Proceedings of the XVII ECSMGE-2019
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3

doi: 10.32075/17ECSMGE-2019-0353

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

Consolidation behaviour of quick-clays. The deposit area at the Follobanen Project (Norway)

Comportement de consolidation des argiles rapides. La zone de dépôt sur le projet Follobanen (Norvège)

T.Mateos
Acciona Ingeniería, Madrid, Spain

ABSTRACT: The Follo Line Project consists of a 22km new double track line between Oslo Central Station and the new station at Ski, south of Oslo. Around 20km are being excavated by TBM along twin tunnels. All the muck material from the excavation will be deposit in a valley close to the main access of the tunnel works. The valley is filled by soft soils, mainly quick clay covered by peat and surrounded by rock outcroppings. Maximum thickness of the quick clay layer is around 15m and the peat layer reaches a thickness up to 1.5m. The total volume to be stored in the area is over 4.1 million of cubic meters.

Due to the poor geotechnical properties of both materials a detailed filling strategy had to be developed to minimize failure risks. All the design fulfills the Norwegian standards for construction over sensitive quick-clays. Moreover, the consolidation behavior of the quick clay is being monitored since the beginning of the construction. Right now, more than 30 months of data have been recorded including horizontal displacements by seven inclinometers, vertical deformation by 30 settlement plates and pore pressure excess development by 30 piezometers located at different positions in the foundation soils. Monitoring data have been analyzed to study actual deformability of the quick-clay layers, the response of pore pressures development during construction and its subsequent time-dissipation.

RÉSUMÉ: Le projet de ligne Follo consiste en une nouvelle ligne à double voie de 22 km entre la gare centrale d'Oslo et la nouvelle gare de Ski, au sud d'Oslo. Le tunnelier fouille environ 20 km le long de deux tunnels. Tout le matériel de boue provenant de l'excavation sera déposé dans une vallée proche de l'accès principal du tunnel. La vallée est remplie de sols meubles, principalement d'argile rapide recouverte de tourbe et entourée d'affleurements rocheux. L'épaisseur maximale de la couche d'argile rapide est d'environ 15 m et la couche de tourbe atteint une épaisseur pouvant atteindre 1,5 m. Le volume total à stocker dans la zone dépasse 4,1 millions de mètres cubes.

En raison des mauvaises propriétés géotechniques des deux matériaux, une stratégie de remplissage détaillée a dû être développée pour minimiser les risques de défaillance. Toute la conception répond aux normes norvégiennes pour la construction sur des argiles rapides sensibles. De plus, le comportement de consolidation de l'argile rapide est surveillé depuis le début de la construction. À l'heure actuelle, plus de 30 mois de données ont été enregistrés, notamment les déplacements horizontaux de sept inclinomètres, les déformations verticales de 30 plaques de tassement et le développement de l'excès de pression interstitielle de 30 piézomètres situés à des positions différentes dans les sols de fondation. Les données de surveillance ont été analysées pour étudier la déformabilité réelle des couches d'argile rapide, la réponse au développement des pressions interstitielles pendant la construction et sa dissipation temporelle ultérieure.

Keywords: quick clay; consolidation; deformability; monitoring; risk.

1 INTRODUCTION

The deposit area is located close to the main entrance for the tunnels construction (Åsland site). The selected valley is a sedimentary basin filled by soft soils and originally excavated by glacial dynamic (Figure 1).

Three main soil types have been described in the area: a *peat layer,* which completely covers the valley. Its thickness would be between 1 to 2m. A *quick clay layer* below the peat; its thickness can reach up to 15m in the central section of the valley, and continental *brown clays* located mainly in the southwest border of the basin and locally over the quick clay.

The basin can be divided in two areas: the main basin and the south basin. The first one is the bigger one; it is elongated along the valley and comprises the thicker layer of sensitive clays (Figure 2).

Figure 1. Location of the deposit area in Åsland site.

The second one is almost completely isolated at the south edge of the valley. The soil profile changes, as there are 2-3 meters of brown clay over the marine grey clay, which –in this area- only shows limited sensitivity.

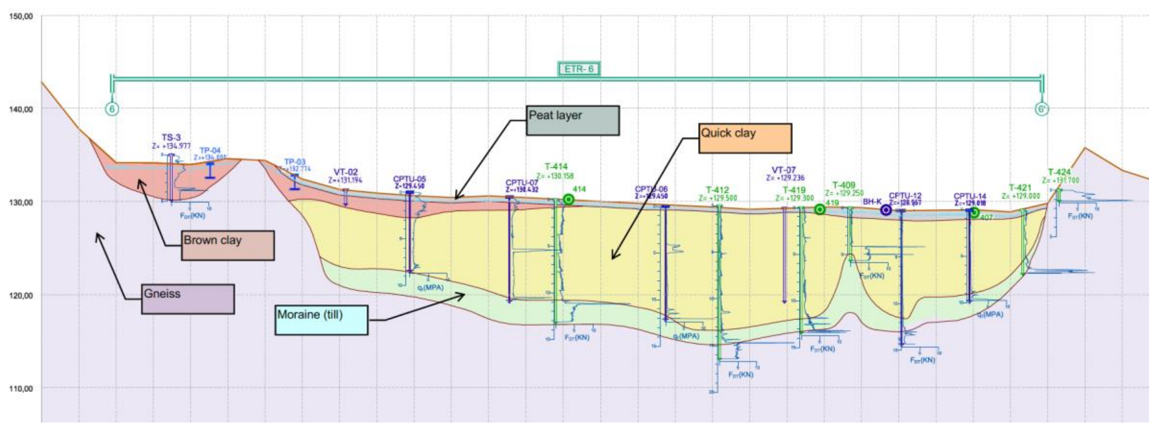

Figure 2. Longitudinal geotechnical profile along the valley (the south basin is on the left disconnected from the main basin by a basement outcrop). The red line in Figure 5 shows the profile position.

The sedimentary layers are located over the rock basement –mainly gneiss- which sorrounds almost completely the valley. There is only a narrow exit –on the nortwestern edge- where the creek cutting the valley finds its way out. Over the gneiss a till layer has been also detected.

The deposit area were selected due to its proximity to the construction tunnel entrance, in order to avoid material hauling. However, the presence of brittle clays and high deformable soils in the area involved a non negligible failure risk.

2 GEOTECHNICAL INVESTIGATION

In order to get an accurate ground profile of the area and to estimate the strength and deformability/consolidation parameters of each soil type, a geotechnical investigation were carried out. The investigation comprised undisturbed sampling, total soundings, CPTu, in situ vane tests and tomography resistivity profiles (ERT).

CPTu data were analyzed based on correlations specially developed for quick clays in Norway (Karlsrud et al, 2005). The results were compared with other test data in order to estimate the undrained shear strength and deformability modulus for the quick clay layers.

The main results are included in the next figure which shows the values for the undrained shear strength estimated from the CPTu data compared with other usual correlations.

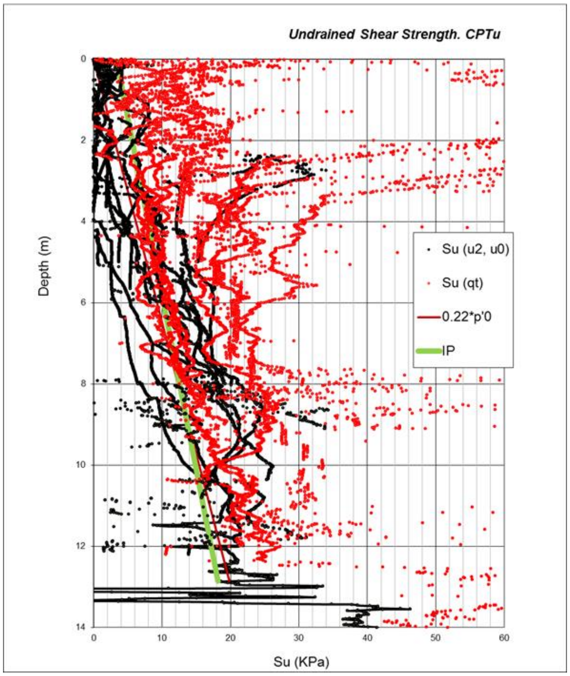

Figure 3. Undrained shear strength from CPTu data.

The green line in Figure 3 was drawn based on Håndbok V220 (SVV, 2014) which proposes to estimate the undrained shear strength increment with depth through a line whose slope depends on the plasticity index of the clay:

$$S_u = \left(a_u + p'_0\right).tan\theta_u \quad (1)$$

$$tan\theta_u \sim 0.4\sqrt{I_P} \quad (2)$$

The average I_p value for the quick clays in the area was around 17% and $p_0´$ is the reference pressure.

The second correlation in Figure 3 –red line- was also proposed for normally consolidated clays:

$$Su = k.p'_0 \quad (3)$$

With k = 0.25 to 0.3 for normal consolidated clay and down to 0.16 for quick clay. A value equal to 0.22 has been selected to compare the result with the data from the study area.

Sensitivity of the quick clay layer found in the area is not extreme, although enough to classify the clays as "quick" according to the Norwegian standards (NVE, 2014). The sensitivity values from fall cone tests are shown in the next picture. It also highlights that clay in the south basin is less sensitive than the clay in the main basin.

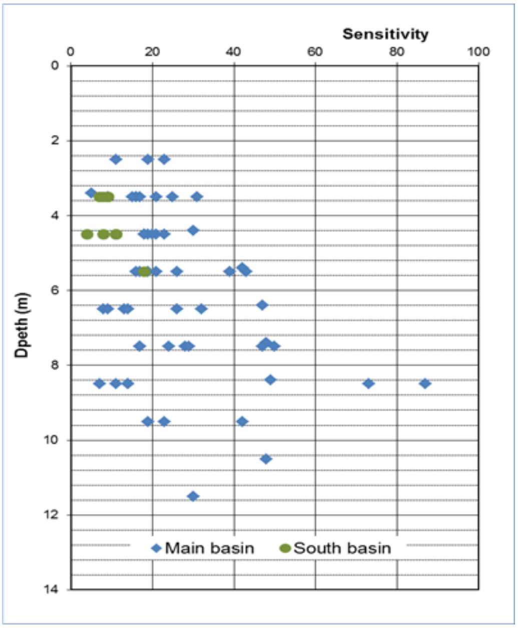

Figure 4. Sensitivity values from fall cone tests.

3 CONSTRUCTION STAGES

The deposit area was initially designed to store one million cubic meters, however during the construction the required capacity increased up to 4.1 million cubic meters of muck material.

The surface of the deposit area, after filling completion, will be approximately 100000m^2. The height of the fill will be between 15 to 30m for the permanent fills and up to 60m for temporary stages required for construction.

To assure stability of the fill and to reduce failure risk related to the sensitive clays, a monitoring plan of the area was developed and installed. The plan involves the installation of settlement plates, inclinometers and wire vibration piezometers.

Moreover, a detailed construction procedure was developed in order to reduce failure risk. It comprised an improvement treatment by wick drains in order to reduce pore pressure development and to speed up its dissipation.

The construction procedure also limited the load applied by machinery working in the area for each different fill layer to be executed. The drainage layer 0.9m thick was carried out in 3 successive sublayers before the wick drains were driven. Then, a reinforcement geogrid was installed and another 1,1m of granular material were extended in successive layers to get a support layer for the deposit of the muck TBM material.

The next table summarized in a simplified way the construction stages in the main basin.

Table 1. Construction staging.

	Fill Thickness	Dates	
Drainage layer	0.9 m	February 2016	
Monitoring installation		07/03/2016	01/04/2016
Wick drains		06/04/2016	19/04/2016
Reinforcement layer 1	0.6m	30/05/2016	29/06/2016
Reinforcement layer 2	0.5m	06/06/2016	07/07/2016
Fill construction	13 to 18m	July 2016	October 2017

Right now, an extension of the deposit area is being carried out. Then, the final settlement magnitude is changing and excess pore pressure is developing again.

4 MONITORING DATA ANALISYS

The monitoring installed consists on settlement plates, vertical inclinometers and vibrating wire piezometer.

Initially 30 settlement plates, seven inclinometers and 6 piezometers were set up. After a year of data recording, another 7 piezometers were installed and some of the initial ones were replaced due to different record problems (Figure 5).

.

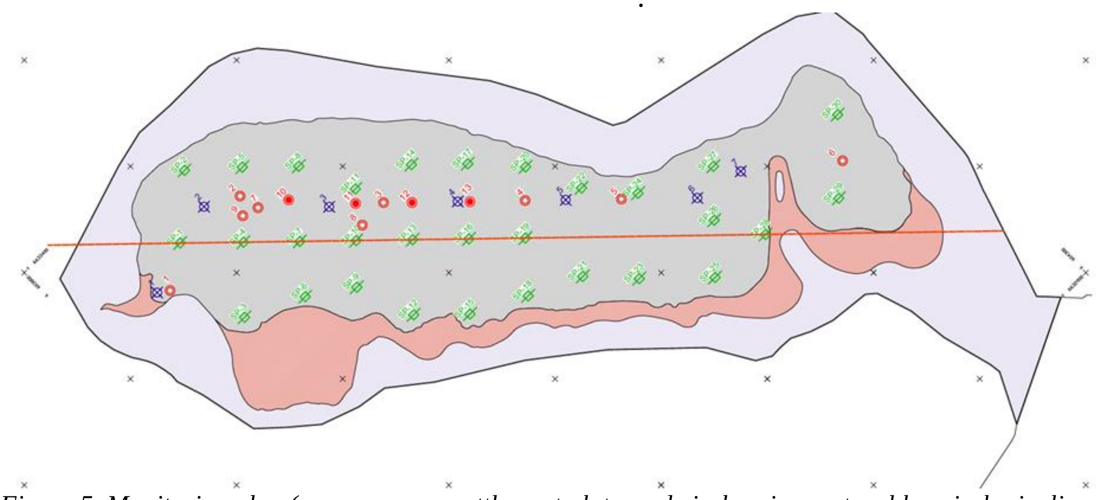

Figure 5. Monitoring plan (green square: settlement plate; red circle: piezometer; blue circle: inclinometer).

The complete series of data analyzed covers two years and a half. The main results showed correspond to piezometers and settlement plates, as these data are relevant to understand the consolidation behavior of quick clays.

4.1 Piezometer

Excess pore pressures have been recorded since the beginning of the filling of the deposit area.

Next figures show the values obtained since the main filling end –by summer 2017- to December 2018.

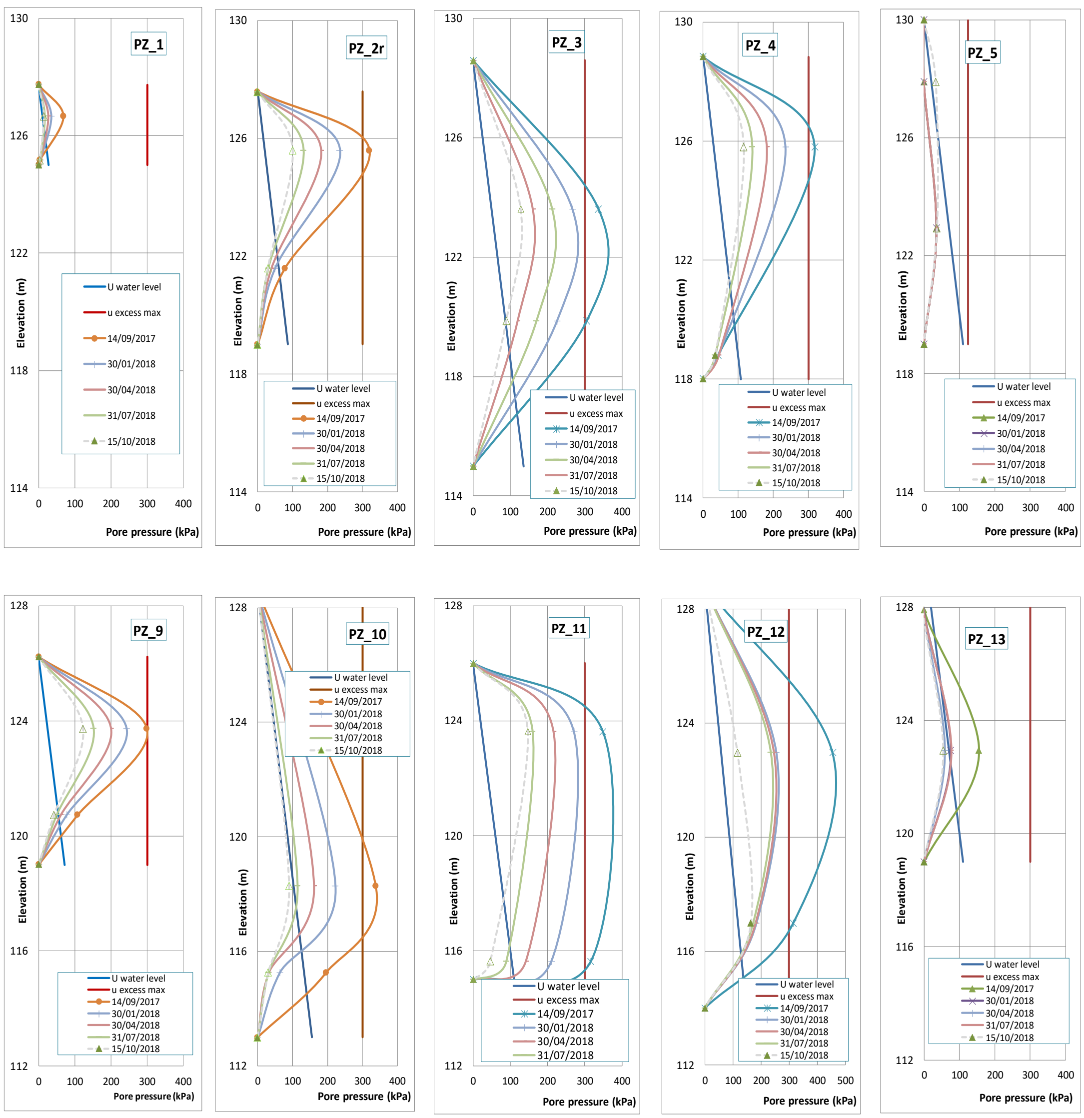

Figure 6. Isochrones obtained from the piezometer records.

The interpretation implies that the quick clay layer is being drained both by its upper and bottom limits. The shape of the isochrones obtained from the two piezometers installed in each point, also support that idea.

The zero excess pore pressure at the bottom of all sections has been drawn at the depth where the rock was located in each point during the geotechnical investigation.

The isochrones show how consolidation is still happening although is almost over in areas where the soft soil depth is smaller and where the loading first finished, as in piezometer PZ_1.

The excess pore pressure recorded in most locations would point out that drainage has happened by the upper limit of the soft soil -through the drainage layer built with that purpose- and also by the moraine layer located between the quick clay and the gneiss. Only the deepest piezometer at location PZ_11 shows values excessively high for that hypothesis. This result might be related to a deeper position of the moraine in the area that the one derived from the available data or to a more impervious layer inside the moraine sediments.

4.2 *Settlement plates.*

The next figure shows data from the settlement plates located in the central part of the valley, where the thickness of the quick clay layer is bigger. The maximum settlement recorded is around 4m. The current fill height in the area is 15m higher than the initial; it means that the overload corresponds to a fill up to 19m high, taking account of the maximum settlement recorded.

The shape of the settlement curves points out that primary consolidation is almost over in all the points. The worst one is settlement plate 11, however this plate is located close to the new fill which is being currently constructed and probably the results are influenced by this new construction stage or maybe due to a deeper position of the moraine layer, as the piezometer data also denote.

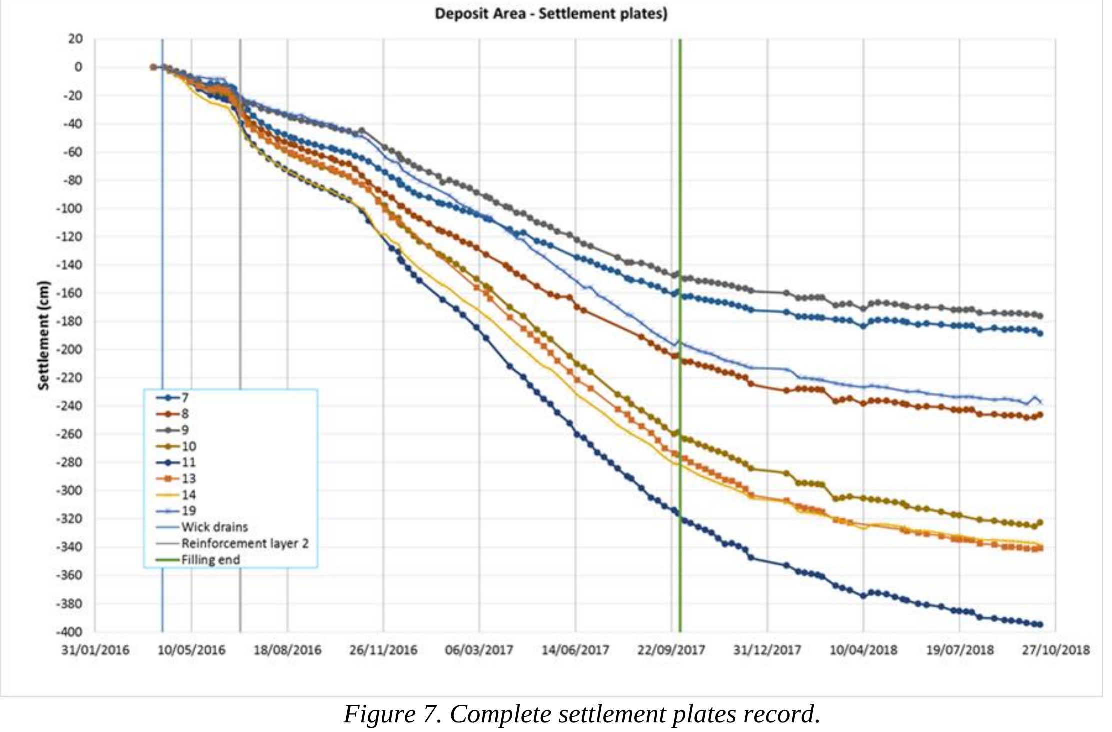

Figure 7. Complete settlement plates record.

The data from the settlement plates have been analyzed by the Asaoka method (Asaoka, 1978). The main results are shown in the next table. The value of both the horizontal and the vertical consolidation coefficients have been estimated for two periods of the fill execution: the first one corresponds to summer 2016, between the execution of the reinforcement layers and the beginning of the construction of the TBM muck layers (Figure 8). The second one resembles the final section of each curve, once the main filling of the landfill was over (green line in *Figure 7*).

Moreover, during the geotechnical investigation, eight oedometric tests were carried out. The average value obtained for the vertical consolidation coefficient was 1.5.10^{-7}m^2/s, which is similar to the usual value for quick clays in technical publication. See, for example, Håndbok V220 (SVV, 2014)).

The main results of the analysis have been summarized in

Table 2, where the first columns show the results of the initial stages after the execution of the reinforcement layers and the last columns the results of the last period since the filling end. In both cases, the horizontal and vertical consolidation coefficients have been estimated to compare the changes in consolidation behavior

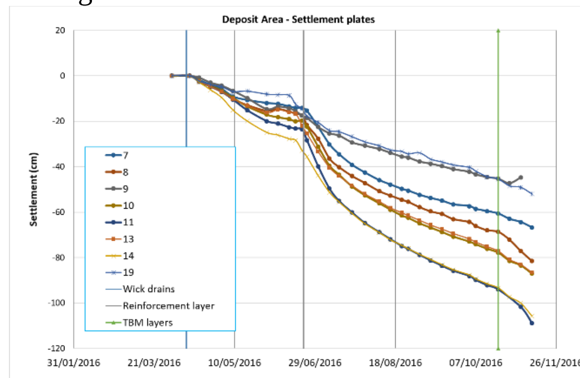

Figure 8. Initial settlement plates record.

The horizontal coefficient would represent a consolidation process were the wick drains would have worked properly. On the contrary, the vertical coefficient would be the most representative of the consolidation development if the wick drains would not have worked correctly.

Table 2. Settlement plates consolidation analysis

	June to December 2016				July 2017 to October 2018			
	Final Settlement	Consolidation degree	Ch	Cv	Final Settlement	Consolidation degree	Ch	Cv
	cm	%	m^2/s	m^2/s	cm	%	m^2/s	m^2/s
SP-7	62	96	3.79E-07	1.41E-06	191	96	1.03E-07	3.85E-07
SP-8	72	95	3.14E-07	2.39E-06	248	97	1.21E-07	9.21E-07
SP-9	51	88	2.08E-07	1.58E-06	180	95.5	9.62E-08	7.31E-07
SP-10	81	95.5	3.17E-07	2.92E-06	336	97	8.84E-08	8.13E-07
SP-11	97	97	3.43E-07	3.75E-06	370	100*	8.13E-08	8.89E-07
SP-13	91	85	1.92E-07	2.86E-06	358	90	8.89E-08	1.36E-07
SP-14	115	81	1.60E-07	8.21E-06	350	93.5	1.86E-07	2.39E-07
SP-19	58	79	1.46E-07	7.48E-06	245	97	9.53E-08	1.22E-06
		Average	2.57E-07	3.82E-06		Average	1.08E-07	6.67E-07

Next table compares the ratio between the consolidation coefficients obtained by the settlement analysis and the value estimated by the oedometric tests (value 1 in the table). Both values decrease from the very first stages to the current stage.

Table 3. Consolidation coefficients ratios.

	Ch	Cv
oedometer tests		**1**
June to December 2016	1.72	25.49
July 2017 to October 2018	0.72	4.45

The results seem to indicate that the influence of the wick drains was greater at the beginning of construction and that consolidation was taking place according to a horizontal consolidation coefficient twice the laboratory values.

Currently the consolidation is slower and probably only controlled by a vertical consolidation coefficient around 4.5 times the laboratory value, although it could be also related to a reduction in permeability due to the consolidation process.

5 CONCLUSIONS

The excavation of the TBM tunnels of the Follobanen project generates a huge amount of muck. A deposit area was selected by the property (BaneNor) to store all this material, in spite of the fact that the ground in the area is composed by soft soils: mainly quick clay and peat.

Monitoring data since April 2016 to October 2018 have recorded the consolidation behavior of quick clay layers. Maximum settlement recorded reaches up to 4m in the area where the quick clay layer is around 15m thick. That means that around 25% of the initial water content in these materials has been drained out.

Piezometer data have been analyzed. The isochrones curves derived point out that drainage during consolidation happens both by the upper and bottom limits of the soft soil layers.

The analysis of the settlement plates seems to indicate that wick drains worked properly during the initial consolidation stages. However, consolidation rates during the last year are slower and it seems that consolidation is happening without the help of the wick drains due to their buckling. Moreover, the observed reduction in settlement rate could be also related to a decrease in the clay permeability due to consolidation.

6 ACKNOWLEDGEMENTS

The author would like to thank the Norwegian Railway Administration (BaneNor) for their authorization to publish the monitoring results. I also want to highlight the work of the Acciona-Ghella Joint Venture (AGJV) responsible of the construction of the Follobanen project, which has been in charge of the monitoring device installation and data recording.

Special thanks finally to my colleagues of Acciona Ingeniería; without their help and enthusiasm, the design and development of this project would not have been possible.

7 REFERENCES

Asaoka, A. 1978. Observational procedure of settlement prediction. *Soil and Foundations, Journal of the Soils and Foundations Engineering*, **18(4),** 87-101.

Karlsrud, K., Lunne, T., Kort, D.A. and Strandvik, S., 2005. CPTU correlations for clays, Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering, 693-702, Millpress Science Publishers/IOS Press. Osaka.

Norges vassdrags og energidirektorat (NVE). 2014, Sikkerhet mot kvikkleireskred. *Veileder nr 7*.

Statens Vegvesen (SVV). 2014, Geotecknikk i vegbygging. *Håndbok V220*.