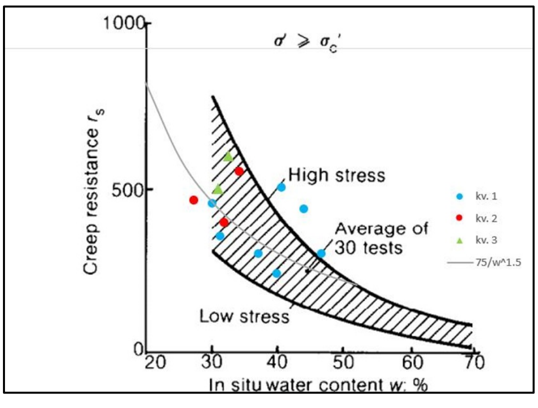

Fig. 3. Interpreted creep resistance factor r_s for IL oedometer tests in circles, shown with empirical correlations between r_s and the natural water content for clay from [11].

3 Design

3.1 Overall design principle

Fig. 4 illustrates the terrain, the embankment and the bedrock in profile 3. Towards Langsteinelva River, the slope at first is 1:8 (for agricultural purposes) and ends with 1:2 slope down to the river. The profile location is given in the plan view in Fig. 5.

Based on these principles, the following calculations have been carried out:

A. Required ctc distance between vertical drains to develop at least 95 % consolidation during the construction phase
B. Stability calculations with lime–cement columns in a ribbed pattern to meet partial factor requirements
C. Long-term settlements calculations (creep settlements) in the permanent phase

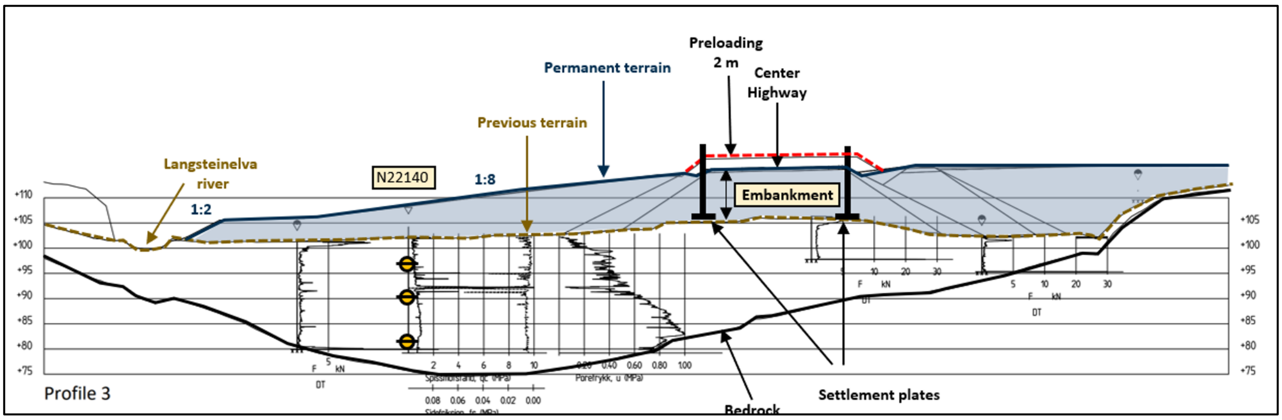

Fig. 4. Illustration of soil profile and embankment (profile 3).

To ensure sufficient stability, prefabricated vertical drains (PVD) have been designed to provide faster consolidation of the clay. In combination with lime–cement stabilized clay in a rib pattern at the toe, this ensures stability. The center-to-center (ctc) distance between the vertical drains is chosen to ensure approximately 100 % of the primary consolidation settlements occur during the construction phase.

The use of vertical drains combined has two purposes: to increase the undrained shear strength of the clay and to accelerate the primary settlements.

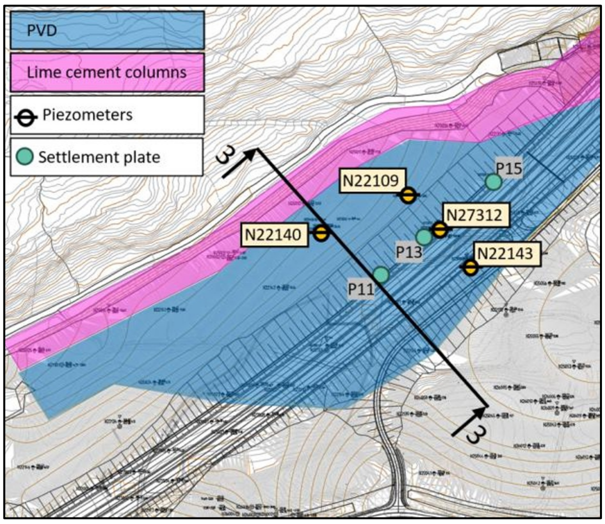

Fig. 5. Plan view with locations of the sensors for monitoring.

3.2 Design of vertical drains

3.2.1 Principle

The design principle entails that 95 % of the primary consolidation settlements are completed during the construction phase. In the design, the planned filling rate resulting from rock extraction during tunnel driving was an input parameter to the design. The schedule showed that tunnel driving would continue for about 20 months. With a maximum fill height of 14 m, the filling rate would be around 0,7 m/month, conservatively rounded up to 1 m/month. A resting period of 6 months was assumed to allow for general uncertainties, even though the planned resting period was about 1 year.

The required PVD ctc distance is determined according to the following criteria:

- To achieve at least 95 % theoretical degree of consolidation within approximately 6 months of resting time.
- Increase the undrained strength during the filling works to provide sufficient stability during ongoing filling operations.

3.2.2 Assessment of consolidation rate

Preliminary estimates for the consolidation rate were carried out. It was considered unnecessary to perform more advanced calculations because there was significant uncertainty regarding the thickness of the soil deposits and the soil stiffness parameters. The combined average degree of consolidation, U, for vertical and horizontal drainage is determined using the method proposed by Carrillo [8]:

$$U_v = 1 - (1 - U_V)(1 - U_H) \quad (1)$$

U_V is vertical drainage according to Terzaghi [9] and U_H is the radial consolidation according to Barron [10].

The embankment fill causes a stress increase above the original preconsolidation stress, p_c', so consolidation will therefore proceed both in the OC and NC ranges. The consolidation coefficient c_v differs in these ranges because the permeability of the clay decreases somewhat upon compression and because the settlement modulus decreases above p_c'.

Fig. 6 shows the degree of consolidation for various PVD ctc distances. To obtain 95 % of the consolidation during the construction period (conservatively set to 6 months), the figure shows that the required ctc distance spacing is at least 2 m (orange curve).

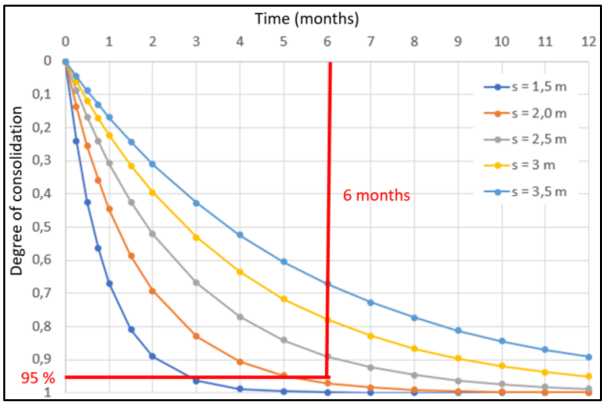

Fig. 6. Theoretical consolidation rate for different PVD ctc distances.

3.2.3 "Stepwise" consolidation of the clay

To calculate the increase in undrained shear strength during the filling works and consolidation, it is assumed, for simplicity, that 1 m of fill is placed with a certain filling rate, leading to a so-called "stepwise consolidation". The filling rate is dictated by the tunnel operation, and assumed to be 1 m per month.

For each step the total load, excess pore pressure, and the increase in effective stress at the start and end of the filling step are calculated. The degree of consolidation is calculated in the same way as for vertical drains. The consolidation rate depends on the PVD ctc distance and the filling rate. For example, from Fig. 5, following the curve for ctc distance 2.0 m, the degree of consolidation after 1 month is 0.43, which means that 43 % of the excess pore pressure dissipates during this period. Then, the increase in effective stress and the thus undrained shear strength (using the SHANSEP principle, with S = 0.29) at the start and end of each step can be calculated.

Fig. 7 shows the theoretical increase in shear strength at the start of each filling step, for different PVD ctc distances. For example, the figure shows that immediately after 10 m fill (with PVD spacing 2 m) the increase in undrained shear strength is around 40 kPa. For full drainage this corresponds to a fill height of 7-8 m. This principle has been used to estimate the increase in undrained shear strength due to consolidation in a stability profile during ongoing filling works.

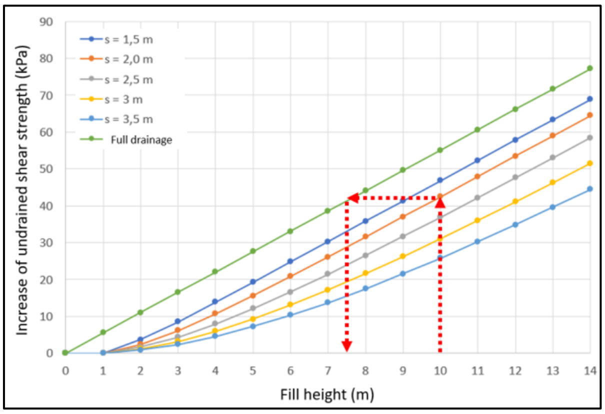

Fig. 7. Theoretical increase in undrained shear strength immediately after each fill level, by using the method of «stepwise consolidation», ($c_{v,NC}$ = 8 m^2/y and fill rate 1 m/month, SHANSEP factor S = 0.29).

In Fig. 8 the theoretical excess pore pressure is shown as a function of the fill height. The fill rate is 1 m per month. The graph illustrates that the excess pore pressure theoretically never will exceed 45 kPa. This value was used to control the fill rate during the construction phase.

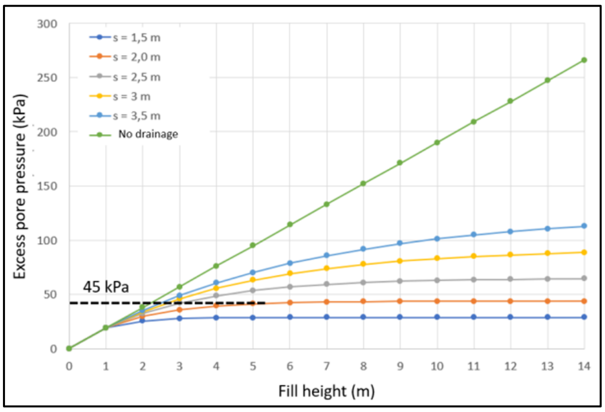

Fig. 8. Calculated excess pore pressure immediately after filling ($c_{v,NC}$ = 8 m^2/y and fill rate 1 m/month).

3.3 Stability assessment

The increase in undrained shear strength by consolidation is calculated according to the SHANSEP principle, with a factor S = 0.29. Undrained shear strength in the permanent situation is calculated as cu = S × (p0' + γ'H), where H is the fill height. During ongoing filling, it is calculated as cu = S × (p0' + γ'H'), where H' is the equivalent fill height at full drainage.

Fig. 7 shows how H' is determined, with an example of 10 m fill height and PVD's ctc distance s = 2 m. The clay is then fully consolidated to an equivalent of 7-8 m, which gives an increase in cu of 40 kPa.

To ensure the required stability towards Langsteinelva River, lime–cement stabilization must be used in a narrow band of a stripe of 10 m along the river.

The strength for this area has been calculated based on the degree of coverage of the lime-cement columns.

3.4 Long-term settlement (creep)

Calculation of creep settlements shall comply with the requirements given by the Norwegian public road administration, which states that the total settlement must not exceed 40 cm for roads with a speed limit of 110 km/h. The maximum permitted differential settlement is 40 cm over a length of 80 m, i.e. 0.5 cm per metre.

Creep settlements were based on the time resistance concept presented by Janbu [11], which is calculated using the equation:

$$d\varepsilon_s = \frac{1}{r_s}\ln\frac{t-t_r}{t_p-t_r} \tag{2}$$

where r_s is the time-resistance factor, t is the time counted from the loading time, t_r is a reference time and t_p is the consolidation time. For the calculations, $r_s = 300$ has been chosen, which is in the lower range of interpreted values and agrees with the empirical correlation, shown in Fig. 3.

The calculations show that the maximum settlement after 40 years is approx. 30 cm, i.e. below the requirement of 40 cm, shown in Fig. 9. The differential settlement has been calculated over lengths of 25 m, 50 m, 75 m and 100 m with different depths to bedrock. The calculated creep settlements are within the requirement for differential settlements.

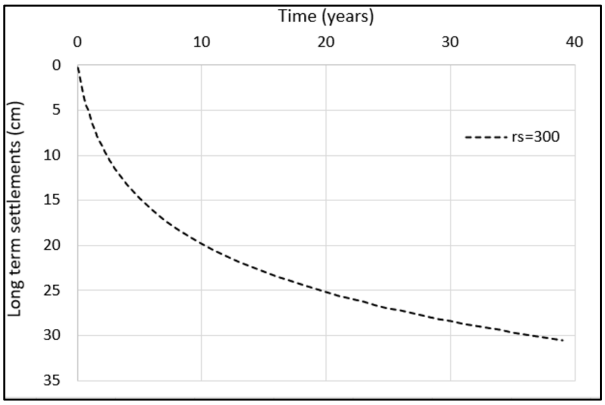

Fig. 9. Calculated long-term settlement (creep).

4 Construction phase and results from monitoring

4.1 Instrumentation and monitoring plan

The settlements during the construction phase were monitored by installing settlement plates in pairs every 50 m along the road (in total 26), shown in Fig. 10. The settlements were measured every 2 weeks during the construction phase and after the preloading was removed, resulting in about 2 years of settlement data. In addition, piezometers were installed at 4 locations with 2–3 gauges at different depths, location shown in Fig. 5. The piezometers were continuously monitored remotely. During the construction phase some of the pore pressure sensors stopped functioning.

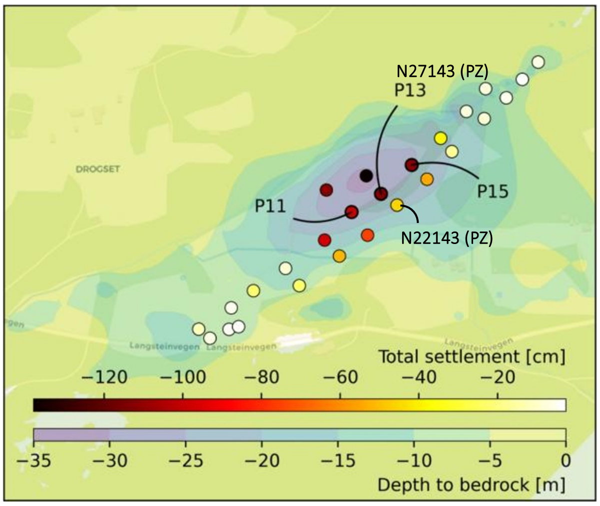

Fig. 10. Location of settlement plates and measured total settlement and depth to bedrock.

4.2 Results and data from construction phase

4.2.1 Pore pressure

Fig. 11 shows the measured pore pressure response for sensor N22143. The location is shown in Fig. 10, and the installation depth is 8 m. Fill height is plotted on the secondary axis and the theoretical pore pressure is shown in dotted line. The increase in pore pressure is given by fill height and an assumed soil weight of 19 kN/m^3. The theoretical dissipation, in dashed plots, is based on the theory given by Eq. (1) and PVD ctc distance 2,0 m.

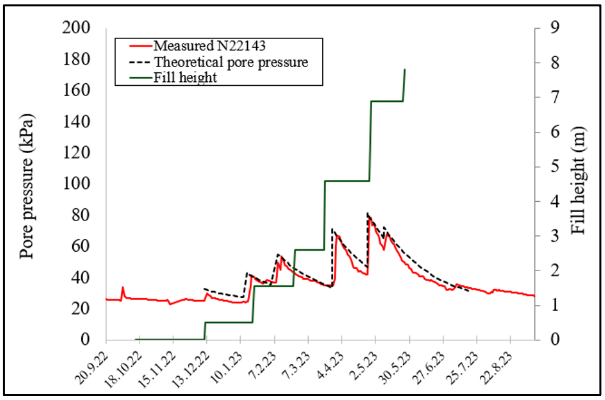

Fig. 11. Measured pore pressure, fill height and theoretical pore pressure based on equation (1).

During the construction phase, the measurements were used to control the fill rate to keep the excess pore pressure below the theoretical value, in this case 45 kPa, as shown in Fig. 8.

Measured excess pore pressure versus embankment height is shown in Fig. 12. The theoretical curve for PVD ctc distance 2 m and fill rate of 1 m/month is shown in orange. Measured pore pressure is shown for

4 sensors. The result show that up to 6 m fill height the pore pressure is below the theoretical value. From fill height 7–10 m, the results from N22143, installed at 8 m depth, shows somewhat higher pore pressure, with a maximum excess pore pressure of 60 kPa. This is mainly due to increasing filling rate during this period, caused by surplus rock from the tunnel extraction.

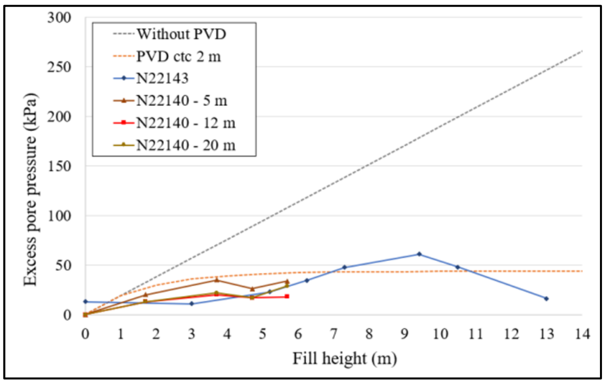

Fig. 12. Measured excess pore pressure vs. embankment height.

4.2.2 Settlement

Fig. 10 shows the maximum measured settlement for all the 26 settlement plates at the last measurement 10th December 2024. The total measured settlement ranges from 10 cm (white circles) and up to 150 cm (black circles). The total settlement is related to the depth to bedrock, also shown in the figure.

Fig. 13 shows the settlement rate at the last measurement 10th December 2024, 3 months after removing the preloading. For most of the plates the settlement rate has decreased to below 0,3 cm/month, while for 5 settlement plates, the rate is around 0,5-1 cm/month.

Only sensors P13 and P15 are considered further since they show the largest settlement development, being placed in an area with the largest soil thickness, as shown in Fig. 13.

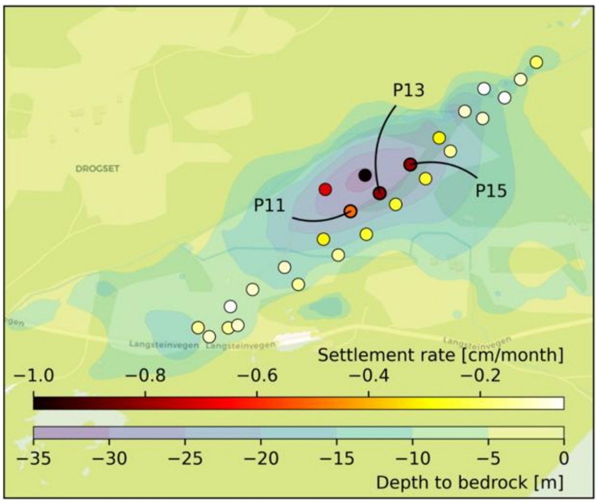

Fig. 13. Settlement rate at 10th December 2024.

Fig. 14 show the settlement time series for P15 with fill height. The maximum measured settlement is 120 cm in dec. 2024, around 2 years after installation. The settlement rate for P15 is shown in Fig. 15. The rate is up to 16 cm/month at the start of the filling works, and reduced to 2,5 cm/month. After adding the final preloading (2 m high fill, assumed to be 38 kPa) the rate increases to 4 cm/month, but decreases quickly to 1 cm/month in dec. 2024.

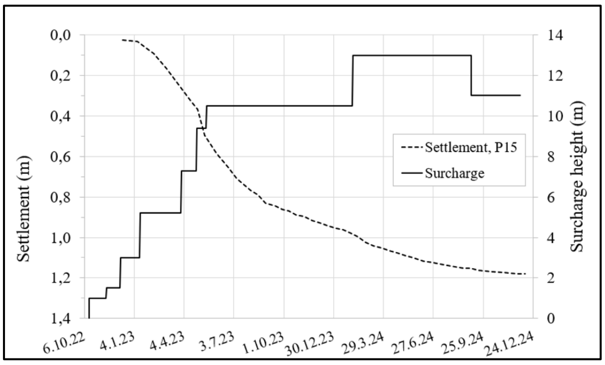

Fig. 14. Settlement time series for plate P15.

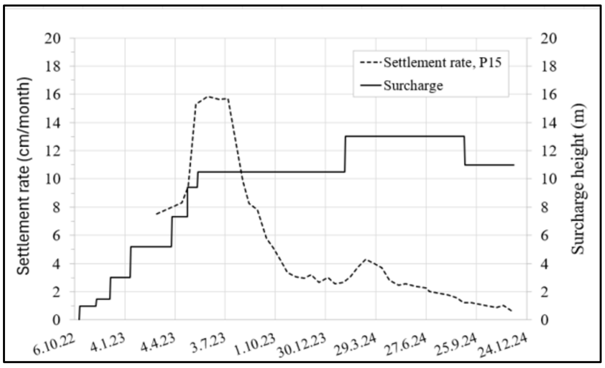

Fig. 15. Settlement rate for plate P15.

Fig. 16 shows the pore pressure for piezometer N27312 versus settlement rate for P13. The figure show that the settlement rate is around 2 cm / month at the same time as the excess pore pressure (Δu) is 0. Theoretically, the settlements from primary consolidation should be completed by this time and only secondary settlements (creep) remaining.

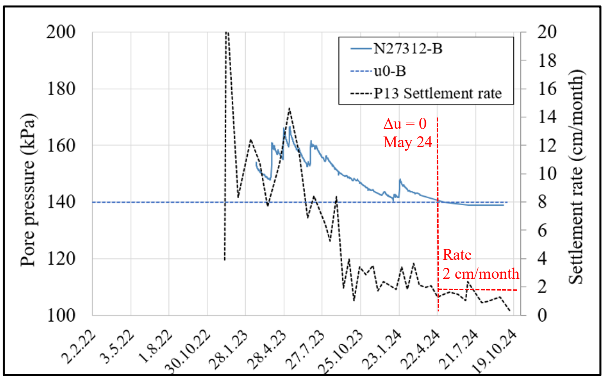

Fig. 16. Pore pressure vs. settlement rate.

As shown in section 3.4, the settlements from creep after 40 years were calculated to 30 cm, which is below the requirements (40 cm). In Fig. 17, the measured settlement rate is compared to the calculated creep rate from the design phase. Month 0 corresponds to the time in Fig. 14 where the excess pore pressure is 0 (May 2024). The measured settlement shows a better fit with a creep calculation with rs = 150, which is 50 % of the interpreted rs in the design phase.

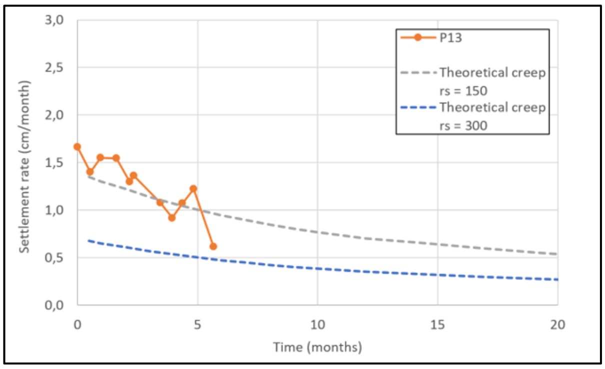

Fig. 17. Measured settlement rate compared to the calculated creep rate from the design phase.

With r_s = 150, the theoretical creep settlement after 40 years is 60 cm (50 % over the requirement of 40 cm). The neighbour settlement plates have no expected remaining settlements (as shown in Fig. 11), therefore the requirement regarding differential settlements along the road also is a concern.

Given in Fig. 3, the interpreted r_s values from 12 relevant oedometer in the area tests ranges between 250 – 600, with an average of 400. To match the measured settlement rate (in Fig. 16), r_s has to be reduced to 150, which is 40 % of the interpreted value of 400. It is therefore likely that the settlements still are governed by primary consolidation, even at the time when 2 piezometers indicate excess pore pressure = 0.

5 Applying probabilistic models to predict future settlements

Due to the uncertainty of the remaining settlements based on the creep calculations with updated rs in chapter 4.2.2, there was a need to use another method to predict the final settlements. The auto-regressive probabilistic method proposed by Asaoka [14] was therefore applied to the time series data from the settlement sensors. Nøst et al. thoroughly describes this method in [15]. A summary is presented in this article.

Using the Bayesian formulation of the method, we can quantify the uncertainty in settlement predictions based on recorded history. Furthermore, the method allows for identifying whether the total settlement requirement is met with an uncertainty. In addition, we can identify when the acceptable rate of settlements is met.

Visualization of the training and the testing measurement data for sensor P13 is shown Fig. 17. The results show that the predicted total settlement within a 95 % prediction interval) for P13 is 125 cm, while the last measured value was 115 cm. This leads to a prediction of the remaining settlement of 10 cm, which is well below the given requirements of 40 cm.

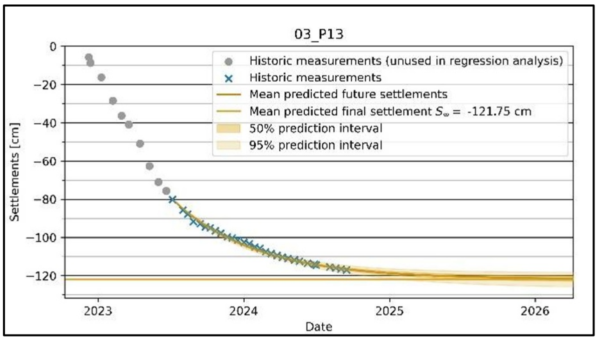

Fig. 17. Settlement prediction using the Bayesian Asaoka method on settlement sensor P13. Mean predicted final settlement S = 121 cm, while the predicted final settlement with 95 % prediction interval is S = 125 cm.

6 Discussion and concluding remarks

This article presents a case study of constructing a 14 m high embankment on soft clay of 30 m thickness for a future highway in Norway. Ground improvement was used in the form of vertical drains and lime–cement columns, both to satisfy stability and settlement requirements. To limit the extent of lime–cement columns, the vertical drains were designed to ensure 95 % consolidation would be completed during the construction phase. In addition, the principle of "stepwise consolidation", which follows the progress of rock extraction from the nearby tunnel, was used to achieve the required stability of the embankment. During the construction phase pore pressure and settlements were measured, both to control that the fill rate was acceptable given the assumptions on consolidation rate, and to verify the design principles.

Pore pressure measurements show that pore pressure dissipation follows the theoretical dissipation curves for the given spacing between PVDs. Generally, there is less response in excess pore pressure than theoretically expected for the certain loading. This may be due to factors such as layering and depth to bedrock, which will affect the stress distribution.

During the construction phase some of the pore pressure sensors stopped functioning. In the end only two reliable sensors remained, making it difficult to obtain an overall understanding of the pore pressure situation in the area. In these sensors the in situ excess pore pressure (Δu = 0) was reached at a time when the settlement rate was higher than expected. The measured settlement rate was compared to the calculated creep rate and the creep resistance factor r_s was adjusted to match the measured rate. The factor had to be reduced to 50 % of the interpreted value from oedometer tests. It is likely that a considerable part of primary consolidation remained at that time. More pore pressure sensors at other locations and soil depths would probably have given a better picture of the development.

Settlement measurements show that the settlements calculated in the design were underestimated. A maximum total settlement of 100 cm was calculated, whereas measurements show up to 140 cm. The design did not include the additional load from the fact that the embankment was sinking. Yet, taking this additional load into account, would still have underestimated the total settlement. This may be due to several factors, i.e. lower stiffness in the in situ clay than interpreted, a reduction in stiffness due to installation of PVDs, and the so-called "smear zone" and others.

To verify that the settlement rate is within the requirements set by regulations given by the national road authorities, a probabilistic analysis was performed using Asaoka's method. Even though the settlements were higher than calculated in the design, this method showed that, within a 95 % confidence interval, the settlement requirements at completion will be met.

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