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On the design of a deep secant pile wall

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ABSTRACT
The Norwegian public road administration is building a new four-lane tunnel between Sandvika and Rud, partly as an open cut & cover tunnel with deep excavation. At Mølla, the new tunnel is located 30 meter below terrain making the retaining wall for the excavation one of Norway's deepest. This area has demanding soil conditions with a 15m layer of soft clay over 15m hard moraine on top of the bedrock, which has large variations in depth over the area. Further, the ground water level is high and a heavy trafficated road is close by.

Because of the difficult ground conditions, secant pile wall with a pile diameter of 1.2 m is chosen for its flexibility and the need of establishing the pile foot into bedrock. Support of the 28 meter high secant pile wall is done by seven levels of tie back anchors. The horizontal load acting on the wall were, during design, predicted to be high and measures for reducing it are taken. In particular, it was predicted that, by lowering the ground water level, the water pressure acting on the wall could be reduced by at least 30%. This is ensured through continuous pumping and the reduction has been verified by measurements.

Due to the some uncertainties in actual horizontal loads acting on the secant pile wall, the complexity of the project as well as the novelty of the design for Norwegian projects, an extended monitoring program has been introduced. The measured pore pressure, wall deformation and anchor loads will be compared to the theoretical predictions in order to validate the design methodology. The excavation has started and will continue until late summer 2016.

Keywords: retaining structures, deep excavations, secant pile wall

1 INTRODUCTION

The Norwegian Public Road Administration is building a new four-lane tunnel between Sandvika and Rud. The tunnel is located outside Oslo and is a part of an upgrade of existing E16 between Sandvika and Wøyen. The tunnel is approximately 2.3 km long and consists in northern part of two cut & cover concrete tunnels, respectively 90 and 500 meters long, with excavation depths of up to 30 meters. The excavation site at Mølla has demanding soil conditions with soft clay over hard moraine on top of the bedrock. The area has large variations in depth which gives challenges related to the design of retaining wall construction. Furthermore, the ground water level is high and a heavy trafficated road is close by.

Due to the large excavation depth and special challenges in the area is this project followed with great interest from the academic community in Norway. This article covers the design of the excavation, with emphasis on engineering assumptions and technical solutions. At the moment, the excavation has started and will continue until late summer 2016.

2 PROJECT DESCRIPTION E16 SANDVIKA – WØYEN

E16 between Sandvika and Hønefoss is the most traffic congested two lane road section in Norway with an YLT of 35,000 vehicles. It has long been in need for an upgrade and the work has been going on since the 1990s. This includes a new four lane road between Wøyen and Bjørum which opened in 2009.

The last part of the E16 towards Sandvika, and connecting to E18, has not have any work done since it was built in the early 1980s, creating major traffic problems. An upgrade of the road has been long awaited, but has been postponed several times due to trace election and political processes.

The Norwegian public road administrations project E16 Sandvika – Wøyen, a 3.5 km new four line main road, consists of 3 major contracts:

- E16 Bjørnegård tunnel from Sandvika to Rud
- E16 Rud - Vøyenenga
- Upgrading the local road system, Sandvika - Emma Hjort

Engineering started with the road planning in 2008 with completed tender documents for the first part of the road, E16 Bjørnegård tunnel, in 2014. The Norwegian public road administration started the construction work for the new tunnel in February 2015 and the expected completion of the road section Sandvika - Wøyen is November 2019. The last part of the project, upgrading the local road system in Sandvika where today's E16 is located, is expected to be completed during 2021. The entire project has a budget of approximately 4 Billion NOK.

The first part of the road will go in two parallel tunnels until the intersection with Bærumsveien, where the road will go in daylight towards Vøyenenga. The northern part of the road tunnel consists of two cut & cover tunnels in deep excavations. At Mølla, where the deepest excavation is located, it will be up to 30 meters from the current ground level down to excavation level for establishment of the two concrete tunnels.

3 PLANNING STAGE – PRELIMINARY INVESTIGATIONS AND ESTIMATES

Due to a fault zone in the bedrock under Statnett's transformation station at Hamang and Franzefoss industrial plant, it was performed a lateral displacement of the tunnel route to the west early in the road planning. Moving the tunnel route, resulted in the need for establish a deep cut & cover concrete tunnel at Mølla, close to existing E16 and high-voltage lines crossing the site 20 meters above terrain. In the next planning stage it was studied various solutions for crossing the local deep area. The options were:

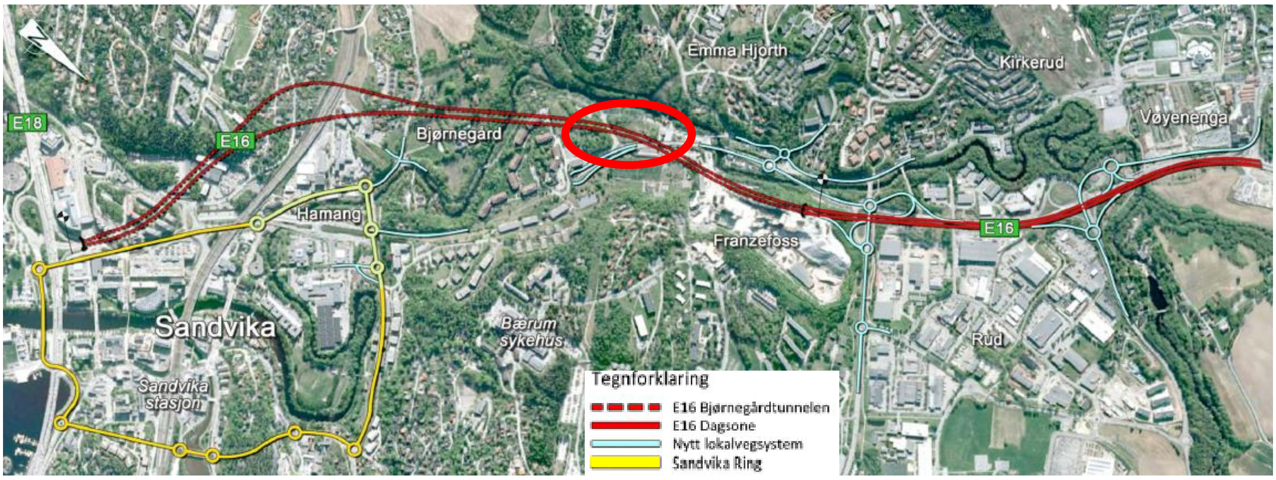

Figure 1 Overview of the project E16 Sandvika - Wøyen

- Open excavation using a supported wall.
- Ground freezing of soils and tunnel driving through frozen material zone
- Tunnel driving securing the tunnel face with a screen

In the process selecting the solution, it was focused on using a robust solution with good feasibility. Use of an open excavation was preferred for its robustness and it has the lowest risk in relation to the technical implementation, although the complexity is uncommon. An important advantage using an open excavation is that the rock tunnels and the concrete tunnel can be built independently. An unforeseen problem on a subset does not give direct consequences for neighboring elements.

3.1 Site investigations

It has been carried out extensive site investigations in connection with the different planning stages, as well as some surveys in previous study phases and building of the existing E16. It is performed total soundings, CPT, piston samples for laboratory testing and piezometeres for water pressure measurements are installed.

The site investigations show layered soil conditions with soft clay and silty clay over hard and stony moraine. It is also registered a sand layer between the clay- and moraine layer in center part of the pit. The thick moraine layer decreases from north to south along the excavation area. Depths to bedrock varies from rock in day to 30 meters depths, and the inclination of the bedrock is steep in each end. The bedrock also falls steeply across the area under existing E16. In the central part of excavation area where the rock level is at its deepest, the ground conditions consisting of 10-15 meters of soft clay over 10-15 meters with hard moraine. A section of the construction pit with stratification and plot of performed soundings is shown in Figure 2.

3.2 Special geotechnical challenges

Establishing an excavation in loose fill of this size has not been performed previously in Norway. Demanding ground conditions with thick layers of soft clay and stone rich moraine, high groundwater level and steep rock stream provides an outer limit of what is technically possible. Also the existing road E16 close by will give strict requirements for deformation of the wall.

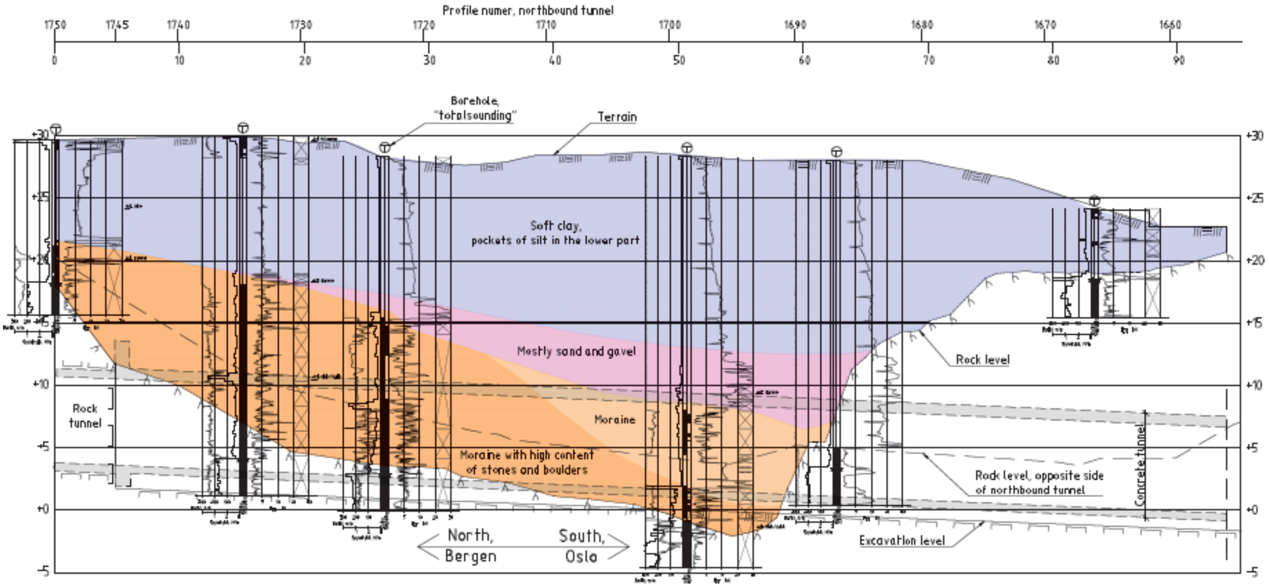

Figure 2 Figure with longitudinal section of the excavation pit showing stratification and plots of performed soundings.

Installed piezometers shows hydrostatic pore pressure with groundwater level at +25 (approximately 3 meters depth), which provides a huge pore pressure on the wall. With a highly permeable moraine layer, there is a desire to establish a dense wall to avoid problems with water intrusion due to excavation as well as limiting deformation at facilities nearby. The great excavation depth gives large horizontal loads acting on the wall with related vertical forces at pile foot in use of back tie anchors. Steep inclination of bedrock, with possible vertical overhang at local parts, bad / fractured rock in the upper part gives the challenges of transferring the forces to bedrock. This is especially a challenge in the area where the bedrock in front of the wall must be removed due to the depth of the tunnel. This gives intersections tightly into the supported wall.

4 DESIGN OF SUPPORTED WALL AT MØLLA

Several types of retaining walls were considered before secant pile wall was selected. Arguments for using secant piles were:

- Rigid and dense wall with large moment-, shear- and axial capacity.
- Possibility of drilling into bedrock that gives great flexibility in the area of steep bedrock surface and gives a secure pile foot.
- Possibility of use of a smaller machine when establishing secant piles under existing high-voltage line.
- Good experiences from other projects in Norway with difficult and varying soil conditions.

The secant pile wall is placed as close to the concrete tunnel as possible to minimize the height of the wall giving substantial cost- and time savings. This because of the drop of bedrock surface across the excavation area, and location far away as possible from the diverted E16 gives the opportunity for lowering the terrain before installation of the wall. The disadvantage would be a short distance between the secant pile wall and blasting contour with up to 10 meters height. Location of the wall and plan for lowering the terrain before installation is shown in Figure 3. The height of the wall is reduced to approximately 28 meters.

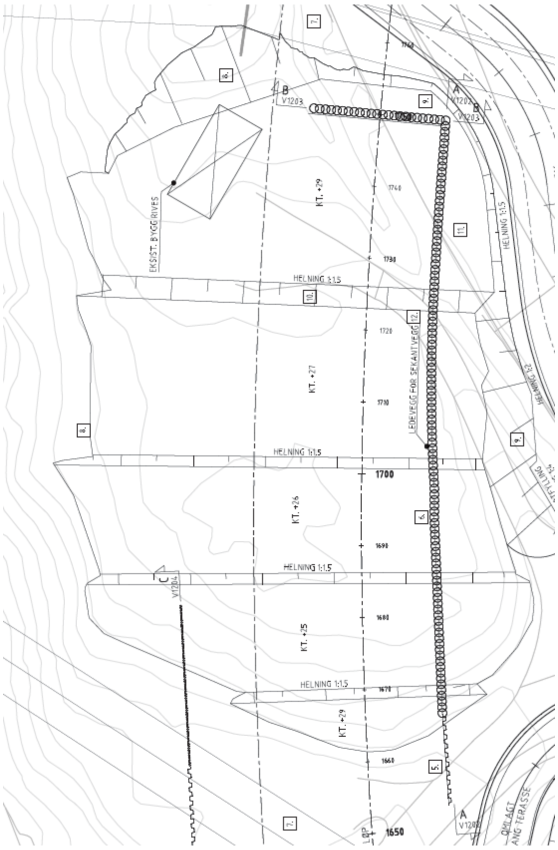

Figure 3 Construction area at Mølla with location of the secant pile wall and excavated terrain level before installation.

With only 1meter distance between the secant pile wall and the blasting contour, while the bedrock surface drops across the excavation area, there will be parts where the secant piles stands on smaller rock surfaces. During excavation the axial force on the rock surface will be large and it was therefore chosen to transferring the axial forces to below bursting level using steel core piles. Figure 4 shows the problem with bursting in front of secant pile wall and the large drop in bedrock level across the excavation area.

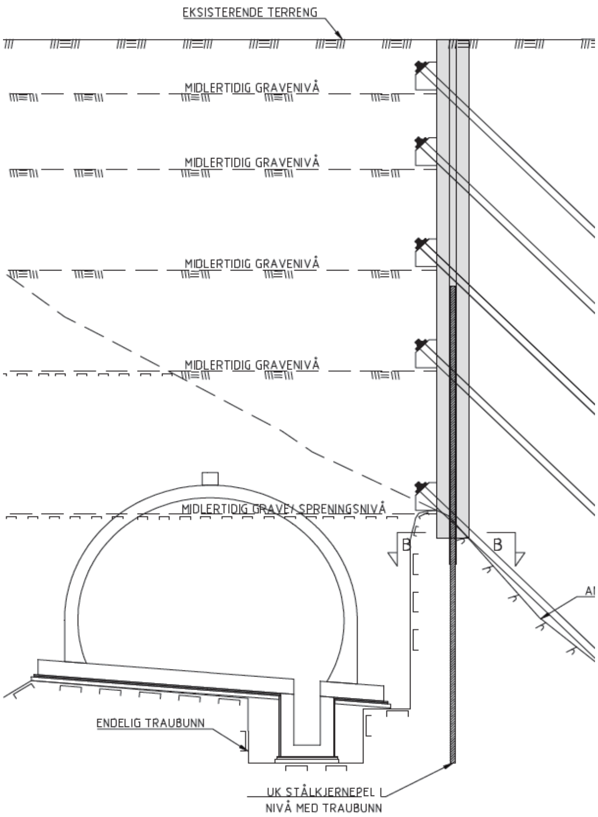

Figure 4 Cross sectional view shows the situation where the axial force at secant pile is transferred to below bursting level with steel core piles.

Other challenges resulting from the presumed steep rock surface and possible vertical overhang was skidding of secant pile and lack of good pile foot in rock at installation. With uncertainties in result of pile foot installation in the steepest area, it was prepared for the use of jet grouting under, in front of and back of the pile foot in addition to insert multiple rows with tie back anchors. Establishing a blasting contour near the secant pile wall also gave uncertainty about the rocks capacity to accommodate the large horizontal forces at the pile foot. The solution here is to pre-stress tie back anchors at pile foot to horizontal equilibrium.

It was in early stage of planning considered to drill the secant pile into rock with stop at bursting level to avoid the above-mentioned issues. But this was considered to be too insecure when there was little experience in Norway with drilling secant piles deep in to bedrock. Since there was uncertainty about the solution and it would increase both the costs and time consumption, a solution that were not dependent on long drilling in bedrock was chosen.

Preliminary calculations showed naturally very large forces on the wall and caused many rows with tie back anchors. Especially when there was a desire to have continuous horizontal pad rows across all computing section and the need for the use of reinforced concrete pads. The starting point was to use 1200 mm secant piles since the piles had great rigidity and capacity for optimization of the support of the wall. Optimizing went on reducing the number of rows with tie back anchors and center distance between the ties. This for taking advantage of the great stiffness and moment capacity of the secant pile wall and reduce construction time, when establishing each back tie level is time consuming due to long anchors and use of reinforced concrete pads. It is considered that only the reinforced secondary piles are taking the forces acting on the wall. The primary piles are filling material to ensure a tight wall.

5 UNDERLYING ASSUMPTIONS FOR DIMENSIONING OF SECANT PILE WALL

In early phase a simpler calculation program was used to get a sense of forces acting on the secant pile wall. Furthermore, it was only the work of FEM simulation program to account for oblique stratifications, slanting rock stream and the impact on the wall from the underlying hillside traffic load from E16. In the drained layers the soil model *"Hardening Soil - Small Strain"* was used with empirical data for input of strength- and deformation parameters. For the soft clay the soil model *"Hardening Soil - Small Strain"* is used in effective stress analysis (long term) and "*NGI-ADP*" in total stress analysis (short term). The soil model "*NGI-ADP*" is developed especially for marine clays.
For a support wall structure of such a dimension that is to be established at construction pit Mølla, there is no possibility of using conservative parameters in the dimensioning. It is dependent on using realistic strength- and stiffness parameters in calculations. Therefore is "soil tests" performed in the FEM program to determine

parameters for the clay which corresponds with the real triaxial tests.

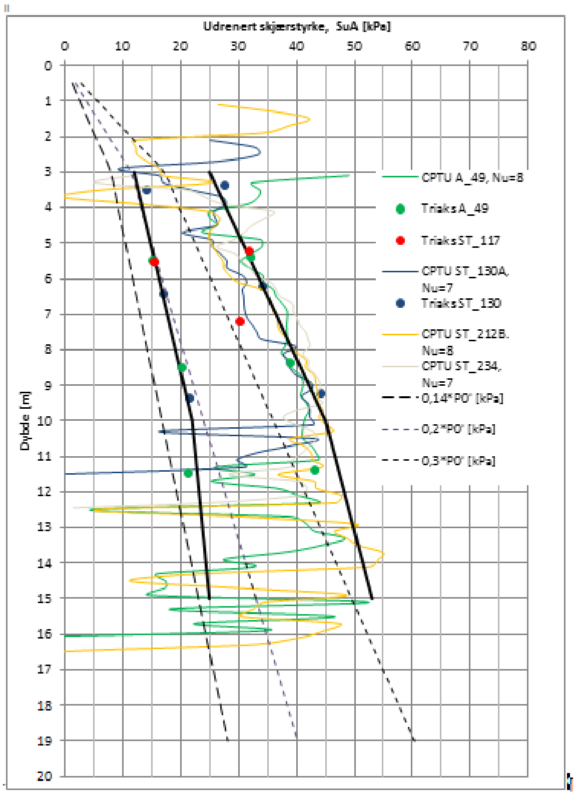

Figure 5 Selected shear strengthening profile for clay layer.

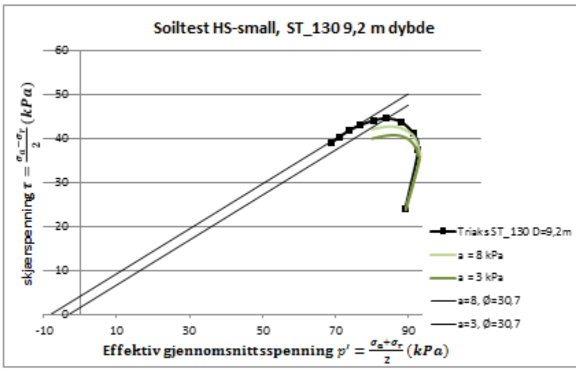

Figure 6 Assembly "soil test" and triaxial testing, ST_130 at D = 9,2m - Small strain.

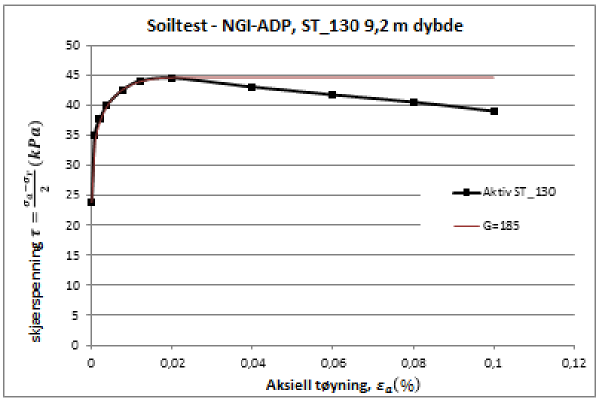

Figure 7 Assembly "soil test" and triaxial testing, ST_130 at D = 9,2m - NGI-ADP.

With high groundwater levels and hydrostatic pore pressure with depth, the large part of the forces acting on secant pile comes from water pressure. Normal practice for design of retaining walls of this type in FEM simulation programs, is to multiply the characteristic forces from the calculation with a load factor, in this case equal to $\gamma_{t\,=}$ 1.40. This means that the contribution from the water pressure acting on the wall will be multiplied by the load factor, when it is not possible to separate the water pressure as a separate load. Designed forces acting on the secant pile wall will be unnecessarily large as the uncertainty related to the water pressure is minimal. At least in relation to the choice of soil parameters, which the load factor actually is intended for. Calculations are therefore performed with reduced water pressure in the ultimate limit state (ULS). It is additionally performed calculations with full water pressure acting on the wall, but this as an accident limit state (ALS).

With four calculating sections, several soil models, variations in water pressures, testing various number of rows with tie back anchor and center distance of the ties, numerous calculations was performed. With an optimized solution in terms of the number of tie back rows and center distance between the ties, results of the calculations show that the secant pile wall has sufficient capacity in ULS also for full water pressure. The tie back anchors have only sufficient capacity in ALS for full water pressure. When dimensioning the anchors in ULS, the water pressure is reduced to 70% of the original hydrostatic pore pressure.

To reduce the water pressure behind the secant pile wall, several wells will be established to lower the ground water table/water pressure behind the wall during the excavation. Since there was some uncertainty of the permeability of the soil, a full-scale pumping test was performed to verify the effects of groundwater pumping.

6 FULL-SCALE PUMPING TRIALS

The full-scale pumping test was performed with two pumping wells. A total of 6 piezometers, each with indicators at 5 different depths, were installed close to the pumping wells before the beginning of the test. An immediate response was detected by the piezometers when the pumps were started. After only one day of pumping, the desired level of 70 % of the original water pressure level reached. The trial went on for 72 hours and the original water pressure went back quickly after ending the test. The response in the clay layer was slower, as expected, but also here the desired water pressure reached. Figure 9 show the location of wells and piezometers. Figure 8 and Figure 10 results from the pumping test

Figure 9 Location of the pump wells and piezometers in connection with the full scale pumping trials.

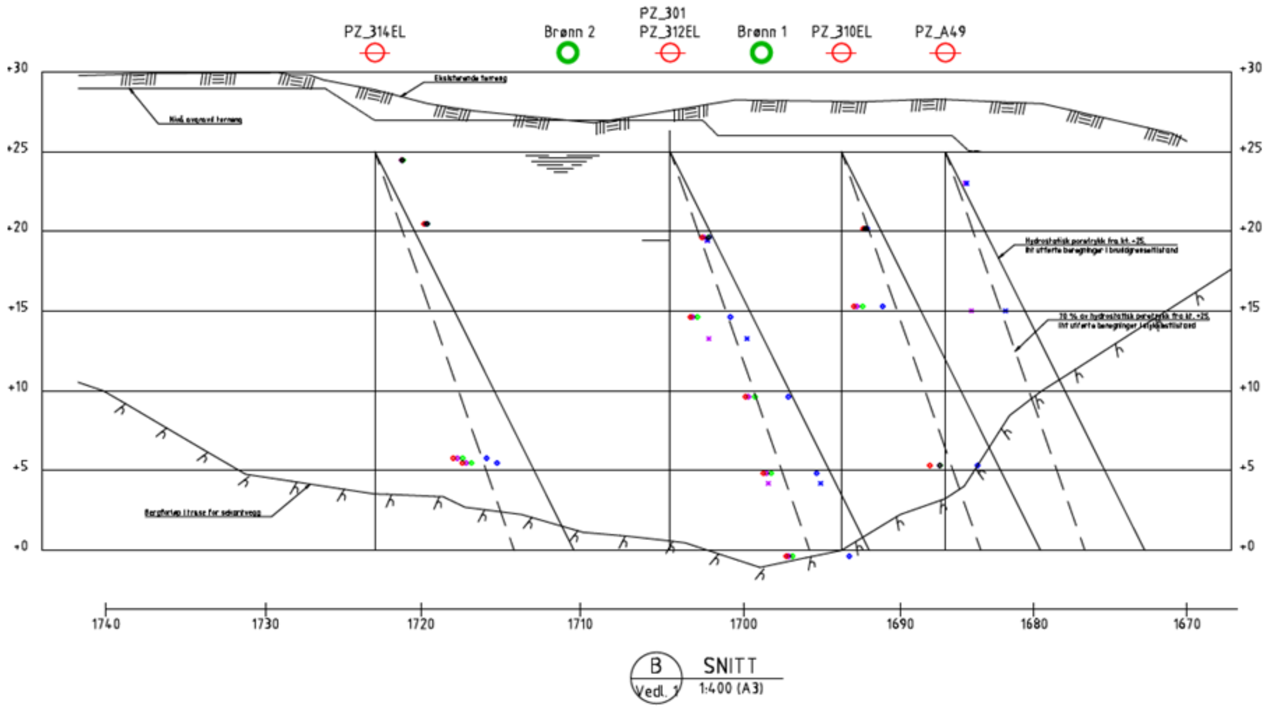

Figure 8 Section along secant pile wall with result from the full-scale pumping test. Blue marker indicates water pressure before pumping, and the other colour shows water pressure after 24, 48 and 72 hours of pumping.

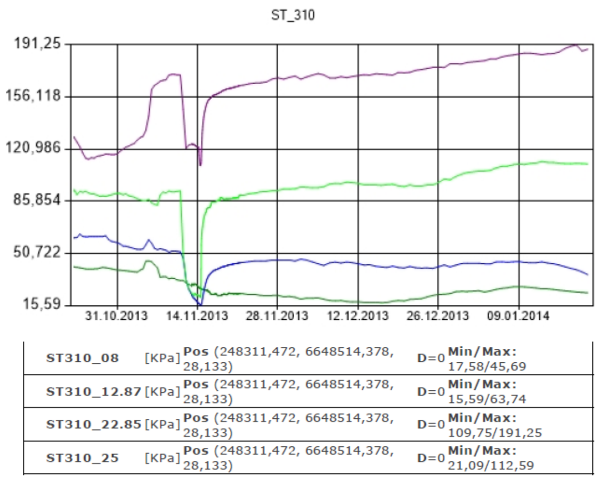

Figure 10 Results from trial pumping at piezometer ST_310. Graphs showing variation in pore pressure at different depts.

The conclusion of the full-scale pumping test is that the soil is permeable and lowering the ground water table to desired level behind the secant pile wall is achievable. Due to the immediate response to original pore pressure level, the construction is dependent on reliable pumps that will continue pumping during the construction phase without interruption. It will therefore be installed two more pumping wells behind the wall for a more robust situation.

The area nearby the excavation site has a thick layer of soft clay over moraine layer and is sensitive for ground deformation due to pore pressure changes. To minimize the effect of the surroundings, and especially the transformation station at Statnett’s facility, two or more infiltration wells will be installed. While the ground water table/water pressure is lowered behind the secant pile wall, water will be infiltrated into the ground at the infiltration wells below the transformation station.

7 INSTRUMENTATION OF EXCAVATION PIT MØLLA

The cut and cover excavation at Mølla is unique in Norway and there is little experience in the dimensioning of supporting structures with such large excavation depths. Especially with the challenging soil conditions with soft clay over hard moraine. When it is assumed in the design that the water pressure behind the wall is reduced, close monitoring is important. An extensive instrumentation program of the secant pile wall will be installed. Deformation of the secant pile wall, anchor forces and pore pressure will be followed up close during excavation. All data will be remotely monitored and limits will be set with alarm levels and SMS warning. Considering the surroundings nearby the construction pit, deformation sensors are installed.

For the secant pile wall, deformation monitors will be installed in five sections and load cells mounted on the bars in all levels in three. To verify measured values from remotely monitored load cells, supplementary load cells with manual sensors are used in addition. For control of pore pressure distribution at the wall, already installed piezometers from pumping test will be used. Longitudinal section of secant pile wall with all the remote reading instrumentation installed is shown in Figure 11.

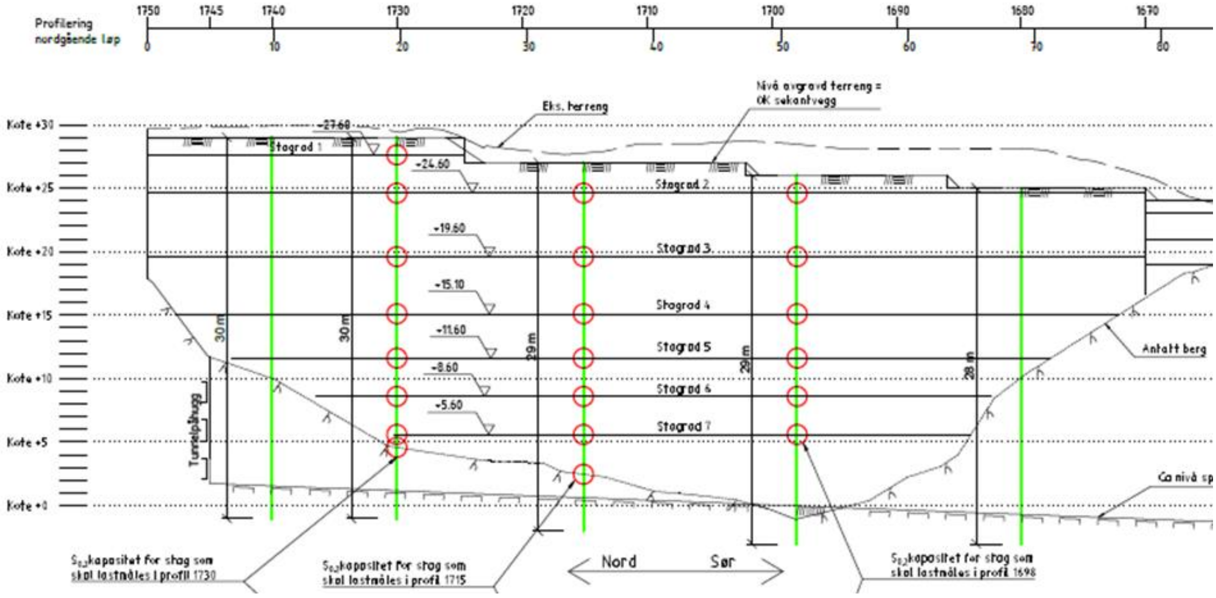

Figure 11 Longitudinal section of the secant pile wall showing remote reading instrumentation. Green lines are location of the displacement monitors and red circles are planned load cells.