REMEDY
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BegrensSkade/REMEDY

Risk Reduction of Groundwork Damage

Deliverable 3.3

Numerical hydrogeological modelling of drainage to an excavation

Work Package 3 – Hydrogeological methods, drainage and grouting

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Summary

The report regards the conceptual and numerical modelling of drainage scenarios to a planned excavation in a deep clay deposit in Oslo, Norway. The modelling was performed in SEEP/W 2D. Ground investigations indicate that the clay lies directly on top of the bedrock, without a layer of moraine between them. The upper metres of the bedrock is assumed to be weathered and serve as a draining layer. The drainage of groundwater to the excavation along steel core piles, and the mitigating effect of cut-off walls of various lengths, were modelled. There were made several versions of the numerical model, with a variation of conductivity and upstream boundary conditions. The simulation results were compared with empirical pore pressure data presented by the R&D project BegrensSkade I (2012-2015). The simulation results deviated notably from the empirical data, leading to the conclusion that the model itself is unreliable. Although there were found weaknesses with the model that could have been improved and resulted in better compliance, some major uncertainties are considered to remain and to affect the results to a significant degree. The uncertainties are in particular related to the modelling of the (weathered) bedrock conductivity, i.e. how the groundwater flows horizontally, vertically and across large distances within the bedrock joints. The discussion of these uncertainties is only introductory, and there is first of all a need to study the literature in greater detail to find out whether these issues are as important as argued here, and whether researchers already have studied them sufficiently. If the literature study comes to the conclusion that more research is needed, two modelling tasks are proposed with the aim to get a better understanding of how the groundwater flow could have been modelled more realistically in a continuum two-dimensional model.

Contents

Appendix

Review and reference page

1 Introduction

Urban ground works such as deep excavations performed in soft clay may cause damage to neighbouring buildings and structures. Drainage causes pore pressure drawdown, followed by consolidation settlements. The costs related to settlement damage can be substantial and there is a considerable potential for reducing these costs.

Data and observations presented by the Norwegian R&D project BegrensSkade, suggest that drainage to excavations is one of the main causes of settlements and damage. One of the reasons for this is that the problem is not well understood (Baardvik *et al.*, 2016).

The risk of drainage and pore pressure reduction can be reduced in the early design phase of a project by undertaking the necessary type of investigations to understand the hydrogeological and geotechnical conditions at the site. In addition, one may select construction methods and mitigating measures to reduce the risk of drainage. A way to

Figure 1 illustrates ideally how the groundwater level in the surrounding soil is lowered as the construction work in an excavation proceeds, and how the groundwater level may be restored after construction has been finalized.

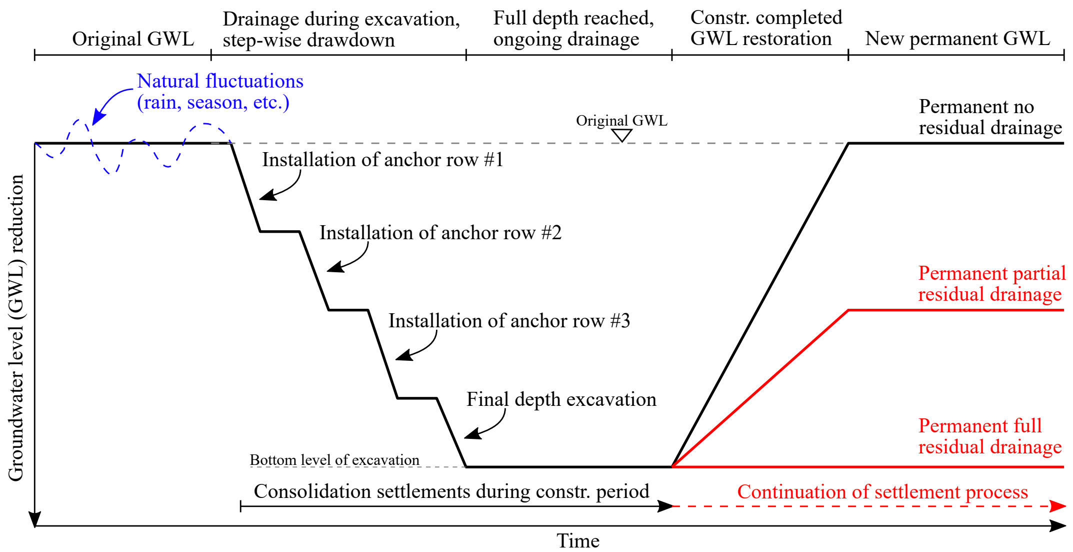


Figure 1: Illustration of impact on pore pressure drawdown in permeable deposits before, during and after a deep excavation project. The red lines indicate a new permanent lower groundwater head compared to the original head, resulting in a continued settlement process even after project completion. Note that vertical scale of drawdown is exaggerated.

1.1 Background

Experience from case histories of deep excavations in typical Norwegian ground conditions reveal that (1) the drawdown is often equal to the elevation at which aquifer drainage was initiated (e.g. the bottom of the excavation during piling), (2) even small seepage rates may cause significant drawdown and (3) once drainage is initiated, there is a persistent challenge to permanently re-establish the initial head levels, even when considering remedial measures.

In order to predict the settlements that may be caused by a certain design and location of an excavation, the engineer must decide on a realistic value of pore pressure reduction caused by drainage to the excavation. The reduction can typically be determined in two ways:

1) By using empirical data from similar projects (i.e. similar ground conditions and excavation/foundation methods).
2) By mathematical analyses of the ground water flow due to excavation and mitigation measures. Often, this would include numerical modelling.

As part of BegrensSkade I (2012-2015), pore pressure data from 17 construction projects throughout Norway was compared with respect to the measured pore pressure draw down at various distances from the excavation, the excavation characteristics and the mitigating measures used. Figure 2 shows these data presented as pore pressure reduction normalized with respect to the excavation bottom level below the groundwater potential surface, as function of the distance from the excavation. These curves may be used by the engineer to arrive at an estimate for the pore pressure reduction at various locations around the excavation.

Hydrogeological modelling enables the simulation of the effects of excavation and foundation works, as well as the ground conditions and the remedial measures with the purpose of reducing groundwater inflow and subsequent reductions of pore pressures surrounding the excavation. One of the possible remedial measures is installing cut-off walls using pre-excavation grouting with cement in the bedrock below the retaining wall so that the low-permeable barrier is extended downwards.

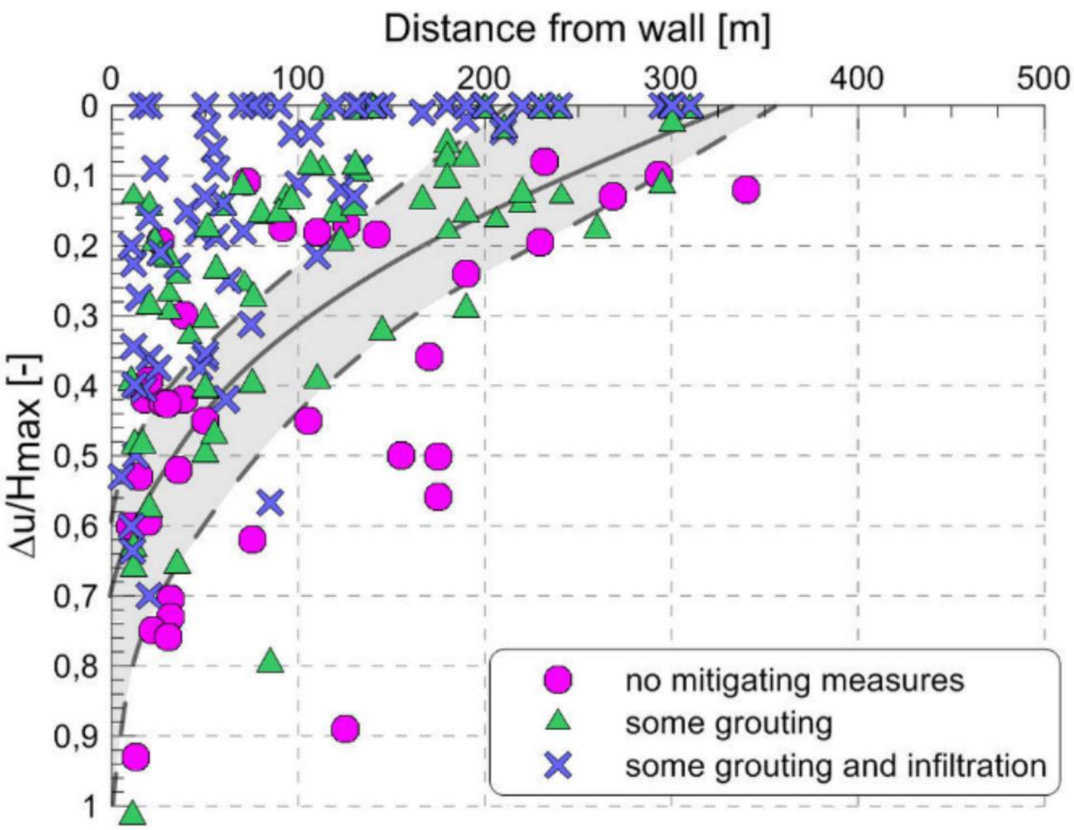


Figure 2: Measured pore pressure reduction Δu plotted against distance from the excavation wall. Δu is normalized with respect to the largest possible pore pressure reduction H_{max}, which equals the excavation depth below the hydraulic potential at bedrock.

1.2 Purpose of work

This study investigates the use of hydrogeological modelling as a tool to assess the effect of drainage, as an alternative to the use of empirical data.

The overall purpose of the work was to gain a better understanding of the applicability of numerical simulations of a 2D-model to predict the influence an excavation, piling and the use of injection cut-off walls have on the pore pressure levels in the surrounding soil. A key question was whether the ground water flow could be modelled realistically or not. A secondary aim in this regard, was to study what influence modelling effects such as boundary condition and bedrock conductivity had on the simulation results.

1.3 Structure of work and report

The work summarized in this report consisted mainly of three steps:

1. Developing a conceptual model of the soil and ground water flow conditions in the project area.
2. Developing a numerical 2D model based on the conceptual model.
3. Run simulations on the numerical model and continuously analyse the results with respect to empirical data

The report is structured similarly:

Chapter 2 – Description of the case study
Chapter 3 – Development of the conceptual model
Chapter 4 – Development of the numerical model
Chapter 5 – Main results
Chapter 6 – Discussion
Chapter 7 – Conclusions

2 Case study – Campus Ullevål Deep Excavation

Norwegian Geotechnical Institute (NGI) is as of early 2022 planning to dismantle its office building in Oslo, Norway and construct a new, larger building that will host both NGI and other companies. This project is called Campus Ullevål.

2.1 Description of area

The site lies just south of the highway Ring 3, between Sogn colony garden and Ullevål national football stadium, see Figure 3. The terrain is planar, and the site is at an elevation of +97 to +98 masl. The site is located on top of a sediment basin mostly consisting of clay and some coarser layers. The ground conditions are detailed in Section 3.3.

Figure 3: Overview of the area, with approximate property boundaries of Sognsveien 72.

2.2 Description of construction plans

The planning is still in preliminary stages, and the exact solutions and design of the excavation have not yet been decided. However, the dimensions of the excavation and the foundation principles are quite clear at this point. The building will have one floor below the terrain level, resulting in a required excavation depth of approximately 5-6 m (91 masl.). Figure 3 shows the property area as a red dotted line. The lateral extents of the excavation will almost equal the property area. The width will range from 70-100 meters. The building will be founded on piles, with the use of at least some steel core piles bored to bedrock. The excavation will be supported by sheet pile retaining walls with inner struts. It has not been settled whether the sheet pile wall itself will be rammed to bedrock or designed to float in the clay, or a combination of the two. The construction period is assumed to be one to two years, for which the excavation may cause pore pressure drawdown.

3 Conceptual model

The conceptualization of the problem consisted of gathering information about the soil and ground water conditions, and then using this to understand how the water likely flows in the area surrounding the excavation. This was done with help of tools like ArcGIS Pro as well as Leapfrog Works. This chapter describes the process of developing the conceptual model. Conceptualization of a hydrogeological system is described further in general terms in the state-of-the-art report by Kahlström and Langford (2022).

3.1 Background data

Throughout the last century, the neighbourhood to the site have seen high construction activity. A natural consequence of this is that there have been gathered a substantial amount of data from ground investigations. Most of the data have been collected by the City of Oslo and digitalized in "Undergrunnsarkivet" (UA), an archive of ground investigation data from around Oslo. Key parameters, such as interpreted depth to bedrock (or bedrock elevation), is available to the public and may be requested from the Department of Geodata in the Planning and Building department. During the work with this project, such a request was put forward for the area shown in Figure 4.

In addition to the geotechnical soundings and readings of exposed bedrock received from UA, the database used in this project included data from a few boreholes drilled by NGI and others. Among those data is piezometer readings in the nearby area to the site. ETRS 1989 NTM Zone 10 and NN2000 was used as spatial reference for all handling of geodata.

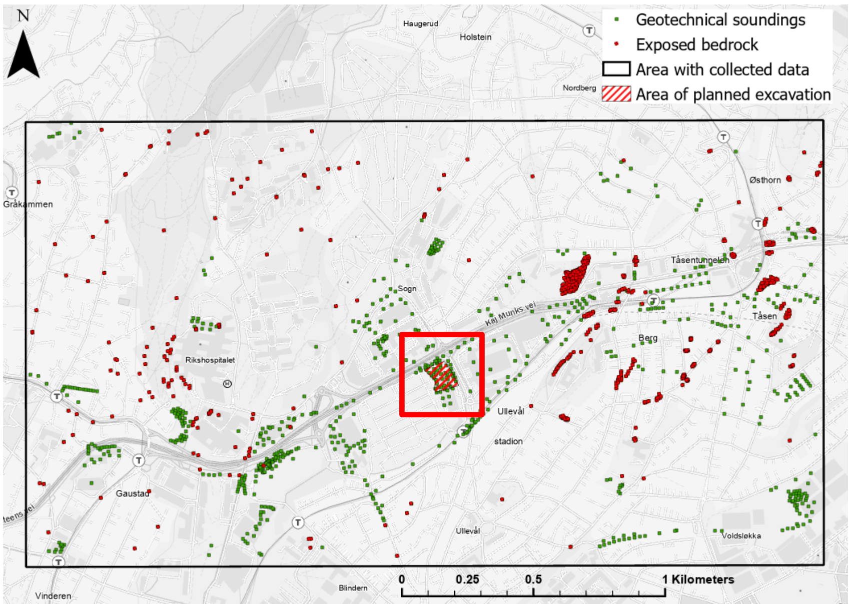


Figure 4: Map showing the area where the ground data are collected from. Received data from UA is shown in red (bedrock without soil cover) and blue (geotechnical boreholes). The red frame encloses Sognsveien 72.

3.2 Ground model

The terrain level coordinates (X, Y) and the interpreted elevation of bedrock (Z) from the geotechnical soundings and readings of exposed bedrock were imported from ArcGIS/Excel to Leapfrog Works along with a digital terrain model (DTM). In Leapfrog, the (X, Y, Z)-points were used to generate a model of the bedrock surface with the same lateral extents as the DTM (frame in Figure 4). A raster map (with cells 5m x 5m) of the soil thickness was then generated based on the bedrock surface and DTM in ArcGIS. The map is shown in Figure 5 along with the data it is based on.

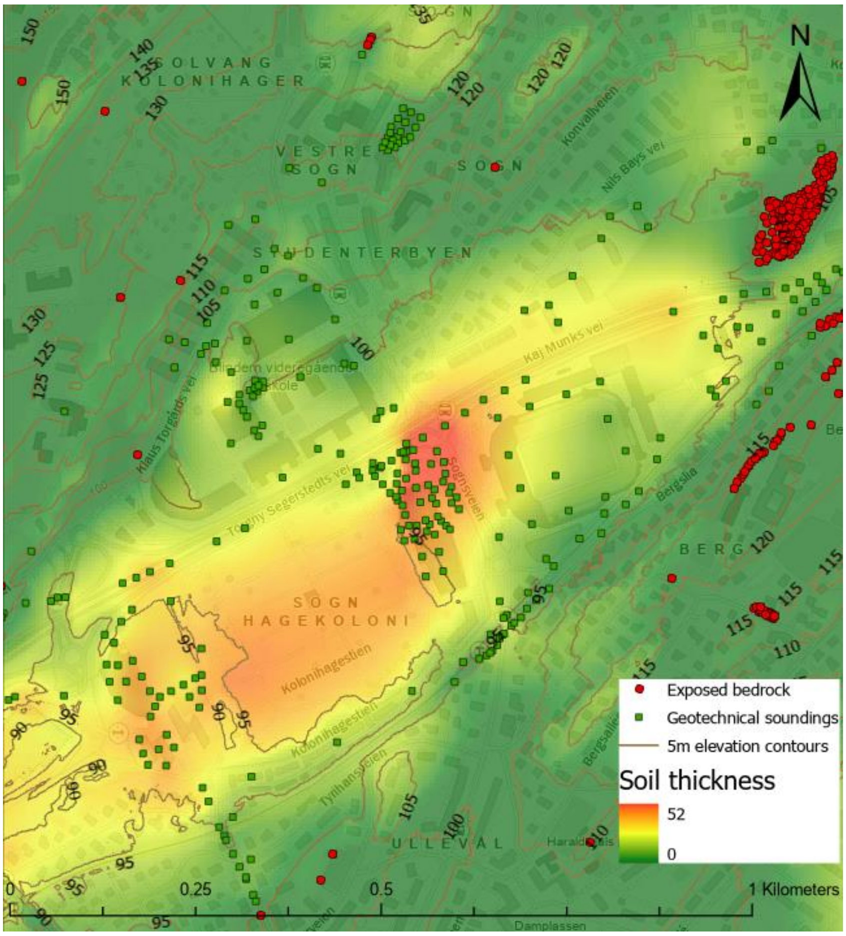


Figure 5: Map that shows the modelled soil thickness around Sognsveien. Data points used for the interpolation of the rock surface is included to visualise the uncertainty.

3.3 Geological setting

As the soil thickness map in Figure 5 illustrates, Sognsveien 72 is located on top of a 20-50 m deep sediment basin. The basin stretches from north-east of Ullevål stadium in a south-westerly direction towards Blindern. It is confined by a bedrock ridge in

southeast and the hill towards Sognsvann in northwest. The marine limit in the Oslo area is approximately 220 m (Norwegian Geotechnical Institute, 1971; Nakrem, 2005).
Around the site, below two to three meters of filled masses and dry crust, the soil stratigraphy consists mainly of marine clay of varying coarseness and sensitivity (Langford, 2018). As is indicated by the soil thickness map, there is a depression in the bedrock surface beneath and north of the site. In this area, the soil thickness varies from approximately 30 m towards 50 m. The upper ten meters or so, determined to be of postglacial age, is mostly firm with a low sensitivity. The glacial clay below has a higher sensitivity and is quick at several depths. Throughout the entire profile, the clay includes interposed layers of silt, sand and gravel. The inhomogeneity in the different parts of the profile may have different explanations. Changes in the depositional environment seems like the most important explanation. These changes may have been caused by changing climatic conditions as well as the regression of the sea after the crust began to rise. However, in their internal report on the varying ground conditions within Oslo, Norwegian Geotechnical Institute (1971) notes that the inhomogenity seen in the upper 2-5 meters of the soil profile at Ullevål stadium may be partly explained by the soil including masses from old slide events. Although slide activity may also have contributed in part to the variations seen at greater depths, the high sensitivity indicates that this clay have retained its original grain structure, as inherited from the deposition of flocculated grains in the saline water. Changes in the depositional environment are therefore more likely to explain the variations in the deeper part of the profile.

Unlike what is the case at several other areas in Oslo, NGIs' geotechnical soundings from Ullevål do not indicate a layer of moraine lying on the bedrock surface. Thus, the groundwater transport below the clay can be assumed to occur through joints in the bedrock. The bedrock in the area consists for the most part of alternating sequences of shales and limestones (NGU - Geological Survey of Norway, 2021). These sedimentary rocks are of Ordovician age and are locally interrupted by dykes of younger, Permian eruptives. As part of the planning, tunnelling and operation of the Tåsen road tunnel, located about half a kilometre west of Sognsveien, the bedrock stratigraphy was carefully investigated and analysed. Most dykes were found to stand almost vertically, while a few interrupted the bedding horizontally.

3.4 Hydrology

The catchment area to Sognsveien 72 is shown in Figure 6 along with an extension of it towards Oslo Science Park. The catchment areas are generated in NVEs hydrology analysis tool (NEVINA). The site receives runoff mainly from the north valley side where the catchment extends up towards the east-side of Sognsvann. The catchment area with outlet at Sognsveien 72 is 2.4 km^2 and has a specific discharge of 17.4 l/s*km^2 (560 mm per year) according to the NEVINA analysis. It receives about 800 mm precipitation yearly.

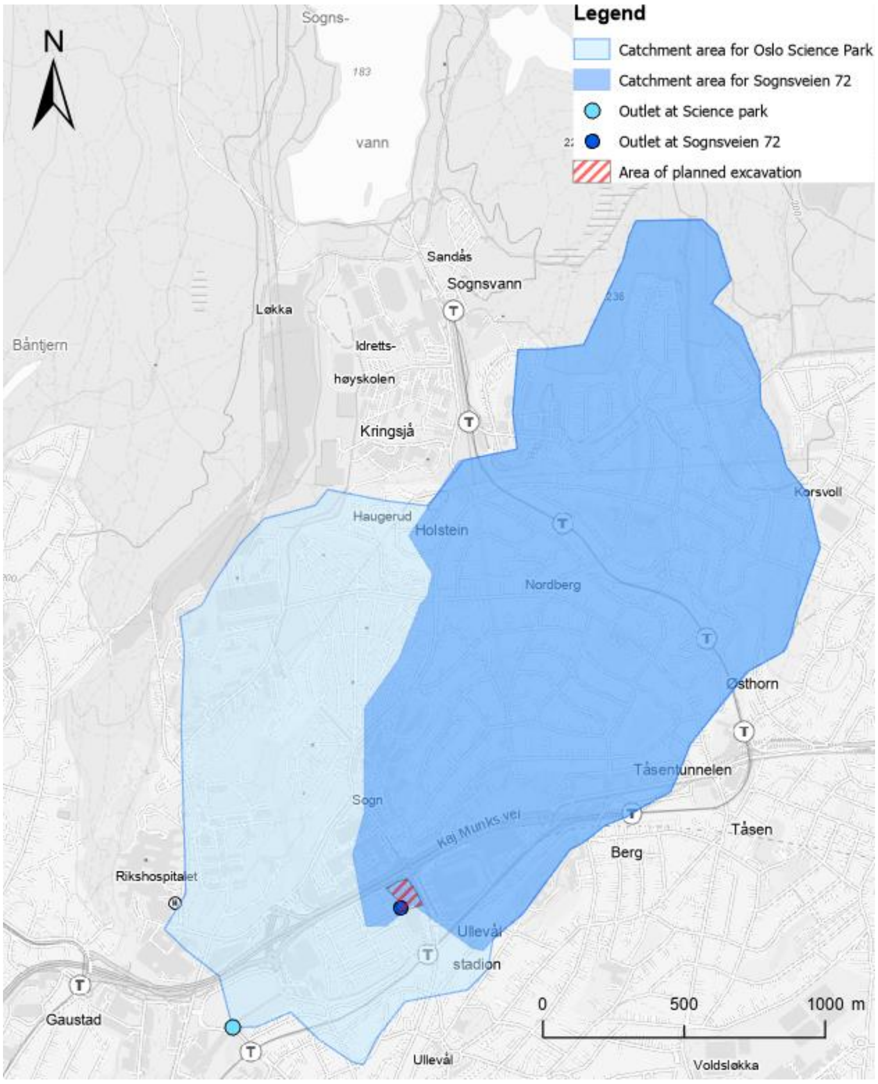


Figure 6: Relevant catchment areas. The catchment area with outlet at Sognsveien 72 has a surface of 2.4 km^2 and is incorporated in the larger catchment area with outlet at Oslo Science Park (A=3.75 km^2).

3.5 Hydrogeology

The hydrogeological conditions at and east of Sognsveien 72 are fairly well known. Figure 7 shows the location, terrain level and the total ground water head level of the piezometers that have currently been used to assess the ground water flow in the area. In connection with instalments of energy wells at Ullevål stadium in 2007-2008, 8 piezometers (PZ1-8) were installed in the vicinity to the stadium to monitor the pore pressure fluctuations during the construction work. All piezometers were installed with the tip at bedrock level. In addition to these, the collection of pore pressure data includes readings from three newer piezometers; SVV, N08 and N25. N08 and N25 were placed by NGI in September 2021. N08 is only measuring in the clay, not at bedrock level.

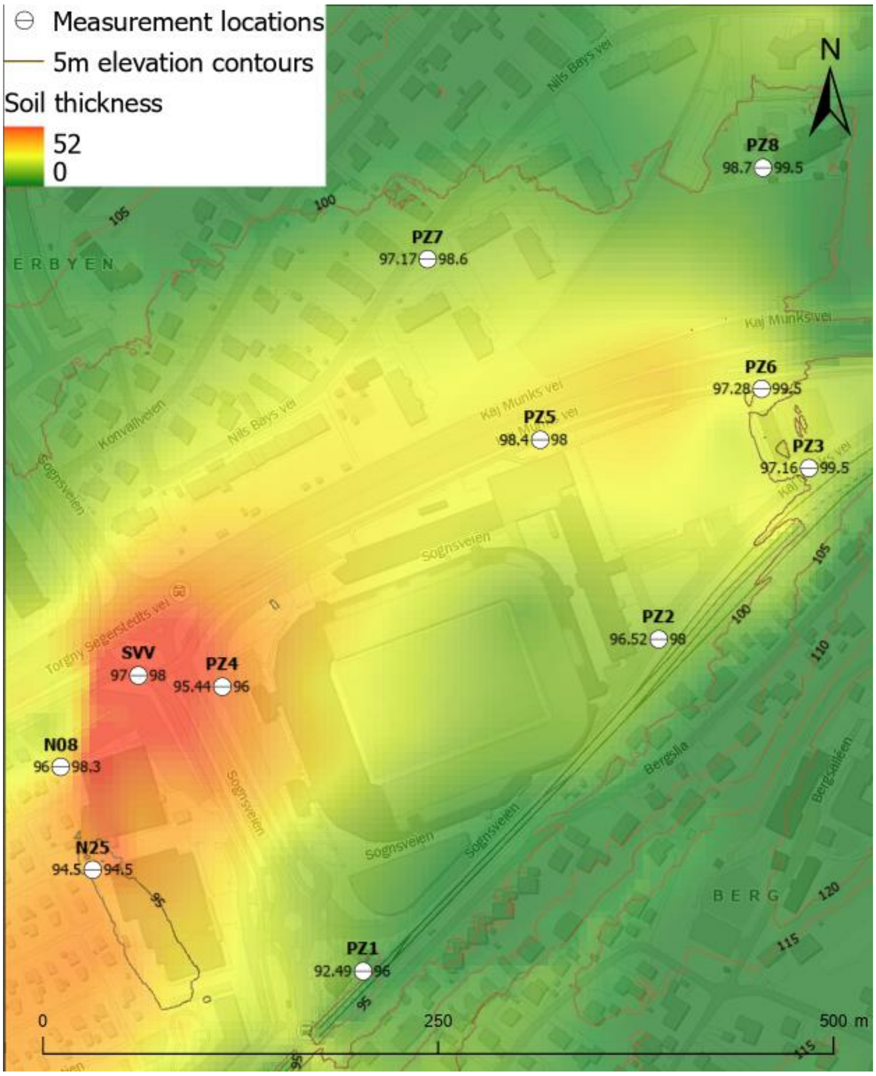


Figure 7: Map of Ullevål with the locations of the piezometers placed in the vicinity to Sognsveien 72. The value to the left of the piezometer symbol is representative for the total head through the year. The value to the right is the terrain level.

The pore pressure measurements indicate a slightly artesian water pressure east of Ullevål stadium. At Sognsveien 72, where the terrain level is approximately the same as around the stadium (97-98 masl), the groundwater potential is located around 1-2 meters below the terrain. Although there are some uncertainties in the total head estimates due to seasonal variations, it appears clear that the groundwater potential decreases south-westwards in the basin. Further, note the low potential (92.5 masl) at PZ1 and other indications that the pore pressure also decreases notably directly southwards. There is some uncertainty associated with the PZ1-value, but it is not significant enough to affect the assertion that the groundwater flow around Sognsveien inclines towards the bedrock ridge that acts as a barrier in the southeast.

The piezometer readings around Ullevål are not distributed well enough to enable the generation of a realistic groundwater equipotential map which covers the extent needed for numerical modelling. As supplement to the piezometer points, there were therefore placed points manually at strategic places both inside and outside the relevant extent, including along rivers and exposed bedrock. The points were attributed with a value thought to represent the groundwater potential at the location. First, twelve points were placed along the nearby rivers Sognsvannsbekken in the west and Akerselva in the east. Equipotential lines were then generated by using the "contour" tool in ArcGIS. These lines did clearly not represent the actual situation across the southeastern bedrock ridge. Therefore, an additional set of points were added to the ridge, locating the groundwater level at terrain. Also, PZ4 and PZ5 were removed. Although no errors could be found in their readings, and none were commented in the associated internal reports, the author believed that the actual groundwater conditions around Sognsveien 72 are better visualised by equipotential lines which disregard PZ4 and -5. Figure 8 shows the resulting equipotential lines.

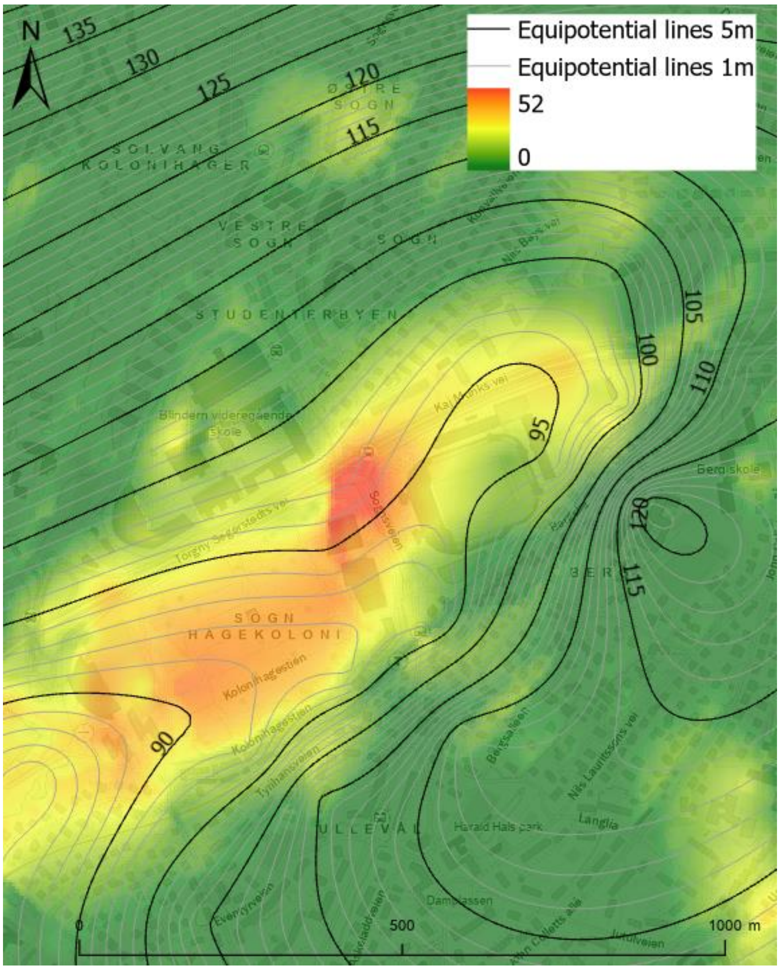


Figure 8: Map which shows the equipotential lines along with the soil thickness.

4 Numerical model

4.1 Software

The numerical 2-dimensional model was developed in SEEP/W, a program that is part of the GeoStudio package. SEEP/W is a finite-element software product for modelling groundwater flow in porous medias. It allows for modelling of both saturated and unsaturated groundwater flow. In this case study, all materials were modelled as saturated. The 2019 version of the software was used.

4.2 Geometry

Originally, it was intended to simulate the groundwater conditions in two profiles: one parallel to the basin and one parallel to the flow direction at the excavation site, i.e. perpendicular to the equipotential lines. However, it was decided to focus on the profile along the basin. The advantages and disadvantages of the two profiles will be discussed in Chapter 6.

The left (west) and right (east) boundary were set 500 m away from the excavation pit. The large distance was assumed to reduce the influence of the boundary conditions on the pore pressure close to the excavation. In addition, both boundaries are close to locations where the pore pressure at bedrock is known. Figure 9 shows the cross-section line A-A', while Figure 10 shows the model.

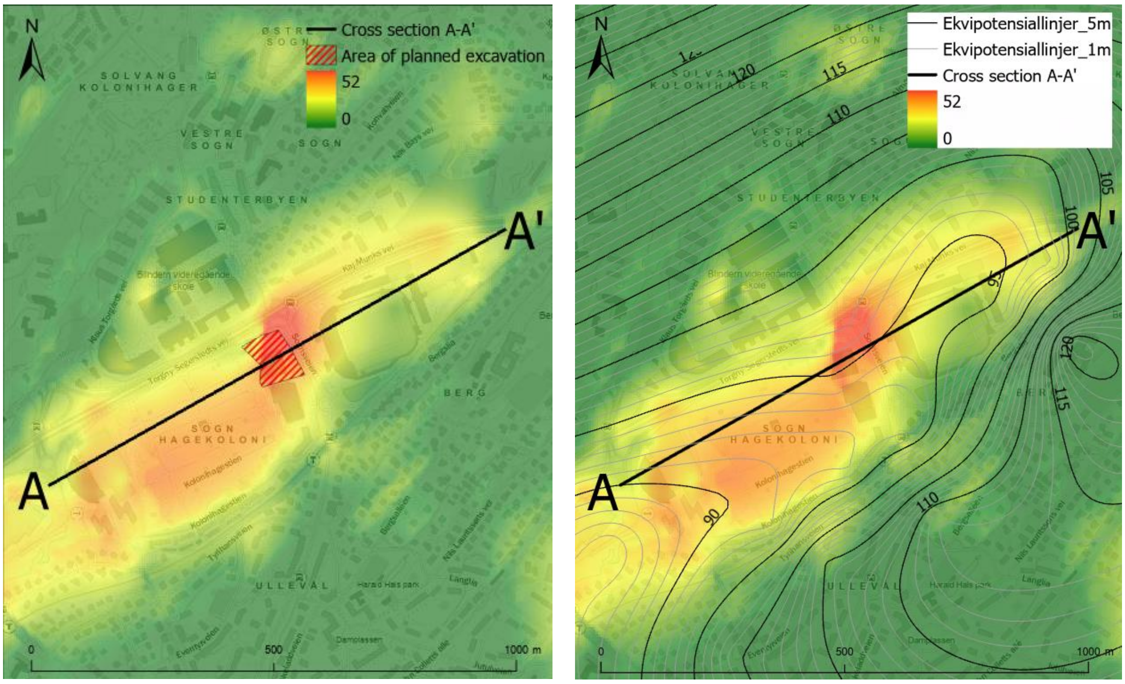


Figure 9: Cross section A-A'.

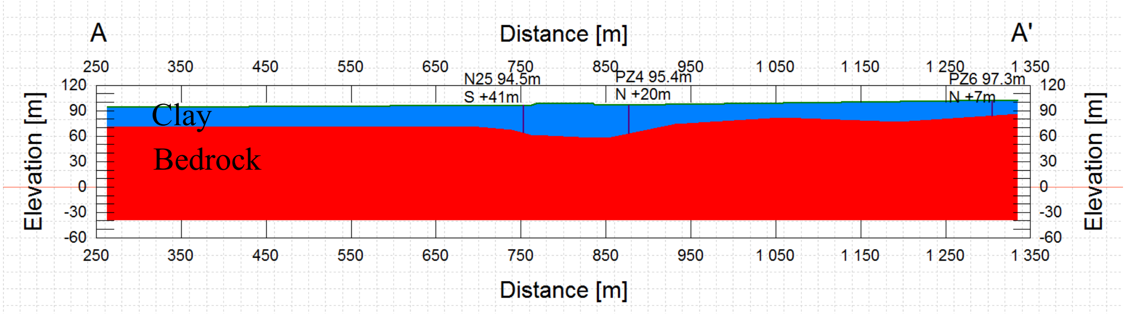


Figure 10: Cross section A-A' as modelled in SEEP/W. The blue material is clay, the red is the bedrock.

4.3 Ground profile

Figure 11 shows the depth profile of the material model at the right, upstream boundary. The ground was modelled as two layers:

1. Clay from the ground and 12-40 meters down. Lowest point at +58 masl.
2. Bedrock down to -40 masl.

The inhomogeneous clay was modelled as one layer due mainly to two reasons. First, because it is assumed that the bedrock has a higher conductivity than the clay, most drainage will happen from bedrock level and up along the steel core piles, not through the pores of the clay. Therefore, as long as the average conductivity of the clay is modelled reasonably, variations in it is not that important for the groundwater challenge believed to be most prominent. Second, the aim of this work is to assess the modelling method itself and to study how different lengths of cut-off walls and variations of the material properties and boundary conditions affect the simulation results. Potential groundwater challenges caused by overpressure in drained interposed layers in close vertical proximity to the excavation floor is not in the scope of this work. Therefore, the conductivity of the upper soil, and the variations of it, is not that important to model accurately in this work. Regardless, the complexity of the clay with interposing layers of silt, sand and gravel is practically impossible to model exact, and thus, simplifications are completely necessary. The filled coarse masses on top of the clay are not important for the groundwater challenges that is studied.

The bedrock was modelled as one material with a thickness of 100-120 metres. Initially, it was intended to model a 1m thick draining layer above the bedrock. However, the calibration of the model was performed both with and without this draining layer. It showed negligible difference in results, and therefore, it was decided to model the bedrock as a single layer, not differentiating between the weathered and non-weathered parts of the bedrock.

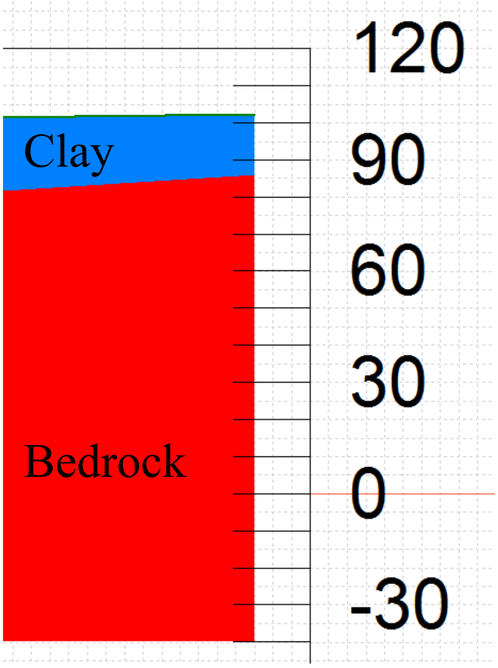


Figure 11: Depth profile of the material model at A'.

4.4 Hydraulic boundary conditions

As discussed in Section 3.5, the groundwater around the planned excavation is likely to flow towards south-southwest because of the pressure differences present. In the 2D profile from northeast (A') to southwest (A), it will therefore flow from right to left. Table 1 presents the right and left boundary conditions.

Table 1: Hydraulic boundary conditions. Note that for the right boundaries, several options was used. Note that the left boundary was changed slightly as the bedrock conductivity was changed (90.3m with k=5e-06 m/s, 90.0m with k=5e-07 m/s). The right boundary was modelled both as constant flux and constant head, but only for the k=5e-07 bedrock conductivity.

State	Left boundary	Right boundary
Steady, initial conditions	H = 90.3m/90m	H = 99.5m
Transient, after excavation	H = 90.3m/90m	Q = 4e-10 $m^3/s/m^2$ or H = 99.5m

At steady-state, both the downstream (left) and upstream (right) boundaries were chosen to be modelled as constant total head H. The values were determined by calibration. H_{left}, H_{right} and $K_{bedrock}$ were changed until the steady-state analysis produced a groundwater potential around the excavation that were similar to the measured potentials. Section 4.6 describes the calibration process further.

At transient state, the right boundary was modelled both as water flux Q ($m^3/s/m^2$) and total head H. Q was set to equal the average water flux along the cross section corresponding to the steady-state H_{right}. The left hydraulic boundary was modelled with the same constant head value as at steady-state for both models (constant Q_{right} and H_{right}).

The boundary condition at terrain was modelled as the water flux corresponding to the conductivity of the clay. This was set equal to 1e-9 m/s, thus giving $Q_{terrain}$ equal to 1e-9 $m^3/s/m^2$. The modelling implied groundwater level at terrain. The boundary at the bottom of the bedrock layer was set as impermeable (Q=0).

4.5 Parameters

Table 2 gives the material parameters used in the material model.

*Table 2: Material parameters. * denotes values that were determined based on the calibration.*

Parameter	Clay	Bedrock	Pile leakage	Unit
Hydraulic conductivity K	1e-9	5e-6*/5e-7*	1e-4	m/s
Compressibility m_v	1e-5	0	0	kPa^{-1}
Saturated volumetric water content θ_v	0.7	0.01	0	-

The value of the hydraulic conductivity K_{clay} was based on empirical data for typical Norwegian clays (Karlsrud, Erikstad and Snilsberg, 2003; Statens Vegvesen, 2018). The actual permeability may possibly differ notably from this value, but as it nevertheless is considerably lower than the bedrock below, variance in permeability in the clay will not affect the results noteworthy.

The conductivity of the bedrock, $K_{bedrock}$, was determined based on a calibration of the model. Because of some uncertainties regarding the total head values at the boundaries, simulations were performed with two K-values, see Table 2. The results of this sensitivity analysis are presented in Chapter 5. The hydraulic conductivity values correspond with empirical values for similar bedrock in Norway presented by Dagestad, Hansen and Braathen (2003).

The conductivity of the pile leakage was set determined based on observations at excavation sites in Oslo showing practically immediate groundwater drainage as the piles had been bored to bedrock, thus indicating a high-conductive zone along the pile casings. Simulations with pile leakage equal to 1e-5 m/s gave negligible difference in results.

The compressibility coefficient m_v and saturated volumetric water contents θ_v are values that are typically used for similar bedrock. Variation in the values within reasonable limits have little influence on the results.

4.6 Calibration

The model was calibrated before the various scenarios were solved. The calibration included changing H_{left}, H_{rigjt} and $K_{bedrock}$ until the analysis of the initial conditions produced values of the total head H that were similar to the measured (real) values. Figure 12 shows the points where the resulting values of H was compared with real, measured values.

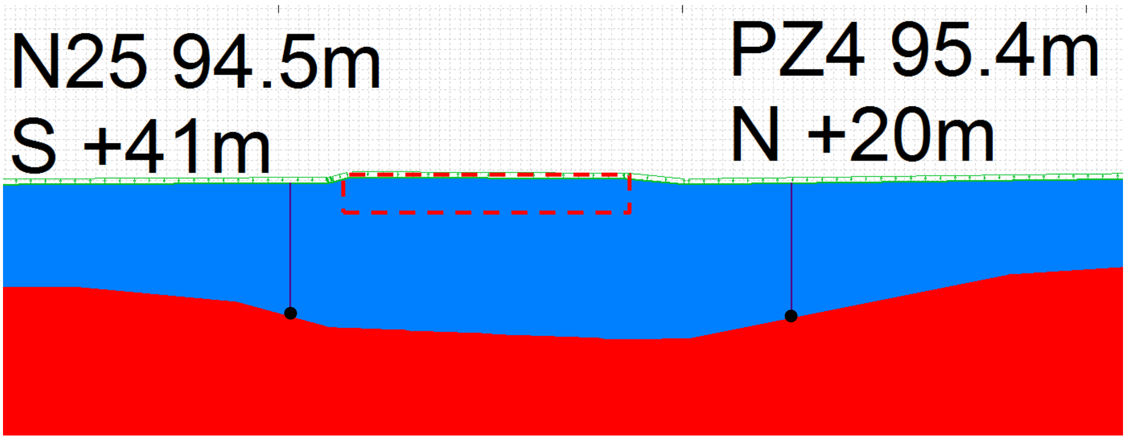


Figure 12: Clip of the model at initial conditions, showing the two points where H was measured and compared with nearby piezometer measurements N25 and PZ4. N25 was measured perpendicularly 41m south of the cross-section, whereas PZ4 was measured 20m north of the cross-section – both at bedrock. The red, dotted line shows approximate extents of the excavation that is modelled in the transient stage.

In essence, the calibration process was as follows:

1. Choosing a best-guess $K_{bedrock}$.
2. Set H_{left} and H_{right} similar to the values in the equipotential map.
3. Run a simulation of the initial steady-state conditions.
4. Compare the simulated H values with those measured at N25 and PZ4.

If the H values at step 4 corresponded sufficiently, the process continued to step 5, if not, step 2-4 were conducted again. If H_{left} and H_{right} had to be set to values not corresponding with the equipotential map to produce realistic H values at step 4, the process would have restarted at step 1, choosing a new $K_{bedrock}$. However, there was no need for that, as the two first-choices of $K_{bedrock}$ appeared to be reasonable.

5. Change the right boundary to constant water flux Q, with the value being set to the average of the resulting fluxes through the right-boundary elements. H_{left} is kept as it is.

6. Run a simulation of the initial state containing the new boundary conditions. This simulation was set to run for 10 000 years.
7. Compare the simulated H values with those simulated in step 4 (with H_{right} instead of Q_{right}).

If the H values from the step 7 simulation corresponded well with those from step 4, the calibration was considered a success. This was achieved at first try.

Table 3 presents the total head values as simulated by the model and as measured by piezometer.

Table 3: Values of total head [m] as measured nearby and as simulated. The values inside the parentheses () in column "Measured H" represents the value retrieved from the equipotential map at the exact location on the modelled cross section.

Location	**Measured H [m]**	**Simulated H [m]**	
		K = 5e-7 m/s	K = 5e-6 m/s
10m downstream (N25)	94.5 (95)	94.6	94.5
PZ4	95.4 (95)	95.8	95.8

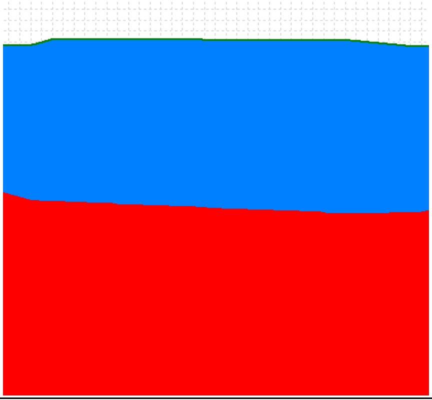
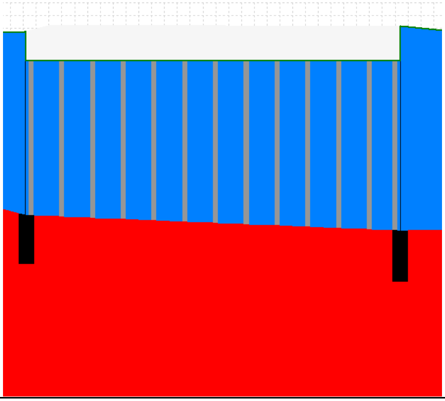
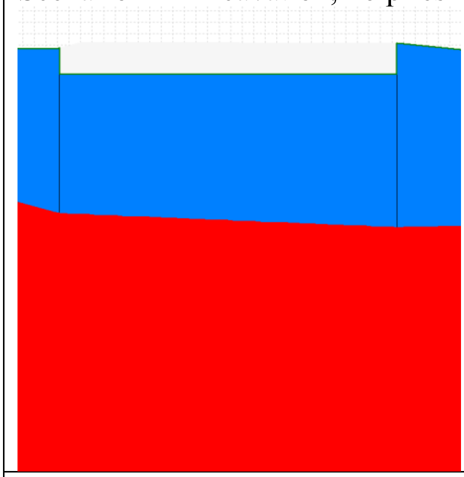
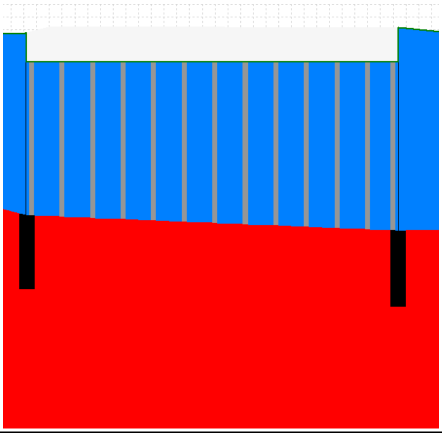
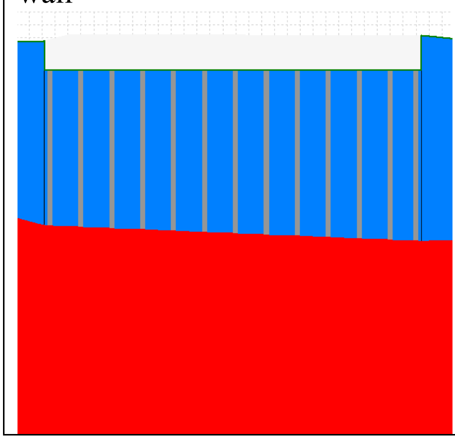
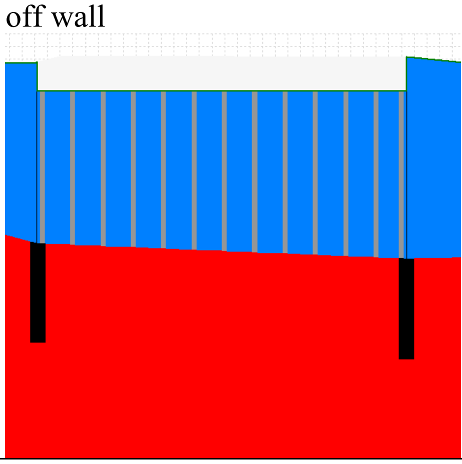
4.7 Scenarios

To study the effect of piling and the use of cut-off wall, five scenarios were simulated. Table 4 shows which features were active in each scenario. All scenarios were simulated with both values of the hydraulic conductivity $K_{bedrock}$ and with the two different upstream boundary conditions. The modelled scenarios are shown in Table 5.

Table 4: Simulated scenarios for the transient state. All scenarios also include the excavation.

Scenario	Piles	Sheet pile wall (SPW)	Injection cut-off wall
1	-	To bedrock	-
2	Yes	To bedrock	-
3	Yes	To bedrock	10m
4	Yes	To bedrock	15m
5	Yes	To bedrock	20m

Table 5: Close-up of the model construction area for the different scenarios.

Initial state 	Scenario 3 – Excavation, piles, 10m cut-off wall 
Scenario 1 – Excavation, no piles 	Scenario 4 – Excavation, piles, 15m cut-off wall 
Scenario 2 – Excavation, piles, no cut-off wall 	Scenario 5 – Excavation, piles, 20m cut-off wall 

4.8 Stages

As expressed in Figure 1, the excavation and the related ground works is in reality carried out in stages. However, for simplicity, the entire excavation and construction process was modelled as one stage. The scenarios with pilings will not have the scenario without pilings as parent stage. It will simply continue from the initial stage before the excavation is implemented. This is of course a major simplification, but the resulting final pore pressure reduction will be the same. Hence, the simulation of all scenarios will include two stages:

1. Steady-state (no change in groundwater level with time) which represents the initial in-situ conditions before excavation
2. Transient state (change in groundwater level with time) after the pit has been excavated. Begins directly from the steady-state conditions.

The excavation will be open in a two-year period. To examine the effect of leaving the excavation open longer than planned and ensure that the groundwater flow reaches equilibrium, all transient stages lasted for 10 years.

5 Results

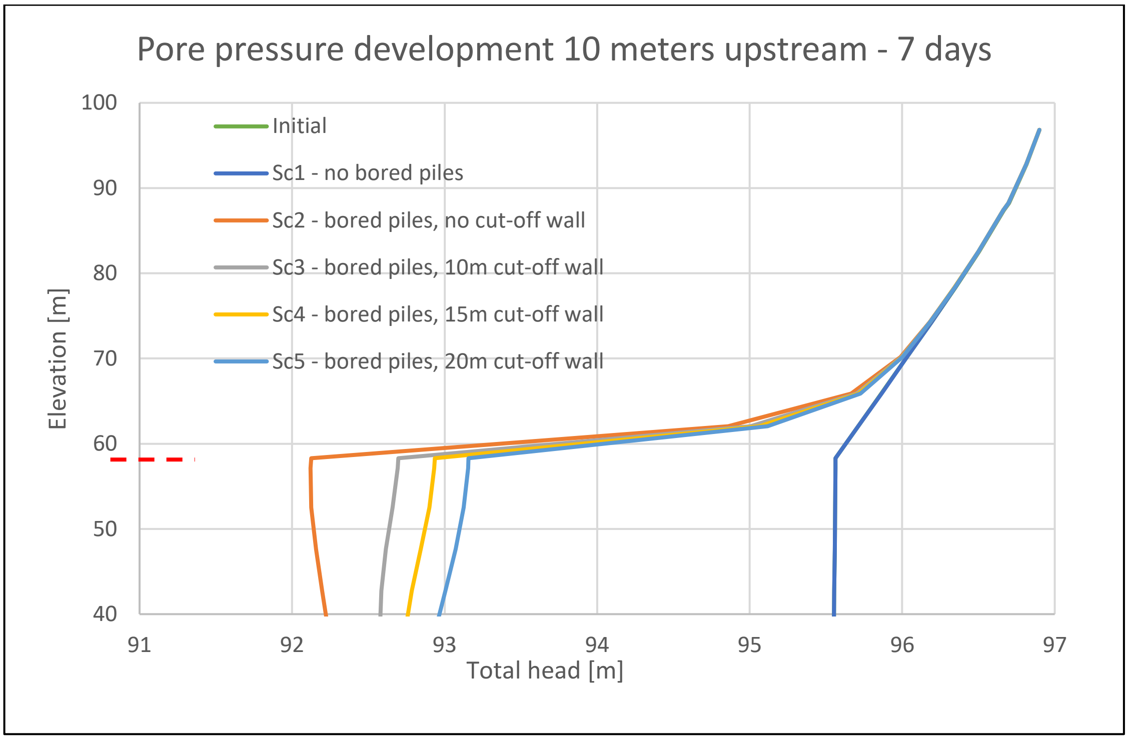
SEEP/W allows reading of a large number of different parameters at user-defined time steps. The single most important parameter for this case study is the pore pressure reduction. In this report it is for the most part given in terms of variation of the total head. It is the reduction of pore pressure, not the volumetric flow into the excavation, that is needed to estimate settlements. The following chapter presents main results related to the effect of bored piles and cut-off wall, how these effects change with time after start of drainage, and the sensitivity of the results with respect to the conductivity.

5.1 Effect of bored piles and cut-off wall

The total head (relative to sea level) was registered along ten different depth profiles from the terrain and some 10-20 meters down into the bedrock. Five profiles downstream of the excavation, and five upstream. As predicted, the pore pressure reduction decreased with distance from the excavation both downstream and upstream. The absolute effect of the cut-off walls, given in terms of pore pressure difference, thereby also decreased away from the excavation. Furthermore, both the negative effect of the bored piles and the positive effect of the cut-off walls were slightly lower downstream than upstream. Although there were some differences from profile to profile, the differences were only minor, and the overall effect of bored piles and the cut-off wall may be illustrated by results from one of these profiles. For this, the results from the upstream profile closest to the excavation (10 m) are chosen because the effects were largest at this location.

Figure 13 shows the total head along the profile 10 m upstream of the excavation. Elevation is plotted on the vertical axis. Comparing scenario 1 and 2, the effect of the leakage along the bored piles is quite large, being around 35 kPa after 7 days. After 1 year, the pore pressure reduction has increased to almost 40 kPa. With the excavation being around 5.5 m deep, and the hydraulic head at bedrock being equivalent to represent a groundwater level located around 1.5 m below terrain, 40 kPa represents the maximum pore pressure drawdown H_{max} that is possible for this leakage scenario.

In the beginning of the drainage period, the effect of the cut-off walls (scenario 3-5) are considerable. The difference in pore pressure at bedrock from scenario 2 (piles, no cut-off wall) is 5 kPa for 10 m cut-off wall, and 10 kPa for 20 m. However, the effect decreases with time, and after 1 year, the difference is only 3 and 6 kPa.



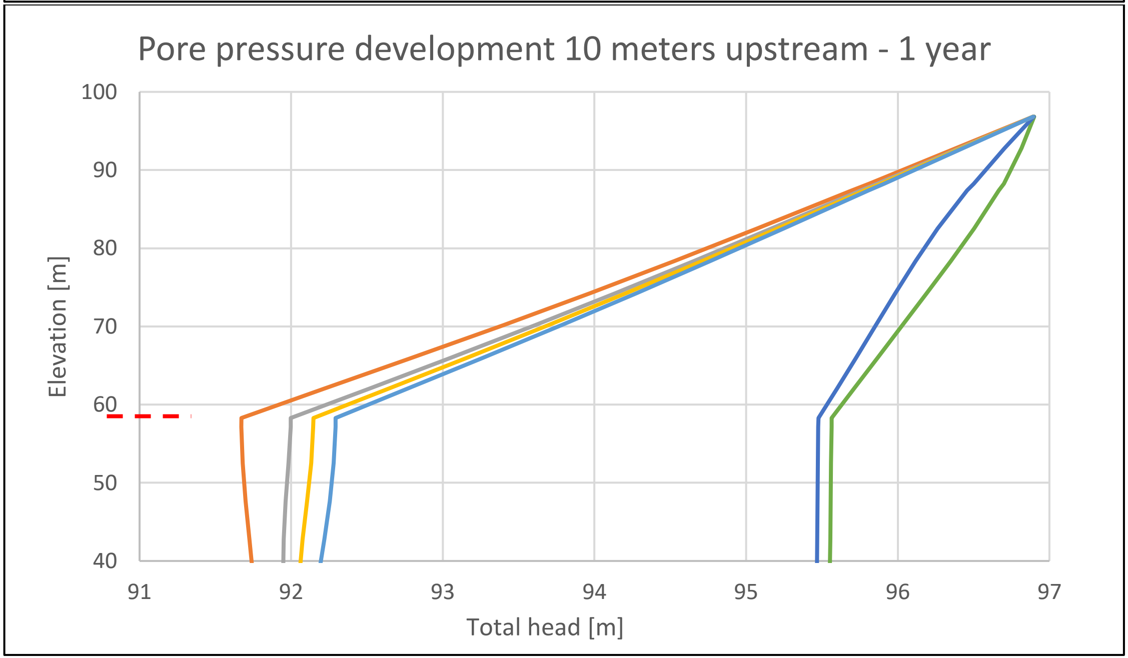


Figure 13: Total head vs. elevation 10 meters upstream (northeast) of the excavation after 7 days and 1 year. The bedrock is at approximately 57 m elevation, illustrated by the red, dotted line. Note that the under-hydrostatic conditions are caused by the groundwater level in the model being at terrain, while it actually is located approximately two meters below terrain. This discrepancy is believed to have little or no effect on the results in terms of pore pressure reduction at bedrock.

5.2 Sensitivity analysis: hydraulic conductivity K

Figure 14 presents 10 m upstream results from simulations of scenario 2 and 5 on the model where $K_{bedrock}$ was 5e-7 and 5e-6 m/s. The difference in pore pressure drawdown is small, only about 1 kPa. This is also the case for locations longer upstream and downstream.

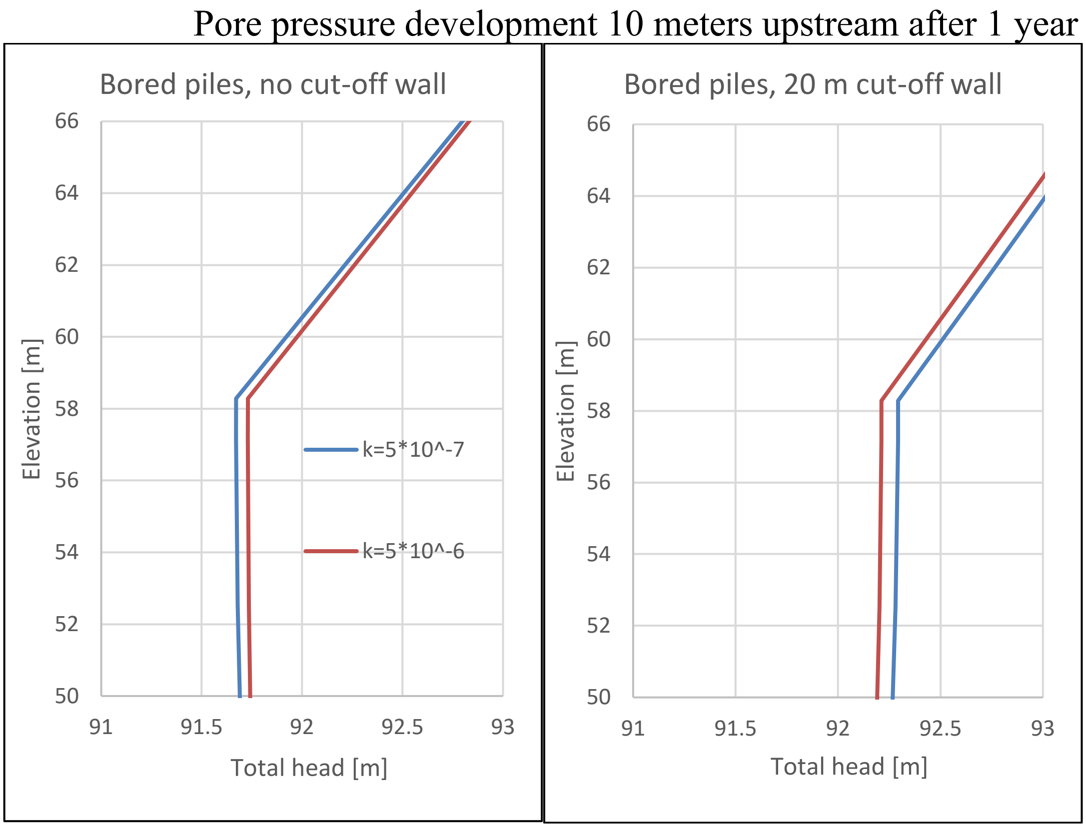


Figure 14: Pore pressure development right below and above bedrock at 10 meters upstream after 1 year for scenario 2 (left) and 5 (right).

6 Discussion

As noted in the introduction, the primary aim of the case study has been to evaluate the usability of hydrogeological modelling as a tool to assess the influence an excavation and related construction methods has on the pore pressure levels in the surrounding area. The most important question in this respect is:

Can the model accurately predict the pore pressure drawdown that a certain excavation and related construction work will cause at a location with certain ground conditions?

Furthermore, the question that ultimately has to be answered is:

Can the engineer rely on modelling results when deciding on design and mitigation measures for a planned excavation?

These two questions will be of main focus in the following discussion. First, they will be answered with respect to the specific case before some of the main modelling uncertainties will be discussed in more general terms. One cannot conclude these important questions based on only one case and only one model of that case. But, most of the weaknesses with this specific model will be challenging to overcome for similar models. Thus, there may be made general assertions regarding challenges that will be relevant for engineers working on similar construction projects.

6.1 Reasonableness of the model and results

In order to discuss the reasonableness of the modelling results, it is useful to compare them with data from similar projects. Figure 15 presents simulation results from scenario 2 and 5 for both models (upstream constant head and constant) together with empirical data presented in BegrensSkade I.

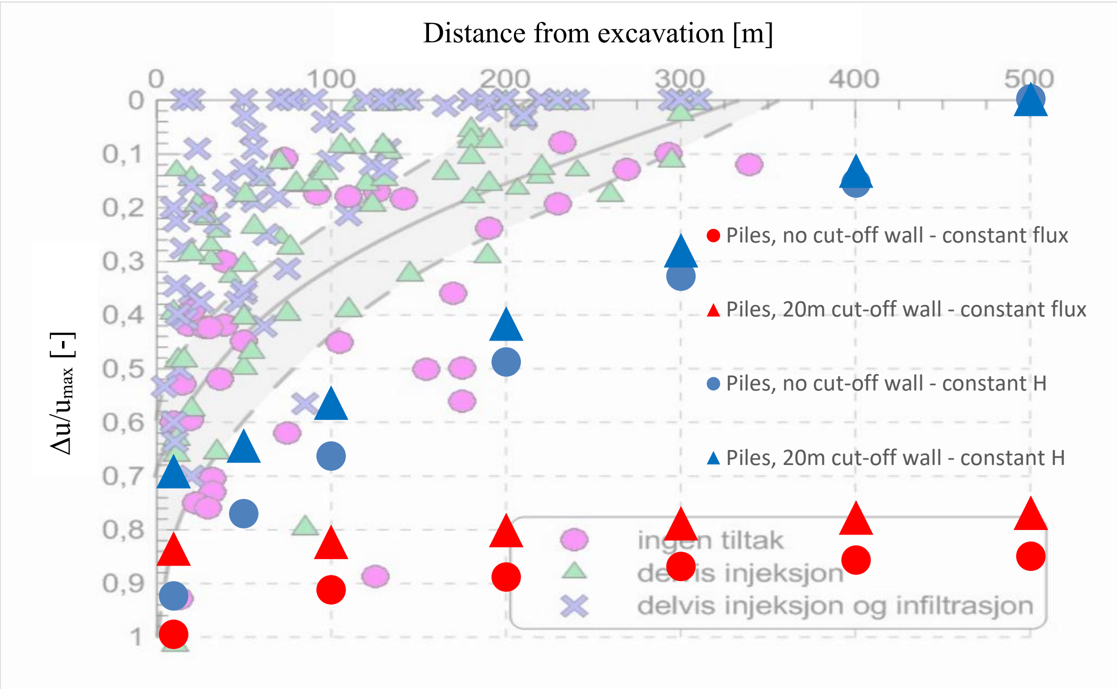


Figure 15: Simulation results of the pore pressure drawdown at bedrock at various distances from the excavation plotted along with pore pressure measurements compiled in BegrensSkade I. The pore pressures are normalized with respect to the pore pressure representative for the depth of the excavation below the groundwater level, u_{max}.

Constant flux
With constant flux at the upstream boundary, the pore pressure drawdown is almost the same at 100 m as it is at 500 m from the excavation. This seems unlikely to actually happen. It could have been the case if there was a highly conductive fault zone stretching in the same direction as the modelled cross-section, but as this has not been modelled, the results from the constant flux model may be disregarded, at least for distances above 10-20 meters from the excavation.

Constant head
Initially, the results seem much more promising for the constant head boundary. But the results are highly influenced by the geometry of the model. If the distance from the excavation to the upstream boundary had been 300 m instead of 500 m, the results would suddenly have seemed much more reliable. Such a decision could have been argued for as the length of the sedimentary basin upstream from the excavation is about 300 m. That the surrounding bedrock has not been eroded to the same elevation as the bedrock

in the basin, indicates that the surrounding bedrock is stronger and less fractured. However, it is only an indication, and it would have to be substantiated by data (e.g. from Lugeon tests) before such a modelling decision could have been made if the results were to be used in decision-making in a construction project. Nevertheless, if such tests had been performed and the boundary had been set at 300 m, the development of the pore pressure reduction between the boundary and the excavation would still had been a major uncertainty. The results in Figure 3 show almost linear development from the excavation, especially for the scenario with a cut-off wall. The reasonableness of this is difficult to evaluate from comparison with the empirical data as the results are quite case-dependent and relies on the ground conditions. This issue, which depends on how the water flows in the drainage material, is discussed in general terms in 6.3.

Effect of the cut-off walls

The effect of the cut-off walls on the pore pressure reduction is generally quite modest. With the use of constant head at the upstream boundary, the effect 10 m upstream after 1 year is around 10 kPa, but it quickly decreases with distance from the excavation. It is not possible to read out the exact effect of mitigation measures from the empirical data in Figure 3, but it is a common notion that (1) cut-off walls often do have a noteworthy effect even at large distances from the excavation, and (2) the effect of the cut-off wall may be highly dependent on its length (depth). The effect is difficult to prove, as cut-off walls are usually installed before construction that causes major leakage paths into the excavation. But, assumed that the model is under-estimating the effect of the cut-off walls, there is a couple of aspects that may contribute to such an under-estimation:

1. **Anisotropic conductivity**: The bedrock conductivity in the model is set to be isotropic ($K_z = K_x$). However, the literature indicates that, in fractured bedrock, K_z is often lower than the horizontal conductivity K_x (Welch and Allen, 2014). A cut-off wall will alter the flow-path such that the groundwater will have to flow downwards and around the wall. If $K_z < K_x$, the downwards flow would be reduced. This would in turn result in a lower pore pressure drawdown than if $K_z = K_x$ (as in the model).

2. **Change in conductivity with depth**: The shallow, weathered zone of the bedrock is usually more jointed and have a higher conductivity than the bedrock at larger depths, thus resulting in a decrease of conductivity with depth (Dagestad, Hansen and Braathen, 2003; Welch and Allen, 2014). A 20m cut-off wall may therefore in reality lead the groundwater flow into a less permeable material than the 10m cut-off wall will. Hence, the groundwater flow will be slowed down more by the 20m cut-off wall than by the 10m wall.

These deviances may offer some of the explanation as to why the effect of the cut-off wall itself, and the length of it, is predicted by the model to be lower than what has been observed in construction projects. Both alterations will lead to less conservative results in terms of the effect of the cut-off wall, and the bedrock conditions will vary significantly from site to site. Therefore, it would probably be necessary to gather case-specific data to substantiate the decision of anisotropic conductivity ratio and how the bedrock should be divided into sequences of changing conductivity. For this case study, such data was not present.

Recharge of groundwater

As part of this type of hydrogeological modelling work, it is common to compare the modelled inflow with the groundwater recharge to ensure that the inflow is realistic. This has also been done for this case study, but it is not that straight-forward because of the uncertainty regarding how thick the draining layer is.

The catchment area upstream of the right boundary of the model, i.e. 500 m away from the excavation, is about 2 km^2. The following calculations assumes that precipitation from the entire catchment contributes equally to the groundwater recharge. The specific discharge for the catchment, is about 17.5 $l/s/km^2$. This means that about 560 mm of the precipitation is discharged as surface runoff or as groundwater infiltration. The recharge of the groundwater in the upper, jointed bedrock is however not easy to predict in terms of flux because we don't know the thickness of the water-bearing zone which feeds the excavation. The flux at the right boundary is about 3.5e-9 m/s for the $K_{bedrock}$ = 5e-7 m/s constant flux model. Given that the actual draining layer have a thickness of 2m and a width of 200m (across the basin perpendicular to the cross-section and flow), the equivalent recharge from precipitation is 22 mm/year. This equals almost 4% of the specific discharge. With a thickness of 5m, it equals 47 mm (8.4% of the specific discharge). A thickness of 10m gives 110 mm (20 %). Based on recharge data and calculations from other sites (Stav, 2020), the thickness range between 5m and 10m draining layer seems reasonable. However, there are large uncertainties especially regarding the actual area which contributes to the groundwater infiltration.

Key results that correspond with empirical data

Although the results overall seem unreliable, there are a couple of aspects of the results that is in line with the empirical data and/or seem reasonable.

First the drainage to the excavation in scenario 1, with no use of piles, is negligible. Although the pit is excavated to almost four meters below the hydraulic potential at bedrock, there is almost no groundwater draining upwards through the clay because of the low permeability. This complies with experience from similar projects where the layer of clay has not been "punctured".

Second, when the layer of clay is punctured by the piles, the groundwater begins to drain seemingly instantly, and the majority of the pore pressure reduction at bedrock happens within the first week. A new equilibrium of flow is also achieved quite rapid at all distances from the excavation, e.g., for all scenarios, the pore water pressure is almost the same after 60 days of drainage as it is after 10 years.

Third, the pore pressure is slightly less affected of the drainage downstream of the excavation than upstream. This seems reasonable as the gradient towards the excavation will be lower than it will for the upstream region. Fourth, and related to the third aspect, the effect of the cut-off walls is lower downstream than upstream. This aspect has been noted by several construction projects, and costs have been saved by only establishing cut-off walls upstream.

6.2 Other weaknesses regarding the case study

In addition to those uncertainties already mentioned regarding the boundary conditions and the modelling of bedrock conductivity, there are some more weaknesses regarding the modelling of the pile leakage and the flow direction.

Conductivity of the pile leakage

Before the pile leakage conductivity of 1e-4 m/s was decided both models (constant flux and constant H) were run with various conductivities to study its influence on the results. Varying the conductivity did not alter the hydraulic potential at bedrock between the excavation and upstream boundary noteworthy. With k=1e-5 m/s, the hydraulic potential upstream was increased with about 0.1 m (or 1 kPa) for all scenarios. This indicates that the conductivity of the leakage paths does not matter much as long as it is considerably higher than the conductivity of the clay. However, to be sure, it should have been studied in more detail and especially testing with conductivities about the same as the bedrock.

Flow direction

The actual groundwater flow in the area cannot be accurately modelled in 2D. The equipotential map, which is also associated with uncertainties itself, indicates an overall flow direction approximately along the modelled cross-section. However, close to the excavation, the groundwater is indicated to flow perpendicular to the cross-section. The initial flux in this area would therefore be lower than what is modelled (for the northeast-southwest cross-section). Furthermore, there may be fracture zones in the area which crosses the modelled section at an angle. How this would affect the results is not clear. Nonetheless, the main objective of this case study was not to achieve results that could be used directly in the Campus Ullevål construction project, but to study the modelling of such cases in general. In other words, it is not actually a weakness of the case study, but it would possibly have been a weakness if the results were to be used for dimension and design of solutions for the construction project.

6.3 General remarks about boundary conditions and the regional conductivity in fractured bedrock

This section discuss 2D boundary conditions and modelling of groundwater flow in fractured bedrock in general terms. The case study is used as basis for the discussion, but the regarded challenges will also be necessary to address for engineers working on projects with similar construction problems and ground conditions.

Boundary conditions

The implications that follow the two different hydraulic boundary options are summarized in Table 6. The reasoning is based on Darcy's law ($Q = k*i*A$) applied on flow from N to M in Figure 16.

Table 6: The implications regarding the two different types of boundary conditions. Based on flow from N to M in Figure 4.

Implications on	**Constant Head**	**Constant Flux**
Flow	When the leakage begins, the pore pressure at M will decrease, whereas the pore pressure at N is kept constant. Thereby, the hydraulic gradient will increase, which in turn will cause Q_{right} to increase.	When the leakage begins, the pore pressure at M will decrease. The gradient between N and M then "desires to" increase. But, because Q_{right} is kept constant, so is the gradient, and the hydraulic potential at N must decrease.
Influence area	When one decides the length of the model upstream, one also decides important aspects of the influence area. E.g., if the total head for the transient stage is set to be the same as in the initial steady-state condition (before excavation), then the maximum radius of influence will be lower than the upstream boundary. Hence, the radius of influence is "pre-determined" by the total head value at the boundary and the length of the model. This is exemplified by the blue points from the constant H 500m model in Figure 15.	The pore pressure drawdown at N will almost equal the one at M. If the distance between M and N are large, this will most likely be unrealistic. The reason is that the communication between the permeable fractures close to N and those close to M may be over-estimated when the entire bedrock is assigned the same conductivity. This is exemplified by the red points in Figure 15.

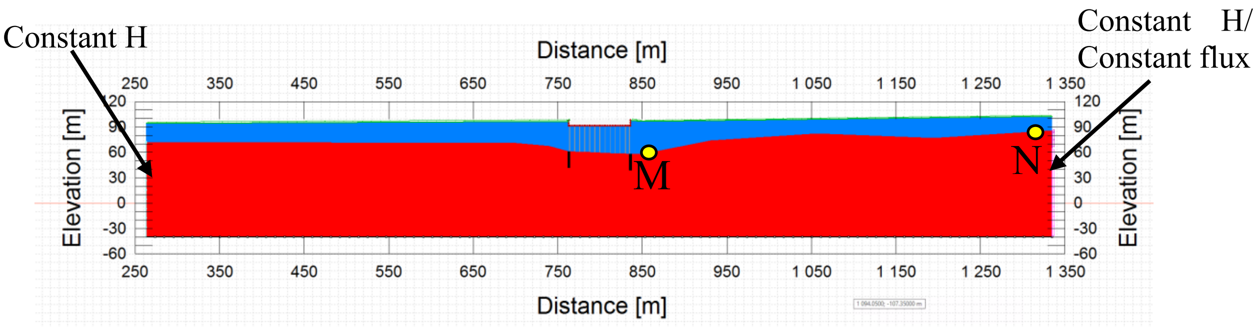


Figure 16: The numerical model. The initial groundwater flow is from right to left. M and N denotes two locations at the bedrock surface.

Regional conductivity in fractured bedrock

With "regional conductivity" it is meant the conductivity between two locations that are farther apart than a hundred metres, for example the conductivity between M and N in

Figure 16. The hypothesis is that the conductivity between M and N is noteworthy lower than the conductivity measured by Lugeon tests in boreholes at the two locations. Thi may be the cause if only some of the water-bearing joints surrounding N are communicating with the joints at M. Figure 17 illustrates this notion.

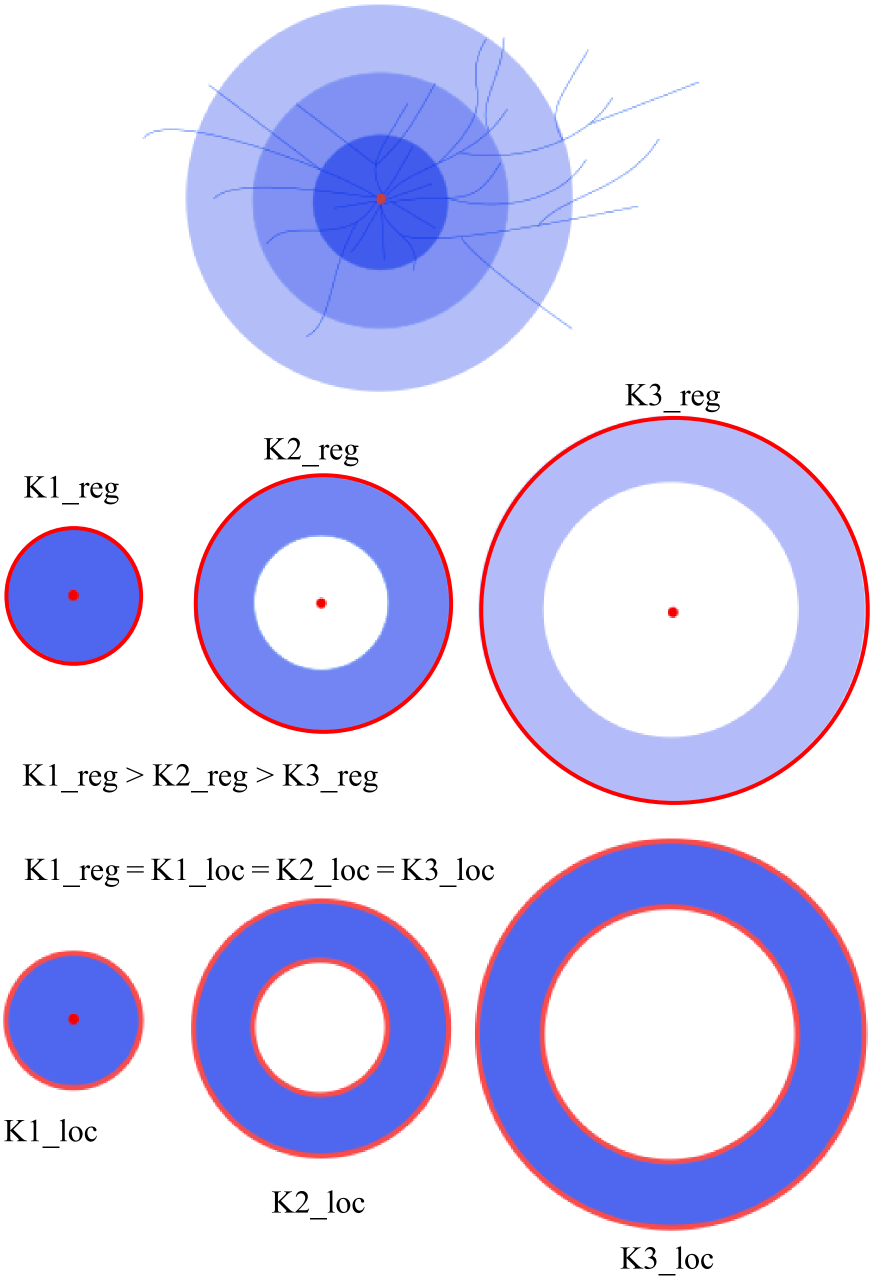


Figure 17: Illustration of the difference between regional and local conductivity (K_reg and K_loc). The upper figure shows a 2D horizontal plane with the imagined extents of joints that are communicating with the midpoint location. The two lower figures illustrate regional and local conductivity when considering the flow from the outer to the inner red line (or point).

In the Campus Ullevål model, the groundwater 500 m from the excavation flows as easily towards the excavation as the groundwater 10 m from the excavation. The validity of such an assumption may vary greatly from site to site depending on the interconnection of the joints. But in cases where the hypothesis is true, the assumption made in this case study may be a major, conservative simplification. Because the overall bedrock flow would have been lower, it seems likely that the constant flux model would have yielded more realistic results if the bedrock had been modelled as vertically divided sections with decreasing conductivities towards the excavation. Deciding on the sections and conductivities seems like a difficult task. If the modelling results were to be used in the decision-making in a construction project, the decisions would have to be substantiated by data. Because of this, it is not apparent to the author whether such an approach would have been practically useful or not. The costs of ground investigations would have been high, and one would still have uncertainties regarding the bedrock flow.

7 Conclusions

In conclusion, neither of the two models (constant H and constant flux) were trustworthy. The results did not coincide with empirical data. Even if the upstream boundary had been altered such that the results had been within the range of empirical data, the development of pore pressure drawdown with distance from the excavation would likely be linear. A linear development may be the case for some sites, but not for others. The bedrock model could have been made more realistic by reducing the K_z/K_x ratio and/or differentiating the bedrock into sections both horizontally and vertically with different conductivities. However, deciding how to choose the ratio and sections would have to be substantiated by case-specific data. Lugeon tests at different depths in the bedrock, as well as pumping tests with monitoring wells at different distances along the modelled cross-section, could have produced the necessary data. However, such tests are costly and must be planned in detail in order to yield useful results.

The author concludes that for modelling results to be used in the decision-making regarding mitigation measures that depends on the groundwater flow in the bedrock in some way, there would have to be gathered more information about the nature of the bedrock groundwater flow then it has for this case study. Further, it is suggested that when using a continuum model to represent the groundwater flow in bedrock, it should be assessed whether the various features of the bedrock conductivity is modelled realistically enough without differentiating the conductivity horizontally and/or vertically. In the case study, empirical data such as those presented in Figure 3, would have been trusted more than the modelling results.

The Campus Ullevål model had several simplifications regarding the bedrock groundwater flow. There are made suggestions on how to avoid these simplifications. However, more ground investigation data would have been needed in order to make the necessary modelling decisions, which overall would have made the results less conservative but probably served as a better foundation for the decision on cut-off wall and the length of it. There is made no conclusion as to whether the modelling approaches should be used or not. This depends on the complexity of the specific project and the available budget. The aim with these discussions were on the other hand to highlight the simplifications that are made in such models and to emphasize their possible effect on the results.

It is proposed that for all construction projects with a planned excavation that may drain groundwater, the assumed reliability of the results from a potential model should be compared to using empirical data before deciding on whether to perform modelling or not. The assessment of the model reliability should be based on which simplifications that will be made in the model.

8 Further work

As mentioned, the modelling performed in this case study could have been adjusted so that the results would have aligned better with empirical data. However, it is considered to be a strong advantage if the conductivity of the bedrock could have been modelled more realistically. The need to represent the conductivity of the draining layer as realistically as possible is emphasized. First, the report proposes to perform a literature study on the modelling of groundwater flow in the weathered bedrock zone. After that has been performed, if the uncertainties raised in the discussion are still present, the report proposes two modelling tasks for cases where the fractured bedrock is the primary draining layer or cases where cut-off walls will guide the groundwater from the moraine down into the fractured bedrock. The aim of both is to investigate the effect of two (or three) different modelling options that take the difference between soil (pore) and bedrock (joint) conductivity into account:

1. Make a 2D model where the fractured bedrock is modelled as distinct materials horizontally, with different conductivities. The conductivities should decrease away from the excavation in order to take into account discontinuous fractures which contributes to the measured conductivity in a nearby borehole, but not necessarily to the "experienced" conductivity for a leakage point a certain distance away. This is expected to result in a more realistic (non-linear) development of the pore pressure reduction with distance from the excavation. Note that this modelling task may also be appropriate for cases where the drainage layer is a discontinuous moraine deposit as the conductivity in such a deposit may be similar to fractured bedrock conductivity. The problem should be modelled both with constant head and constant flux as the upstream hydraulic boundary.

2. Make a 2D model where the fractured bedrock is modelled as distinct materials horizontally, with different conductivities. The conductivities should decrease downwards to take into account the decrease of fractures and their aperture with depth in the bedrock (Welch and Allen, 2014). The same model should also be simulated with a scenario where the vertical conductivity is lower than the horizontal. Both adjustments are expected to result in a more realistic (and greater) effect of the cut-off wall.

It must be emphasized that in order to use the results from the models proposed above for decision-making in a construction project, the options to differentiate the conductivity a certain way would have to be based on data, preferentially site-specific.

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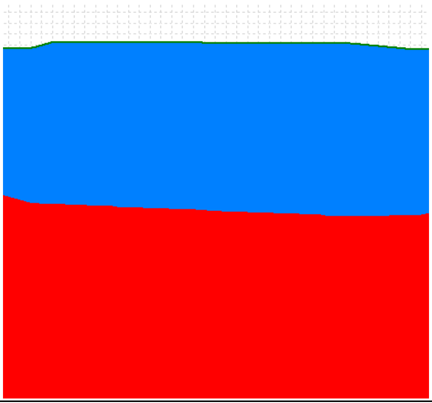
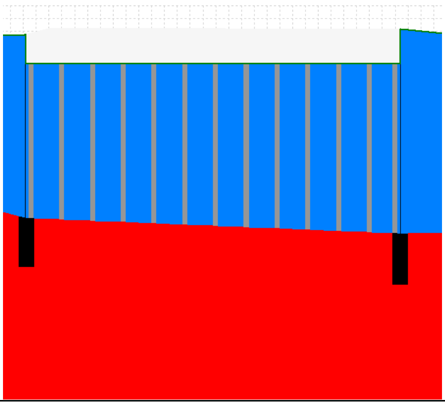
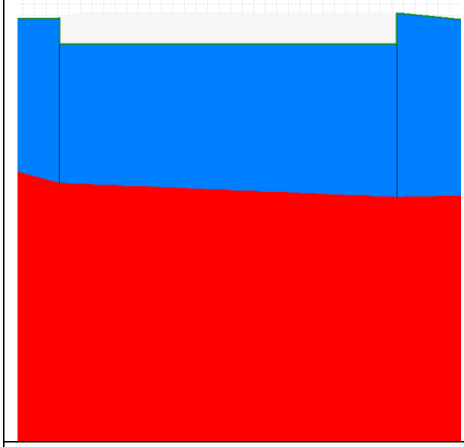
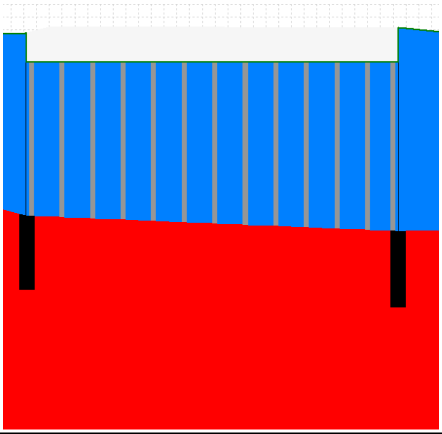
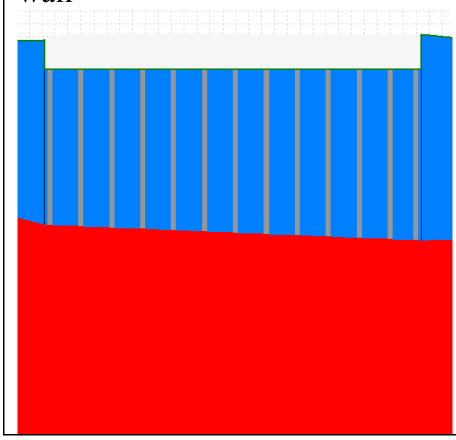
Appendix A

Close-up of the construction area of the model for the different scenarios.

Initial state 	Scenario 3 – Excavation, piles, 10m cut-off wall 
Scenario 1 – Excavation, no piles 	Scenario 4 – Excavation, piles, 15m cut-off wall 
Scenario 2 – Excavation, piles, no cut-off wall 	Scenario 5 – Excavation, piles, 20m cut-off wall