BegrensSkade/REMEDY

Risk Reduction of Groundwork Damage

Deliverable D3.1

Drainage to Excavations – State of the Art Report

Work Package 3 – Hydrogeological methods, drainage and grouting

Deliverable Work Package Leader: NGI

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REMEDY
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Summary

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

The risk of settlement damage caused by drainage and pore pressure reduction can be reduced during the early design phase of a project by undertaking the correct type of investigations and understanding the hydrogeological conditions. Furthermore, one may select construction methods, which reduce risk of drainage. In case the selected construction solution yields an unacceptable risk for settlement damage to surrounding buildings and infrastructure, remedial measures may be designed to mitigate the effects, followed by implementation and monitoring during the construction phase.

This report provides State-of-the-Art related to hydrogeological investigation methods, hydrogeological modelling and numerous measures to mitigate the effects of drainage to excavations. In addition, governing Norwegian rules and regulations are discussed, as well as the causes of drainage to excavations in Norwegian ground conditions.

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1 Introduction

1.1 Background

Ground works such as deep excavations and foundation works performed in soft clay can cause damage to neighbouring buildings and structures. Drainage causes pore pressure lowering, 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 (Karlsrud et al, 2015), 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.

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. Furthermore, it is crucial to follow up on the execution of the ground works and monitoring during the construction phase.

1.2 Contents

This State-of-the-Art report summarizes the current Norwegian best-practise on evaluating and mitigating effects drainage to excavations. Chapter 2 provides a brief background on the cause and effects of drainage to excavations and in Chapter 3, the current rules and regulations in Norway are detailed. In Chapter 4 a brief outline is given into different ground investigation methods usually applied in Norwegian problems to determine hydrogeological information. As numerical modelling of hydrogeological problems become more common, Chapter 5 details the philosophy and concepts of numerical modelling in hydrogeology. Finally, Chapter 6 provides examples of mitigating measures that may be applied for deep excavations to control and reduce drainage and ground water drawdown.

2 Effects of drainage to tunnels and excavations

2.1 Drainage to bedrock tunnels

Experience from tunnels (Karlsrud et. al, 2003) and ground water pumping, shows that even small amounts of leakage into a tunnel, can result in substantial decrease in pore pressures at bedrock level (at bottom of clay layer) (Figure 1). The main reason is the very limited recharge that comes through a low-permeable soft clay deposit, which results in a confined aquifer.

Analysis of data from numerous tunnel projects in Norway (Karlsrud, et al. 2003) show that the leakage rate into a tunnel needs to be limited to approximately 3-8 l/m/100 m tunnel to limit pore pressure decrease to 10-30 kPa at the bedrock level (Figure 2) above the tunnel. The data also shows that the pore pressure decrease can extend as far out as 200-400 m from the tunnel (Figure 3). The relatively large scatter in the results is related to difference in hydrogeological conditions, magnitude and duration of the leakage for the different tunnels.

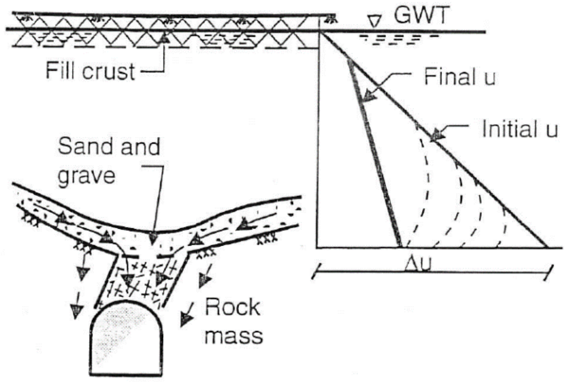

Figure 1 Leakage to bedrock tunnels overlain by clay deposits (from Karlsrud et al., 2003).

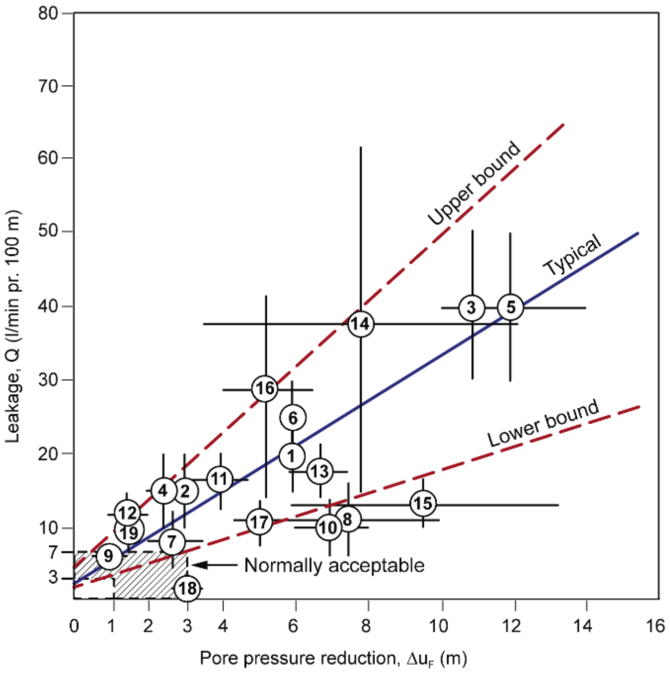

Figure 2 Measured rate of leakage to tunnels plotted against monitored pore pressure reduction at bedrock right over the tunnel (from Karlsrud et al., 2003).

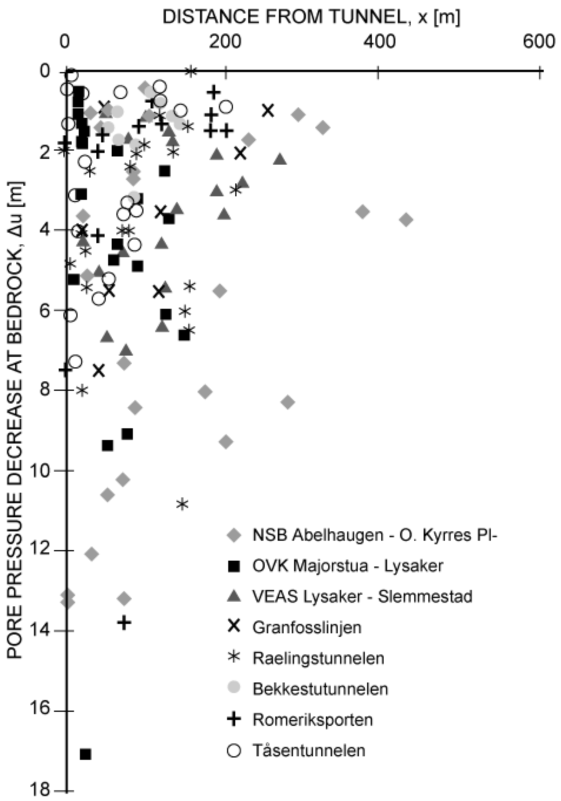

Figure 3 Measured pore pressure decrease at bedrock plotted with distance from excavation (from Karlsrud et al., 2003).

Based on the data, there are well-established recommendations on procedures for determining the allowable inflow rates to tunnels (Karlsrud et al, 2013; Karlsrud et al, 2003). For deep excavations however, the complexity is increased, and no best practises are currently adopted to address this issue.

Negative effects that need to be considered consequently from leakage to excavations and pore pressure lowering are:

- Consolidation settlements in clay due to pore pressure decrease
    - The sensitivity to settlements depends on the foundation type of buildings and structures, the distance to the excavation, the type of building, the depth to bedrock and the geotechnical properties of the clay, especially the pre-consolidation pressure. In addition, previous settlements may result in a larger sensitivity to damage.
    - Settlements will also cause negative skin friction on piled foundations.
- Decay of organic material due to groundwater lowering, for example cultural layers, wooden rafts and wooden piles.

The vulnerability of an area needs to be evaluated with respect to the potential for settlements and the consequence for buildings and structures. In addition, consequences for underground infrastructure needs to be considered

2.2 Drainage to excavations

The drainage situation for excavations is essentially the same as for tunnels (Figure 4), which implies that for an excavation of dimensions 100 m × 100 m significant pore pressure reduction can occur at the base of the clay outside the excavation, if the leakage from the confined aquifer or bedrock exceeds about 5-10 l/min in total. However, the precise magnitude of pore pressure reduction, and the lateral extent of the area subjected to a reduced pressure is difficult to predict and dependant on the local hydrogeological conditions.

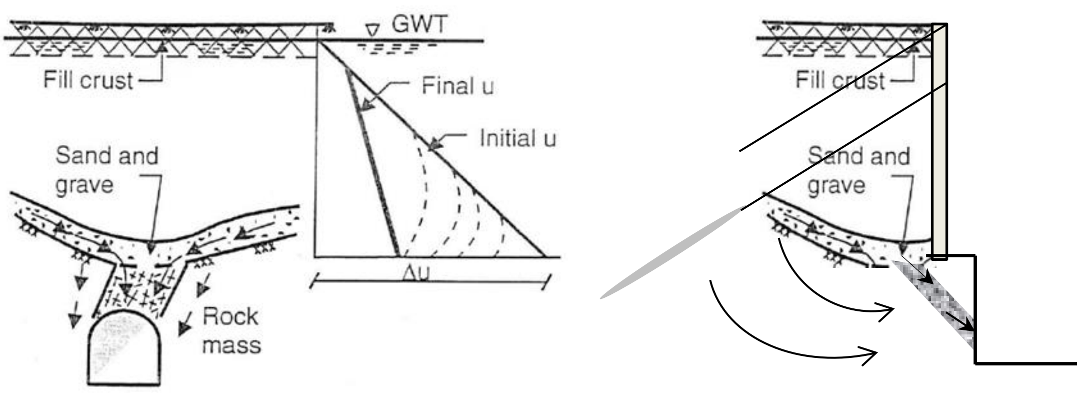

Figure 4. Comparison of drainage situation to tunnels and excavations.

The causes of drainage to an excavation can be many and complex. However, the main leakage scenarios for an excavation in soft clay, as illustrated in Figure 5 are:

- Leakage through the sheet pile wall
- Leakage through gaps between the toe of the sheet pile wall and the bedrock surface
- Leakage through cracked bedrock
- Leakage during drilling for tieback anchors or piles (through the casing or the gap between soil and casing)

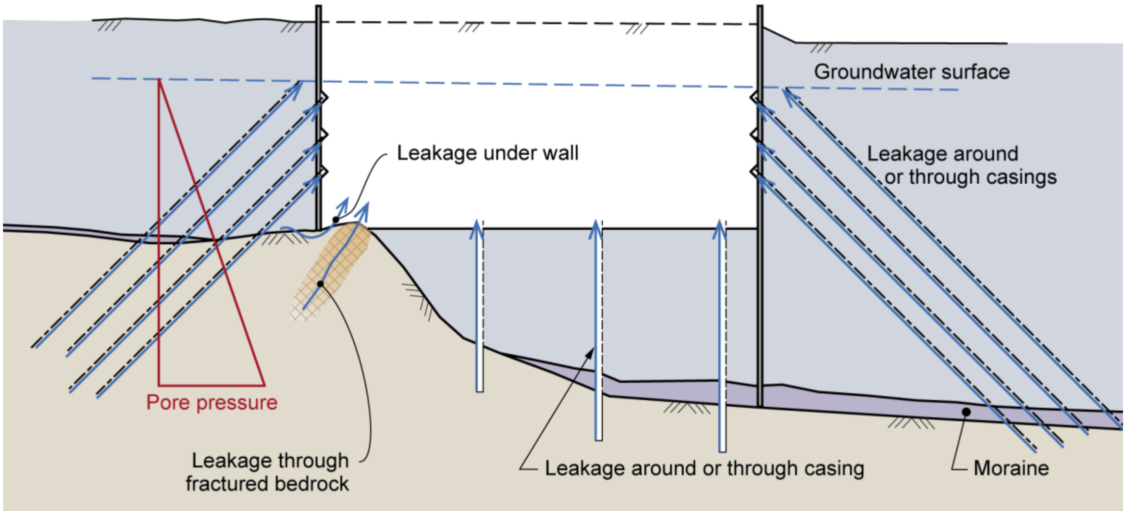

Figure 5: Illustration of the main causes of drainage of groundwater to deep excavations in soft clay.

Leakage through the sheet pile wall mainly occurs during the cutting of holes for drilling of tie-back anchors (Figure 6). In addition, leakage can occur through unsealed or poorly sealed locks between the individual pile sections. If the soil behind the sheet pile wall is clay, leakage will be limited to permeable layers.

If the depth of the excavation reaches the bedrock level, resulting in an uncovered bedrock surface, there is a large potential for leakage through fractures. Uncovering the toe of the sheet pile wall also cause a large potential for leakage, especially if the bedrock surface is steep and there is a permeable soil layer on top of the bedrock.

Drilling for installation of piles and tie-back anchors has a potential for leakage when performed from a level below the ground water level or under artesian conditions. The leakage can occur through the gap between the installed steel casing or through the casing itself (Figure 6 and Figure 7).

Figure 6 Example of leakage of water around and through casings for tie-back anchors.

Figure 7 Example of leakage of water around casings for steel core pile at bottom of excavation (photo: Jernbaneverket).

As a part of the Begrens Skade-project (Karlsrud et al, 2015) several case studies have been analysed to investigate the most common causes for extensive settlements. All projects are deep sheet pile wall supported excavations, carried out in normally consolidated soft clays. One of the main causes of settlement is drainage to excavations from the confined aquifer and pore pressure lowering.

To better understand the effects of drainage to excavations in soft clays, the Begrens Skade-project has collected and interpreted pore pressure data from 17 case histories, together with previously published data from Braaten et al (2004), Johansen (1990) and Karlsrud (1990). The results are shown in Figure 8. In the plot, the measured pore pressure reduction at bedrock level, Δu, is normalized with respect to the depth of the excavation below the original ground water surface, H_{max}. The data are plotted against the horizontal distance of the piezometer from the excavation. This database is currently being updated as part of the REMEDY project.

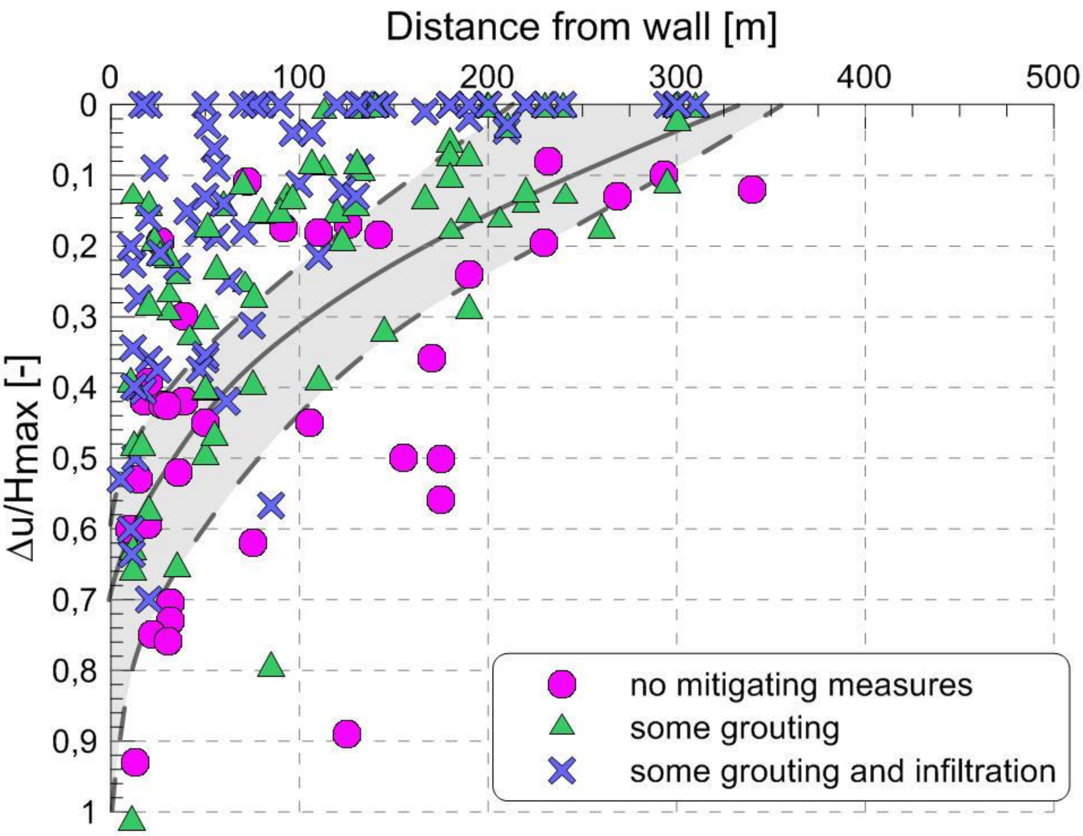

Figure 8. Observed normalized decrease in pore pressure at base of clay layer as function of distance from the excavation based on case records from Norway

The data shows a relatively large scatter, which is related to varying hydrogeological conditions, amount and duration of the leakage, use of different construction methods and varying mitigating measures. However, some general conclusions can be drawn by organizing the data accoring to the mitigating measures that were undertaken.

Pink symbols show cases where no grouting or recharging (infiltration) of water was undertaken, green symbols show cases where some grouting at the toe of the wall and into bedrock was undertaken, and blue symbols represent cases where some grouting as well as infiltration was undertaken.

The figure suggests that even when performing systematic grouting and infiltration, the maximum pore pressure reduction close to the excavation could correspond of 20-50% of the depth of the excavation below the groundwater level. Furthermore, a reduction may extend as far as 300-400 m laterally from the excavation. It can be concluded that it is challenging to maintain the pore pressure levels, even when mitigating measures are undertaken.

Dashed lines in the figure indicate which range pore pressure drawdown can be expected. It is important to note that the lines are rough estimates. The lower bound could be applied for cases where both infiltration and grouting is performed, and the higher bound can be taken as a worst-case scenario where no mitigating measures are undertaken. However, some projects experience an even larger pressure drawdown, caused by unfavourable conditions at the specific sites.

Analysis of settlement data (Karlsrud et al, 2015) indicate that drilling for piles are associated with a greater risk of leakage than drilling for tie-back anchors. The reason is likely that piles are generally drilled from a deeper level that the anchors. In addition, systematic leakage-testing and grouting for anchors are generally performed to ensure enough tension capacity of the grout body. This is generally not undertaken for pile, unless designed as tension pile.

3 Rules and regulations

The requirements on temporary and permanent groundwater and pore pressure levels for underground construction projects are set based on acceptable criteria on the surrounding environment and neighbouring areas. This chapter summarizes the requirements in current Norwegian regulations and standards.

3.1 Norwegian regulations

Requirements on drainage to excavations with respect to surrounding areas is given in the Norwegian regulations by the *Vannressursloven* (Lov om vassdrag og grunnvann, LOV-2000-11-24-82) and *Grannelova*, (Lov om rettshøve mellom grannar, LOV-1961-06-16-15).

Lov om rettshøve mellom granner (Grannelova) - Aktsomhetsplikt
§2 *"Ingen må ha, gjera eller setja i verk noko som urimeleg eller uturvande er til skade eller ulempe på granneeigedom. Inn under ulempe går òg at noko må reknast for farleg."*

§5 *"Ingen må setja i verk graving, bygging, sprenging eller liknande, utan å syta for turvande føregjerder mot utrasing, siging, risting, steinsprut, lufttrykk og anna slikt på granneeigedom. "*

The paragraph §2 states that no one may execute work, which can cause harm or inconvenience to neighbouring properties. Hazardous acts are considered an inconvenience.

The paragraph §5 states that no one may start excavation, construction, blasting or other activities, without executing mitigating measures for stability, vibration, rockfall or other consequences for neighbouring properties.

The "Vannressursloven" has §45 *konsesjonsplikt* (duty of commission), §5 and §43a *aktsomhetsplikt* (duty of caution), §6 *forholdet til naboer* (relation to neighbours).

Lov om vassdrag og grunnvann (Vannressursloven)– Aktsomhetsplikt
§5 *"Enhver skal opptre aktsomt for å unngå skade eller ulempe i vassdraget for allmenne eller private interesser.*
Vassdragstiltak skal planlegges og gjennomføres slik at de er til minst mulig skade og ulempe for allmenne og private interesser. Denne plikten gjelder så langt den kan oppfylles uten uforholdsmessig utgift eller ulempe. Vassdragsmyndigheten kan ved forskrift fastsette nærmere regler om planlegging, gjennomføring og drift av bestemte typer vassdragstiltak.
Vassdragstiltak skal fylle alle krav som med rimelighet kan stilles til sikring mot fare for mennesker, miljø eller eiendom".

The paragraph above states the requirement of caution, which is new since 1. January 2018. Any activities, which affect watercourses or groundwater, need to be planned and executed in a way, which minimizes the damage or disadvantage to public or private interests.

This obligation applies as far as it can be met without disproportionate expense or inconvenience. The Water Authority may authorize regulations on the planning, implementation and operation of certain types of water resource measures. Water resource measures shall fulfil all requirements that may reasonably be provided for protection against danger to humans, the environment or property.

NVE (the Norwegian Water Resource and Energy Directorate) is the supervisory authority for the management of Norway's water and energy resources. The regulations of the Vannressursloven is further clarified NVEs guideline 1-2017 (NVE, 2017). The document clearly states that activities such as excavation, tunnelling, energy wells and groundworks need to be evaluated with respect to the requirements of caution, as they can affect the groundwater conditions (groundwater levels, groundwater flow and groundwater quality). This means that if the requirement of caution is not met, it is necessary to prepare an application to the NVE to get a concession for the planned project.

In the Regulations for Planning- and building (Plan- og bygningsloven) Chapter 4 on requirements for construction plans, a couple of paragraphs apply for effects of drainage to excavations.

- §4-2 Regional and count plans should include a consequence assessment with respect effects on environment and society.
- §4-3 requires a vulnerability analysis of the plans for construction with respect to neighbouring areas, which would include an assessment of the effects of drainage on pore pressures and settlements.

Plan- og bygningsloven
§ 4-2 **Planbeskrivelse og konsekvensutredning**
"For regionale planer og kommuneplaner med retningslinjer eller rammer for framtidig utbygging og for reguleringsplaner som kan få vesentlige virkninger for miljø og samfunn, skal planbeskrivelsen gi en særskilt vurdering og beskrivelse – konsekvensutredning – av planens virkninger for miljø og samfunn."
§ 4-3 **Risiko- og sårbarhetsanalyse**
"Ved utarbeidelse av planer for utbygging skal planmyndigheten påse at risiko- og sårbarhetsanalyse gjennomføres for planområdet, eller selv foreta slik analyse. Analysen skal vise alle risiko- og sårbarhetsforhold som har betydning for om arealet er egnet til utbyggingsformål, og eventuelle endringer i slike forhold som følge av planlagt utbygging. Område med fare, risiko eller sårbarhet avmerkes i planen som hensynssone, jf. §§ 11-8 og 12-6. Planmyndigheten skal i arealplaner vedta slike bestemmelser om utbyggingen i sonen, herunder forbud, som er nødvendig for å avverge skade og tap".

3.2 Design standards – Eurocode

The current revision of the Eurocode 7 (CEN, 2004) have limited requirements regarding groundwater applicable to the design of geotechnical works:

- The ground water flow and pore pressure distribution should be determined before construction starts and that observations are sometimes necessary at far distances from the construction site (4.3.2(5)).
- The effect of the construction works, including groundwater lowering, should be observed. The monitoring program should cover structures that might cause changes in groundwater flow and levels due to drainage, especially in fine grained soils. Possible structures that might cause drainage are listed, such as tunnels, larger underground facilities, deep basements, supported excavations and cuts.

There are currently no requirements or guidelines on the extent of the monitoring program or what depths the piezometers should be installed.

4 Ground investigations

4.1 Introduction

Before starting groundwork such as deep excavations and foundations, it is necessary to perform thorough ground investigations. The purpose of the ground investigation campaign is to establish a good-enough understanding of the geotechnical, geological and hydrogeological conditions of the project site and its surroundings.

No established guidelines exist for defining the extent and types of ground investigations needed to evaluate the risk of groundwater drainage to deep excavations. The Eurocode 7 standards states that the extent of investigations depend on the complexity of the geology and consequences in case of damage.

Important outcomes from a ground investigation campaign, to determine the potential for drainage to the excavation, include the mapping of any clay-filled depressions (*dyprenner*), the presence and hydraulic conductivity of permeable soil layers below the clay and bedrock fault zones with water-bearing fissures. Figure 9 shows seepage of ground water through open steel core pile casings.

Combining results from different investigation methods, interpretations and extrapolating known information is important to achieve an optimal understanding of the hydrogeological conditions.

Figure 9: Seepage of ground water through open steel core pile casings.

4.2 Geological investigation methods

Geological investigations, such as core drilling and borehole geophysics, are not commonly performed, or not performed with satisfactory level of detail, for deep excavation projects. When there is a risk for reduction of pore pressure at bedrock level, investigations to characterize the hydrogeological properties of the bedrock should be performed. These investigations should especially focus on uncovering the presence and orientation of any water-bearing fissured zones in connection to the excavation. Geological field mapping with emphasis on jointing and orientation of fault/weakness zones is one of the most important parts of an investigation programme (Holmøy, 2008). Also, core drilling with water pressure tests has given valuable information.

For example, the risk for reducing the pore pressure at bedrock level is increased in case of drilling for steel core piles and tie-back anchors due to seepage along or within the casing tubes. Also, when uncovering bedrock during the excavation works, groundwater can drain freely into the excavation through water-bearing fissures if no injection grouting is performed prior to excavation. Further information on geological investigation methods is provided in appendix A.

4.3 Geotechnical investigation methods

It is concluded that drainage to deep excavations can cause pore pressure reduction at large distances from the excavation, due to the typical Norwegian ground conditions with confined aquifers overlain by deposits of soft, low permeable clay. Investigations to evaluate the vulnerability to neighbouring structures, buildings and infrastructure therefor need to be undertaken for an area much larger than the excavation itself.

Performing in-situ soundings for the determination of soil stratigraphy and depths to bedrock in the zone of influence surrounding the deep excavation is important, especially in areas with at-risk buildings and infrastructure. Depth-to-bedrock and a coarse characterization of soil stratigraphy can commonly be determined by the Norwegian Total Sounding method (NGF, 2018).

Also, the investigations need to include the determination of settlement properties for clay layers, including CPTU-soundings (NGF, 2010), high quality sampling for determining index parameters (w_n, I_P, S_t) and CRS-testing (pre-consolidation pressure/OCR, modulus and hydraulic conductivity of clay).

To determine the OCR of the clay layers, it is necessary to evaluate the in-situ pore pressure distribution. The next chapter covers methods for measuring pore pressures.

4.4 Hydrogeological investigation methods

4.4.1 Pore pressure monitoring

It is crucial to establish in-situ pore pressure profiles for all clay filled depressions typically within a circumference of 200 m to 300 m around the excavation Karlsrud et al (2015). Pore-pressure levels govern the in-situ effective stresses of the soil and thus how sensitive the clay is with regards to additional pore pressure reductions before exceeding the pre-consolidation stress, which then may lead to significant settlements. The difference in pore pressure levels at various locations around the excavation will also give an indication of the in-situ groundwater flow through that area.

Pore pressure sensors should be installed in permeable layers and fault zones, to establish seasonal variations and identify any artesian pore pressures. To establish the in-situ stress profiles it is also necessary to measure the pore pressures in the upper part of the soil, and possibly also at several levels in deeper clay deposits, as illustrated in Figure 10. Total soundings, or preferably CPTU soundings, should be performed prior to detect any permeable layers and accurately decide on the depths at which to install the piezometers.

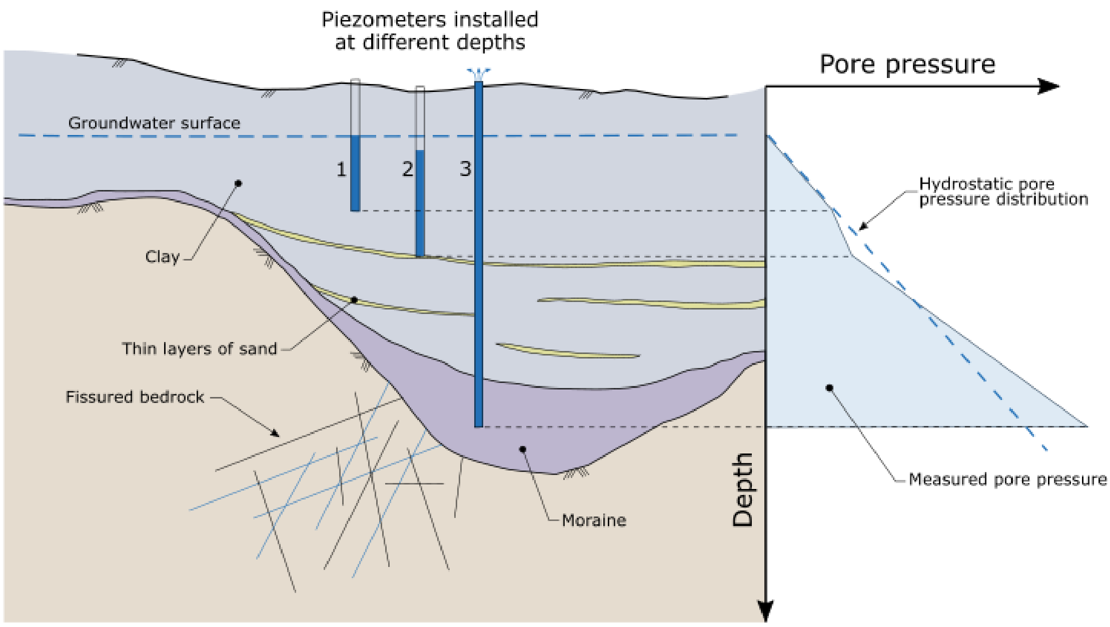

Figure 10: Measuring pore pressure at varying depths revealing (1) hydrostatic pore pressure, (2) drained pore pressure and (3) artesian pore pressure.

Most projects in Norway today use electronic transducers (such as vibrating wire sensors) to monitor pore pressures. These devices measure pressure with a high degree of accuracy, rapid response time and the possibility to record the data at a distance from the borehole. They can be installed with conventional geotechnical drilling rigs to large depths if the ground conditions are not too firm. In modern large-scale projects it is common to install real-time measurement systems allowing live updates on a web portal.

This is an excellent way to allow monitoring and follow-up of pore water pressures during a construction phase.

4.4.2 Hydraulic conductivity tests

There are many methods to determine the hydraulic conductivity in-situ with varying applicability and level of complexity, presented in Table 1. Some of these tests are commonly performed for Norwegian tunnelling projects but should be a part of any ground investigation campaign for a deep excavation in contact (directly or indirectly) with bedrock. Further details on the different methods to measure the pore water pressure and determine hydraulic conductivity is provided in appendix B.

Table 1: Methods for determining in-situ hydraulic conductivity.

Investigation type	Information type / comments
Variable head ("Slug") tests	Also known as falling or rising head test, slug test or Le Franc tests is a short time test evaluating "point" permeability in a bore hole and provides a low-cost assessment of the hydraulic conductivity.
Lugeon test	Also known as packer test. It is a method to investigate in-situ hydraulic conductivity in boreholes in bedrock. The test is performed by pumping water into the isolated borehole section under constant pressure and measure the water loss over time.
Flowmeter	High resolution impeller flow meter is used to locate permeable zones in the borehole and to map the vertical water flow in bedrock boreholes.
Pumping test	This is a standard hydrogeological test procedure to assess the aquifer type, boundary conditions and hydraulic conductivity of an aquifer. The test consists of a borehole well where water is removed by a pump, and several surrounding piezometers or observation wells.

4.5 Geophysical investigation methods

Geophysical ground investigation methods are widely used in addition, or as a substitute, to traditional drilling methods. The measurements are to detect anomalies and identifying different properties in a material as compared with its surrounding media. As an example, a water-filled zone of fractured bedrock conducts electricity better than its surrounding, sparsely fractured, bedrock. Similarly, seismic waves travel slower in a fracture zone than in solid rock. In this manner, geophysical investigations can be applied to obtain information on the hydrogeological conditions.

It is important to select a geophysical method suitable for not only the expected ground conditions but also other factors, such as the presence of buried cables and infrastructure, which may cause disturbance.

Table 2 provides an overview of commonly available seismic and electrical geophysical ground investigation methods and their areas of application. Further details on each geophysical method is described in appendix C.

Table 2: Geophysical ground investigation methods for mapping of hydrogeological situation

Investigation type	**Information type / comments**
Refraction seismic	Give seismic velocity of the uppermost 5 to 10 m of a soil deposit. Find the thickness of soil and bed rock quality. Identify weakness zones / fractured zones.
Electrical resistivity tomography (ERT). Induced Polarization (IP).	Resistivity section of the subsurface can be imaged in 2D or 3D. Best resolution is achieved for the uppermost 50 to 60 m. Low resistivity in rock mass can be a result of increased porosity due to fractures. (IP) is a method of its own and can be carried out with the same equipment as ERT. ERT may be disturbed and provide ambiguous results in case of buried cables/constructions. Zones parallel to the measurement line are difficult to detect.
Induced Polarization (IP).	IP measures electrochemical responses (polarization) of subsurface materials (primarily clays) to an injected current. IP measures are applicable for locating geological structures and disseminated deposits. The measures can be carried out with the same equipment as ERT.
Electro Magnetic survey (EM)	Electromagnetic induction (EMI) is used to locate disturbed moisture-bearing soil or weakness zones in the rock. Gives information of soil (layers with different properties), groundwater, minerals and bedrock. Method can cover a large area in very short time (easy to carry)

Ground Penetrating Radar (GPR)	Measures reflections of boundaries of variable dielectric permittivity (related to water content) and electrical conductivity. 3D geo-radar measurements can be carried out consisting of an array of antennas parallel to each other. Heavy equipment – machinery is needed.
Self-potential (SP) and Mise a la Masse (MAM)	SP measure natural occurring electrical potentials in the subsurface. The mechanisms governing the SP signal are in general of electrochemical nature. A leakage path or other hydraulic permeable weakness zones can be located. Can for example be used for detecting discontinuities in sheet pile walls.

In addition to the geophysical ground investigation methods reported above there exist different geophysical methods for boreholes (also called well logging or wireline geophysics) summarizing high resolution measurements in existing wells or boreholes. The method can easily be undertaken in boreholes from rock coring. A summary of the methods is given in Table 3. Geophysical borehole logging with hydraulic testing (well capacity and percentage distribution of water ingress in the borehole) has given reliable predictions of zones with high hydraulic conductivity (Holmøy, 2008).

Table 3: Widely used borehole probes for geotechnical applications.

Method	Principle	Condition measured	Applicable in cased boreholes
Optical Televiewer	High-res image	Continuous image. Structural information, lithology, bedding and fracturing, casing inspection, core orientation	no
Acoustic Televiewer	Ultrasonic beam	Continuous image. Fracturing and dipping of beds, lithology, thin beds	no
TCN-Probe		Temperature, electric conductivity of fluid and natural gamma of rock	
Sonic	Seismic wave	Rock strength, lithology, fracturing, shear- and bulk modulus, Poisson's ratio	yes
Resistivity (Laterolog)	Resistivity	Lithology, fracturing, water quality and aquifer thickness	no
Gamma Ray	Natural radiation	Uranium concentration (e.g. blackshales), clay thickness and clay content	yes
Flowmeter	Water flow	Vertical- and horizontal flow (heat pulse flowmeter), transmissivity, fracture flow rate	yes/no
Density logging	Gamma ray (Cs-137)	Bulk density, porosity, rock strength, aquifer thickness	no

4.6 Combining ground investigation methods

Often a combination of geophysical measurements and conventional drillings give a comprehensive overview of material properties and hydrogeological conditions. The following sections detail several project examples for combining different ground investigation methods.

4.6.1 Electrical Resistivity Tomography (ERT) combined with Total Soundings (TOT).

With regional geophysical investigations it is possible to identify inhomogeneous layers in soil or weakness zones in rock mass. The results will help optimize the extent of geotechnical investigations, such as total soundings or cone penetration test (CPTU). Figure 11 shows an ERT profile showing a prominent clay deposit (blue soil = low resistivity) and bedrock (yellow/red = higher resistivity). Along the ERT profile a series of Norwegian Total Soundings were performed to verify the bedrock interpretation. A more accurate bedrock profile was then obtained (shown as black dots in bottom of drilling columns in the figure). The ERT measurements also revealed the presence of an aquifer above bedrock with a higher resistivity, interpreted as a fine sand, which may impose a risk for settlements on the clay overburden in case of drilling for i.e. steel core piles or tie-back anchors.

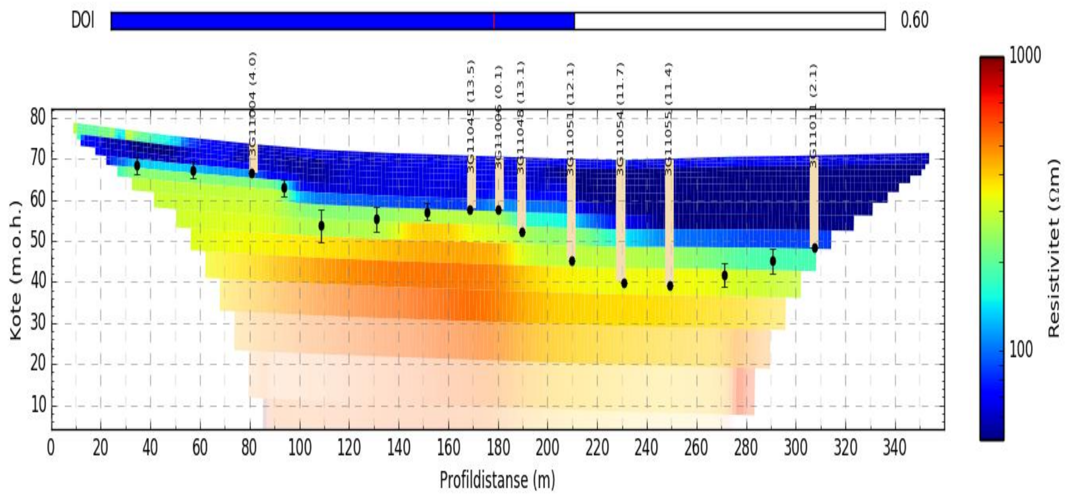

Figure 11: Results from ERT measurements combined with results from Norwegian total soundings providing a detailed bedrock interpretation and soil type identification.

4.6.2 Electrical Resistivity Tomography (ERT) combined with seismic investigations

A drilling campaign was carried out showing variations in bedrock depths between 1 m and 10 m on a 250 m transect. To decrease the uncertainty of the bedrock model in between boreholes, several geophysical methods were carried out.

A combined geophysical survey (ERT and seismic) was performed, resulting in the identification of two previously unknown weakness zones. Based on IP-measurements the sediments could be characterized as clays and an overall more detailed bedrock models was obtained. Figure 12 shows the resistivity result.

- Drillings are shown as vertical lines with ID KS1024 and KS 1030.
- The black dotted line is the final interpretation of the bedrock surface which could be mapped with an accuracy of +/- 0.5 m depth. The white line represents the seismic reflector (seismics not shown).
- The two weakness zones around at profile 100465 and 100725.
- Variations in the bedrock resistivity from more than 10000 ohm.m to less than 3000 ohm.m (change from purple to orange) from profile 100750 to 100600 indicate a decrease in bedrock strength due to tectonics (different bedrock type).

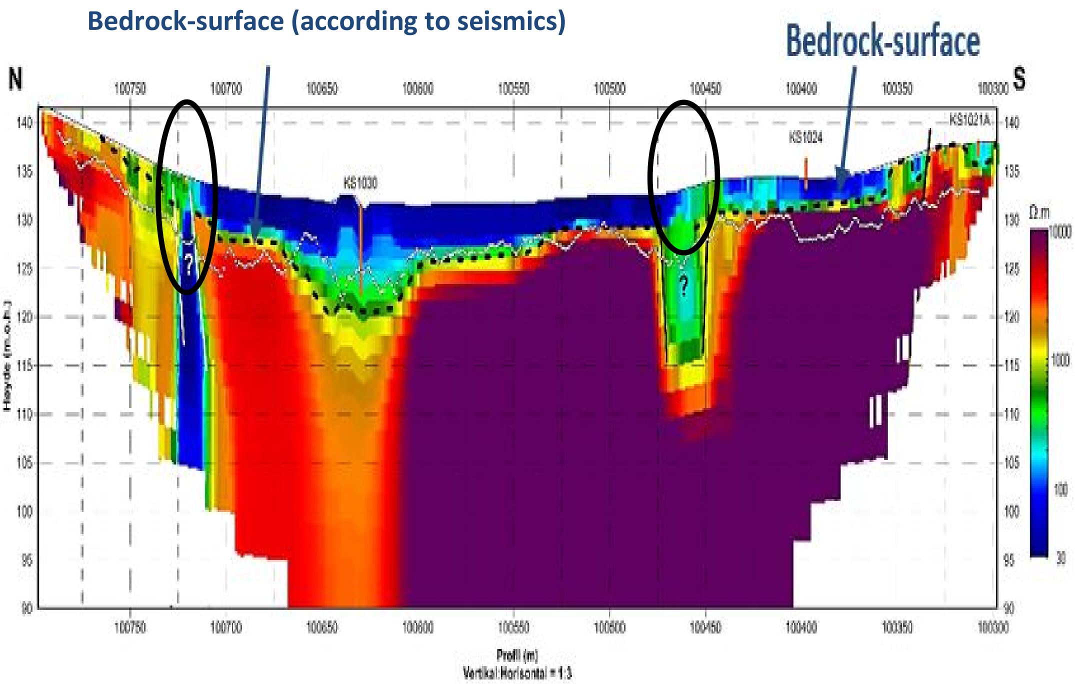

Figure 12: Combined results from ERT, seismic survey and Norwegian total soundings (source: NGI).

4.6.3 Electrical Resistivity Tomography (ERT) combined with Misse a la Masse (MAM) and CPTU soundings

For the construction of a road tunnel the client wanted to utilize the underlying natural aquitard (clay layer) to reduce costs for a geotextile or grouting below the tunnel floor. CPT soundings and core sampling showed that the thickness and distribution of the clay layer varied significantly across the 30 000 m^2 project area. A geophysical survey was carried out to map the thickness of the clay layer to identify areas which could lead to a potential groundwater leakage, as shown in Figure 13. The survey was carried out iteratively:

- Application of ERT along 16 profiles.
- Based on the ERT results and 58 CPT sounding and core samples, a ground model was developed for the project area. Two potential leakage zones were identified in this phase for further follow-up.
- Verification of the potential leakage zones (below 3 m clay thickness) with additional drillings and construction of four groundwater wells.
- Investigation of these zones with a Mise a la Masse (MAM) survey to evaluate the groundwater leakage potential
    - Injection of current in two different depths to investigate leakage of target layer in the four wells established for the survey.
    - Figure 13**Error! Reference source not found.** shows the result of the MAM. Please note that the potential leakage zone could be reduced significantly.
- Final grouting of the relevant leakage zones (400 m^2) and cost savings over ten million NOK

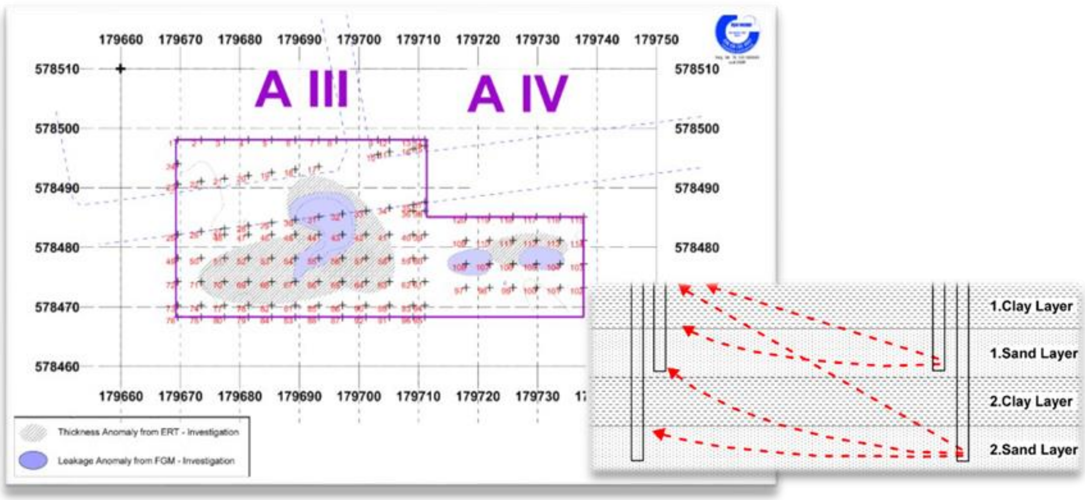

Figure 13: Left - MAM result for a potential groundwater leakage based on ERT results. Right – principle of MAM investigations within the project (figure courtesy: Texplor Group (Texplor, 2019). Crosses indicate location of sensors.

4.6.4 Quality control of grouting (combined SP and MAM survey)

After installation of a diaphragm wall in soft sediments the excavated area showed leakages along the barrier. To identify the location of the leakage paths (through or below the diaphragm wall) a combined self-potential surveying (SP) and Mise a la Masse (MAM) survey was carried out. Figure 14**Error! Reference source not found.Error! Reference source not found.** shows two MAM measurements, before (upper part) and after (lower part) injection. As can be seen the injection has been widely successful but the quality control (QC) showed that further work had to be carried out.

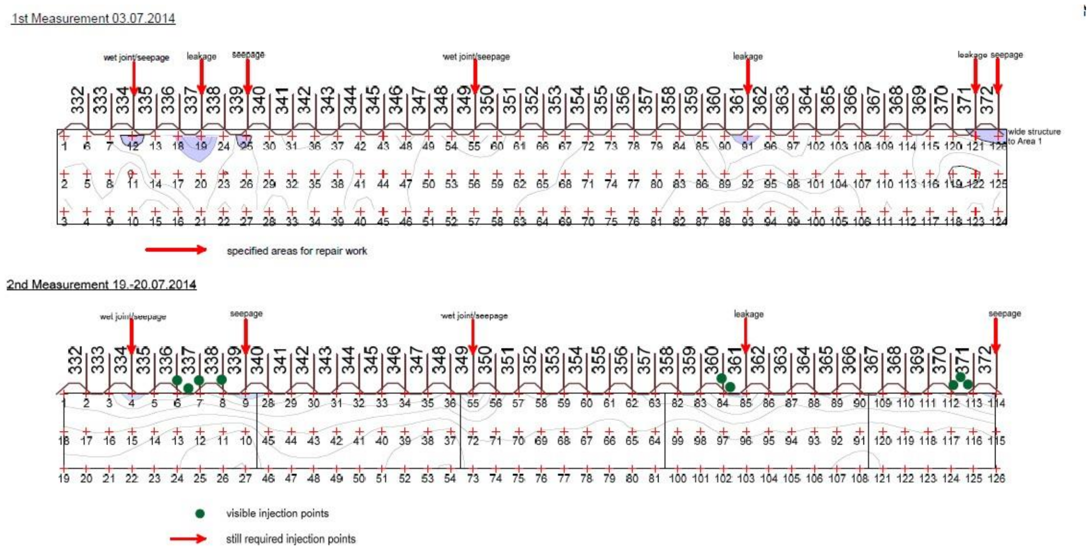

Figure 14: MAM result (top view) of leakages along diaphragm wall. The upper part shows the leakages before injection. The lower part shows the QC of the injection works (figure courtesy: Texplor Group (Texplor, 2019). Crosses indicate location of sensors.

5 Hydrogeological modelling

Hydrogeological numerical modelling is the art of compiling data regarding the hydrology and geology of a selected field site, visualizing the three-dimensional problem, then representing this conceptual model in a simplified manor with numerical terms that still reproduces the field data during varying conditions.

5.1 Why model?

Numerical modelling is as enlightening for the modeler as it is others reviewing the results. As soon as an acceptable model is established, new simulations of various conditions (drainage, pumping, infiltration, etc.) can easily be conducted quite quickly.

The positive sides of modelling often include:

- Compiling data for a holistic evaluation
- Enlightening when the conceptual model does not reproduce field measurements
- Illustrates the need and areas for additional field investigation
- Predict groundwater potential change after excavation or other changes in the field conditions
- Review consequences over varying time periods

Some potential negative sides of modelling:

- Modelling is time consuming, however it will generally be cost effective.
- The results are only as good as the quality of the input data
- Results are interpreted with an excessive degree of confidence

There are various types of modelling-work. Modelling can be used to predict the effects and consequences of various changes in the hydrogeological conditions. It can also be used to interpret possible field conditions or field investigations. Modelling can also be used to analyse generic hydrogeological conditions, to better understand hypothetical hydrogeological systems (Anderson & Woessner, 2007).

5.1.1 When should you model?

When is it appropriate to model? As mentioned earlier, modelling is time consuming and therefore can be expensive if the problem is small and relatively simple. Then an analytical solution and/or empirical data, using an appropriate equation, would solve the question at hand. It is therefore very important to ask what is the problem that is to be solved and how should it be modelled. Is it a one- or two-dimensional problem, which can be solved for a new or present steady-state condition, then an analytical equation may be enough for that particular case. If the problem is 2- or 3-dimensional, and especially if the solution needs to be during a transition-period to a new steady-state, then a numerical model is probably the most efficient method for analysing the problem, as illustrated in Figure 15 and Figure 16.

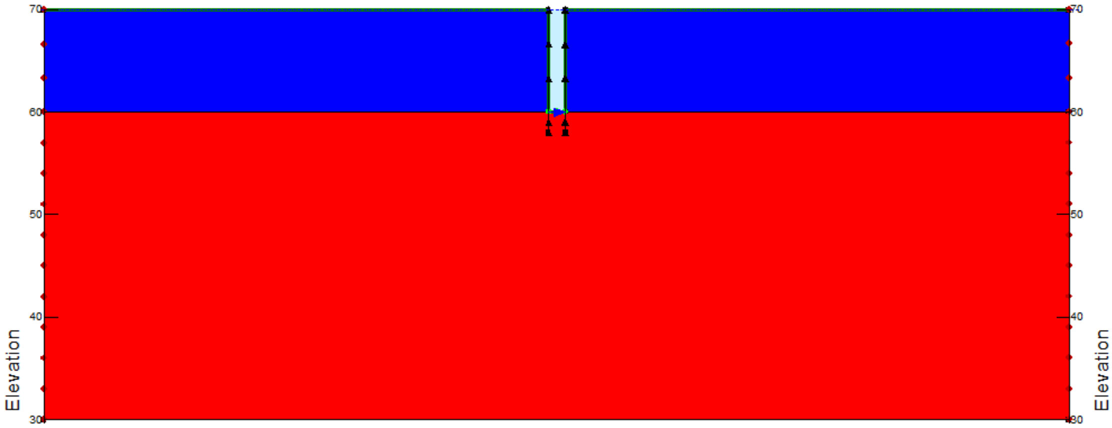

Figure 15: 2-D numerical model of 10 meters of clay deposits on bedrock, with construction pit contained by sheet piling and grouted bedrock below sheet piles (K.J. Tuttle).

An example could be a construction site, where there is an excavation pit that is constructed below the groundwater level. If the question at hand is the initial amount of water that will be needed to be pumped from the open pit floor, then an analytical solution is appropriate. This is because the question at hand is consistent with the initial conditions, i.e. the groundwater pressure at the start of the excavation process. Whereas, if the question is the amount of water to be pumped during the construction period and the change in groundwater pressure outside the pit during that time period, then a numerical model is more appropriate. Here the question is time dependent and the answer is changing with time.

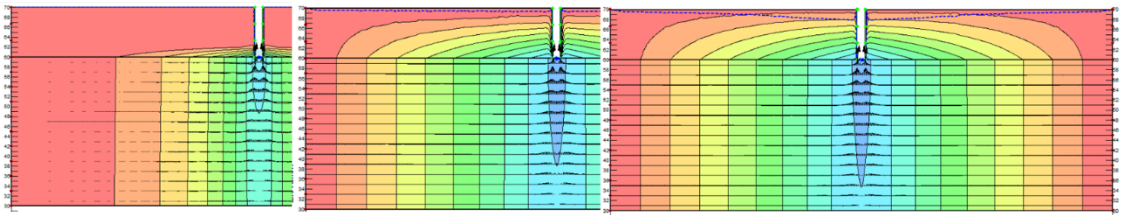

Figure 16: Time sequence og drainage of groundwater through the bottom of a construction site illustrated in figure 1. The figure on the left represents the extent of groundwater pressure reduction after one day, the next figure after 1 year and 23 days, and the last figure to the right represents drainage of the site after 10 years (K.J.Tuttle).

5.1.2 Types of models

Considering cases where construction sites are the focus point of concern, several numerical programs are available on the market that could be used to model various conditions. Figure 17 shows the two main types of models, one that models a continuous porous medium and another that models discrete factures. In general, the continuous porous medium model is best at representing groundwater flow in sediments while the other models flow in discrete fractures in bedrock. Although representing spatial variations of permeability is difficult in both types of models, it is most difficult to create a realistic 3-dimensional model of individual fractures and their permeability.

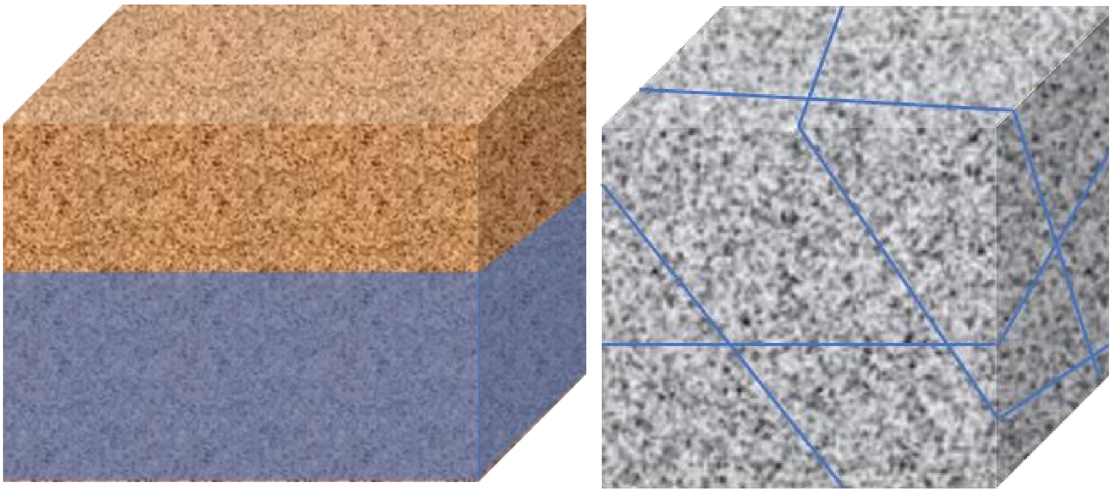

Figure 17: Volume of porous sand in left cube and fractured bedrock in right cube.

It is possible to model groundwater flow in bedrock using a continuum model if cell sizes can function as representative elementary volumes (REV) that contain equivalent porous medium (EPM) parameters representing the hydraulic conductivity distributions of the fractures in the REV. This method can work if the fracture distribution and connectivity is dense enough to allow model cells to be small enough to give enough detail to the problems in the case. These models should be considered as regional results, and not over-interpreted.

Examples where continuum models that have been used include simulations along tunnels to evaluate either the degree of grouting needed to limit groundwater ingress and/or the groundwater pressure reduction due to a given grouting activity. Parameter studies for excavation pits considering the groundwater ingress when pits are constructed solely in clays or when they are excavated further through the clay and into an underlying fractured bedrock. Also, simulations of bedrock caverns surrounded by clay deposits and culture heritage deposits dependent on high groundwater levels have been conducted both in Bergen and in Tønsberg. The practice of groundwater modelling is becoming more usual and varied and help in answering numerous questions in complicated cases.

5.2 Data input and results

It’s long been said about modelling that poor data in gives poor results out. That is of cause true, but what is poor data? Modelling is a tool that enables the modeler to comprise many different parameters together to give a combined result afterwards. In generic modelling, the case problem is often fictive, as well as the input data. Poor data in this case would be parameter values included in the model that were not naturally related, giving an unexpected/unnatural result. Input values that are physically coherent would give a more plausible result. Modelling is therefore helpful and important even when there isn’t large amounts of hard field data.

Input data that is necessary for a groundwater model includes:

- Surface topography
- Hydrostratigraphic layering (lateral and vertical extent)
- Hydraulic boundaries (infiltration, water bodies, discharge rates, water divides)
- Groundwater level / pressure
- Values for hydraulic conductivity, porosity/storage, etc.

5.2.1 Data acquisition

Generally, well documented boreholes is the best method for acquiring detailed data about ground conditions, including stratigraphic layering, groundwater level, bedrock fracture frequency and orientation. Borehole logging tools render detailed visual and digital resolution that enables the hydrogeologist to interpret in which fractures groundwater is flowing into and out of the borehole. This increases the accuracy of borehole pumping tests to determine the hydraulic conductivity for individual fractures and help understand in which areas of the bedrock there is higher or lower groundwater pressures.

Geophysical profiling methods can be a good indirect method for acquiring field data, and especially regarding bedrock topography and sediment stratigraphy, weakness zones in the bedrock and groundwater table. Geophysical methods are however sensitive to various ground conditions, and they need to be verified by more direct methods like borehole investigations. The combination of drillings and geophysical methods can allow for 3-dimentional interpretations of the ground conditions and groundwater level.

5.3 Conceptual model

Prior to building a numerical model of your site, it is important to create a conceptual model of the hydrogeological conditions to be modelled. This can be sketched on paper or more preferably in a GIS, as shown in Figure 18. Several numerical programs today use shape files from GIS directly in the model to define the geometries of the hydrogeological problem. The conceptual model should include the hydraulic boundaries of the area and the internal geometries of the hydrostratigraphic units to be modelled. This exercise aids the modeller in setting borders for the case area.

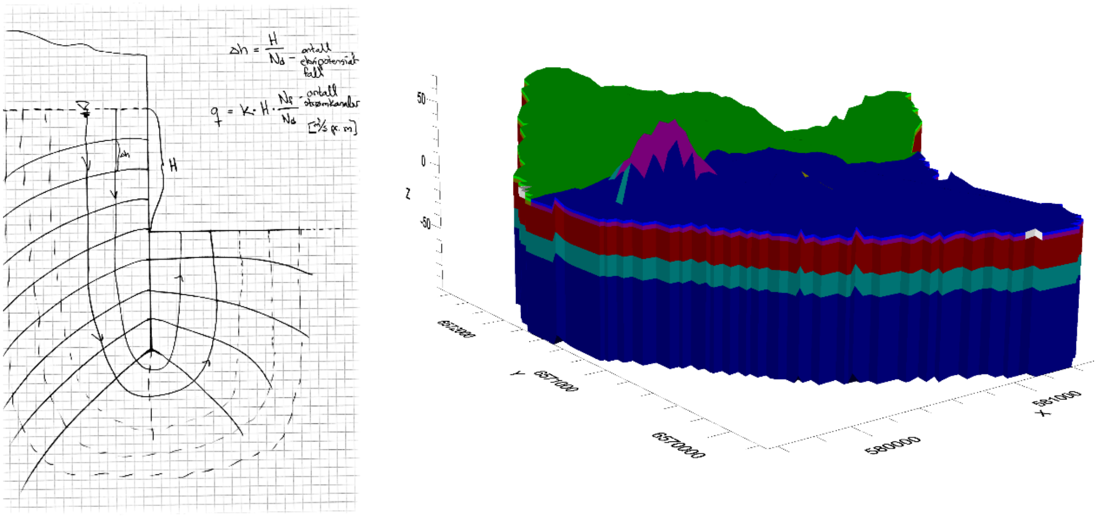

Figure 18: Example of a conceptual model for drainage of groundwater from the bottom of an open pit with sheet piling driven below the bottom elevation of the excavation pit (sketch: V. Brandvold) and a GIS-based 3-D conceptual model with several layers with varying hydraulic conductivity (E.Halvorsen).

Other than identifying the geometry of the hydrogeological conditions, making a conceptual model also aids in considering the form of discretization needed for the model. The form of the cells and their contact with other cells in the direction of the groundwater flow is important for the numerical stability of the program and the ability of the program to converge on a balanced solution. Cell mesh generation is becoming more automated in most programs, but the modeller needs to review the generated mesh to consider if the model should simulate the conditions properly.

5.4 Your toolbox

Calculating groundwater drainage and change in pressure potentials can be done by several methods and models. In simple 1- and 2-dimensjonal cases, where a steady-state solution is sufficient, analytical equations can give quick and easy answers. Often though, problems are more complex and require 2- and 3-dimentional solutions, often considering a time aspect instead of a final steady-state result.

5.4.1 Analytical solutions

There are a large range of analytical equations for various situations, and problems that can be described analytically if local conditions require it. There are however several limitations to analytical solutions, and one of the most important conditions is that they all give steady-state results. It is important to realize that these results can influence the interpretation of the results. An example could be calculating the groundwater drainage to a tunnel or an excavation pit. The drainage will be defined by the initial groundwater potential, which does not change in the analytical equation. Drainage results will be correct in the absolute initial moments of the drainage, but over time, the actual groundwater potential outside the pit or tunnel will be reduced by the drainage, and the subsequent drainage will therefore be reduced. The analytical solution will then give a correct answer for the initial moments of drainage, but not for the time that follows.

5.4.2 Numerical models

One of the oldest and most widely used groundwater modelling programs is MODFLOW, developed in the early 1980's (Holzbecher & Sorek, 2005). Today there are several Pre- and Post processers that use MODFLOW and that also combine several other modules other than 3-dimensional groundwater flow, including unsaturated flow, heat flow, contaminant transport and density flow. MODFLOW is widely accepted and therefore used in modelling also effects of tunnelling, deep excavations and dewatering of areas. One of the draw-backs with MODFLOW is that it isn't coupled with other soil and rock mechanic software for modelling ground material stability or deformation. Settlements are often calculated using the 1D settlement module GeoSuite, which could include groundwater pressure change from MODFLOW-results

GeoSlope Inc. has a suit of modules that can handle 2-dimensional flow in both unsaturated and saturated soils. The modules can simulate interactively both pore water pressure change and soil settlement, with groundwater drainage.

There are also 3-dimensional models, for example Plaxis 3D and Rocscience (RS3), that can model interactively pore water pressure and settlement with groundwater drainage. These types of numerical models would give the most reliable results for excavation cases and will be able to simulate time-dependent coupled conditions.

5.5 Keeping the model simple

5.5.1 What is the question?

When modelling, it is important to keep in mind the questions that are to be answered. The questions will determine the data necessary for the model, and the type of model that needs to be used. The simpler the model is, while still including representative boundary conditions, the easier it will be to reach a solution and understand which parameters are dominant in that case. Results will be easier to interpret due to the limited conditions that are included in the model. At the same time, it is important to include the defining conditions so that the model gives representative results. An example of a simplified model that under several circumstances would give a correct representation would be combining overlying hydrostratigraphic layers that have very similar hydraulic characteristics. Instead of having two layers, one lay can represent both, reducing the number of layers in the model and significantly reducing number of cells in the model. Excessive cells cause the simulation to take more time and creates large data files.

5.5.2 What are the determining conditions?

In keeping the model simple, limiting the simulations to the determining ground and construction conditions is important in making relevant interpretations of the results.

This can include creating a groundwater model that is used to simulate individual construction conditions that are of interest, illustrating the effects of each individual scenario. This may also shed light on the dominating scenario. After understanding the consequences of each scenario, a site-specific combination of the various scenarios can then be better understood. The combined effect of these scenarios will give a new result, and each individual scenario will have a changed result from a stand-alone scenario. Again, this shows how modelling is an intuitive instrument also for the modeler, resulting in a better understanding of the whole situation.

5.5.3 Which resolution is necessary?

Large models with many layers and cells make models run slower and create very large data files. Combined with large changes in hydraulic conductivities, groundwater gradients and boundary conditions, the model often becomes less stable and/or taking longer time to converge on an acceptable solution. The ultimate setup is a model with low resolution in areas that are not of very importance to the case questions, as long as the simulation isn't influenced by poorer resolution in areas distant to the question at hand. By changing the cell resolution to reflect the need for more or less detailed calculations within the model, the total sum of cells can be kept relatively low and make the modelling process easier.

It is sometimes possible to increase the cell resolution in a model after initial runs, but this can often cause problems too with future convergence of the simulation, so it is best to reach an optimal cell configuration at the first try. Generally, spending a bit more time in the beginning of a model project, determining model boundaries, input parameters and cell resolution will reduce the problems in simulations later.

5.5.4 Nested models

In some cases, it is necessary to model a large regional area to both understand the actual regional conditions playing on the local site of interest, and/or the importance of the regional scale conditions that define local conditions. In these cases, the model may become too large to simulate conveniently when it is necessary to review both the regional conditions as well as more detailed conditions at the local scale. In these circumstances, it is possible to divide the problem into two separate simulations, one covering the entire area, the other consisting of a much smaller area, with a much higher cell resolution at the site in question.
The nested model, or the smaller model that focuses on a much smaller area, includes the results of the larger model in its smaller boarders, using the results of the larger model at boundary conditions. In this way the nested model is able to build up the cell resolution needed for the local conditions that are not available in the larger model.

5.6 The art of modelling

5.6.1 Hard and soft data

Hard and soft data are here defined as actual field measurements for hard data and estimations of parameter values based on experience from similar field conditions for soft data. Soft data is also important for the model since there is always a limited amount of hard data available from the field. Hard data are generally kept unchanged when included in the model, but soft data can be adjusted within a range considered typical for the individual parameters.

Soft data is generally defined from the conceptual model of the area. When the initial numerical model is created, results from the simulation will probably not give an acceptable representation of the groundwater head. This indicates that the conceptual model is not representing the field conditions adequately and needs to be revised in order to give a better simulation of the aquifer. The process of adjusting the soft data parameters to better the conceptual model and parameter settings is called calibrating the numerical model.

5.6.2 Model calibration

Almost all modelling projects are inverse models, where the modeler often knows more about the groundwater water level than the actual aquifer parameters and boundary conditions. The groundwater level is easier to measure and often does not change as radically as the other field parameters might. The model is then calibrated around the known groundwater level by adjusting the other parameters and boundary conditions, within a reasonable range of values. A forward model is when one has the known field-parameters and is trying to estimate an unknown groundwater level.

Calibration is a trial-and-error process, where the initial model encompasses the conceptual model’s geometry, parameters and boundaries, as well as the measured groundwater levels as reference measurements. After running the initial model, deviations from the simulated groundwater level and the actual field groundwater level is compared. Almost always, the modeler will find that the simulated groundwater level does not compare with the measured level satisfactorily. Generally, the difference between the simulated and the measured values should lie within 5% of the Root Mean Error or standard deviation.

In refining the model after the initial simulation, the modeler will want to adjust the parameters that are of largest uncertainty, but within the range of probable values. This iteration of adjusting model parameters continues until the model shows a satisfactory comparison with the measured groundwater levels. It is this process of calibration that the modeler experiences the voids of her/his understanding of the hydrogeological conditions in the field. This process will also indicate what areas and parameters that should be investigated more and how the various parameters are influencing the results of the simulations.

5.6.3 Testing your conceptual model

During the calibration process, the model may not converge to give a steady numerical solution. The combination of the various parameters, combined with the spatial cell resolution, may not give a numerical result. In these cases, the modeler will need to re-think their conceptual model and perhaps even re-build the model from scratch. The positive side of this tedious exercise is that the modeler will better understand the holes in their conceptual model and in which areas the model is in need of additional hard data.

5.6.4 Additional data

At this point in the modelling exercise, additional data may be required to understand adequately the actual field conditions. Additional drillings to map the hydrostratigraphy, hydraulic conductivities, groundwater heads and flow gradients may be necessary. Boundary conditions may also need to be verified in the field. Although this may sound unnecessary, there is arguably a lack of understanding of the groundwater conditions, and it will be necessary to understand the actual conditions during construction and use/maintenance.

5.6.5 Sensitivity test

A sensitivity test should be run on the successfully calibrated model so that the uncertainty of the parameter and boundary values can be quantified. There are actually several combinations of values within the parameters and boundary conditions that could result in the very similar results, but each parameter has a different impact on the results. The sensitivity analysis quantifies the impact each of the major parameters has on the results of the model. The parameters that should be tested are the hydraulic conductivity (vertical and horizontal), recharge and storage coefficient and boundary conditions (Anderson & Woessner, 2007).

The type of simulations described in this chapter are called deterministic modelling, where the most likely parameter values are used directly in the model. The sensitivity test looks at the effect of changing one parameter value at a time to find the sensitivity of each parameter in the model. Another type of simulations is stochastic modelling. Stochastic modelling uses a probable distribution function for hydraulic conductivities in the aquifer to generate probable fields of K in the model. New fields are generated, each as probable as the other in numerous Monte Carlo simulations. This results in a sort of sensitivity analyses, giving more probable solutions than other. This method is very straining on the computing capacity and demands a considerable amount of data handling afterwards. It isn’t always resulting in hydrostratigraphically sensible results either.

5.6.6 Model validation

Considering that even with a satisfactory calibration and acceptable sensitivity analysis, the uncertainty in the model needs to be tested against a known change in the parameters.

The last test of a model is the validation of the model to show that it is able to replicate a known change, in order to indicate if the model will give acceptable results in other conditions in the aquifer. This can include typically a pumping test in the aquifer or a well-documented change in recharge and groundwater level. Since the calibration of a model is in theory one of many combinations of parameter values, good results from changed stresses in the model, in the form of a pumping test or well documented recharge episode (e.g. rain), will indicate that the values chosen were good representations of the field parameters. This gives greater confidence in the subsequent model results when simulations predict future conditions.

In the case of the pumping test used as validation, a pumping borehole and observations boreholes are included in the model, along with the pumping rate and observation levels. The model is run to simulate the stress event in the aquifer and the resulting simulated groundwater levels are subsequently compared to the field measurements. If there is a good match, the model should give predictions on other stresses which are of interest to the project. If the results do now compare well with the field measurements, then the parameter values of the model probably do not describe the field correctly but were one combination of values that were able to give the same initial groundwater levels. In this later case, the work with calibrating the model needs to be conducted again, including a following new sensitivity test.

6 Measures to prevent and control drainage

Data shows that small volumes of water leaking into an excavation may cause significant reductions in pore water pressure at bedrock level and initiate a consolidation process in the soft clay overburden. When planning a deep excavation project, measures to prevent and control the potential for leakage needs to be considered and included in the design. Preventive measures for drainage, for example installing a grout curtain underneath a retaining wall, is often a costly endeavour and can also be time consuming. However, the toolbox of available measures when a leakage has occurred are often limited. Experience from Karlsrud et al (2015) reveal that only projects where mitigating measures where applied, such as grout curtains and water injection wells, managed to limit the pore pressure reduction in the surroundings.

6.1 Awareness of drainage and damage potential

All excavation which include work underneath the groundwater table has a potential of lowering the ground water table or pore pressures over large areal extents. For each of these projects the potential for damage due to consolidation settlements in clay, degradation of wooden rafts or piles, increased skin friction on piles and effects on entrances and infrastructure needs to be assessed.

Many factors need to be considered when assessing the level of risk related to damage on surrounding third party interests caused by settlements. These factors include among others:

- Geotechnical, geological and hydrogeological ground conditions
- Depth of excavation
- Retaining wall type and support system.
- Foundation type
- Vulnerability of surrounding buildings and infrastructure

Previous projects show that the costs related to damage caused by drainage to excavations often are substantial. In addition, the impact on private property owners is strenuous and processes result in negative publicity for the developer. The Nabolov and Vannressurslov also requires the developer to be cautious. In densely populated areas it is evident that mitigating measures are necessary to reduce the risk for damage.

6.2 Strategy for drainage prevention and control

Experience show that drainage is one of the main causes of settlements that are not accounted for in design. History has shown that to manage an unacceptable risk for damage from settlements, the remedial measures must be taken from the early planning stage until construction is finished.

6.2.1 Early design phase

It is important to undertake feasibility studies and assessment of ground conditions including analysis of geotechnical (especially the over-consolidation ratio) and hydrogeological conditions (artesian pressures, sensitivity analysis to drainage), to evaluate the risk of pore pressure decrease with respect to the design depth of the excavation and the distance to the bedrock surface. Time for construction and possible duration for an open excavation pit needs to be evaluated and space available on the construction site for any potential drainage measures.

At this stage, impact of previous and ongoing construction (ongoing settlements from drainage to tunnels, excavation pits, earthworks and fills, ground water lowering) need to be considered. In addition, alarm limits for pore pressure reduction and requirements for maximum allowable settlements (including creep) should be evaluated, depending on the sensitivity to damage for buildings, structures and infrastructure surrounding the excavation. The risk for pore pressure decreases due to drilling for tie-back anchors and piles should be evaluated against alternative methods (internal struts and driven piles).

6.2.2 Detailed design phase

During the design phase, measures to decrease the impact from the excavation and foundation works need to be described:

- Use of temporary packers in casings, grouting and sealing of boreholes in bedrock.
- Assess and choose drilling methods considering risk of erosion, disturbance and drainage.
- Request logging of data during drilling.

The hydrogeological conditions should be assessed, with respect to permeable layers over bedrock or in the clay, to enable evaluation of the influence of drainage. In addition, hydrogeological tests, geological mapping of fractures and weakness zones can be valuable. Describe measures to prevent drainage into the construction pit, as well as wells for water infiltration to uphold pressure levels.

6.2.3 Construction phase

A plan for monitoring needs to be established to document the impact of the excavation. Measurements and data acquisition need to start well in advance of construction, to capture seasonal variations in pore pressures, as well as ongoing settlements.

The monitoring of pore pressure should be undertaken in a zone of at least 200 – 300 m, from the excavation to capture the possible effects of drainage. It is crucial to install piezometers in the intersection between clay and bedrock and in permeable layers in the clay, to be able to capture the quick response on pressures caused by drainage.

It is necessary to undertake quality control at the construction site by geotechnical engineers or other qualified staff, including verifying the contractor's procedures and execution for drilling and sealing/waterproofing. Finally, requirements should be made to ensure that the contractors have the requested skills and expertise.

6.3 Grouting as a drainage mitigation measure

Grouting is considered one of the main mitigation measures to reduce the risk of unacceptable reductions of pore pressure and subsequent settlements in the surrounding area. To put it simply, grouting means that air and water-filled voids (fractures and pores) in soil and rock are filled with a liquid material that after a certain time hardens and turn into solid form. The split spacing technique is often adopted where the number of grout holes are increased progressively until the desired water-tightness is obtained.

6.3.1 Different types of grouting

The term ‘grouting’ could be regarded as a common name for a variety of different methods used to change the properties of the ground for sealing and/or stabilizing purposes, for example:

Permeation grouting. The most common grouting method, where the grout is injected into existing joints and pores in the ground, the grout pressure is held sufficiently low to prevent the creation of new fractures

Fracture grouting. Compared to permeation grouting, during fracture grouting a pressure is used that causes some propagation of fractures and deformation of the formation

Compaction grouting (displacement). High pressure and a stiff grout material is used when compaction grouting is performed, and the aim is to cause densification of the ground around

Jet columns (replacement). Jet grouting utilizes a special drill bit, which excavates columns in the ground using high pressure water jets and thereafter fills the columns with grout material.

Artificial ground freezing. Freezing of the ground differs in that it does not involve the introduction of a material into the ground but freeze the existing water in the ground to impermeable high strength screens in the soil.

6.3.2 Selection of grout material

The grouting result, expressed as the spreading of grout, is affected by the grouting pressure, the rheology of the grout material and the pore structure of the formation to be grouted. Furthermore, the result is affected by possible dilution of the grout and, also, the grout can be transported away by the groundwater flow (Borchardt, 1996).

Necessary results from a ground investigation campaign to perform a detail design of the grouting operation include the soil layers, grain size distributions, relative density and permeability. Based on these findings, the detail design includes for example the position of grout holes (distance, length) and choice of grout material and grouting pressure.

Littlejohn (1985) states that a ground investigation is only satisfactory if it gives enough information to answer the following questions:

Can the ground be grouted? *Is grouting a suitable method for sealing or stabilizing the ground, can the required spreading be obtained?*

For ground treatment what types and amounts of grout are required? *Which properties are the most important for the grout to possess, would it be cost effective?*

What strength increase or permeability reduction can be anticipated? *What results have been obtained through grouting earlier in similar soils, evaluated from laboratory tests if possible, or field observations?*

Grouting is ultimately a craftsmanship, and to achieve a good sealing result it is important that the work is performed in the right manner. Experienced grouters can observe the response of the injection fluid and control the execution underway, so that the risk of plug formation in the hollow bar or for erosion/flushing away of soil mass is minimized.

6.5 Grouting with cement

Cement is an economical and well-established grouting method used primarily for sealing of fractures in rock (Figure 19). Cement-based grouts are suspensions, i.e. solid particles (cement grains) suspended in a fluid (water). Depending on the alignment of fractures relative to the ground it may be beneficial to drill non-vertical grouting holes to maximise the coverage of fractures.

Bedrock grout curtain. Cement is commonly used to create a watertight barrier in bedrock. This is used when bedrock is uncovered in the excavation pit, or to seal of drainage paths within the bedrock below the excavation. It is a precondition to achieve necessary spreading to all water bearing fractures that a certain grouting pressure can be used, alternatively that the grout hole pattern is very close.

Using polyurethane in combination with cement (combi-grouting). As reported by Huth & Wien Engineering AS (2010), this combination may reduce the consumption of cement, and due to the reaction pattern of the polyurethane a more watertight result is also obtained, compared to the use of cement only. It is especially beneficial where there are large losses of grout in problematic areas, where it is difficult to reach counter pressure and generally when the cement consumption is very high.

Further reading on cement grouting is available in appendix D, and NGF (2019).

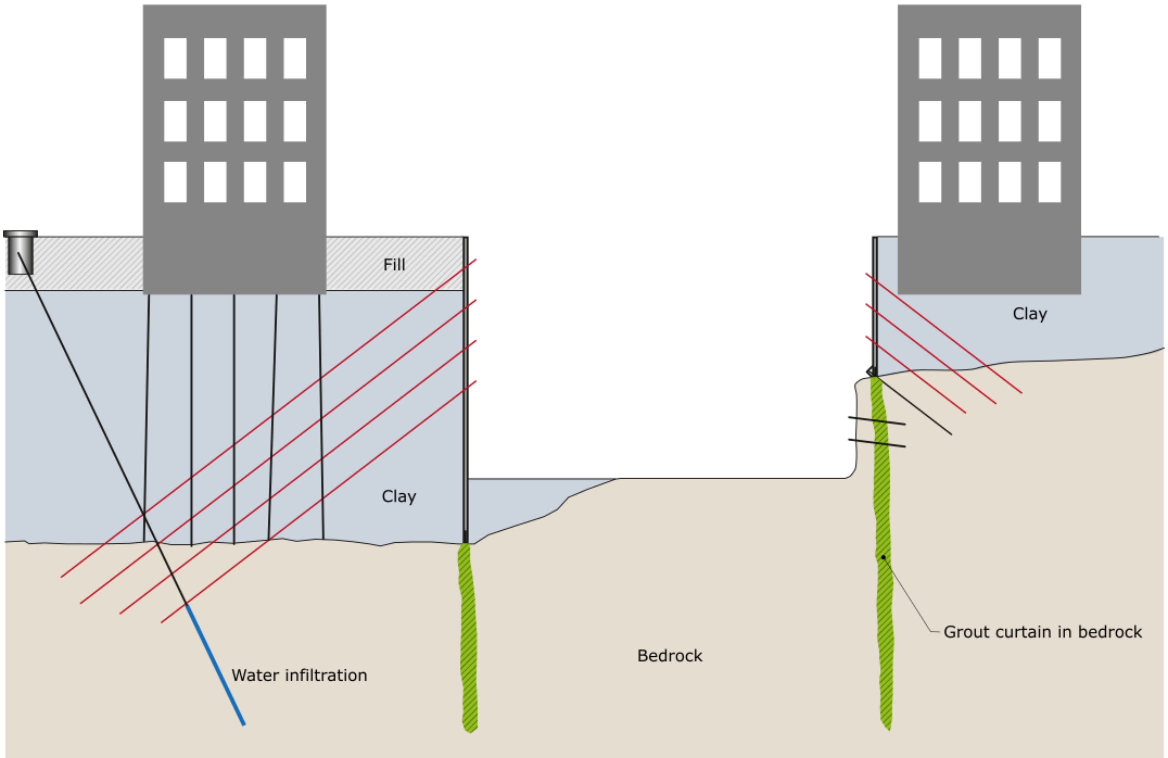

Figure 19: Grout curtain below sheet pile walls to seal against drainage to the excavation from fractures in the bedrock (NGF, 2019, figure modified to English).

6.6 Grouting with polyurethane

Polyurethane (PU) is an expanding grout material that can be used with light grouting equipment and is not so affected by freezing (non-aqueous). There are many different types of polyurethane, both water soluble and not water soluble, as well as one-component and two-component. The suppliers in the market have different products to choose from depending on the application at hand, for example regarding expansion ratio and reaction time.

The following section provides a brief overview on a few of the various areas of application for polyurethane grouting, also illustrated in Figure 20. Most importantly, PU does not require a high counter pressure, and can thus be utilized to seal leakages even after soil in the excavation pit has been removed.

Sealing of holes in sheet pile walls (when drilling for tie-back anchors). In several projects, polyurethane has been used to seal leakages around anchor holes. For this application, rags and textile fibres soaked in TACSS (a 1-component PU product) and pushed in place around the anchors. When in contact with water, the polyurethane reacts in the hole and expands around the anchors, effectively creating a watertight barrier. See also Figure 21.

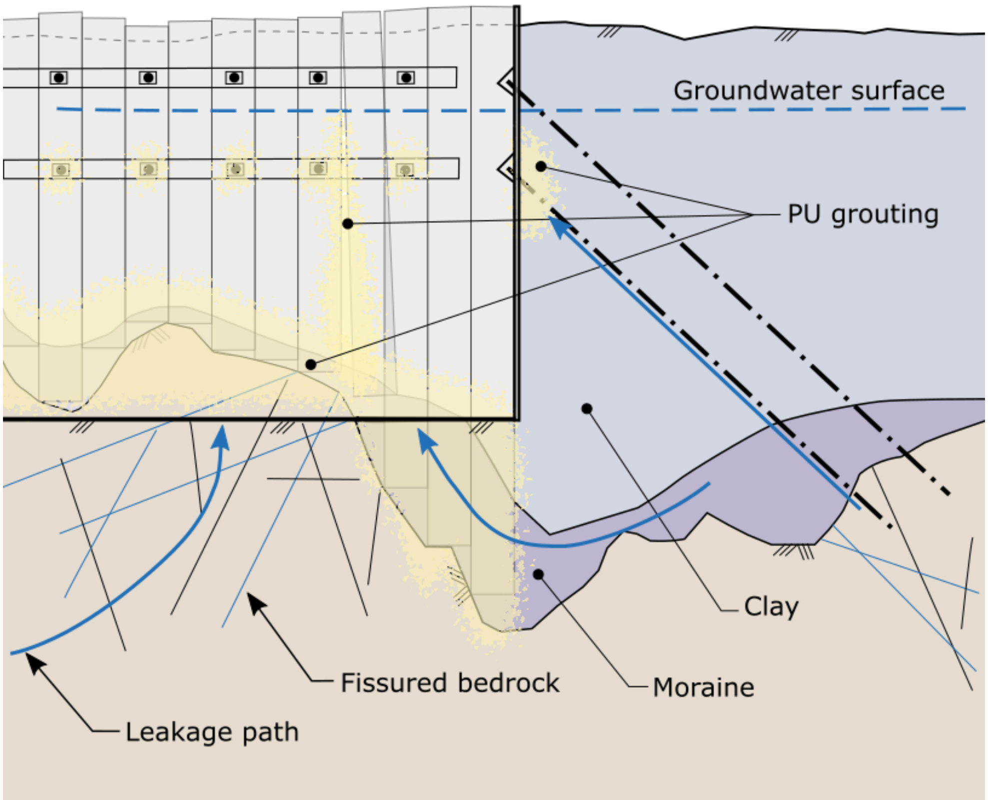

Figure 20: Sealing of gaps in retaining structure below groundwater table with polyurethane (PU) grouting to prevent drainage into the excavation.

Sealing of gaps between toe of sheet pile and bedrock after excavation. Polyurethane is a good solution for sealing against leakages through any gaps between the sheet pile footing and bedrock surface. Sealing can be performed if leaks are detected after excavation has been completed, which would be very difficult and time-consuming to seal using cement grouting. Also, here the split-spacing grouting technique is commonly adopted.

Sealing of leakage through the sheet pile wall locks. When leakages occur in the sheet pile locks or are visible during the excavation of the building pit, small holes can be drilled into the actual lock and the grout can be injected in the connection between the sheet pile sections. This is performed as a time and cost saving measure, as an alternative to welding the locks at the building site. It is common to use polyurethane as grout material in this case. This sealing method is performed rather quickly by two people with light pump equipment. The grouting efforts can be adjusted to the size of the water leakages and the conditions at the building site.

Sealing of leakages when drilling for steel core piles. Leakages can occur along the outside, or inside, of casings for steel core piles. In such case, polyurethane can be used as a grouting method in order seal such leaks.

Further reading on polyurethane and other chemical-based grouting methods is available in appendix E, and NGF (2019).

For safety reasons, it is important not to heat the polyurethane product (*e.g.* due to welding), since the poisonous hydrocyanic acid gas is formed.

Figure 21: Left: Sealing of an opening for anchors with polyurethane grout (Picture: Huth & Wien Engineering AS). Right: Detail of opening for anchors after sealing (Picture: NGI)

6.7 Jet columns

A jet column is a concrete cylinder casted in the soil without the need for excavating. The principle includes the use of high-pressure water and air jets that erode the soil while simultaneously the soil is replaced, or mixed with, a cement slurry.

According to Peleveiledningen (NGF, 2018b), the jet grouting technology can be divided into two categories:

- Geo-concrete columns (i.e. *compaction grouting*, soil mixed with cement slurry)
- In-situ concrete columns (soil is replaced, cement slurry only)

When jet columns are to be installed in contact with the bedrock, the pilot drilling is extended into the bedrock until the erosion nozzle is flush with the bedrock surface. With the erosion nozzles installed perpendicular to the drilling rod, the contact with the bedrock will depend on the slope of the surface.

To obtain a watertight barrier, it is important to avoid gaps between columns. If the centre of the pile is higher than the surrounding bedrock, an untreated are between the bottom of the column and the bedrock might exist, as made visible in Figure 22. In cases like this, the void area will be treated during the installation of the next column due to the overlap. The c-c distance between the columns determines the contact between the columns and is controlled to give enough overlap. Depending on the requested level of sealing, one or more rows of jet columns might be installed.

Jet columns are typically adopted to provide sealing for:

- Transition between sheet pile toe and bedrock surface
- Below bottom of excavation (a plug)

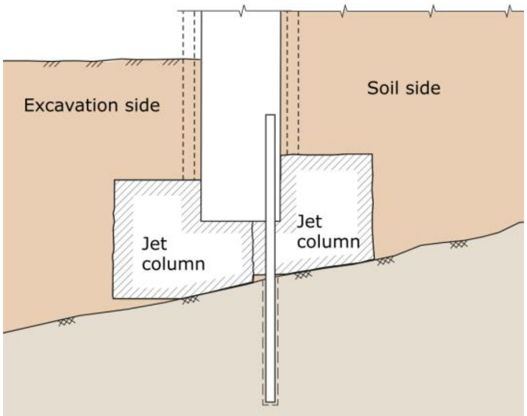

Figure 22: Sealing of transition between bedrock and sheet pile (NGF, 2019).

One key benefit with jet columns is that they may provide several functions such as a load-bearing structural member in combination with its purpose of sealing against water leaks. Additional information on jet grouting is detailed in appendix F, and NGF (2019).

6.8 Artificial ground freezing

Temporary sealing and stabilization of in-situ rock and soil through artificial ground freezing (AGF) may be an environmentally friendly option compared with the methods previously discussed (Johansson, 2009). The temporary construction consists of frozen water which creates a hydraulic seal and bonding agent between soil and rock particles, effectively holding these together, conceptually shown in Figure 23. When the temporary frozen soil construction has served its purpose, the ground is thawed, naturally or artificially, such that the groundwater can once again flow free.

For sealing purposes around an excavation, the frozen zone may be targeted to areas where drainage is expected, i.e. in the transition zone between the sheet pile and bedrock surface or sealing of prominent permeable soil layers.

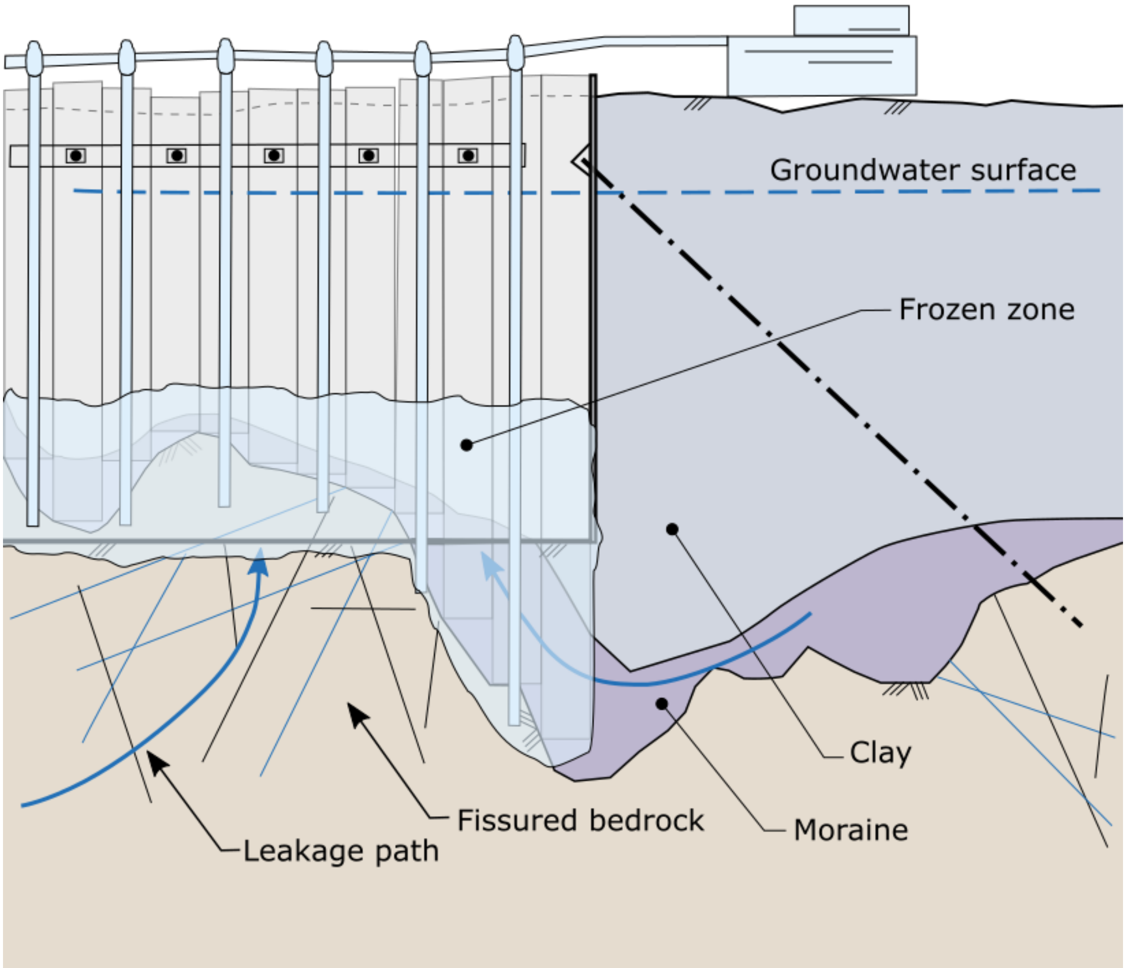

Figure 23: Artificial ground freezing in transition zone between sheet pile and bedrock surface.

Artificial ground freezing is relatively insensitive to soil composition provided the moisture content exceeds approximately 10% (Johansson, 2009) and may thus be applied in most ground condition. Fine grained soils may experience thaw consolidation related to excess pore pressure build-up during the thawing process and reduction of the clay water content after thawing (NGI, 1994, Johansson, 2009).

Freeze pipes are commonly installed inside a drilled casing and the pipes can be installed either vertically, tilted or horizontal depending on the capabilities of the drilling equipment. Freezing of the ground is not rather complicated, and primarily obtained using one of the following techniques:

Indirect freezing is a closed system between the cooling system and freeze pipes, powered by electricity, in which a brine is circulated having a temperature of approx. -40 °C. The cooling component in the brine is often ammonia. This artificial freezing technique is suitable for projects where a large volume of ground over a longer period (NGF, 2019). Figure 24 shows some equipment used for this method.

Direct freezing is an open system using liquid nitrogen (-196 °C) as cooling component. As this is an open system, the liquid nitrogen is consumed and requires re-fill if the freezing period is active. Due to the much larger temperature gradient the ground can be frozen approximately five times faster compared to brine freezing (NGF, 2019). Direct freezing also requires less equipment to mobilize, although it needs to have vehicle access to re-supply the liquid nitrogen. As such, direct freezing using nitrogen is more suitable for smaller projects of limited a time-frame.

Figure 24: Freezing container and equipment for brine freezing. (Image courtesy of Anne-Lise Berggren, Geofrost AS).

Additional information and project examples on the artificial ground freezing method is detailed in appendix G and NGF (2019).

6.9 Water injection wells

The procedure of water injection involves drilling injection well into an aquifer and introduce water into the aquifer my means of pumping. The injected water helps to maintain the pressure in an aquifer subjected to pore pressure reductions due to leakage. The wells may be strategically placed near a deep excavation to shield specific vulnerable buildings or infrastructure objects. Water injection wells should be a part of any well-planned mitigation measure program to control settlements of surrounding buildings and infrastructure caused by drainage. It is important to establish the wells in advance of the construction works. Monitoring of real-time pore pressure levels by means of pore pressure sensors (as detailed in 4.4.1) is important.

A fully functional water injection program relies on detailed ground investigations including the mapping of bedrock fractures, any presence of bedrock depressions and the natural groundwater gradient. NGF (2019) report positive experience with injection water into bedrock, whilst the experience related to injecting water directly into soil deposits is not as promising.

Experience from several projects show that directly injecting water from construction site runoff easily clogs the wells due to the presence of fine-grained sediments. This is also the case even if there have been sedimentation tanks on site prior to injecting. If the well is badly clogged it may be required to replace it by drilling a new well. The inclination of the well must be based on the fracture direction to maximise contact to the fracture system, as shown in Figure 25.

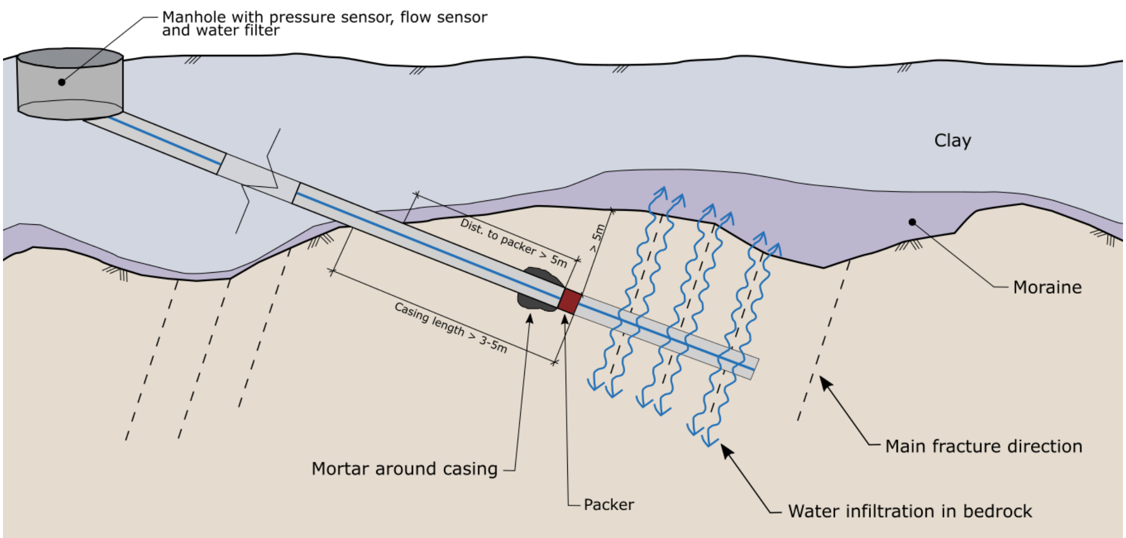

Figure 25: The principle of a water injection well in bedrock (NGF, 2019, adopted and translated).

6.10 Packer and plate sealing

Openings in the sheet pile wall will naturally always be a possible source for leakages into the building pit. It is however necessary to make holes in the sheet pile wall where anchors are to be drilled or where for example pipes or cables shall be inserted. There are also holes in the sheet pile that is used when it is lifted. Holes should systematically be sealed as soon as possible. Also, old holes in used sheet piles should be sealed by welded-on steel plates before the sheet pile is driven down. In addition to i.e. polyurethane grouting, the following measures can normally be used to seal holes in sheet piles (NGF, 2019):

- "Packer solution" with rubber plate
- Welding (welding on steel plates, to create a water-tight connection between the casing and the steel pile wall), shown here in Figure 26.

Figure 26: Steel plate welded onto sheet pile to cover hole opened during drilling of anchors (NGI).

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