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Ground Settlements caused by Drilling of Geothermal Energy Wells in Norway

-A Porepressure Reduction Analysis

Master's thesis in Civil and Environmental Engineering
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Preface

This Master thesis in geotechnics at NTNU is written in the course TBA4900 during the spring semester of 2022 as part of the MSc in Civil and Environmental Engineering. The course is rewarded with 30 ECTS.

The idea was proposed by NTNU, Asplan Viak and Oslo Municipality. The work has been carried out in cooperation with my supervisor Professor Rao Martand Singh (NTNU IBM), Randi Kalskin Ramstad (Asplan Viak) & (NTNU IGP) and Jenny Ingelöv Eriksson (Oslo Municipality).

This work will look into the subject with a numerical analysis of hypothetical geothermal energy well installations and other similar bored ground works, finding the ground settlement due to porepressure reductions at bedrock. Different soil profiles and pore pressure profiles are simulated, presenting time and area of influence relations and the mitigation measures available. In addition, an excel tool is made available for the industry, for quick risk and settlement assessments for 1 dimensional drainage. Keep in mind that this is a masterstudent's work with limited field data due to pending court cases, and that the work should be treated as such.

Trondheim, 2022-06-10

Jonathan Marson Våpenstad

Acknowledgment

I would like to express my appreciation to my supervisor Rao Martand Singh for helpful guidance and commitment during this master thesis. I would also like to thank Einar John Lande (NGI), Elisabeth Gundersen (SVV), Eivind Wiik Ånes (NGI) and Sondre Sagmoen (NGI) for valuable discussions and insights surrounding the topic. Steinar Nordal (NTNU) and Seyed Ali G. Amiri (NTNU) provided valuable and forthcoming discussion and help regarding the models, which is greatly appreciated.

The topics are interesting and much has been learned, which will surly be helpful in my professional career which will begin after the summer. Last, but not least I want to thank Kine for her invaluable help and support.

Abstract

Geothermal energy well systems are increasing in popularity for heating and cooling buildings in Norway and the rest of the world, and are one viable solution to high energy prices and climate change. However the recent years court cases in Norway regarding ground settlement damage due to energy wells, have raised the need to properly assess the long term effects on the settlements when done in areas with soft clays. Possible causes due to drilling and installation of general ground works from the gathered literature and case studies were found to be overboring, disruption of the soils and pore pressure loss. Risks related to ground works is in particular higher for air pressure drilling in coarser soils with high hydraulic gradients. Pore pressure reductions can also be due to increased seepage to bedrock by fractures caused by drilling or through a bad plug in the soil bedrock transition. Simulations of hypothetical scenarios of energy well installation to find the parameter and settlement relationships was done for different overpressure and hydrostatic pore pressure profiles.

The one dimensional analytical solution and PLAXIS 2D used in this thesis was to give time vs. settlement curves for general soil profiles based on Oslo clay due to a permanent/long term pore pressure loss at bedrock for one dimensional drainage. The results for 1D drainage show that a pore pressure loss below hydrostatic is dependent on the depth to bedrock, the thickness of dry crust/ weathered clay and stress history of the clay. An equal pore pressure loss in an overpressure linearly above hydrostatic for the entire length below the water table drained to hydrostatic is also dependent on stress history, but is more severe relative to the below hydrostatic pore pressure loss and much less dependent on the depth to bedrock. Increasing the depth results in greater settlements long term. The difficulty in 2D axisymmetrical modeling and unknown boundary conditions however, makes the mapping of this 3D drainage and influence area difficult and resulting in figures that are non-general. The depth to bedrock, inflow of water as well as extent or lack of a moraine/permeable layer determines the area of influence.

The results show that the general clays in Oslo can give substantial settlements if normal consolidated and the presence of a deeper soft soil above bedrock amplifies this. A soft clay or clay with lower permeability makes the area of influence more dependent on the outer boundaries/infiltration into the moraine. A clay trench/depression with the bottom filled with moraine increases the risk of differential settlements and thus damage to buildings and their transitions.

A large 3D model where the soil layering (especially moraine) and bedrock topography is mapped should be done on a case by case basis, as the soil layering, boundary conditions and depth to bedrock varies too much to use a general model for more than one dimensional drainage. Using the produced 1D analytical solution for measured pore pressure reductions in permeable soils above bedrock is recommended for quick assessments of one dimensional drainage in clays. The presence of a drainage/permeable layer as moraine is deemed necessary for one dimensional drainage and its omission can result in slower, smaller settlements and influence area.

Sammendrag

Geotermiske energibrønnsystemer øker i popularitet for oppvarming og avkjøling av bygninger i Norge og resten av verden, og er en løsning på høye energipriser og klimaendringene. De siste årenes rettssaker i Norge angående setningsskader på grunn av energibrønner har imidlertid reist behovet for å vurdere de langsiktige effektene på grunnsetninger, spesielt når de gjøres i områder med bløt leire. Mulige årsaker på grunn av boring og installasjon av grunnarbeider fra innsamlet litteratur og casestudier er overboring, forstyrrelse og poretrykks-tap. Risiko knyttet til installasjon av grunnarbeid er spesielt høyere for lufttrykksboring i grovere løsmasser med høye hydrauliske gradienter. Poretrykksreduksjoner kan også skyldes økt tilsig til berggrunnen via bergsprekker, eksisterende eller forårsaket av boring, eller gjennom dårlig plugg i foringsrør-berg overgangen. Simuleringer av hypotetiske scenarioer for installasjon av energibrønner for å finne parameter- og setnings forhold ble gjort for ulike overtrykk og hydrostatiske poretrykkprofiler.

Den endimensjonale analytiske løsningen og PLAXIS 2D ble brukt for å gi tid vs. setningskurver for generelle løsmasseprofiler basert på Oslo-leire på grunn av et permanent/langvarig poretrykkstap ved berg for endimensjonal drenering. Resultatene for 1D-drenering viser at et poretrykkstap under hydrostatisk trykk er avhengig av dybden til berggrunnen, tykkelsen på tørrskorpe/forvitret leire og spenningshistorikken til leirene. Et likt poretrykkstap i et overtrykk lineært over hydrostatisk for hele lengden under grunnvannslinjen drenert til hydrostatisk, er også avhengig av spenningshistorien, men er mer alvorlig i forhold til det under hydrostatiske poretrykkstapet og mye mindre avhengig av dybden til berggrunnen. Økende dybder gir større setninger i det lange løp. Vanskeligheten med 2D-aksesymmetrisk modellering og ukjente grenseforhold gjør imidlertid kartleggingen av dette flerdimensjonale drenerings- og påvirkningsområdet vanskelig og resulterende figurer ikke-generelle. Dybde til berggrunn, infiltrasjon samt utstrekning eller mangel på et morene/permeabelt lag over berg bestemmer influensområdet.

Resultatene viser at de generelle leirene i Oslo kan gi betydelige setninger dersom de er normalt konsolidert og tilstedeværelsen av dypere bløte leirlag over berggrunnen forsterker dette. En myk leire eller leire med lavere permeabilitet gjør påvirket område mer avhengig av ytre grenser/infiltrasjon i morenen. En svakhetssone / forsenkning med bunnen fylt med morene øker risikoen for differensialsetninger og dermed skader på bygninger og deres overganger.

En stor 3D-modell der jordlagdelingen (spesielt morene) og berggrunns topografi kartlegges bør gjøres fra sak til sak, da jordlag, grensebetingelser og dybde til berggrunn varierer for mye til å bruke en generell modell for mer enn én dimensjonal drenering. Bruk av det produserte 1D analytiske løsningsverktøyet for målte poretrykksreduksjoner i permeabel jord over berggrunn anbefales for raske vurderinger av endimensjonal drenering i leire. Tilstedeværelsen av et drenerende permeabelt lag som morene anses nødvendig for endimensjonal drenering og dens utelatelse kan gi langsommere, mindre setninger og påvirkningsområde.

Contents

Abbreviations

BC Boundary condition

CPTU Cone penetration test with pore pressure measurement

DTH Down the Hole

GSHPs Ground Sourced Heat Pumps

GWL Ground water line/level

NGU Geological Survey of Norway

NVE The Norwegian Water Resources and Energy Directorate

OCR Overconsolidation ratio

POP Pre overburden pressure

SVV The Norwegian Public Roads Administration

1D One dimensional

2D Two dimensional

3D Three dimensional

List of Figures

List of Tables

Chapter 1

Introduction

Norway and the rest of the world are experiencing a growing demand for ground sourced heat pumps (GSHPs) for cooling and heating purposes. GSHPs are referred to as energy wells throughout this thesis, as it fits the Norwegian term better. Energy wells, in addition to saving electrical powerconsumption, have the added value of being beneficial for mitigating climate change by potentially cutting greenhouse emissions. Zhang et al. [2014] shows that GSHP systems can be utilized in a large scale to meet the cooling and heating requirements in urban areas. It can additionally be placed in the immediate vicinity of the user, saving other infrastructure costs and space. In Norway the depth to bedrock is usually shallow, allowing for greater effect and less expenses. In total almost 50 000 energy wells have been installed in Norway, with approximately 3000 wells installations every year for the last decade, see Figure 1.1. Further increase in the amount of energy well installations is expected due to the increasing prices for electricity, even in Norway which has enjoyed low electricity bills historically.

Ramstad [2011] shows that the theoretical demand for all heating in Norway could be met by energy wells and thus cutting the heating demand up-to 90 %. Norsk Varmepumpeforening [2022] puts the usual heat saving to 60-80 %. A relative new phenomenon are bigger well-parks and these are increasing in number, as bigger parks have even more cost advantages. However, when built in urban areas with soft clays there is a need to properly assess the long and short term effects on ground settlements. In recent years the amount of court cases regarding ground settlements after the construction of energy wells in neighbouring buildings, could be an indication that there will be an increase in cases in the future. Investigation of the severity of the settlements, what went wrong and the mitigation measures available, are vitally necessary to ensure that the same mistakes are not made again and the public trust remains unbroken. Such knowledge is very valuable to help further develop drilling and legal regulations that might have become outdated.

This master thesis aims to numerically model hypothetical scenarios of energy well installation to find settlements. Possible causes due to drilling, installation and operation are from the case

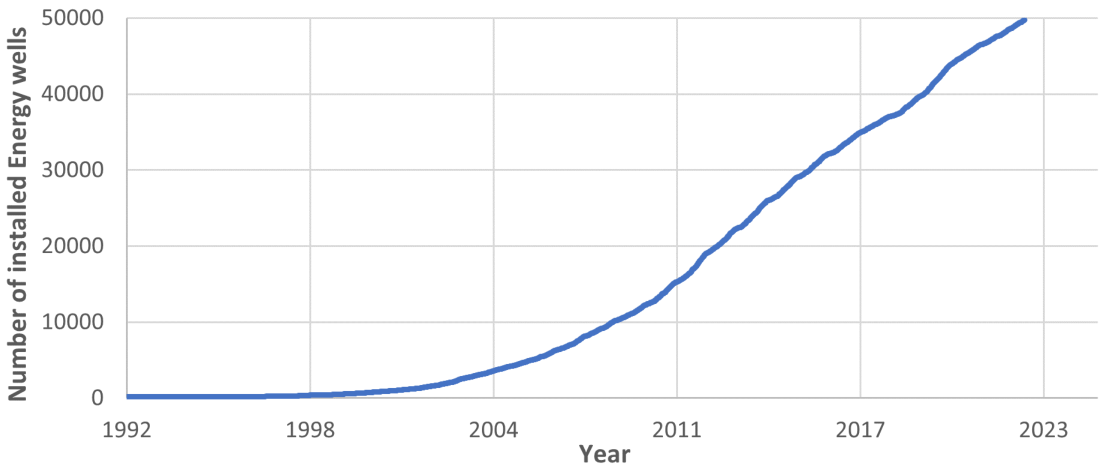


Figure 1.1: The graph shows around 3000 energy wells installed per year in Norway for the last decade, based on data from the national groundwater database [GRANADA, 2021].

and literature study in Våpenstad [2021] presented in Appendix B. The possible causes were found to be overboring, disruption of clay and due to pore-pressure reduction. Down the hole (DTH) air pressure drilling is ill advised in moraine or silt layers above bedrock and sensitive clays (e.g., [Lande et al., 2020], [Lande et al., in press] and [Baardvik et al., 2016]). Possible pore pressure reduction can be due to waterflow on the outside or inside of the casing, through a bad plug in the soil-bedrock transition or fractures caused by drilling increasing the seepage to bedrock.

1.1 Background

With more and more energy wells, bigger well parks and underground thermal energy storage systems installed, the risks related to ground settlements are increasing and each ongoing court case should be assessed critically. These recent court cases' outcome and final rulings could shake the public opinion of energy well safety and the technology's worth in a negative way, leading to losses both economically and environmentally. Not only for the energy well business and industry, but also for society as a whole. The Geological Survey of Norway (NGU) requires coordinates for all well drilling to be documented [Olje og Energidepartementet, 1996] and Oslo Municipality in particular, is experiencing more problems and public attention. This has resulted to a requirement from June 2020 that a geotechnical assessment must be made before two or more energy wells are drilled and installed. These geotechnical assessment tests and reports can quickly become expensive. This will increase the costs, cutting into the profitability [Iglebæk, 2021] and therefore a proper investigation into the matter is warranted.

One damage event that occurred in Oslo is due in court in August 2022 (T. L. Kibsgaard, personal communication, December 13, 2021) and another in Sandefjord finished recently in October 2021 [Vestfold Tingrett, 2021], although this ruling has been appealed (G. S. Solli, personal communication, December 2, 2021 ; A. Dagestad, personal communication, December 9, 2021). Both of these damage events and another third case settled outside of court are covered in Appendix B.2. In Oslo march 2017 there were almost 4000 registered energywells, of which the total worth can be estimated to be 260 million Norwegian Kroner (NOK) [Cerdeira et al., 2019]. In Singsaker, Trondheim in May 2020 a high pressure artesian layer was punctured as a energy well was being drilled, requiring a relief well and more grouting. The grouting was done directly into the waterborne layer on the outside of the pipe-casing and solved the issue.

A settlement map is planned for Oslo, and such a map together with the action map for tunnel planning by the Norwegian Geological survey (NGU) Baranwal et al. [2016] can give a better risk assessment for settlements due to ground works. A settlement analysis due to energy well installation or any bored ground works effect on the pore pressure was proposed to further back up these assessments.

1.1.1 Previous work

No previous work is done regarding pore pressure loss due to energy wells. The Gundersen and Haugen [2021] report is regarding total sounding geotechnical investigation boreholes, but can be applied for ungrouted energy wells and bored piles too. In this report several boreholes had to be properly sealed or resealed with grouting, as they had previously been leaking, producing surface water and reducing pore pressure underground. This will lead to settlements as the effective stress increases. The aforementioned report was a consequence to the settlements around Grønli, Fredrikstad. In an earlier report regarding Grønli it was found that leaking geotechnical survey boreholes was the cause. The risk of salt "washed" out of the clay, producing quick clay, was also noted [Gundersen, 2018].

The risks of pore pressure drawdown is known (e.g., [Holmøy et al., 2019], [Sundell, 2019], [Hauser, 2020] and [de Beer, 2010]) and is most extreme for the Bjørvika and Breivikeidet bridge [Gundersen, 2019] cases. The latter case was extreme due to the washing out of mass (massloss) and artisan conditions, stopping the construction. Settlements due to pore pressure reduction has been done for tunnel-leakages in Karlsrud et al. [2003]. Literature regarding pore pressure reduction due to bored ground works are presented more in detail later.

Veslegard and Simonsen [2014] tested piles and tie back anchors in building pits and for different drill techniques. Some paragraphs however, are devoted to energy wells in particular. Energy wells are lower in diameter in general than piles, but the same issues are there in principle for drilled piles and tieback anchors. Further testing was done on drilled piles and tie-back anchors in Lande et al. [in press] and Lande et al. [2020].

Sagmoen [2017] did PLAXIS 2D simulations of unwanted drilling effects. It is shown Majuri [2018] that the most common problems for Energy wells in Finland are collapsed boreholes and artesian layers or high water yield. The data was collected by questionnaire study among Finnish GSHP practitioners. A questionnaire study in Norway Syljuåsen [2020] has also shown that the most common problems are the same as in Finland. Sinkholes caused by energy wells are not common in Norway. The only documented case is Papirberedden in Drammen, and was due to chiefly two reasons: A bad/leaking plug in the soil to rock transition and a too high re-injection pressure in the open loop system [Vik, 2018].

1.2 Objectives

This master aims to provide charts for risk assessment of pore pressure losses above bedrock for energy wells in general Oslo clays, such that drillers and entrepreneurs avoid damage to buildings due to ground settlements. It it hoped to help in geotechnical assessments for general groundworks in soft soils and for the assessments that are required in Oslo Municipality for 2 or more energy wells installations. The problem is divided into the following objectives:

1. Gather field data from bored ground works' effects on ground settlements
2. Build the analytical solution for settlements due to pore pressure reduction
3. Build numerical models for settlements due to pore pressure reduction
4. Make charts and show correlations and factors/parameter effects

1.2.1 Limitations

The three cases in the case study in Våpenstad [2021] are current court cases or have been recently settled out of court, therefore most technical reports regarding these cases are not publicly available. Thus this work cannot be used for these cases. A broader and general approach was therefore decided, but there are problems with assuming general boundary conditions. The infiltration rate, ground water level (GWL), soil strength and homogeneous, isotropic hydraulic properties and bedrock topography can vary from the assumptions and simplifications made here. A linear pore pressure profile and no creep is assumed. The pore pressure reduction in the calculations are set to be permanent vs. time, and not variable. Building pits and tunnels are not a focus and will likely have higher settlement risks due to bigger hydraulic gradients if below GWL. The effect of quick clay present or drill technique chosen are simplified out in the models, but can exacerbate the pore pressure change and settlements due to overboring and disruption of clay. A proper geotechnical study with pizeometer installations, odeometer testing, etc. should be done for precise settlement calculation on a case by case basis.

Not many tests have been conducted and the tests that are the most similar, are for drilled piles and tie-back anchors. The lack of data to verify a model is present and unfortunate, especially long term settlement measurements. More testing or data gathering and backcalculation is recommended for drilled piles and energy wells in urban areas to better verify the models.

1.2.2 Delimitation of the Problem and Approach

Since the topic is related to Norway, closed loop energy wells are the main focus due to the overwhelming prevalence of closed loop over open loop, and that the recent court cases are regarding closed loop systems. While more data for the case study in Appendix B was attempted to be gathered, it was mostly unsuccessful, due to pending court cases. While originally only about energy wells, bored piles and tieback-anchors with similar diameter can pose the same problems and thus will be included in this work. This broader scope will yield more data, but ultimately supporting the conclusion in Appendix B.

Case 1 in Appendix B.2 show signs of a water table drawdown or at least a pore pressure reduction at bedrock. Case 2 and case 3 has little available data but shows settlements in InSAR after energy well installation. Case 3 is more long term and slower and more circular in area of effect. As the settlements were long term and for 2 / 3 cases not in a circular fashion, pore pressure drawdown long term is the cause of ground settlement investigated. A 2D axysymmetrical PLAXIS model is to be made in order to map the area of effect and show the risks for different parameters/factors at play, and to compare to the 1D models made. Mitigation measures to look at are welded lid, sealment/grouting of fractures/bedrock should stop seepage downwards while grouting on the outside of the casing will stop artisan water, seepage upwards.

The lack of data gives the work a general and broad starting point, with the limitations that entails as discussed above. A hypothetical scenario modelling is envisioned to give results regarding which factors are important. These might be known for experienced hydrogeologists and geotechnical engineers, but the hope is that the drillers and entrepreneurs will have use for these graphs. Freeze -thaw is also not included because of the long time frame as well as being a known problem that the industry is familiar with. They put the wells sufficiently apart to prevent this. Overboring is variable and most notable in silty clay /moraine with artesian conditions. Drill techniques and energy wells are presented in Våpenstad [2021] and Begrens Skade projects (e.g., [Veslegard & Simonsen, 2014]) and will not be elaborated further in this work, but are presented by Chapter 2, 4, 6 and 7 from Våpenstad [2021] in Appendix B. Building loads are simplified out to avoid the danger of over or underestimating the settlements [Sundell, 2019].

1.3 Structure of the Thesis

The rest of the thesis is structured as follows:

- Chapter 2 accounts for the relevant theory
- Chapter 3 accounts for literature regarding bored ground works
- Chapter 4 presents the models
- Chapter 5 presents the 1D results
- Chapter 6 presents the 2D axisymmetrical results
- Chapter 7 discusses the results and compares to available data from case studies
- Chapter 8 presents the conclusions and proposals for further work

Chapter 2

Theory

Parts of this chapter are borrowed and improved upon from chapter 3 in Våpenstad [2021].

2.1 Settlement Formulas

Overboring will result in volume loss ΔV, and disturbance of soils can lower material strength and stiffness E, resulting in settlements. An undrained soil has no volume change due to excess pore pressure p_w, while "drained" does. Commonly E ≠ E', where ' signifies excess pore pressure drained (Karlsrud and Hernandez-Martinez [2013] and Nordal [2020]). As the calculations are long term, drained is the soil condition focused on.

$$\Delta\sigma = \Delta\epsilon E \tag{2.1}$$

Equation 2.1 is referred to as Hooks law. E is the Youngs modulus, and can be calculated by equations using E_{oed} found by an odeometer test. E_{oed} is also referred to as M, $M = \frac{\Delta\sigma^{'}}{\Delta\epsilon}$. Stress is $\sigma = \frac{N}{A}$, where N is the force perpendicular on the surface-area A.
Simplified into 1D the strain $\epsilon = \frac{\Delta V}{V} \rightarrow \epsilon = \frac{\Delta h}{H}$. H is the thickness or height of the soil initially and Δh is the change in the height of the soil, $H - \delta = \Delta h$.

$$\Delta\sigma^{'} = \Delta\sigma - \Delta u \tag{2.2}$$

$\Delta\sigma^{'}$ and $\Delta\epsilon$ are change in effective stress and strain respectively. Δu is change in hydrostatic pore pressure and $\Delta\sigma$ is the change in total stress. σ is calculated by $\gamma_{soil} \times z$, where z is the height or thickness of the soil and γ_{soil} is the unit weight of the soil, often set to 20 $\frac{kN}{m^3}$ for clay. Pore pressure u, is calculated in the same way but with $\gamma_{water} \times z_{gwl}$, where z_{gwl} is the height of the groundwater level (GWL). γ_{water} is ≈ 10 $\frac{kPa}{m^3}$. While the hydrostatic pore pressure is not always equal to pore pressure between the grains, due to for instance capillary suction forces, this is not further discussed due to simplicity. See Figure 2.1 for an example for settlements caused by reducing the pore pressure. The resulting volume loss caused by pore pressure reduction is due to the reduction of pore volume in the soil. If fully saturated, it is just the expulsion of water (as opposed to both air and water) allowing for the consolidation ΔV, the time of which is a function of the porosity and permeability. The settlements are highly dependent on the OCR (over-consolidation ratio) and type of soil [Janbu, 1970]. An overconsolidation ratio $OCR = \frac{\sigma'_p}{\sigma'_0}$ and pre-overburden pressure POP=$\sigma'_p - \sigma'_0$ profile is shown in Figure 2.2. A normal consolidated (NC) soil is situated to σ' currently roughly to the same degree as it was historically previously $\sigma'_0 \approx \sigma'_p$, giving an $OCR \approx 1$. An over-consolidated (OC) soil is consolidated to a higher effective stress σ'_p than it is currently in situ experiencing σ', resulting in a $OCR > 1$. The σ' could have been higher due to glacial loading under the last ice age, lower groundwater level or to overburdening soils that have since been eroded or excavated away. The over-consolidated (OC) and pre-consolidated terms are often used interchangeably. Soils can also be OC due to consolidation not directly from compaction of the soils, but rather by solidifying processes that bind the grains tighter. A medium can also be under-consolidated, like in a new filling, but this is a short term situation. Depending on the M_{oc}, the risk of settlements can be much greater when the effective stress exceeds the preconsolidation pressure, causing a higher consolidation [Bjerrum, 1967].

The total settlements or deformation $\delta = \Delta h$ is calculated by equations from Janbu [1970] for a 1 dimensional drainage:

$$\delta = \int_0^H \epsilon \tag{2.3}$$

Where H is the height or thickness of the layer that is subjected to $\Delta\sigma'$.Janbu [1970] uses residual strain and not the hydraulic gradient i alone to drain a saturated clay with excess pore pressure during consolidation. Janbu [1970] equation for settlements:

$$\epsilon = \frac{1}{m}\int_{\sigma'0}^{\sigma'} \left(\frac{\sigma'}{\sigma_a}\right)^{a-1} d\frac{\sigma'}{\sigma_a} = \frac{1}{ma}\left[\left(\frac{\sigma'}{\sigma'_a}\right)^a - \left(\frac{\sigma'_0}{\sigma'_a}\right)^a\right] \tag{2.4}$$

Where σ_a is the constant reference pressure 1 bar, 100 kPa. a is the exponent of stress and m is the material modulus number, the latter two are functions of porosity Figure 2.3.

- EE, equivalent elastic model a = 1 appropriate for rock or overconsolidated clay
- EP, equivalent elasto plastic model a = 0.5 appropriate for sand and silt
- PL, equivalent plastic model a = 0 appropriate for NC clay and slit
- ES, extra sensitive model a<0

Resulting in for OC clay, M=$M_{oc} = m\sigma'_a$ =constant

$$\epsilon_p = \frac{\Delta\sigma'}{M} \tag{2.5}$$

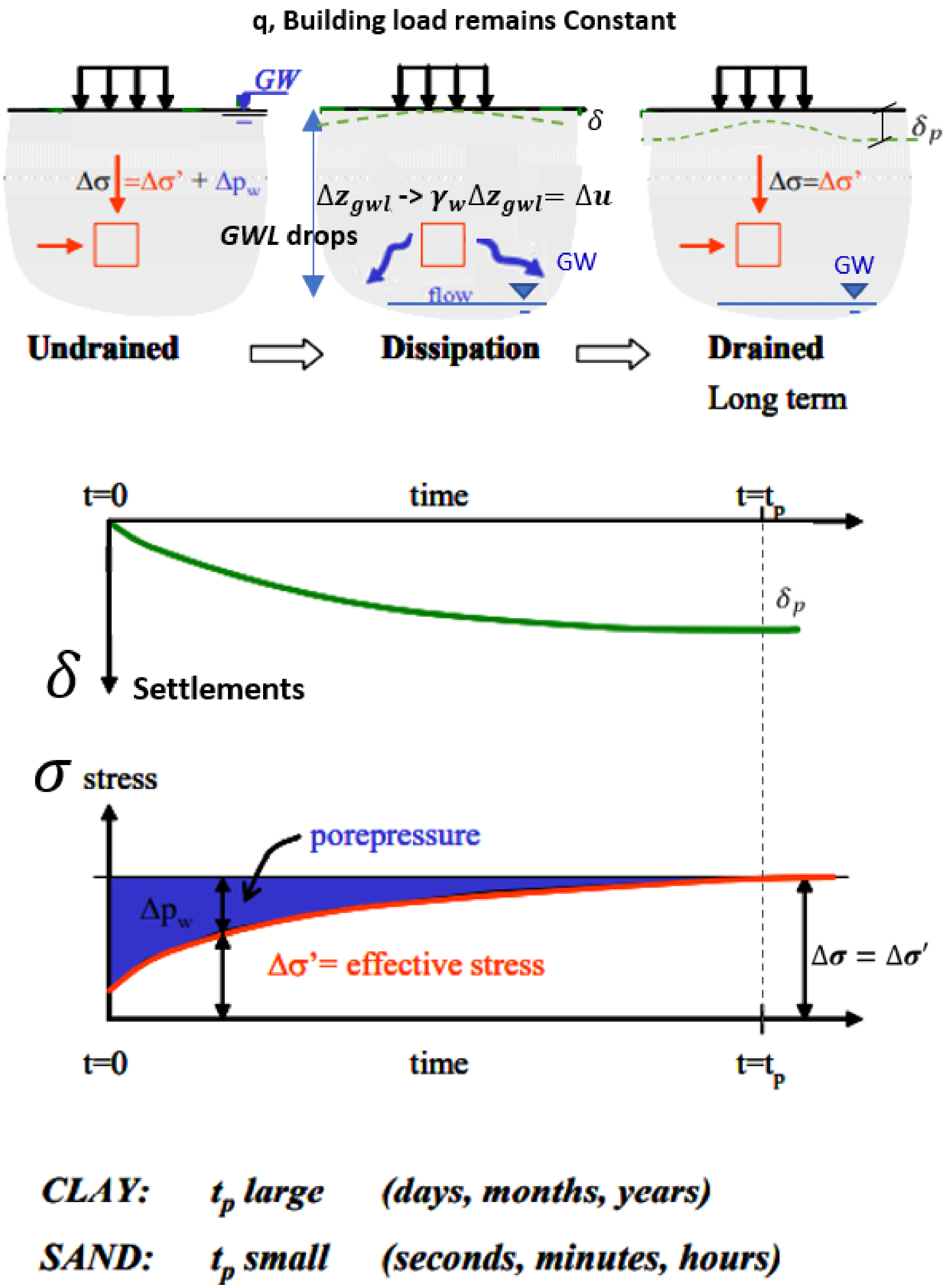


Figure 2.1: Transition from undrained to drained condition due to pore pressure dissipation. If the building load is ≈ 0, the settlements are horizontal in the top profile. Modified from Nordal [2020].

and for NC clay, $M = m\sigma^{'}$

$$\epsilon_p = \frac{1}{m} ln(\frac{\sigma^{'}_0 + \Delta\sigma^{'}}{\sigma^{'}_0}) \tag{2.6}$$

and for sand, $M = m\sqrt{\sigma^{'}\sigma_a}$

$$\epsilon_p = \frac{2}{m}(\sqrt{\frac{\sigma^{'}}{\sigma_a}} - \sqrt{\frac{\sigma^{'}_0}{\sigma_a}}) \tag{2.7}$$

For stresses going from an OC stress area and into a NC area e.g., a clay with $\sigma^{'}_0 < \sigma^{'}_p < \sigma^{'}$, Equation 2.8 is applied. For a pore pressure loss this might only apply for just the top part of a lightly OC clay, as the $\sigma^{'}_p$ increases with depth for OCR, or the bottom part of the clay if POP is used.

$$\epsilon = \epsilon_{OC} + \epsilon_{NC} = \frac{\sigma^{'}_p - \sigma^{'}_0}{M} + \frac{1}{m} * ln(\frac{\sigma^{'}}{\sigma^{'}_p}) \tag{2.8}$$

An odeometer test is required to find Moc, cv, E_{oedo} and m. An example is shown in Figure 2.4 where q stays constant, but the groundwater level drops. The resulting ϵ- H plots can have triangle, block, and other shapes shown in Figure B.7. Using the corresponding janbucurve(s) B.7, Equation 2.9, Equation 2.11 and Equation 2.10 settlements before T_p can be calculated. The time of primary consolidation T_p is when $\Delta\sigma^{'} \approx \Delta\sigma$, or in other words u ≈ 0. This can take some time, depending on the watercontent w, at start and hydraulic conductivity K, see Figure 2.5. $w = \frac{m_w}{m_s}$, where m_w is the mass of water and m_s is the mass of dry soil. $\epsilon_r = \epsilon_p - \epsilon$ where ϵ_r is the residual strain and p is primary ($t = T_p$) and ϵ is at time t. After t primary secondary settlements like creep exists.

Consolidation time:

$$t = T\frac{H^2}{c_v} \tag{2.9}$$

Where t is the time to consolidation, T is a time factor ≥ 0, with T = 1 when $t = T_p$. If two way drainage, e.g., a clay with permeable draining (Δu) layers on top and under, the height H is halved.

The coefficient of consolidation:

$$c_v = \frac{Mk}{\gamma_w} \tag{2.10}$$

where $M = \sigma^{'}_{avg} \times m$, $\sigma^{'}_{avg}$ = (average $\sigma^{'}_0$−average $\sigma^{'}$)/2 and k is the soil permeability.
Degree of consolidation

$$U = \frac{\delta}{\delta_p} \tag{2.11}$$

Where δ is a function of time and δ_p is the settlement at primary T_p and U_p. Factors of importance for time to T_p are grain size, porosity and water content w. Clay requires the longest time to go from undrained to drained of the soils, see Figure 2.5.

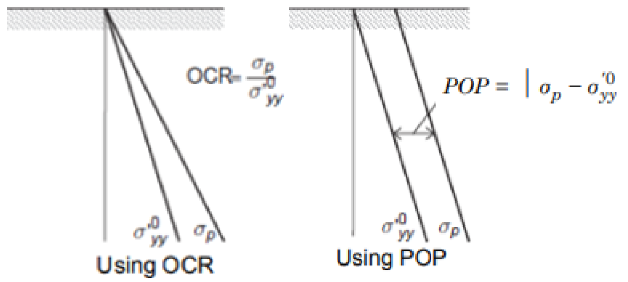


Figure 2.2: Example of an over-consolidation ratio and pre-overburden pressure soil profile. Modified from Bentley [2021a]

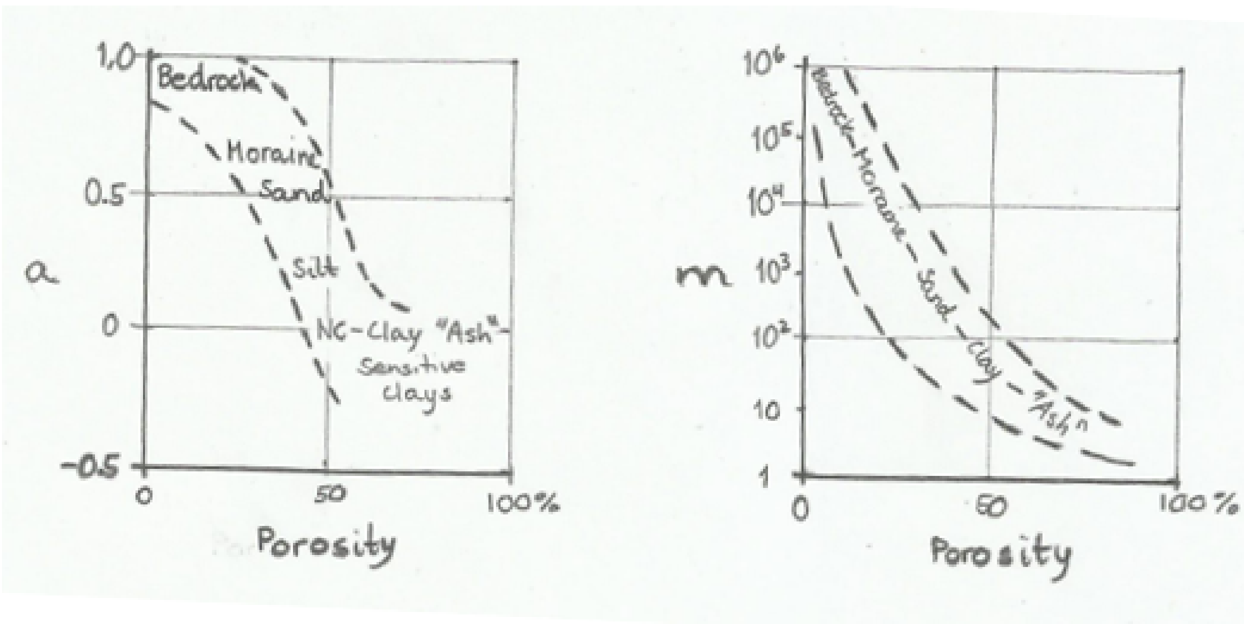


Figure 2.3: Porosity's effect on m and a. Based on Janbu [1970].

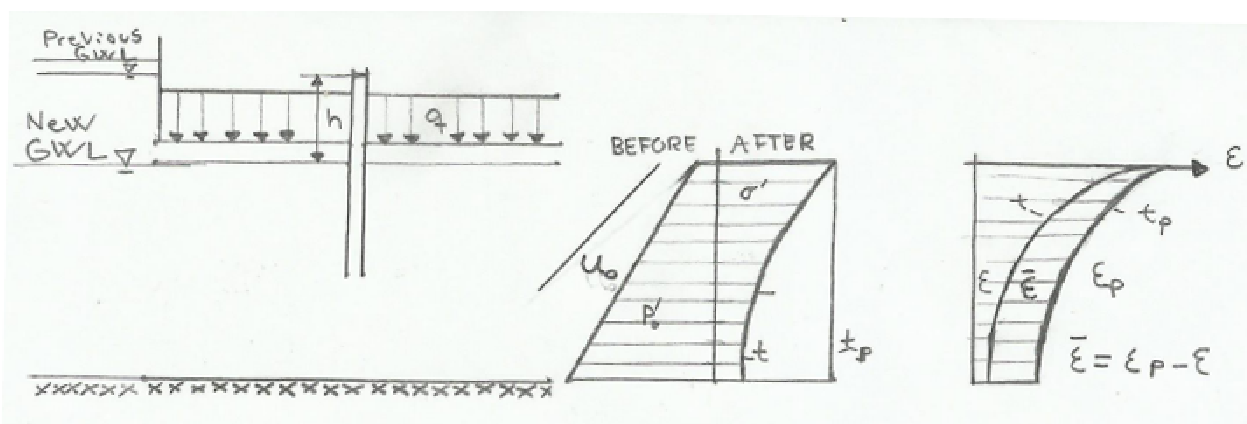


Figure 2.4: Groundwater lowering with loss of pore pressure h under the building and the resulting $\sigma' - H$ and ϵ - H plots. Modified from Janbu [1970].

2.2 Characteristic values

For Norwegian soils the unit weight γ, is in the area of $17-21 kN/m^3$ [Emdal, 2018]. The tables below show characteristic values from experience, but not absolute values and limits for soils. Table 2.1 shows typical material modulus m for NC clays, while Table 2.2 shows typical deformation modulus M_{OC}. Table 2.3 shows characteristic k, permeability ranges. Table 2.4 shows the 1D drainage coefficient and Table 2.5 shows what classifies a soft clay by shear strength value.

Table 2.1: Characteristic values of the material modulus for NC clay, based on Emdal [2018]

	soft	medium	Stiff
m	10	10-20	> 20

Table 2.2: Characteristic 1D odeometer/deformation stiffness of soils, based on Tomlinson [1995]

Clays:	Compressibility	Deformation module $M = E_{odeometer}$ [MPa]
Soft	Very high	< 0.7
Medium/soft	High	0.7- 3
Medium	Medium	3-10
Medium/Stiff	Low	10-20
Stiff	Very low	> 20

Table 2.3: Characteristic permeability ranges of soils, based on Emdal [2018]

Soils:	Gravel	Sand	Silt	Moraine	Clay
k [m/s]	$> 10^{-2}$	$10^{-2} - 10^{-5}$	$10^{-5} - 10^{-8}$	$10^{-6} - 10^{-9}$	$10^{-8} - 10^{-11}$
k [m/day](×8.64)	$> 10^{2}$	$10^{2} - 10^{-1}$	$10^{-1} - 10^{-4}$	$10^{-2} - 10^{-5}$	$10^{-4} - 10^{-7}$

Table 2.4: Characteristic 1D consolidation values of soils, based on Emdal [2018]

Soils:	soft clay	Medium clay	Stiff/OC clay	Silt	Sand
Cv [m^2/year]	< 5	5-15	15-25/-50	up-to 100	> 200

Table 2.5: Characteristic/classification of undrained shear strength S_u for clay, based on SVV [2005] and Nordal [2020]

S_u [kPa]	Clay	Shear strength
< 25	Soft	Low
25-50	Medium	Medium
>50	Stiff	High

2.3 Hydraulic conductivity

$$K = \frac{V}{i} \tag{2.12}$$

Hydraulic conductivity, denoted K is a value with units (m/s) and it denotes how fast a liquid can travel based on both the liquid's and medium's properties perpendicular to to the waterflow when $i = 1$, per unit area [Brattli, 2018a]. Darcy's law $V = \frac{Q}{A} = K * i$, where V is how fast the substance can flow through the pores with units (m/s) and i is the hydraulic gradient. A is the area perpendicular to the waterflow. For a soil an effective area $A_{eff} = A * n_{eff}$ to account for the loss of area (where the grains are), where n_{eff} is the continuous pore volume in a soil as a ratio between 1 and 0. It may be also noteworthy to remark that the water in a soil after GWL lowering may not fully drain due to field capacity. Field capacity denotes the degree or volume of "closed" pores, surface stress and molecular forces is dependent on compaction-level and grain-size, where finer grains increase the field capacity ([Freeze & Cherry, 1979] and [Brattli, 2018a]).

Permeability k, is a value to denote how easily or difficult a liquid can go through a medium with units (m/s). For example k is not affected by temperature, while K is. The permeability is dependent on the grain size and porosity. The porosity, $n = \frac{V_{pore}}{V_{total}}$ denotes how much porevolume there is with air or water/liquid in a medium, while the void ratio $e = \frac{V_{pore}}{V_{solids}}$. V_{pore} is the volume of the pores and V_{solids} is the volume of the soilmass. Consequently, the void ratio e denotes the relation between the free space between loose particles, while porosity n denotes the void directly at the surface of a soil. $e = n/(1-n)$. e is directly affected by compaction-level. A soil can have a constant or changing permeability or hydraulic conductivity be depth as shown in Figure 2.6 or time resulting in different steady state pore pressure profiles. Peat has high settlement risk and anisotropic hydraulic properties, but since the constructions in the case studies are on marine clay this is not considered. Different grain sized soils have varying k, with sand and gravel as more permeable than clay and silt [Freeze & Cherry, 1979]. Silt and especially clay is very little permeable, see Figure 2.5. The following terms should also be noted:

1. Aquifer. An aquifer is a geological formation made up of sufficient permeable material, which allows storage of water and at the same time will allow for movement of water through it under ordinary conditions. For example: sand, moraine or gravel.
2. An Aquiclude can be defined as a formation of relatively impermeable material which while providing storage of water, but is not easily capable of transmitting freely water through it. For instance, the clay in Figure 2.9.
3. An Aquitard is a geological formation of poor permeability, although seepage is possible. Sandy clay is an example.
4. An Aquifuge can be defined as a geological formation of impermeable material that neither contains nor transmits water through it. For example solid bedrock [Brattli, 2018a].

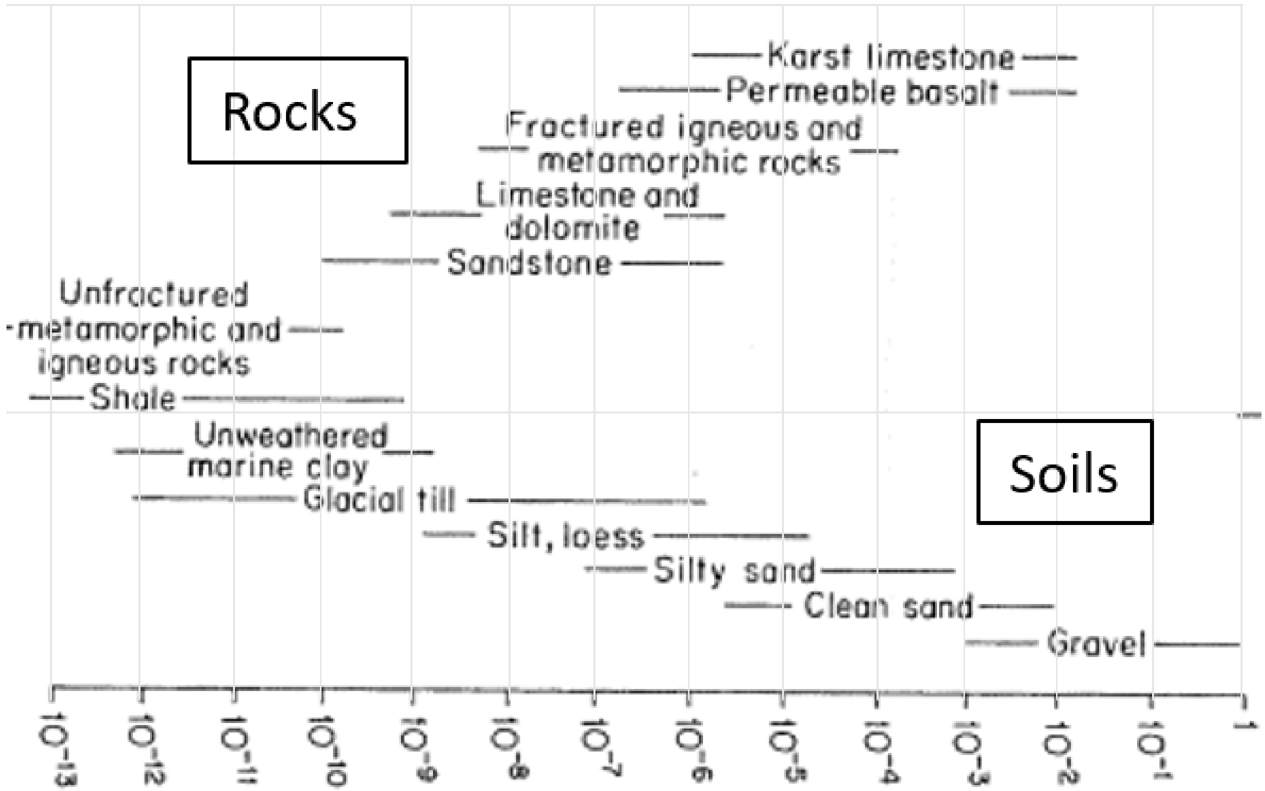


Figure 2.5: Hydraulic conductivity K (m/s) along the x-axis. Modified from Freeze and Cherry [1979]

Transmissivity $T = m * k$, where m is the thickness of the medium, is a denotation of horizontal waterflow through the entire aquifer. The Storage Coefficient of an aquifer is defined as the water volume discharged from a unit prism (a vertical column standing on a unit area $1m^2$ as the water level (open aquifer) or hydraulic head (pizeometer level) in a confined aquifer (aquifer with artesian conditions) decline by one unit depth 1m [Freeze & Cherry, 1979]. For a closed aquifer the compression of the aquifer and expansion of the water due to the loss in pore pressure (increasing effective stress $\sigma^{'}$) results in:

$$S_s = \gamma_w(\alpha + \beta n) \tag{2.13}$$

Where α is the compressiblity of the aquifer, β is the compressibility of water and n the porosity. $\alpha = 1/E_s$ where E_s is the bulk modulus of the aquifer, itself a function of grain size, grain-geometry and level of compression/compaction. $\beta = 1/K_w$, where K_w is the bulk modulus of water. The S_s is very small, around $3E-5m^{-1}$ or less.

$$S = mS_s \tag{2.14}$$

Where the water is produced from the compression of the whole aquifers thickness m, with S being unitless in the magnitude of 5E-3 to 5E-5. For an open unconfined aquifer the storage coefficient S, is equal to the specific yield S_y. Since S_y is magnitudes higher than S_s the latter term can be dropped. Thus the specific yield is a unit for the effective porosity n_{eff}.

$$S = S_y + mS_s \approx S_y = n_{eff} \tag{2.15}$$

However while generally $0.01 < S_y < 0.3$, for very fine grained soils the S_y will become very small and approximate to mS_s in size [Brattli, 2018a]). The transmissivity and storage coefficients can be specified for aquitards as well as aquifers, but in aquitards (e.g., clay) the vertical hydraulic conductivity is more significant than its transsmisivity. The $\alpha >> \beta$ in clay aquitards, thus the $n\beta$ in Equation 2.14 and Equation 2.13 is negligible [Freeze & Cherry, 1979]. The Storage coefficients can be used to calculate the total water volume, and Q water flow when the hydraulic head/GWL fluctuates or is changed by ground works.

$$V_w = SA \times \Delta m (m^3) \tag{2.16}$$

Where V_w is the drained water volume, A drained water area, S the storage coefficient and Δm is the average decline of the GWL or hydraulic head. The difference in Storage coefficient for open or enclosed aquifer results in that a equal Q or V_w extraction will give small changes in the hydraulic potential and "funneling" of the GWL, relative to a closed aquifer. Low S values give bigger GWL/hydraulic head lowering while high S_y values results in slow groundwater flow/response for an aquifer [Brattli, 2018a].

One fracture in rock has Equation 2.12 properties, but assuming parallel plates, stable and laminar (non-turbulent) flow the fracture has properties by Equation 2.17.

$$K_s = \frac{g * e^2}{12v} \tag{2.17}$$

Where e is the width of the fractures, assuming that the fracture is two plane surfaces with the same distance. g is the gravitational acceleration and v is the kinetic viscosity. $v = 1.3 * E{-}3 m/s^2$ for clean water at $10°C$. If the are multiple parallel fractures with width between fractures s, the hydraulic conductivity of rock with fractures becomes

$$K = \frac{e}{s} * K_s = \frac{g * e^3}{12v * s} \tag{2.18}$$

By Equation 2.18 it is evident that increasing the e by 10 times (E1), results in increasing the K by 3 magnitudes (E3). On a general basis, doubling the fracture opening e leads to an increase in waterflow by a factor of 8. This is however simplifying very complex fracture geometry, and a Lugeontest or pumping test is advised for precise calculation of K [Brattli, 2018a]. Further information about these tests and how to conduct them are in Brattli [2018a].

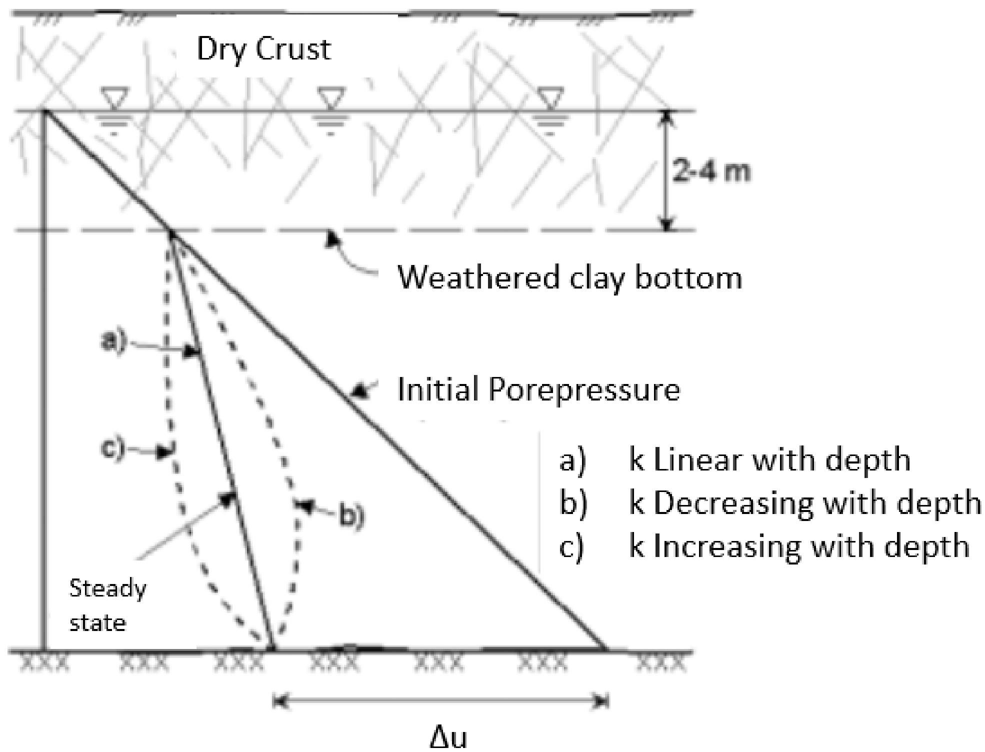


Figure 2.6: pore pressure profile due to pore pressure loss at bedrock. Modified from [Karlsrud et al., 2003]

2.4 Flow and Continuum Equations

Using the conservation of matter, mass in - mass out = Δ mass, to denote the relationship between two sections of a system can be done. Assuming no loss of water the continuity equation becomes:

$$Q_1 = Q_2 \longrightarrow A_1 v_2 = A_2 v_2 \tag{2.19}$$

Where Q is the amount of water per time unit, A the area and v is the velocity perpendicular to the area for points 1 and 2. An incompressible fluid (e.g., water) speeds up when it reaches a narrow constricted (i.e. funnel) section in order to maintain a constant volume flow rate, and this inverse relationship between the pressure and speed at a point in a fluid is named Bernoulli's principle. Equation 2.20 is referred to as Bernoulli's equation and is based on the conservation of energy, assuming friction forces are negligible.

$$P_1 + 1/2\rho v_1^2 + 1/2\rho g h_1 = 1/2\rho v_2^2 + 1/2\rho g h_2 + P_2 \tag{2.20}$$

Where P_1, P_2 are the pressure, ρ is the density of the fluid, g is the gravitational acceleration, v1,v2 the velocities and h1,h2 the heights of the fluid above a reference height at point 1 and point 2. With conservation of mass and energy the sum is constant, and using $\gamma_w = \rho_w \times g$ the

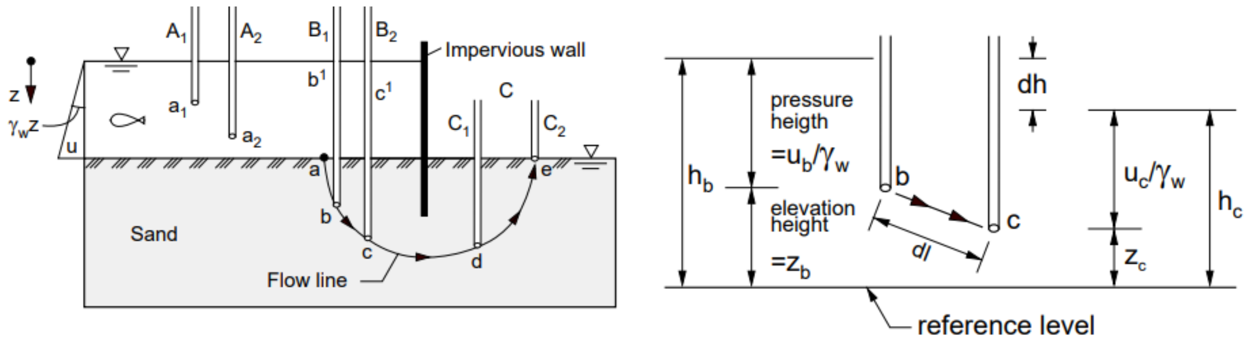


Figure 2.7: Waterflow due to hydraulic head difference. From [NTNU Geotechnical Division, 2017]

simplified equation becomes:

$$P + 1/2\rho v^2 + 1/2\gamma_w h = Constant \tag{2.21}$$

Hydraulic groundwater head, or the potential total energy in water is by conservation of energy:

$$h_t = h + \frac{u}{\gamma_w} + \frac{v^2}{2g} \tag{2.22}$$

Where u is the in situ pore pressure, h is height above refrence level and the latter part $\frac{v^2}{2g}$ being insignificant in laminar, stationary flow and can be simplified out. The hydraulic gradient i, in Equation 2.12 is:

$$i = \frac{\Delta h_t}{\Delta l} \tag{2.23}$$

Where Δl is the travel distance and Δh_t is the hydraulic head, or potential. The gradient is in other words the potential total energy per unit length [NTNU Geotechnical Division, 2017].

By Equation 2.23 the water flows where the head is the lowest, shown in Figure 2.7. In a 3D model putting the Darcy velocities V_x, V_y and V_z into the continuation equation of mass Equation 2.19, the 3 D transient and fully saturated waterflow becomes:

$$\frac{d}{dx}(K_x\frac{dh}{dx}) + \frac{d}{dx}(K_y\frac{dh}{dy}) + \frac{d}{dz}(K_z\frac{dh}{dz}) = S_s\frac{dh}{dt} \tag{2.24}$$

Where d / dx is per unit in the x direction, d / dy is per unit in the x direction, d / dz is per unit in the z direction, d / dt is per unit of time and h is the height of hydraulic potential (the pressure) and S_s is the specific storage-coefficient of the aquifer. If the aquifer/soil is further homogeneous and isotropic (i.e $K_x = K_y = K_z$ =constant) the Equation 2.24 becomes:

$$\frac{d^2h}{d_x^2} + \frac{d^2h}{d_y^2} + \frac{d^2h}{dz^2} = \frac{S_s}{K}\frac{dh}{d_t} \tag{2.25}$$

For stationary flow, steady state the potential heights are constant and the time is simplified out and Equation 2.25 becomes:

$$\frac{d^2h}{dx^2} + \frac{d^2h}{dy^2} + \frac{d^2h}{dz^2} = 0 \tag{2.26}$$

Solving Equation 2.26, called the Laplace equation, requires knowing the boundary conditions values and geometry, in other words the aquifers geometry, groundwater head and/or waterflow at the boundaries [Brattli, 2018a].

2.5 Pore Pressure Reduction

A loss in pore pressure can be due to by several reasons in ground works; leakage, drilling or the pumping out of water. pore pressure reduction can also be caused by hitting an artesian layer that previously was unable to escape due to a impermeable layer above and below (confined aquifer). This aquifer must have an incline with a higher area above, or another higher aquifer in hydraulic contact with it. See Figure 2.8 and Figure 2.9 for an example where water was escaping on the outside of the well casing due to artesian conditions. The shown event happened in 2020 in Singsaker, with an extensive quick-clay slide hazard. The water could have disturbed the quick clay or lowered the salinity, and thus strength. Artesian water can be found with a water chemical composition test to find if the water originates from deep enclosed aquifer or groundwater in an open aquifer. Do note that depending on the water outflow, either where escaping or at terrain, the water pressure might increase, leading to heave. As mentioned in section 2.1 hydrostatic pressure is calculated by: $u_{hydrostatic} = \gamma_w \times z_{water}$ [Brattli, 2018b]. A higher pore pressure than hydrostatic pore pressure means the aquifer has an overpressurized, artesian water pressure, and is likely confined, as opposed to open. An open aquifer has its top in contact with the atmospheric pressure.

In Norway the GWL is generally close to the terrain. Urban surfaces and thus less infiltration might lower this, but urban areas have leaking pipes etc. thus it may be erroneous to think that there is less water infiltration in an urban environment [Sundell, 2019]. With high infiltration expected, the GWL should not drop due to pore pressure reduction at bedrock, unless it is a low depth to bedrock (less than 5-8 m) or narrow "trenches/ravines" [Karlsrud et al., 2003]. The upper clay layer (2-4m) is generally weathered and therefore has a higher permeability relative to the deeper clay layers, meaning that a pore pressure reduction at bedrock often will not lower the pore pressure here [Karlsrud et al., 2003]. See Figure 2.6.

2.6 Foundations and Buildings

When a building is constructed, there are settlements induced by the new building load q, assuming that the new load is higher. $\gamma_{soil} \times z_{excavated} < Q$, where Q is the buildingload (Q= q × B) and B is the average width. A long building length is assumed to approximate the problem into 1D. This new load fits in Equation 2.2 by adding the load such that $\Delta\sigma' = \Delta\sigma + q - \Delta u$. Although q should be lessened with depth to account for the resistant horizontal forces as the load is dis-

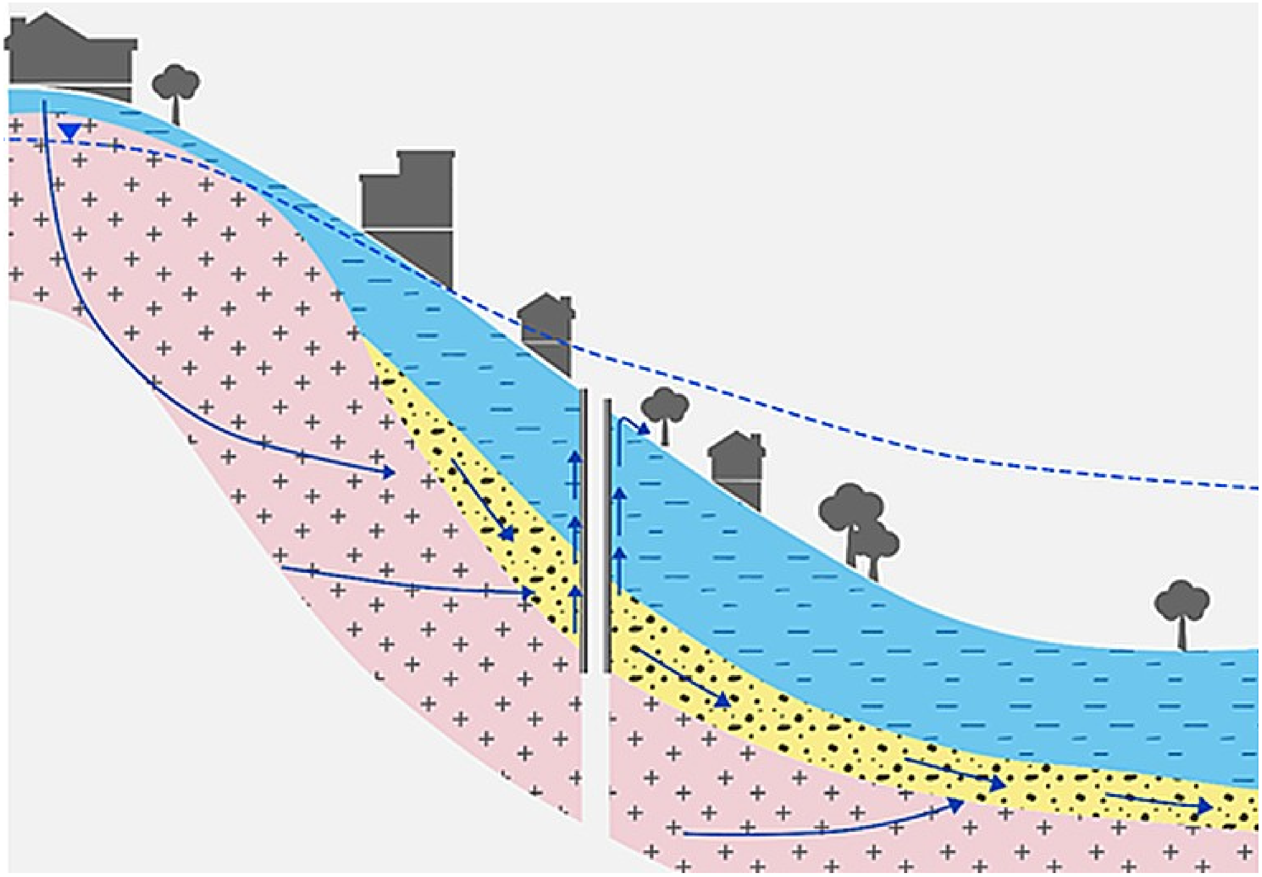


Figure 2.8: Artesian well due to higher hydraulic pressure height (blue dotted line) than terrain. Magnenta denotes bedrock, yellow gravel / sand and blue the marine clay. Taken from [de Beer & Dagestad, 2020].

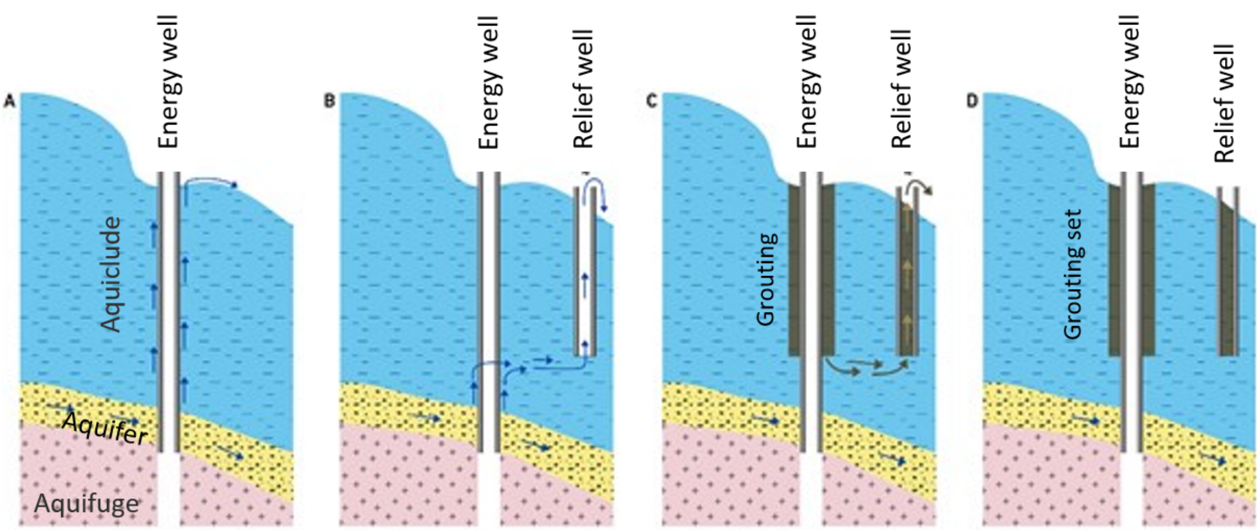


Figure 2.9: Sequence of events in the Singsaker case. A, shows drilling of the well with the resulting artesian water. B and C shows drilling of relief well and grouting. When the grouting is set as shown in D, the issue is resolved. Modifed from [de Beer & Dagestad, 2020]

tributed outward in an inverted triangle area when looking at it in 2D. 1:2 formula is shown in Equation 2.27:

$$q(z) = q_0(1 - \frac{z}{B_{avg}\pi}) \tag{2.27}$$

Where q_0 is the building load at terrain, B_{avg} is the average width of the foundation and z is the depth. This will then give a higher $\sigma^{'}$ and the formulas in section 2.1 to calculate the settlements can be used. In the case study in Appendix B the buildings have been there for a long time such that there are no to little settlements induced by the building load. In other words, way past T_p and assuming that in secondary consolidation the settlements are ≈ 0.

If the settlement under a foundation is universal then the building will likely not sustain damage, although the cables and piping around the area experiencing settlements might be affected. Building-transitions can give damage, i.e pavement - building or brick-buildings next to each other as case 3 in Appendix B.2.3. The difference in settlement between two points $\Delta\delta$ are called differential settlements. This results in foundation cracks and in the worst cases, foundational failure.

> "The differential settlement of the underlying soil beyond the bottom of the raft slab may result in a loss of contact between the soil and the raft slab, thus causing a reduction of the bearing capacity of the raft and potential damage due to flexure of the foundation. In principle, only a highly flexible foundation would conform to the [..] ground movements."
>
> [Zymnis & Whittle, 2021]

The Eurocode 7 (EC7) gives incremental settlements definitions, but for simplicity less than around L/300 differential settlements is a good rough estimate as a criteria of acceptable incremental settlements, where L is the distance between two points. Angular distortion is differential settlements divided by L shown in Equation 2.28, where L is the distance between the two points:

$$\beta = \frac{\Delta\delta}{L} \tag{2.28}$$

Ground settlement will manifest itself initially in buildings as cracks, and Karlsrud [2015] provides figures that indicate different cracks on buildings, the cause of such cracks and how it impacts constructions based on angular distortion. The potential for differential settlements can to varying levels of degree be calculated from the variations in depths to bedrock. This is due to the pore pressure reduction will often be relatively constant under a construction of limited extent, and give less impact on the differential settlements. See figure 4.12 Karlsrud et al. [2003] about freestanding houses damages related to differential settlements. Case 1 in Appendix B had a building with severe structural damage with the total cost for the occupant family of 10 mill NOK. Case 2 and 3 have facade damage, and no data regarding structural damage.

For tunnels, the relation related to settlements and damages show data from clay-filled deep trenches in the Oslo-area, that a pore pressure reduction of 10-30 kPa will generally give small

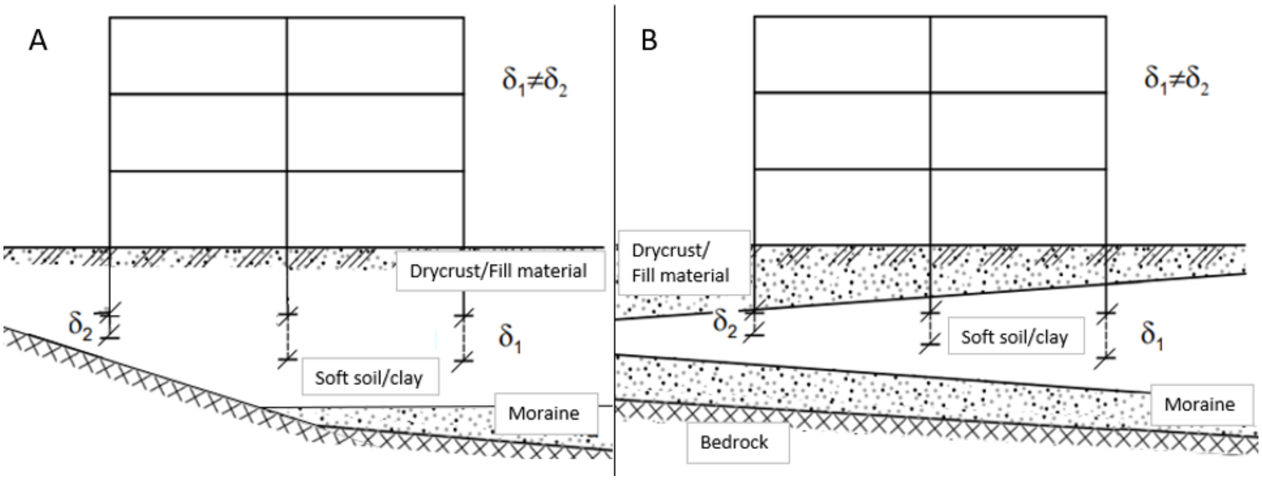


Figure 2.10: Soil layering and bedrock topography effects on differential settlements. Soil layering can vary from case to case with a soft clay layer amplifying the settlements and a deeper basement possibly so, depending on the water infiltration and building load compared to the excavated soil load. Modified from [NTNU Geotechnical Division, 2017]

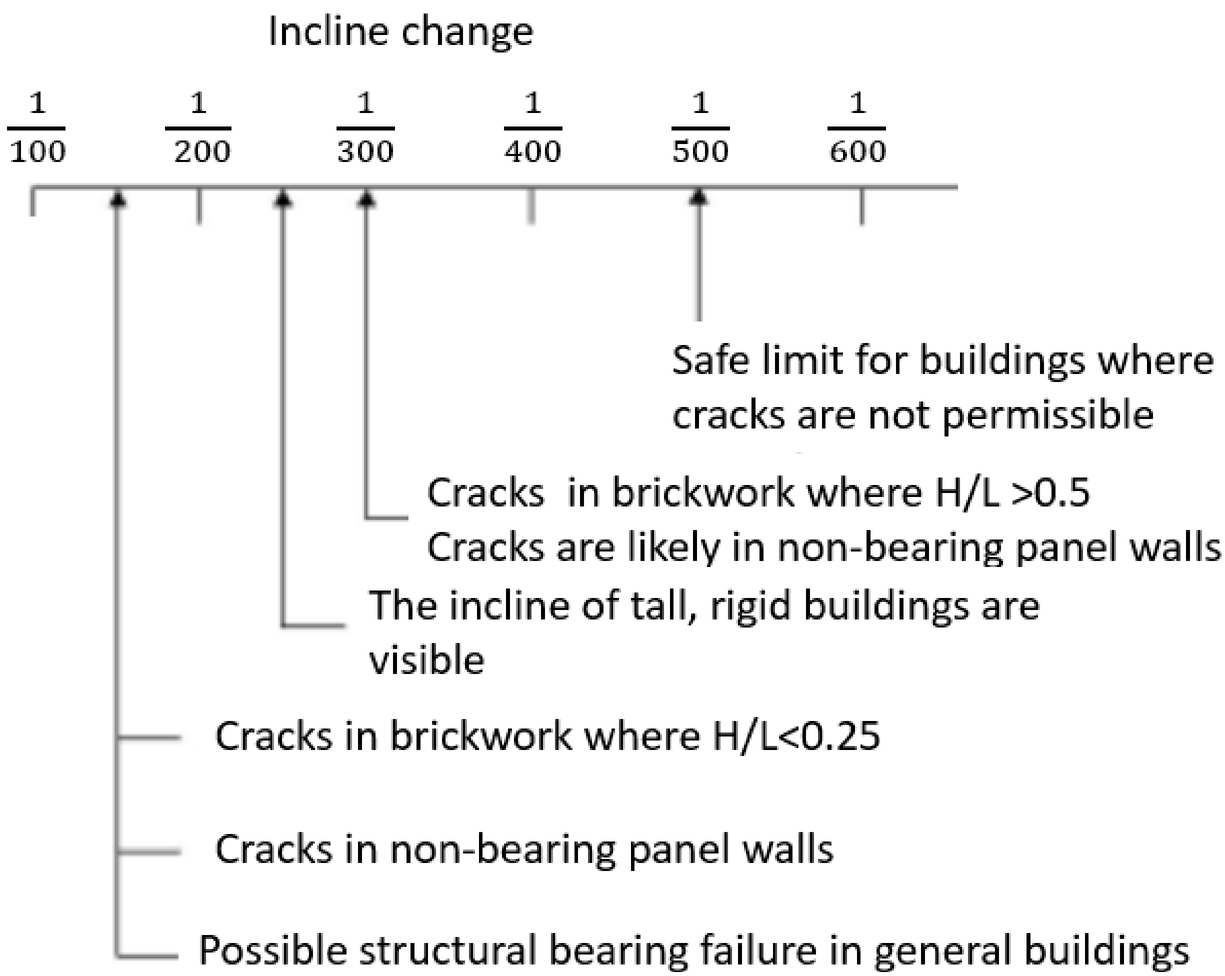


Figure 2.11: Damagepotensial in relation to differential settlement. Incline change is angular distortion $\Delta\delta/L$. Based on [Bjerrum, 1963]

settlements (max a couple of cm) and little probability for serious damage. The measured data show that a 10-30 kPa pore pressure reduction at bedrock gives an acceptable leakage of 3-7 l/min per 100meters. There are examples of apartment complexes that have evenly settled 20-30 cm due to tunnelconstructions, without visible damages. In other examples serious damage has developed from settlements of just 5 cm, due to that the differential settlements have been in the same order [Karlsrud et al., 2003].

Figure 2.10 shows differential settlements where the main factors are thickness of clay (B), and/or bedrock-topography (A). Mirroring Figure 2.10(A) or (B) can give illustrate the effect of a deep weathering trench. The Ground water line (GWL) may be assumed parallel to the dry crust or the terrain as generally the soil layering and GWL follows the terrain, but it can also be non-linear and should be investigated to be certain. [Baardvik et al., 2016] notes that buildings that are low and long are at higher risk for settlement damage, and that brickbuildings are worst due to their inability to be flexible. A rough reference for differential settlement impacts are shown in Figure 2.11, based on Bjerrum [1963], and Baardvik et al. [2016, figure 2.1].

Multiple measures can be done to prevent the $\Delta\sigma'$ increasing. By excavating the soilweight = buildingweight, one in theory can avoid settlements and has been known to work in the past. $\gamma_{soil} \times z = Q$, where Q is the buildingload (Q= q × length), γ_{soil} is the unit weight of the soil and z is the excavated depth [Janbu, 1970]. This is called "kompensert fundamentering", compensated foundation. However, while giving no settlements from building erection, this may put the building at higher risk for additional settlements due to pore pressure reduction since the soil is less compressed, and a higher chance to be subjected to virgin loading. Especially if the soil directly underneath is clay, whereas if it was sand or gravel it would settle less and can settle evenly more quickly if the depth to bedrock is very low and stable across the building length. A tall and compact building is more likely to settle by punching failure, instead of a rotational bearing failure in a lower and longer building, which has a potentially longer L making the β bigger Equation 2.28. The latter has in addition a potential lower building load meaning that the $\frac{\Delta\sigma'}{\sigma'_0}$ ratio can be higher. A bigger building is more likely to have varying bedrock depths and thus differential settlements.

One common measure currently used is to set piles to bedrock or harder soils, distributing the building load to a stiffer area. This is a very stable solution, but expensive. The easier solution is to lower the groundwater table before constructing the building or by compressing by inducing a load. Thus the soil will already experience the higher $\Delta\sigma'$. However in built up areas one must ensure that the groundwater table stays unaffected, therefore, one must be careful not to pump out to much water, because this can affect the surrounding areas negatively. Distribution of the building load is sometimes done by rafting, also called mat foundation, by erecting a steel-reinforced concrete slab. Rafted foundations often have type A $\epsilon - h$ curves in Figure B.7. Rafting was sometimes done on wooden pilings historically (before the modern age), and wooden piling or wooden rafting can start to rot and lose its strength if the groundwater table is lowered, as submerged wood hardly rots due to a lack of oxygen.

2.7 Quick Clay

Quick clay is the term used for a specific type of clay found primarily in Norway and Sweden, and with some existing deposits in parts of Finland, Russia, Canada and Alaska. Quick clay is most known for its bad strength when overloaded or stirred, and the cause for slope instability resulting in devastating retrogressive land slides. Quick clay when stirred acts like a liquid substance with liquid properties. From an undrained shear strength S_u of around 20-40 kPa to when stirred, less than 0.5 kPa S_{ur}, remoulded shear strength. This gives the clay a high sensitivity $S = \frac{S_u}{S_{ur}}$. NVE [2019] defines quick clay as having less than 0.5 kPa S_{ur}. With remoulded shear strength very low, the soil acts like a liquid when overstressed/remoulded. Remoulded is another word for stirred. The soil might not be quick clay yet if it still has a high salinity. It is when the salt is washed out that it can become quick clay. If the salinity is still high, then the clay will remain stiff and have a higher shear strength. It is important to note that soft clay is not synonymous with quick clay. A soft clay has S_u below 25 kPa [SVV, 2005].

When the quick clay is overloaded, the clay structure (similar to a house of cards) will suddenly collapse. Such clay is denoted as "quick". If the clay exposed to sustained loading, is subjected to remoulding from erosion (or other natural causes) and/or by man-made action, the clay masses will suddenly transform into a floating liquid with its own porewater. Quick clay slides can propagate very quickly backwards (retrogressively) over large areas, with the slide debris floating over considerable distances. Rissa (1978) and Gjerdrum (2020) are the most famous examples.

Quick clay is found below the marine level (the present elevation of where the sea level was at end of the last ice age) in Norway. The Scandinavian peninsula (Norway and Sweden) was covered with an ice layer of about 3 000 meters (approx. 20 000 years ago). When the ice started to melt, the small clay particles flowed with the water out to sea and was deposited in the marine environment of salt water along the sea shore (flocculation). The salt transformed the clay particles into a highly unstable structure, often referred to as a house of cards. As the ice melted, the land formation slowly rose due to the loss of the ice weight. Parts of the area with clay, which was previously situated under water, now was elevated above the sea. Currently the marine border is at about 220 - 300 meters above sea level. Over the years after the land elevation occurred after the last ice age, the salt in the clay layers has been washed out due to the fresh water in the ground. When the salt, which created the electrostatic bindings in the clay, is washed away, the strength characteristics of the clay is drastically changed to the worse. The flocculated structure remains however unstable, until mechanical disturbances causes a breakdown and the clay is transformed into a dispersed "liquid" state [Rosenqvist, 1966].

2.8 Geological issues

Deformation along faults in the shallow crust (top 1 km) introduces permeability heterogeneity and anisotropy, which can have an important impact on regional groundwater flow and hydrothermal fluid circulation processes. Bedrock is often assumed to be impermeable, not hydraulic conductive, but can in some areas have fractures due to weathering and/or fault lines leading water flow [Brattli, 2018a]. The Oslofjord area has multiple such fault lines and deep weathering zones [Olesen et al., 2007]. The settlement hazard was noted in Karlsrud et al. [2003]. Faults can have damage zones that water can flow through and Bense et al. [2013] outlines that outcrop observations indicate that fault zones usually have a permeability structure suggesting it acts as complex conduit–barrier systems in which along the fault flow is encouraged and across the fault flow is impeded/constrained. Faults and fractures can be observed as depressions in the landscape, filled with clay, see Figure 2.12. However topside observations should not be the only basis of these considerations, the hydrogeological context of the fault zone should also be included to ascertain the specific impact of a permeability structure [Bense et al., 2013]. This will vary from rock to rock due to e.g., mineral and clastic sediment unlithification or lithification.

> "Fault zones have the capacity to be hydraulic conduits connecting shallow and deep geological environments, but simultaneously the fault cores of many faults often form effective barriers to flow."
>
> [Bense et al., 2013]

Mapping this hazard by e.g., Baranwal et al. [2016] and Lidar data on a case by case basis is recommended. These faults and weakness zones are often where depressions are found, and big variations in bedrock-depth give differential settlement risks. Rock can also fracture due to drilling and explosions [Norsk Standard, 2012]. Even slope or rock stability can fail due to nearby explosions or new pore pressures. Rock with new fracture cracks due to drilling or explosions can create new water flow directions, affecting the pore pressure. An example of such is shown in Figure 2.12. While the figure was made for tunnel risk assessments it demonstrates that such a landscape has potential hazards. Two hypothetical settlement scenarios are shown in Figure 2.13 and Figure 2.14. Temperature, Lugeon or pumping tests can map/approximate fractures by waterflow, Equation 2.18 ([Brattli, 2018a] and [Liebel et al., 2012]).

2.9 Soft Soil Model

The simple and most common soil model is the Mohr Columb model. Few input parameters with linear elastic perfectly plastic stiffness. [Janbu, 1970] and the section 2.1 uses linear elastic perfectly plastic stiffness (stress-strain relationships).

However, the PLAXIS models in this thesis are using the soft soil model (ADV) as it is recommended for NC to medium OC soils for consolidation analysis [Karstunen & Amavasai, 2015]. This advanced soil model takes into account for strain hardening, thus non linear elasticity and plasticity. See Figure 2.15 for different models stress vs strain charts. This leads to not overpredicting the strength in stability and consolidation analyses as the Mohr Columb model risks to do. Both models have the Mohr Columb failure condition (critical state friction angle), dilatancy angle and the cohesion parameter. A dilatancy angle of ≈zero is typical for soft clays [Bentley, 2021a]. The yield surface of the soft soil model is shown in Karstunen and Amavasai [2015, figure

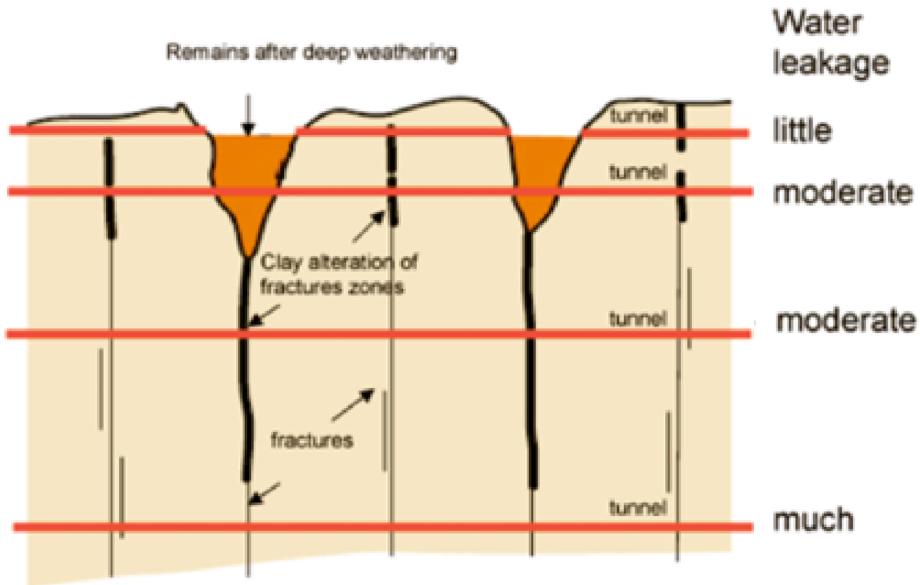


Figure 2.12: Deep weathering in the Oslofjord area with tunnel leakage risk assessments. Taken from Olesen et al. [2007]

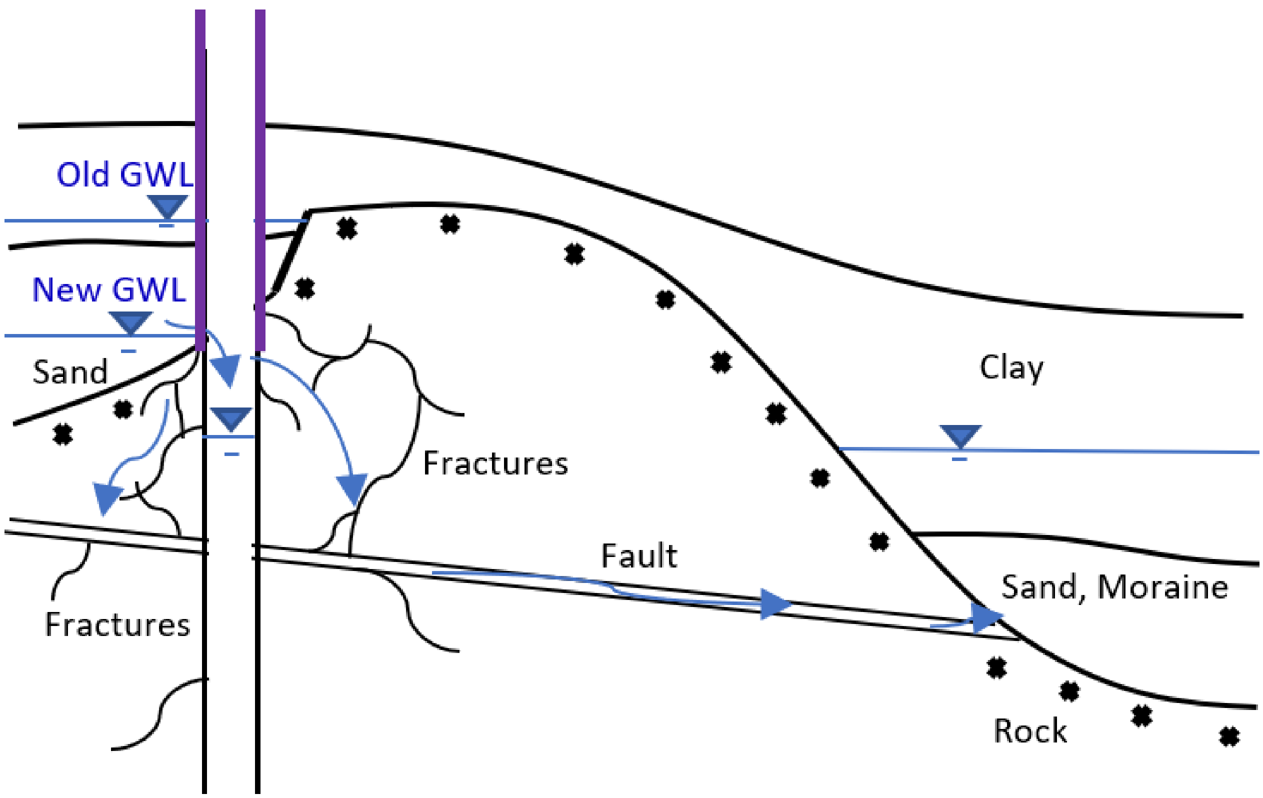


Figure 2.13: Hypothetical significant ground level or pore pressure drop due to bad plug or fractured rock by drilling in an area with varying depth to bedrock. Water flow traveling through fractures to fault or through the well itself. A perched groundwater with a permeable layer of moraine/sand and little groundwater inflow and infiltration increases this risk.

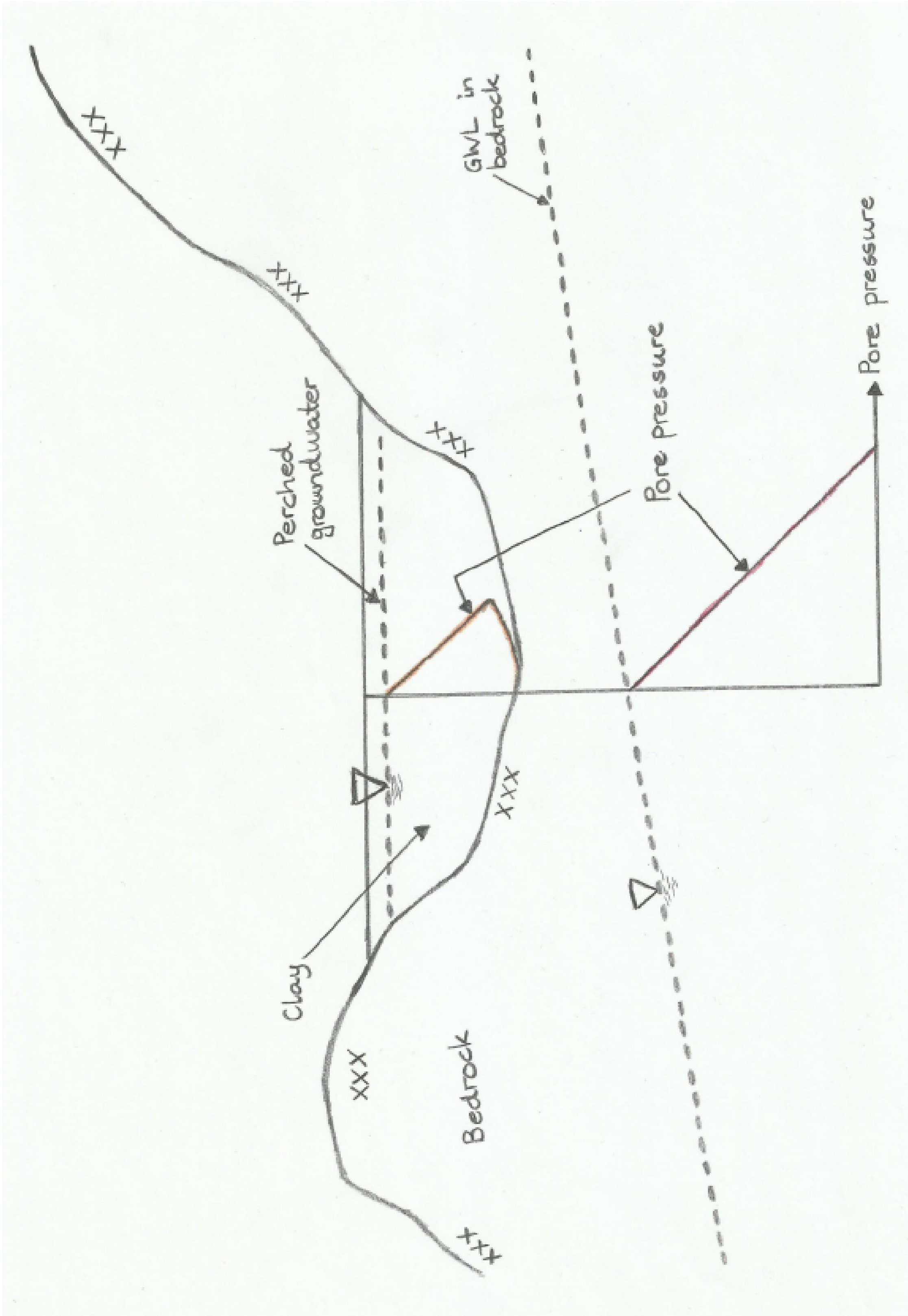


Figure 2.14: Perched groundwater landscape with risk of settlements due to e.g., aquifer perforation. Based on Karlsrud et al. [2003]

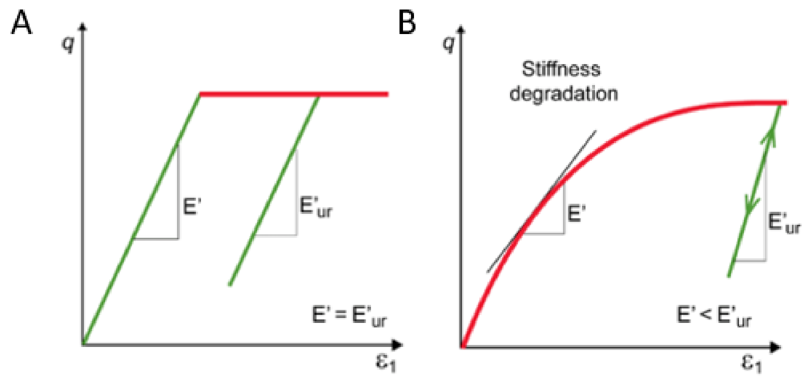


Figure 2.15: Stiffness for different soil models, where $E_u r$ is the unloading/reloading stiffness, q is the deviatoric stress $\sigma'_1 - \sigma'_3$ and ϵ_1 primary strain. (A) shows the mohr-columb model and (B) the strain hardening (e.g., soft soil) model. Modified from [Karstunen & Amavasai, 2015, figure 11]

4]. Instead of the Janbu 1D vertical stress and (residual) strain consolidation, σ_m is used in the soft soil model to take the 3D stress state into account. The mean of the three normal stresses $\sigma'_m = \frac{1}{3}\sigma'_1 + \sigma'_2 + \sigma'_3$ where σ'_1 etc are the main normalstresses in 3 dimensions. Odeometer results give the compression (λ^*) and swelling (κ^*) indexes by a natural logaritmic (ln) stress σ'_m strain ϵ_{vol} relationship, as shown in Karstunen and Amavasai [2015, figure 5], where ϵ_{vol} is the volumetric strain.

Instead of $\Delta\epsilon = \Delta\epsilon_v$, where ϵ_v is the vertical strain, $\Delta\epsilon = \Delta\epsilon_{vol}$ is used such that the strain formula becomes for stress in the OC region ($\sigma'_{m0} < \sigma'_m < \sigma'_{mp}$):

$\Delta\epsilon_{vol} = \kappa^* ln(\frac{\sigma'_{m0}+\sigma'_m}{\sigma'_{m0}})$

And for OC into NC stress region ($\sigma'_{m0} < \sigma'_{mp} < \sigma'_m$):

$\Delta\epsilon_{vol} = \kappa^* ln(\frac{\sigma'_{mp}}{\sigma'_{m0}}) + \lambda^* ln(\frac{\sigma'_m}{\sigma'_{mp}})$

The indexes for 1D strain can be calculated, using the $\Delta\epsilon_v = \Delta\epsilon_{vol}$ relationship and stress coefficient at rest, by the resulting equations:

$$\kappa^* = 1/m_{oc} \tag{2.29}$$

$$\lambda^* = 1/m_{nc} \tag{2.30}$$

where m_{nc} is the material modulus used in section 2.1 for NC clay and $m_{oc} = \frac{M_{oc}}{\sigma'_a}$ is the modulus for the OC condition/stressregion (Equation 2.5), usually 4-10 times higher [Nordal, 2020].

Soft soil creep model is another that could also be used to cover stiffness and settlement behavior for consolidation analysis, but creep is too long term dependent and an assumed relatively small effect on the total deformation, so is simplified out.

2.10 PLAXIS

A finite element method program to approximate the solution numerically is used. The program software is PLAXIS 2 D by Bentley. The element mesh consists of 15 node triangles in the calculations. While anisotropy can be modeled, isotropic and homogeneous soils/conditions are assumed for simplification. The program solves the complex equation series numerically like Equation 2.26 etc. A fully coupled consolidation analysis is done, where fully coupled denotes that the deformation and waterflow processes are intercoupled instead of independently calculated. The Drainage type (undrained or drained) is thus ignored, and the soil response is determined by the Permeability of the material [Bentley, 2021b].

The findings when modeling the Ventuiri effect due to DTH air drilling in PLAXIS Sagmoen [2017] are that these are very hydrogeologically (the boundary conditions) dependent and requires a considerably large model to reach the measured settlements. And that the secondary effect of remoulding due to drilling is possible, but problematic to model with volumetric strain in the affected layer.

Chapter 3

Bored Ground Works

This chapter presents the affects of ground works on ground settlements in previous literature, with bored piles from terrain being the main focus. Bored piles are similar to energy well installation in regard to casing and bedrock grouting/bentonite plug, except that the casing diameter Ø, is greater for most piles and that an energywell can reach 300-350 m depths into bedrock. Energywell casings have approximately Ø12 cm, similar to some tieback anchors and micropiles. A short comparison to the case study in Våpenstad [2021] regarding energy wells is discussed at the end of this chapter, ultimately supporting the same conclusions.

3.1 Bored Piles

A common foundational practise is to have the building load distributed by piles, either by piles in soils or piles into bedrock. While driven friction piles use the resistance in the soils the end bearing bored piles utilize chiefly the bedrock. End bearing piles have the load distributed to bedrock and can thus carry a bigger load. Generally these piles are preferred in areas with limited depths of soft soils overlying solid bedrock [Lande et al., in press]. Bored end bearing steelcore piles is the main focus of this chapter.

As in an energy well installation, an outer steel casing pipe around the drill string is used when drilling in soils. The regulation [Norsk Standard, 2019] says only to be careful when drilling in sensitive/soft clay, not which technique to use. The SVV procedure SVV [2007] advices soly water drilling in soft clay, loose silt and sand.

In the soil-transition a minimum 1 m into solid bedrock is advised [SVV, 2007]. In Norsk Standard [2019] it is not specified length required, but by indication of solid bedrock. Whether air or water drilling into bedrock should be done, is not specified in Norsk Standard [2019], while SVV [2007] denotes conventional odex drilling. Filling up the casing with water is done for 8 hours, with a lower waterlevel than GWL to check for seepage. If any leakage is registered the injection procedure starts straight away without doing the water loss test first [Norsk Standard, 2019]. The water loss test is done by having 1Mpa overpressure and registering the loss with units l/min per borehole-length (Lugeon, L). While no leakage-limits are given in Norsk Standard [2019], it does specify that the pressurereading differential must be less than 10% between two time intervals, an assumed steady flow. SVV [2007] puts the limit at $L < 0.5$ l / min. Before grouting a cleaning of the inside of the casing is done with air or water pressure. The bentonite/injection-mass plug, to stop any leakage must be injected with a reasonable pressure so as to not disturb/deform the soil and bedrock [Norsk Standard, 2019]. The injection-mass/bentonite mix may vary from company from company and is for simplification not discussed in detail. Further information is presented in (e.g., Gundersen and Haugen [2021], Norsk Standard [2019] and Aune and Østvang [2020]).

Further drill length for the uncased pile emplacement in bedrock is not specified, but recommended in both the regulation and procedure to do after the plug is done, see Figure 3.1. There are generally more requirements/recommendations for injection and grouting in SVV [2007] relative to Norsk Standard [2019]. Appendix 11 in Aune and Østvang [2020] compares the different regulations and is recommended to read if further and more in depth insight is wanted, as this section provides a general review and not a comprehensive point by point walkthrough. Furthermore, different drilling companies may have varying procedures/regulations and can thus differ from the installation process described above. For instance, Multiconsult/Hallingdal have a revised procedure discussed in detail later, used in Nyhavna, Trondheim, shown in Figure 3.2.

After the steel-core is installed, the piles are grouted/poured with cement mortar all the way to the top for additional strength. Energy piles are not common in Norway, but may see increased usage in the future. Settlements can give hang-on loads due the compaction of soils. Steel piles are more flexible to take tensile/moment forces compared to concrete piles, putting the latter more at risk. The magnitude of these loads or if the piles are dimensioned to take this extra load is unclear, and likely varies, but it is described as potentially significant for buildings on foundations of piles in Baardvik et al. [2016].

3.2 Literature Review

3.2.1 Nyhavna

This subsection is based on Aune and Østvang [2020], a settlement and overpressure analysis for the case of a housing complex in Nyhavna, Trondheim. An overpressure (10-40kPa) above the hydrostatic, is registered above bedrock in the assumed moraine layer under clay for the general area with a few exceptions. The moraine thickness varies between 1-5 m and has 20-30 m fill material and clay above, the drainage of which leading to settlements up-to 16 cm by calculation, but not actualized/recorded due to successful installation of the piles. Here water pressure drilling with a polymer blended fluid was used in soil, air for the cased borehole and conventional Odex used for drilling in bedrock. The drilling caused a temporary pore pressure loss of up-to 30kPa in the bedrock interface and 6 kPa in the middle of the clay. When

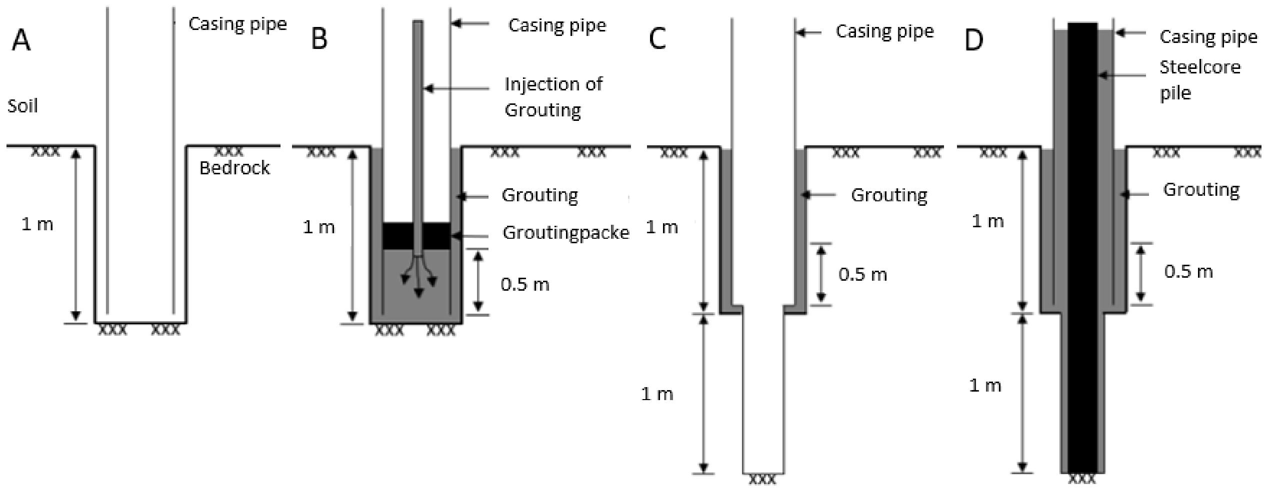


Figure 3.1: Clockwise from top left; Drilling of casing into bedrock (A), grouting/bentonite injection (B), drilling into the cast and bedrock (C), steel core installation and grouting (D). Modified from [Aune & Østvang, 2020]

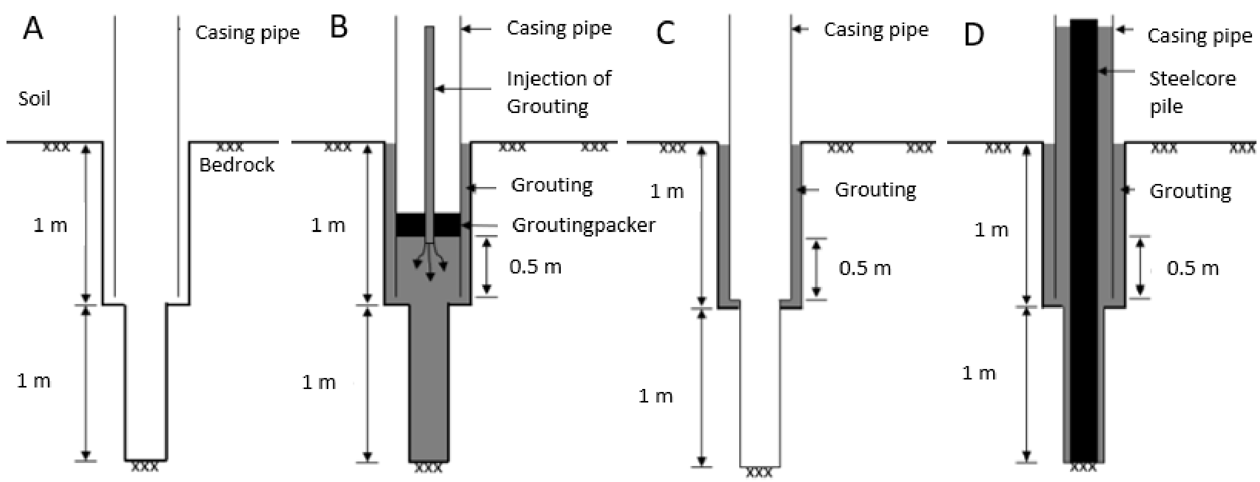


Figure 3.2: Clockwise from top left; Drilling of casing into bedrock (A), grouting/bentonite injection (B), drilling into the cast and bedrock (C), steel core installation and grouting (D). Revised procedure by Multiconsult to achieve better injection of bentonite/grouting. Modified from [Aune & Østvang, 2020]

drilling with radial distances of less than 6 m between the casings, no pore pressure reduction was registered. The polymer blended fluid has likely had a sealing effect, and any leaks, was in the plug-bedrock/soil transition. When casting the plug, the fractures may not get filled as the injected mass takes the least resistant path which is through the bedrock/casing gap. Nevertheless, casting the plug has an effect on sealing the fractures, likely due to the overpressure in the moraine layer outside the casing.

A revised method is grouting after the final boring into bedrock is done as shown in Figure 3.2, leading to more than halving the leaks and only requiring one set of grouting for the ones that still did [Aune & Østvang, 2020, table 30]. Multiple grouting is done if the Lugeon test (water pressure loss per time) shows insufficient sealment, and was needed for the original method up to 3 times for 21 % of the piles. Welded lid at the top of the casing if multiple groutings were still not sufficient [Aune & Østvang, 2020]. Thus doing the uncased bedrock boring before casting the plug and drilling through it yielded better plugs. Also as a sidenote; welded lid is not at the top in [Sveriges geologiska undersökning, 2016] for energy wells, but rather above the plug. Δb denotes the distance between the casing and uncased borehole shown in the frame in Figure 3.3. The frame top right shows that one theory is that low Δb and/or a rounded edge (red circle), gives better grouting effect and thus less leakages, with the columns showing the recorded instances that indicate support for this theory. Δb = 16.5 mm having 31 % leakage after first grouting, and the averages of the other three to Δb = 14.35 mm having 15 % leakages. In other words $\frac{16.5}{14.35} = 1.15$, a 15 % increase in gap Δb gives twice as much leakages. However, more tests in different soil-profiles should be done to conclude conclusively.

3.2.2 Begrens Skade

The research project Begrens Skade provides a comprehensive overview, tests and in-depth discussion related to settlements and ground works in soil. Figure 3.4 shows the increased settlement associated with conventional down the hole (DTH) air drilling due to excessive, compared to water pressure (Wassara). The Venturii and air-lift pump effect is to blame, where a local suction locally round the borecrown can erode mass on its way downward shown in Figure 3.4. In an overpressurized thick moraine or soft clay critically so ([Baardvik et al., 2016] and [Lande et al., in press]). The pore pressure reduction may vary or for a pile it might only exist before grouting etc. The hydraulic height difference Δh and gradient i is important for the risk of overboring and leakage. The drilled piles have a greater risk of leakage if they are bored in artesian aquifers or below GWL, the latter is prone to happen in a building pit. Filling the casing with water/fluid to reduce the pressure helped in Lande et al. [in press], and less moraine was overdrilled. The pore pressure change is registered in distances upto 400 m for building pits. Building pits are also more likely to cause horizontal deformations [Baardvik et al., 2016]. For piles the area of influence is generally much lower around 10 m, but can extend much further depending on the moraine and hydraulic boundaries [Lande et al., in press]. There are three main leakage methods for end bearing piles: outside or inside of casing, by the gap between casing and bedrock or

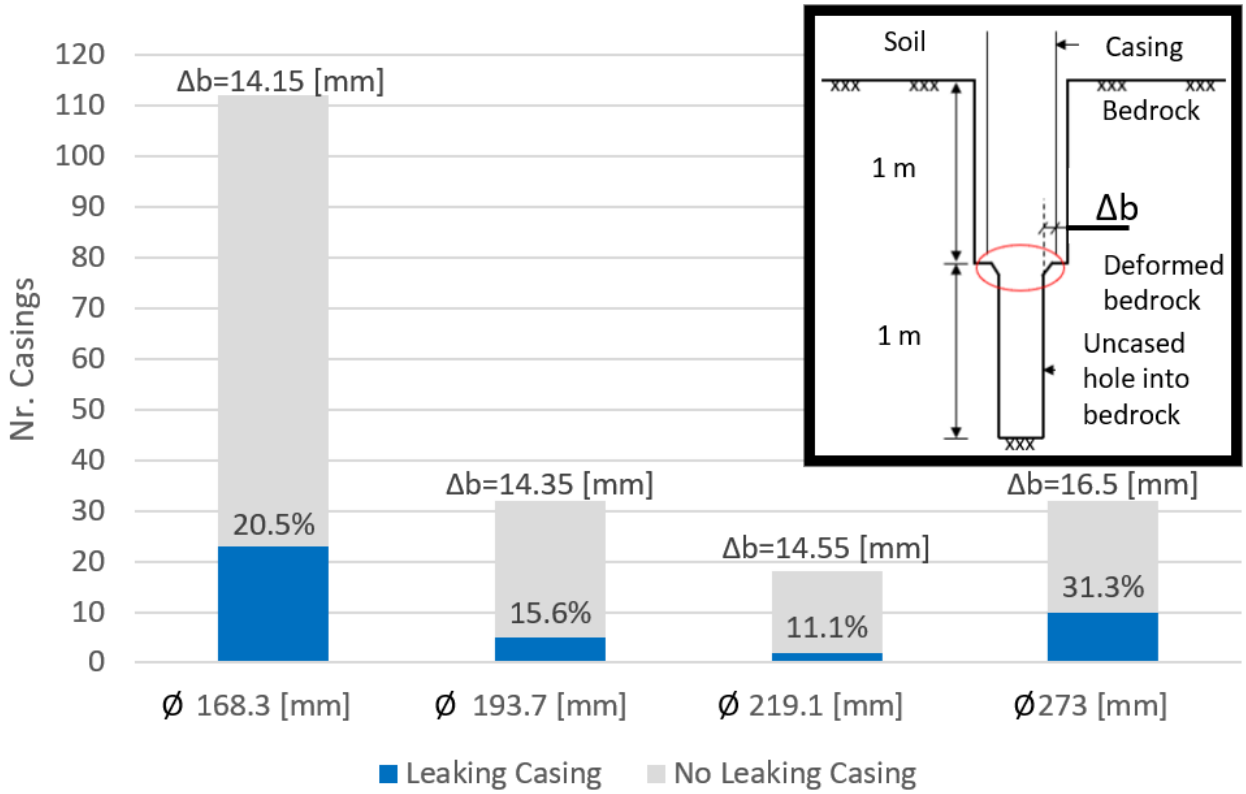


Figure 3.3: Columns denoting number of leaks in casingpipes after the first injection of grout/bentonite for steelcore endbearing piles, based on [Aune & Østvang, 2020]. Frame top right: Endbearing pile into bedrock with possible deformed hole by drilling (red circle) and Δb the gap between casing and inner bedrockhole illustrated. Modified from [Aune & Østvang, 2020].

by fractures in rock [Langford et al., 2016].

For remoulded clay the strength returns rather slowly, and in case of over-consolidated soil layers, only partly [Finnish Road Administration, 2003]. Borchtchev [2015] presents results for shearstrained clay that show expected 8-16 % volume reduction $\Delta\epsilon_{vol}$ in thoroughly stirred clay by remoulding, for in-situ effective stress σ'_v. The results indicate that similar volume reductions can happen in moderately disturbed clay at greater depths where the σ'_v is greater than ≈ 100 kPa. Older tests by NGI [1964] have 10-15 % strain reduction by odeometer (1D) consolidation. While the affected area around the drill-bit is unclear, a greater drill/casing diameter will affect a greater area. An example is shown in Baardvik et al. [2016].

To highlight the trends described above a case is presented , taken from Baardvik et al. [2016] and described in greater detail in Lande et al. [in press]. Here data from DTH air drilling for endbearing steelcore piled foundations for a bridge over a river is presented. For the bridge area the soil profile is deep layers of clay around (10-48m) above thick moraine (1-9m) with varying depths, and an overpressure recorded around 15 % above hydrostatic in the upper confined moraine layer.

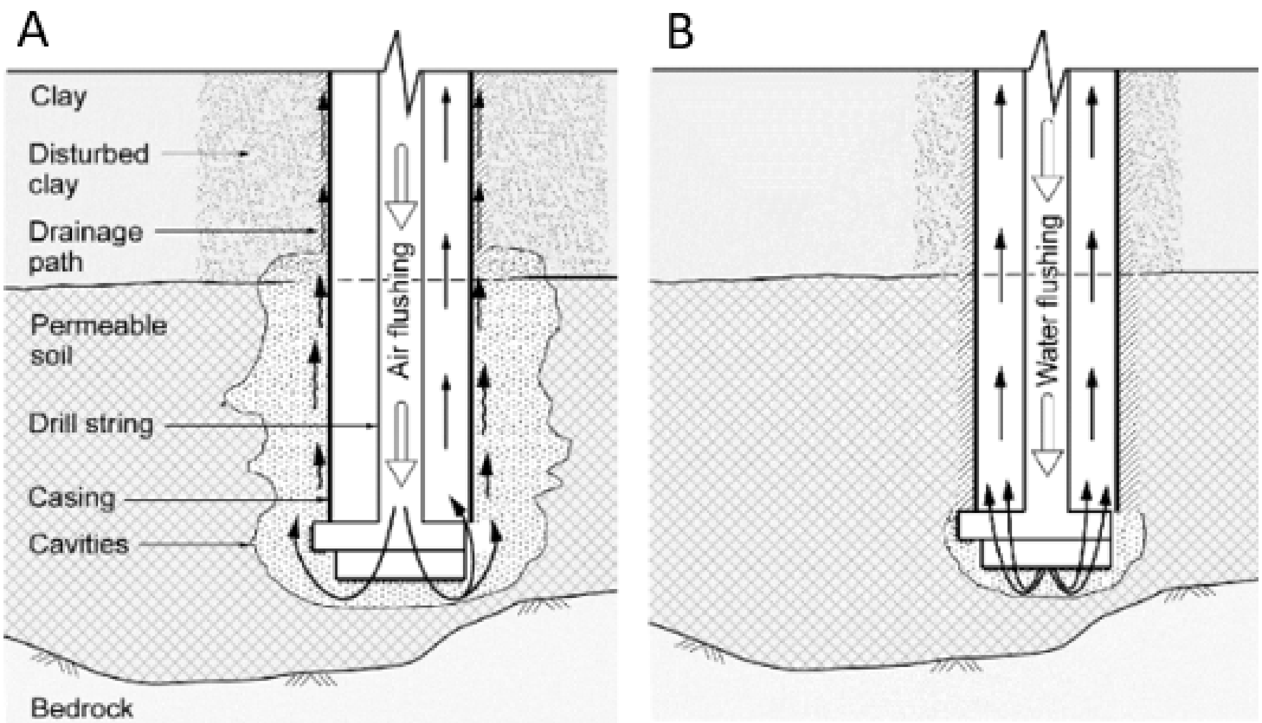


Figure 3.4: DTH air drilling (A), with extensive erosion, disturbance and drainage, and (B) with less effect by water drilling. Taken from [Langford et al., 2016]

Figure 3.5 shows pore pressure variations due to drilling of pile group 4, the closest group, for pizeometers at different depths and distance from the piles. It is important to distinguish that these variations in moraine were generally short (1-2) hours, whereas in clay they were longer. However, this is around 25 kPa below original for pizeometers PZ1,PZ4 (red and black) due to previous pile group drillings and for the PZ2, PZ4 (blue and brown) an around 15 kPa more permanent drop is shown. PZ1 and PZ4 are the closest to the piles drilled, which is why these show the most severe Δu.

Figure 3.6 shows the long term settlement recording for the whole project. The ground settlements that are measured next to pile group 4 (yellow box in Figure 3.6) is likely due to the DTH air pressure drilling in moraine. The abovelaying clay have settled gradually over a longer time period, something that is probably due to stress-redistribution in the clay and suction (pore pressureloss) in the moraine. This is supported by measured pore pressurereductions in moraine and silt [Baardvik et al., 2016].The drilling through clay, dense moraine and into bedrock indicate that drilling with the high pressurized air flushing through the coarse moraine caused erosion and soilvolume loss along the casing. The immediate settlements were recorded up to about 6cm at a depth of 41 m. Additionally, 1-2 cm of settlement due to reconsolidation, is measured during the 3 months after drilling shown in Figure 3.6 [Langford et al., 2016].

In Lande et al. [in press] a design-chart for water pressure and a flow/risk chart for when to use the preferred drill technique is presented. This article has a building pit case with a huge

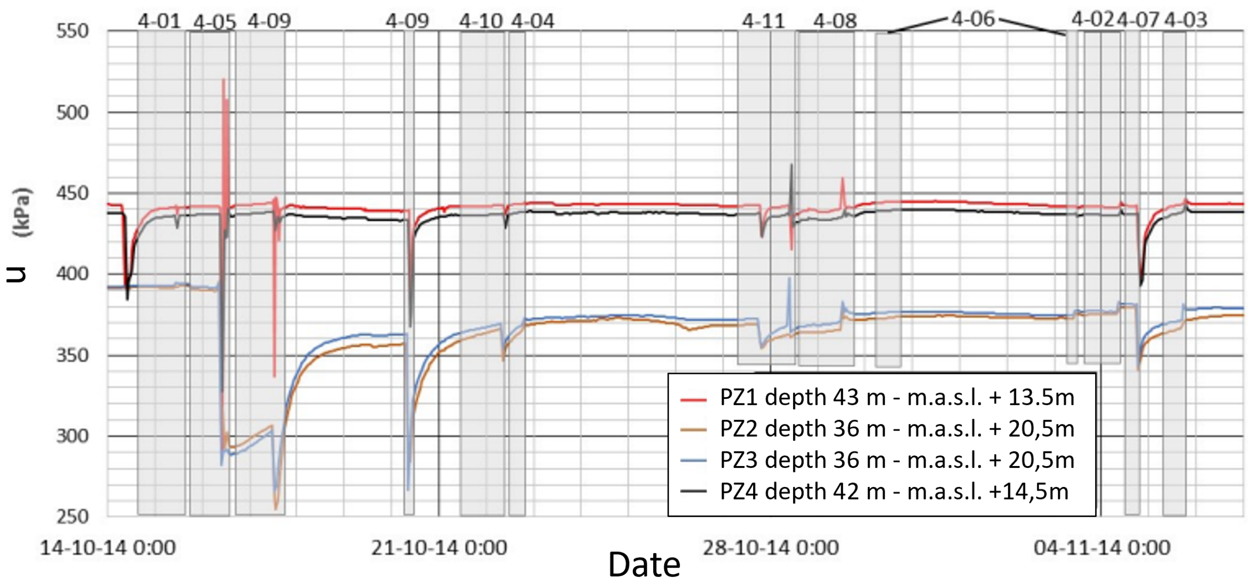


Figure 3.5: pore pressure over time for bored pile group 4, with grey columns signifying drilling of piles. Modified from [Baardvik et al., 2016]

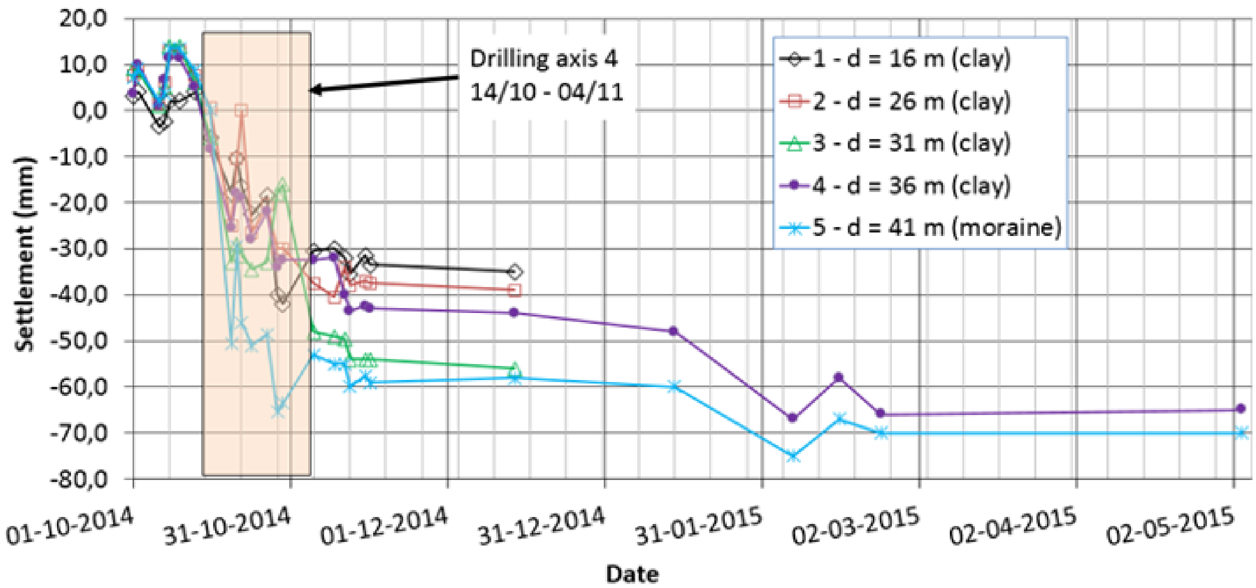


Figure 3.6: Settlements over time throughout the project, with pile group 4 (11x Ø711mm piles) closest to the measuring instruments. Taken from [Langford et al., 2016].

amount of piles below sea level into thick soft/medium stiff clay over moraine. The pore pressure generally stabilized after drilling, but in some areas, to a lower than previous level. The permanent pore pressure reduction in both the bridge and building pit cases are likely due to observed topside leakage (i.e drainage) through the casing before filling it with cement mortar [Lande et al., in press].

3.2.3 Other Ground Works

At Grønlia, Fredrikstad a geotechnical survey for railway construction was done, but these surveyholes were not sealed, and their long term leakage resulted in settlements [Gundersen, 2018]. More info is described in section 1.1.1. Other surveyholes and the sealment procedure is decribed in Gundersen and Haugen [2021]. The survey report Venvik et al. [2019] remarked that it was hard to trace the coarser layers below the clay, and if they were in hydraulic contact with weathered fractured bedrock or not. Here a pore pressure can be built up, and is registered in some survey holes. High porerepssure is important to note in relation to the loading of quick clay. Sheet-piling and piling with pressure and lime-cement stabilization can increase the pore pressure, and subsequently might collapse the quick clay. If there is an overpressure already, a relatively smaller extra increase might potentially be enough to shear and collapse the soil [Venvik et al., 2019].

Breivikeidet bru, Troms is perhaps the most severe ground settlement case in Norway, where construction had to be stopped due to the leakages and mass erotion when piling. The big hydraulic gradient/differential head due to topography, is the main culprit [Gundersen, 2019]. Same case in de Beer and Dagestad [2020] for an energy well puncturing an artesian aquifer in a quickclay-slide hazardous area. Artesian water leaked out of the newly created well, but was quickly remedied by drilling a relief well and using a better grout on the outside of the casing.

A case done with little settlements was the new Munch museum in Oslo [Veslegard & Rønning, 2016], case 2 in Lande et al. [in press]. Pore pressure reduction was registered in the building pit when drilling, but generally stabilized. 88mm settlements due to drilling in excavation pits next to sea level is seemingly very little. The settlements are arguably mostly due to masserotion when drilling. Water pressure drilling fared better, in addition the water/fluid on the inside of the casing counteracted the outerpressure more (lowered hydraulic gradient i). Also in Oslo, Bjørvika is an area with settlements due to all the construction close to the sea in soft/deep clay. Building transitions and the right drill techniques where highlighted of note [Hauser, 2020]. Down-town Oslo has had a few piles leaking upwards in an area susceptible to settlements due to old foundations and significant clay thicknesses. This area had some overpressure measured not in the entire soil profile, but above bedrock, varying with the seasons. This indicates hydraulic conductivity between the rock and deeper soils. The coarser soils above bedrock are found disjointed sporadically [de Beer, 2010].

While the bedrock may fracture easier in some rocks depending on the bedrock quality by drilling, there is denoted in Holmøy et al. [2019] no indication of any correlation between the bedrock type in tunnels and the magnitude of pore pressure reduction. Thus leading to the assumption that fractures/faults may be present, regardless of any bedrock type as expected, and that the permeability in rock has very little relevance, relative to the hydrogeological conditions of the soils. Latter dependant on the hydraulic properties of the coarser soil layer (e.g., moraine) as a "conduit" between bedrock and clay layer, such as that the thickness and areal extent of the coarser grained layer, in addition to the natural groundwater recharge and hydraulic conductivity [Holmøy et al., 2019].

There are figures in (Karlsrud et al. [2003] and Holmøy et al. [2019]) that plot Q / 100m tunnel length vs porerpessure reduction at bedrock, but since these are per 100 m, they cannot be easily converted/used for single piles or energy wells. Still, they show how much water outflow is required to lower the pore pressure. Karlsrud et al. [2003] presents a few time vs. settlement figures due to pore pressure reductions at bedrock due to tunneling, that show that a greater depth to bedrock compared to low depths to bedrock in the short term has less settlements, but in the long term are greater.

Børsum [2018] has data presenting settlements vs time and pore pressure vs time and with and without infiltration wells for two tunnel segments of the Follo project. With pore pressure loss during the drilling, before stabilizing after grouting in possibly 2 up-to 12 months time. The usage of infiltration wells indicate rebounding the pore pressure quicker when comparing the two tunnel segments [Børsum, 2018, figure 38 and 39]. The grouting of tunnels, infiltration well techniques and data are further presented in Børsum [2018].

3.3 Scenarios

To better provide an overview of possible settlement scenarios and mitigation measures, a quick rundown is presented below.

3.3.1 Moraine Layer

Moraine/silt is a coarser grained layer that can be overbored with air DTH drilling and can more easily be drained due to higher hydraulic conductivity. A change in pore pressure for this layer can works its way upwards into the clay. In reality the moraine can vary in which grain size dominates, from glacial till to coarse sand and even seldom gravel or rock fragments.

The Norwegian Geotechnical Society [1994] regarding total sounding tests describes the moraine being generally thicker at depressions in the bedrock topography relative to the topographical highs, where the moraine thickness is limited/thin. A moraine layer might be hard to discern form total sounding tests in some places ([Vestfold Tingrett, 2021] and [Norwegian Geotechnical Society, 1994]). When boring in rock with many fractures and damage/weakness zones, the need to bore more than 3 meters can arise before bedrock detection can be ascertained with certainty. And recently the standard drillingdepth is 3 m into bedrock for total sounding, to better determine bedrock depth. [Norwegian Geotechnical Society, 1994].Experience dictates that it should always be used waterpressure for drilling into rock. This is different compared to energy well and pile installation. Brittle rock is also difficult to map, especially under solid moraine [Norwegian Geotechnical Society, 1994].

It likely is that the case study in scenarios below require a moraine/coarse grained layer. In the latter two scenarios to act as a permeable layer (conduit between the bedrock and clay) in order to happen, or at least that the magnitude is amplified greatly by the presence or size of a more permeable layer like moraine. A pore pressure reduction takes a long time in clay and maybe the leakage path may be more blocked with finer grains.

3.3.2 Overboring and Disruption of Soils

DTH air pressure drilling poses potentially significantly higher risk in more permeable layers of silt/sand/gravel/moraine or sensitive and soft clays. A lower area A, requires a higher velocity, See Equation 2.21 and Equation 2.19, where a lower area requires a lower pressure and higher velocity, producing the Venturi effect, see Figure 3.4.

Usually immediate to short term, with the latter due to remoulding of clay [Ahlund & Ögren, 2016]. In quick clay and other sensitive soil-areas, pile instead. In Statsforvalteren i Oslo og Viken [2020] it is claimed that the casing for the energy well is piled in the sensitive layer, and soils inside the casing is drilled out. There are some piling cases where the pore pressure is only partially stabilized to original level after piling is done, and thus an indication that drilling can lower the pore pressure long term due to suction in the moraine. Since clay has a low permeability a pore pressure reduction in the underlying moraine, which does not have inflow from any other boundary interfaces other than clay, more so at risk. On the other hand there is no cases that indicate settlements long term in the piling cases (except de Beer [2010]), but this could be partially due to not recording at terrain for such a long duration of time.

3.3.3 Upwards Drainage due to Artesian Conditions or top of Casing below GWL

No or insufficient grouting in/around the casing, fractures in bedrock or no welded lid are the possible causes of artesian pressure (confined aquifer) drainage. Veslegard and Simonsen [2014] denotes that in artesian conditions grouting/injection of bedrock must be considered. Figure 2.8shows artesian conditions by higherlaying groundwater in hydraulic contact by permeable layer and/or non-impermeable bedrock. The overpressure might never drain fully out, or just be lowered depending on size of the aquifer, leakage and if mitigation measures are implemented or not. A water chemistry check or by in situ pore pressure measurement relative to hydrostatic pore pressure can be done to ascertain if it is a closed or open aquifer.

3.3.4 Fractures/Bad plug resulting in Increased Seepage/Drainage to Bedrock

Similar to the above except that the leakage cannot be observed at terrain, and that there has to be an connected aquifer below with lower pore pressure. A perched groundwater landscape sensitive to aquifer perforation/drilling is shown in Figure 2.6. Veslegard and Simonsen [2014] describes that fractured/brittle rock, borehole collapse and waterborne layers are problematic when drilling for energy wells into bedrock.

3.3.5 Mitigation Measures

- Infiltration wells.
- Grouting or welded lid as seen in figure 8 Sveriges geologiska undersökning [2016]
- Less pressure or water (e.g Wassare) drilling. For the former the difference in driller expertise/experience plays a role.
- Piling the casing instead in soft clay.
- Better injection into the casing-bedrock gap or fractures such that the plug is properly sealed.

For pore pressure loss below hydrostatic, infiltration wells can be done, but if this contributes to mass and water flow due to overpressure not dissipating it will be detrimental, not mitigating the problem. Therefore overpressure could be good if that means more water inflow, thus no pore pressure reduction. [Sundell, 2019] and [Børsum, 2018]. If however there is a leakage (up or down) it depends on the inflow and outflow of the system whether or not this happens. Massloss due to the water expulsion or washing out the salt are still a problem and should be kept in mind as a hazard. Especially in quick-clay slopes.

Doing it correctly the first time such that a pore pressure decrease never happens is the cheapest solution. For more about grouting materials see Gundersen and Haugen [2021] and Holmøy et al. [2019]. Instead of tie-back anchors, inner struts in the building pit. Grouting/inline the tunnels as well as infiltration wells.

Grouting is the cheapest option. Doing grouting after drilling and casing is done, can be more expensive and give less than desired effect, thus grouting while piling/drilling is recommended as described in Gundersen and Haugen [2021], as is done in other steel core piles. Grout the whole not just the tip, could be done in sensitive artian areas/conditions. Grouting of fractures might be possible but expensive/difficult to do and a overgrouting might raise the GWL or pore pressure leading to other issues like heave. Infiltration wells can mitigate, but should be used with caution as too much can lead to e.g., heave. Lower pore pressure readings from the soil near bedrock is a reliable indicator that settlements will happen. The infiltration rate and in if the moraine layer does not follow terrain but is horizontal or piecemeal present, the mapping of this layer determines the area of effect.

For mitigation of drilling (overboring and remoulding as well as pore pressure change in silty-clay to moraine gravel) it is recommended to use piling in sensitive clay as allegedly done in case 2 [Våpenstad, 2021] and advised Sveriges geologiska undersökning [2016]. A welded cap is also recommended to stop hydraulic flow, see Sveriges geologiska undersökning [2016, figure 8]. Having this above (stopping artisian flow) and below (stopping downwards flow through the borehole) might be an extra step to stop the waterflow if there is fractures or bad plug in the borehole. Different drillers may produce different results by vary flushing and pressure (i.e penetration rates) such that the overboring, disruption and effects on the pore pressure is different. A high penetration rate per time can make the effect worse (Lande et al. [2020] and Lande et al. [in press]) and more the same as piling.A certification of drillers is recommended (e.g., Langford et al. [2016]), but not compulsory, and there exists a school for a certificate of the craft for onshore drilling-operators since 2019 [Daler, 2021].

3.4 Comparison to the Case Study

Possible causes due to drilling are from the literature and case study in [Våpenstad, 2021], presented in Appendix B.1. and Appendix B.2. Possible causes of settlements were found to be overboring, disruption of clay by drilling, and and pore-pressure reduction. Air down the hole (DTH) drilling is ill advised in moraine/silt layers above bedrock and sensitive clays (e.g., Lande et al. [2020] and Ahlund and Ögren [2016]). Pore pressure reduction can also be due to increased seepage to bedrock, seepage on the outside or inside of the casing, through a bad plug in the soil-bedrock transition or fractures caused by drilling.

When referring to cases 1,2 and 3, the casestudy in Appendix B.2 is referred to. By looking at the literature review in this chapter and comparing it to the cases it is difficult to find general conclusions due to the variables involved and the unavailability of data regarding the case study. While water is better than air drilling, the moraine/silt layer and drill operator has much to say. Silt or moraine soils are at greater risk for water-erosion and a washout of mass will increase the settlements. The waterpressure recharge is often unknown and hard to predict. pore pressure might also vary with the seasons. Pizeometer installation in moraine/above bedrock in areas with clays (particularly soft clays) is recommended. Large variations in bedrock is an indication of potential differential settlements. However, the settlements are generally immediate with some remoulding of clay in the short-medium term. The stress redistribution upwards can make it gradual and less for the clay layer, but not so long term as it is in the case study. A general trend in the cases in section 3.2 have a moraine layer above bedrock, and this may be true for all of the cases in Appendix B as well. Permanent drainage is less easy to find data for, explained by that the pore pressure drainage often fixes itself (rechargment after grouting) for bored piles and tunnels. Assuming no leaks leads to only measuring the short-medium term, which explains the lack of long term measurements, and why the plug into bedrock is less covered in detail in most reports/articles. The leakage test is done before the plug is drilled into, for both energy wells and piles installations. A leakage test after the plug is drilled into maybe yields different results.

However, most steel core piles don't leak and if they do, it is short term as the casing is filled with grout after steel core installation. 2 m into bedrock before a plug can be installed is required for energy wells as per NS 3056:2012 Norsk Standard [2012], 1 m more than for piles, and further drilling afterwards is to a much longer depth. Thus requiring a packer/welded lid below as well. But nevertheless the revised method in Aune and Østvang [2020] could be further investigated for use in energy well boreholes. Lowering the Δb is also recommended for energy wells.

While a pile has grouting on the inside, an energy well has not and is drilled much further into bedrock (200-300m), see Appendix B.1.2. Thus, if there is a pore pressure reduction, it is more likely temporary in bored piles, as its insides are cast (concrete and steel piles). Sveriges geologiska undersökning [2016] has a welded lid illustrated above the bottom part of the casing in the bedrock-soil transition. If done correctly this can stop hydraulic connectivity upwards through the casing. Another below would do the same vice versa if there was a downwards seepage and perhaps should be done as well. It may be possible that the bedrock depth was not accurately accessed in the cases, or that the drilled depth into bedrock was insufficient due to bad/fractured rock as discussed in section 3.3.1. Cost analysis says not worth to grout energywells and perhaps other boreholes (e.g., survey hole) due to the low possibility compared to total number of units installed, but for party experiencing settlement damage this line of reasoning is irrelevant. Grouting the whole inside will seal any leaks.

Leakage due to artesian conditions is generally seen at terrain, a big jet of water or rather a trickle of water out of the casing. If on the outside of casing, leakage into the fill/dry crust material, might be harder to detect, leading to possible settlements over time if not fixed. And also the two latter cases in Appendix B.2 have only InSAR data and no proper settlement measuring instrument data or pizeometer data available. Piling down the casing then excavating the insides could be done in soft clay, and the drilling company for case 2, mentions that they did this Statsforvalteren i Oslo og Viken [2020]. All in all to say that the drilling is at fault is difficult, as most often these are short-medium term settlements and more circular in area of effect (see Figure 3.6).

While costs of grouting may exceed damage on the buildings since energywells have such few cases of ground settlement, the third party is affected and thus this cannot be decided through a cost-analysis alone. Case 1 shows that only 1 energy well could have caused a new house to be built for resulting in a total price tag to 10 million NOK [Vestfold Tingrett, 2021].

The few possible cases (with none in court proven at the time of writing) of energy well causing ground settlements indicates the possible cause to be very hydrogeologically dependent, with fractures and/or operator failure (leaking plug) being at fault. The most probable cause of pore pressure drawdown by leaking plug or fractured bedrock causing a permanent pore pressure loss, is the focus in the models. While artesian conditions are not described in any of the three cases, they may be in the deep clay/moraine layer. While more factors could be at play either simultaneously or instead, the case studies regarding energy wells (e.g., InSAR data) indicate long term settlements fitting permanent pore pressure reduction, as shown in Figure 2.1.

3.5 Summary

Drilling in Norwegian soils can pose risks by overboring, disruption and porepressure change in soft clay, silt and moraine. For the bored piles cases the measured settlements are immediate to short term developing in the soils, as the leak gets sealed and/or drilling stops. However for an energy well the borehole is not grouted, and if the plug is insufficiently sealed or there are unsealed fractures in the rock by the plug or below, this possibly can cause settlements over time. Right drill techniques and leak-free checked boreholes are advised when installing energy wells in soft clay above moraine. An additional porepressure/Lugeon check or welded lids installations after drilling into the plug and bedrock can be enacted depending on the court cases outcomes/findings.

Chapter 4

Methods and Models

The 1D and 2D axisymmetrical model, general assumptions, simplifications and soil profiles are presented here.

4.1 Soil Parameters

Every clay is different, but a general model for Oslo clay was made based on parameters from (Karlsrud and Hernandez-Martinez [2013] and Karlsrud et al. [2003]).The clay $m = 12.8$, a typical Oslo clay value, is taken from Karlsrud et al. [2003]. Using Equation 2.29 and Equation 2.30: $\lambda^* = 0.078125$ and $\kappa^* = \lambda^*/5 = 0.156$. Divided by 5 is an assumption, and may not be correct. Cv is however not possible to put in a soft soil ADV model. And while an average M by excel can be calculated and the permeability changed for the different depths, the M and k will change in reality by depth and are not fixed. Since PLAXIS updates the $M = m * \sigma$ for NC and $M = M_{oc}$ for OC, the k was fixed instead. This means our Cv will not be fixed at 4 m/year. Karlsrud et al. [2003] does not mention the variation of m or Cv, but they can greatly variate from clay to clay. From section 2.2 the material parameters ($Cv = 4$) makes it a soft clay, while $m = 12.8$ a medium clay. Lowering the m further could describe soft clay better, but putting the whole clay layer as soft, is excessive for a general model. Further dividing up the clay layer into soft, medium or stiff is advised on a case by case basis where precise data is wanted. From Karlsrud and Hernandez-Martinez [2013] the $k_{clay} = 1.77 \pm 1.14\text{E}{-9} m/s$ where 1.14 is 1 standard variation, an approximately 60% variation due to the large scatter. Converting m/s into m/day for PLAXIS: 1.53E-4, 2.5E-4 and 5.5E-5 m/day. In the PLAXIS 2D manual it is stated:

> "In real soils, the difference in permeabilities between the various layers can be quite large. However, care should be taken when very high and very low permeabilities occur simultaneously in a finite element model, as this could lead to ill-conditioning of the flow matrix. In order to obtain accurate results, the ratio between the highest and lowest permeability value in the geometry should not exceed 10^5."
>
> [Bentley, 2021b]

Table 4.1: PLAXIS soil model and parameters

Description	Parameter	Dry Crust	Moraine	Clay
Soil Model		Mohr-Columb, Drained	Mohr-Columb, Drained	Soft Soil, Drained
Unit weight [kN/m^3]	γ	18	21	20
Modulus of Elasticity [kPa]	E'	5000	20 000	
Poisson's ratio	ν'	0.33	0.33	
Cohesion [kN/m^2]	c'_{ref}	1	10	0
Friction angle [Degrees]	ϕ'	25	40	30
Dilatancy angle [Degrees]	ψ	0	0	0
Permeability [m/day]	k	0.01	1	1.53E-4
Permeability [m/s]	k	1.16E-7	1.16E-5	1.77E-9
Lambda	λ^*			0.0781
Kappa	κ^*			0.0156

In the PLAXIS models the moraine (1 m/day) and clay 1 standard variation lower (5.5E-5 m/day) have a k, permeability difference of 5×10^5, putting this lower bound permeability over the recommendation above. This is the main reason to use the average permeability for clay in Table 4.1, for most calculations. However, the moraine permeability could have been reduced instead. Therefore, soft soils the k might be the lower bound, but in PLAXIS the 1.77E-9 m/s is the one most calculations are done with unless otherwise stated. Void ratio is set at a default $e_{int} = 0.5$ for all soils. It could be increased for soft soils with high clay fractions and water content. However with void ratio inputs it is recommended to use a changing permeability ck in combination the Soft Soil model. In that case the ck-value is generally in the order of the 1D compression index Cc which is = 0.27 instead of the default $ck = 1E15$. The default temperature value $20°C$ is used [Bentley, 2021b].

All layers are homogenous and isotropic $k_x = k_y$ assumed, and anisotropic permeability is not considered in either the 1D analytical solution (handcalculations based on [Janbu, 1970]) and PLAXIS. In reality it might be notably different, but for the 1D settlement analysis it is not important. The moraine and dry crust values are arbitrary chosen and can vary, perhaps most notably the permeability k. The moraine is possibly assigned a too high permeability and is more akin to silt or sand, see Table 2.3. Fully coupled flow is used, thus the option to have drained or undrained in PLAXIS is arbitrary.

4.2 Boundary Conditions and Soil Profile

These vary from case to case, but a flat terrain, soils and bedrock layers are assumed for all models. All pore pressure reductions are assumed to be permanent, but in reality will fluctuate with (time and seasons). Tunnels, steel sheet and core piles can be grouted, while energy wells and other smaller installations are most often not. The grouting can therefore re-establish the pore pressure/the ground water level (GWL) as shown in chapter 3. In Norway the infiltration is

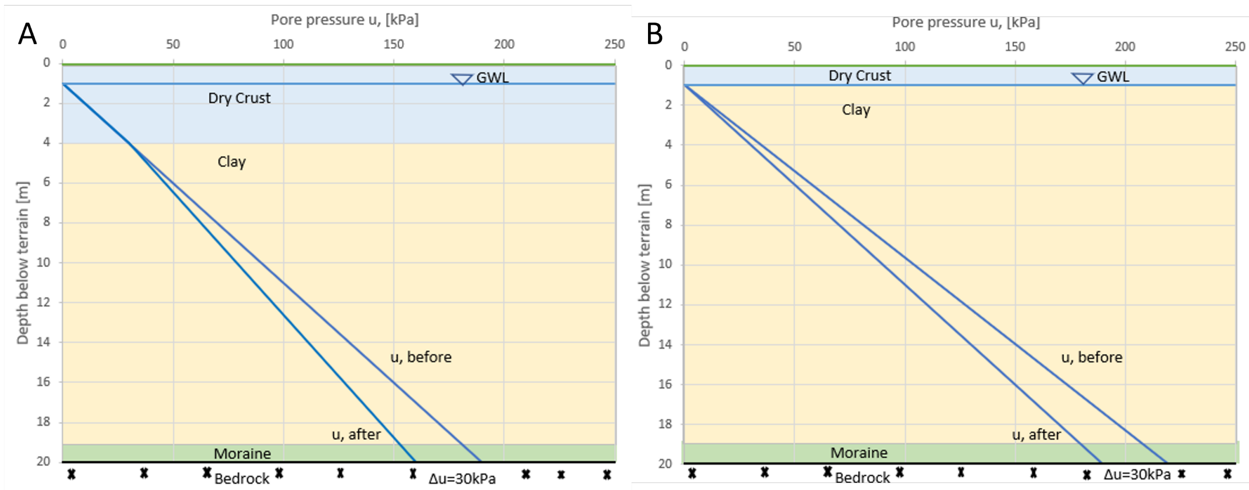


Figure 4.1: 1D model profile for 30kPa (A) below hydrostatic and (B) overpressure to hydrostatic pore pressureloss for 20m to bedrock. The moraine is replaced with clay to be more conservative in the 1D drainage calculations of settlements.

high , and on a general basis the weathered material can be assumed in the first 2-4 m below dry crust or the GWL [Karlsrud et al., 2003]. The pore pressure will thus be approximately constant at 2 m or up to 4 m below GWL due to this increased permeability in the weathered clay, see Figure 2.6. The difference effects are notably for low depths to bedrock, so while in general it might be 2-4 m, each case should be investigated if precise data is wanted. Weathered clay was given the same parameters as clay, save for $k = k_{clay} * 10k = 1.53E-3$ m/day. A silt/silty clay soil was used for pore pressure calculation with a k=2.514E-4 m/day. The pore pressure loss below hydrostatic, is named the hydrostatic scenario, while the overpressure to hydrostatic pore pressure loss is named the overpressure scenario for short.

The unit weight for dry crust chosen is 18, using instead 20 or 19 will give a lower settlement if dry crust is above the GWL. 20 used for clay, 19 or 18 will give slightly lower settlement. The precise unit weights are important for low thicknesses of clay or high dry crust thickness. Changing the $\gamma_{unsaturated}$ lower than $\gamma_{saturated}$ will give higher settlements, however for all models this has been simplified out since unless the GWL drops or the same soil exists both above and below GWL this difference is negligible. $\gamma_{unsaturated} = \gamma_{saturated}$. All models have the GWL at 1m below terrain. If it is instead at 0.5 m the settlements are higher, if it is at 2 m below the settlements are lower. This is due to the effective stress being higher, likewise a higher thickness of dry crust above gwl will give lower settlements. A GWL drawdown can certain cases happen, but is simplified out due to reasons discussed in section 2.5. While the casing/drilling might leave gaps in the soil which will increase the permeability, it is very unknown to what degree and if that effect is with respect to radius around the borehole. Are the welds or plug tight or not? The data is unavailable/unclear from the case studies about energy wells. There could be a gap around the casing, but no mentioning of it or water at terrain, or that there is significant higher settlements around the boreholes due to GWL funneling, although this could be due to a lack of data due to ongoing court cases. There is no mentioning of pore pressure above hydrostatic in case 1, 2 and 3. They show settlements around the borehole in InSAR for case 1 and 2, while case 3 had a backfill and thus the InSAR data is unreliable for the immediate vicinity of the boreholes. Different soil layering will lead to different hydraulic connections. Fractures' and faults' hydraulic properties are hard to predict, and may vary greatly. In the models groundwater head values are used due to lack of data and for simplification. Fixing heads will get the pore pressure difference reading, but stay the same both in time and pore pressure, and not vary. Thus in some models a very large outflow Q of water will happen, and this may be unrealistic on a general basis. This might also be why there are so few energywell cases, as it requires special hydrogeological conditions. Its also an assumption to put that the overpressure is linear above the hydrostatic line for the whole soil profile. The overpressure may also vary with time e.g., seasonally. Building load omitted due to the chance of over or underestimating the settlements. Old buildings can also have bad or no data regarding building load at foundation depth and type. Overestimating settlements if building loads are included but good foundation (e.g., piles to bedrock) and underestimation if they have bad foundations. In the risk map by Sundell [2019] the effect on the end result was not worth the extra computational power. It is recommended to use building load etc. for precise results when foundational data is available.

Depending on the geological history, POP or OCR should be used. OCR is chosen in the PLAXIS model as it is assumed that creep and aging dominate over changes in GWL, i.e that σ'_p increases more with depth than σ'_v. See figure 18 Karstunen and Amavasai [2015]. It is difficult and problematic to assume for a general basis, and odeometer testing for different depths of the clay is recommended to determine whether the clay is NC or OC, and if either OCR or POP should be used. NC clay (OCR=1.0) is used for the majority of the models.

The moraine layer thickness is assumed for the general case to be 1 m as shown in Figure 4.1. However, it is difficult to assume a 1m thickness in general as it will be thickest in the bedrock-topography depressions, but having a 1m moraine layer or omitting it is the more conservative approach as done in Karlsrud et al. [2003]. In the 1D PLAXIS model the moraine is not necessary to facilitate the water flow, but in retrospect perhaps it should have been included all calculations as it is necessary for 1D drainage and greater area of effect. Thus, it would be better to regard/label the results from the 1D models as in not depth to bedrock, but clay thickness affected. Similarly the dry crust in the overpressure to hydrostatic scenario Figure 4.1 (B) could have been 4m thick as (A), but this is not done to be conservative. The Δu is very small at the top, producing not much more settlement difference (unless low clay thickness) but speeds up the drainage.

4.2.1 1D PLAXIS Model

A very fine mesh was used, and the model is shown in Figure 4.3 and Figure 4.2 for an overpressure to hydrostatic and below the hydrostatic pore pressure profile respectively, for a Δu pore pressure reduction at bedrock. Utilizing a PLAXIS head will force the groundwater head given at this boundary. A permeable moraine layer is only included for first models to give pore pressure and strain curves and for the settlement calculations it was dropped to be conservative and give more accurate results when the permeability was lower bound.

The consolidation phase (Phase 1) was split up into 9 phases each with its different time interval in order to make the strain and pore pressure by time graphs. A node at terrain (x=0.5,y=0) was used to calculate the settlement vs time curves.

If the first 3 m under GWL is weathered clay or the saturated dry crust $\gamma_{saturated} = 20kpa$ the settlement of 30 kpa below hydrostatic is 7% lower, 7.42 cm instead of 7.95 cm. Tthe low Δu loss for these top layers is insignificant for medium to large depths therefore the settlement difference is low when the unitweight (both $\gamma_{saturated}$ and $\gamma_{unsaturated}$) of dry crust is kept at $18kPa$. If it is changed to $20kPa$ the difference is greater. The difference in time is however due to the way heads work in PLAXIS. Two way drainage instead of the overpressure being drained downwards, while this is also the case when when it is the same clay for the whole thickness the effect is less noticeable. See Equation 2.10 for the difference. For below hydrostatic this is not a problem as it is one way drainage and fixing the heads at hydrostatic does not change the pore pressure in this scenario. Is this realistic or not? While data gathering was attempted there are no datasets for the whole soil profile pore pressurechange by depth, so the severity of the flaw in the model is unknown. Using a BC of seepage will allow for upwards drainage, while keeping the water-volume and weight. Realistically the water will be drained away long term. This is therefore a possible flaw in the model with regard to time. This was noticed while looking at the results. Since no solution on how to do it better in PLAXIS was found, the model was kept unchanged. Doing the problem precisely/theoretical with laplace or fourier series Terzaghi [1943] or Janbu [1970]) is the only sure way for the overpressure to hydrostatic model with respect to time.

4.2.2 2D axisymmetrical PLAXIS Model

Using an axisymmetrical model makes the model a 3D cylinder. While the soil layering is for the most part kept as the 1D model above, the boundary conditions (BCs) are not. The hydraulic boundary conditions are everything in 3D [Karlsrud et al., 2003, figure 2.16]. Since the model has flat bedrock the moraine layer is flat as well. On a general basis it is difficult to ascertain the moraine extent or thickness, and as such this it is problematic to use this model on a general basis. A fine mesh is used with depth D=20 m and radius R=50 m. Choosing different R/D effects the result, but a large R/D=5/2 was built making the model axisymmetrical around left side and radius extended 50 m out as shown in Figure 4.4. This outer boundary at r=R is referred to as the right boundary. The head at the bottom left is 2 m wide and the right head is 1 m wide as shown. No right head results in an approximately 1D drainage as expected. Especially long term due to the moraine layer draining all the clay. The bottom left head width and value, right head height/length and the soil layering is tweaked. E.g., the top 3 m of clay is weathered in some hydrostatic models, increasing the permeability $k = k \times 10$ being the only soil parameter change. Increasing the dry crust thickness to 4 m or applying the right head=-1 m for the entire

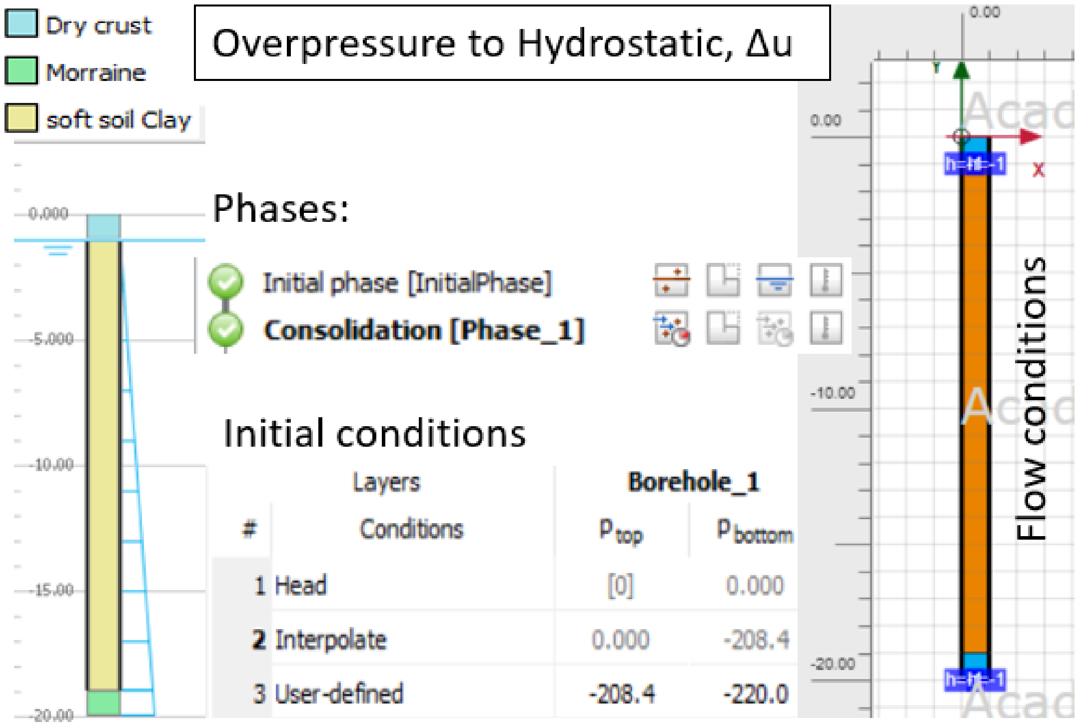


Figure 4.2: Overpressure reduced to hydrostatic PLAXIS model setup for 1D drainage, here shown a 30kPa loss and 20m

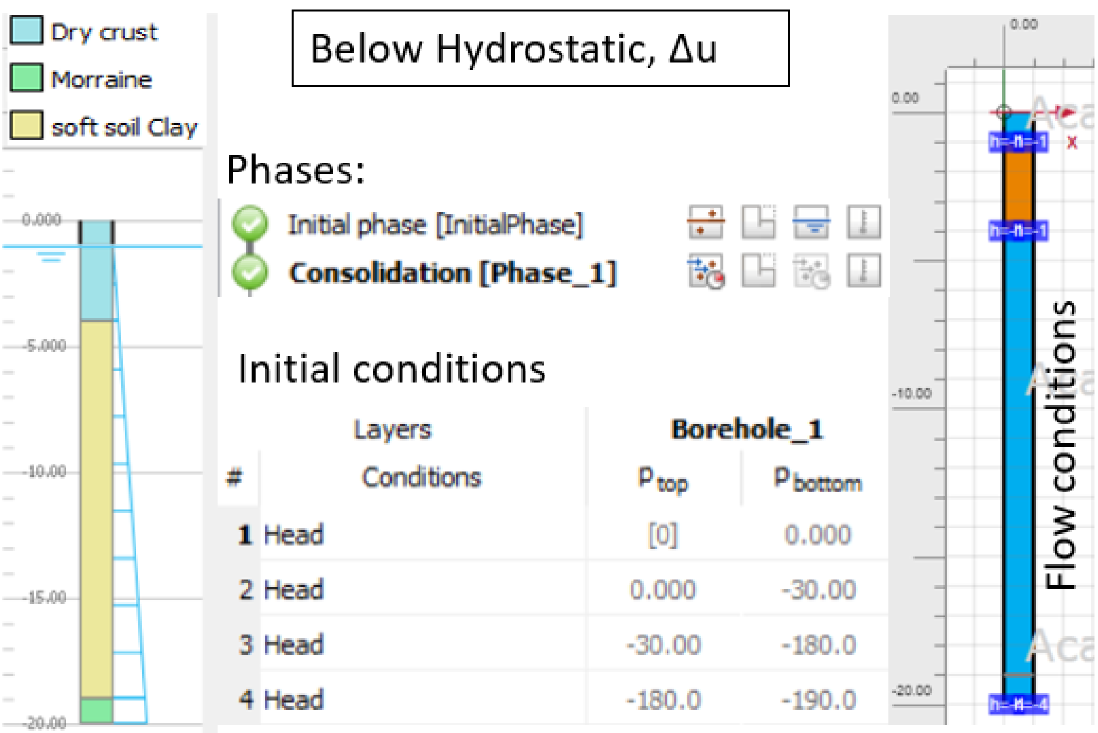


Figure 4.3: Below a hydrostatic pore pressure PLAXIS model setup for 1D drainage, here shown a 30kPa loss and 20m

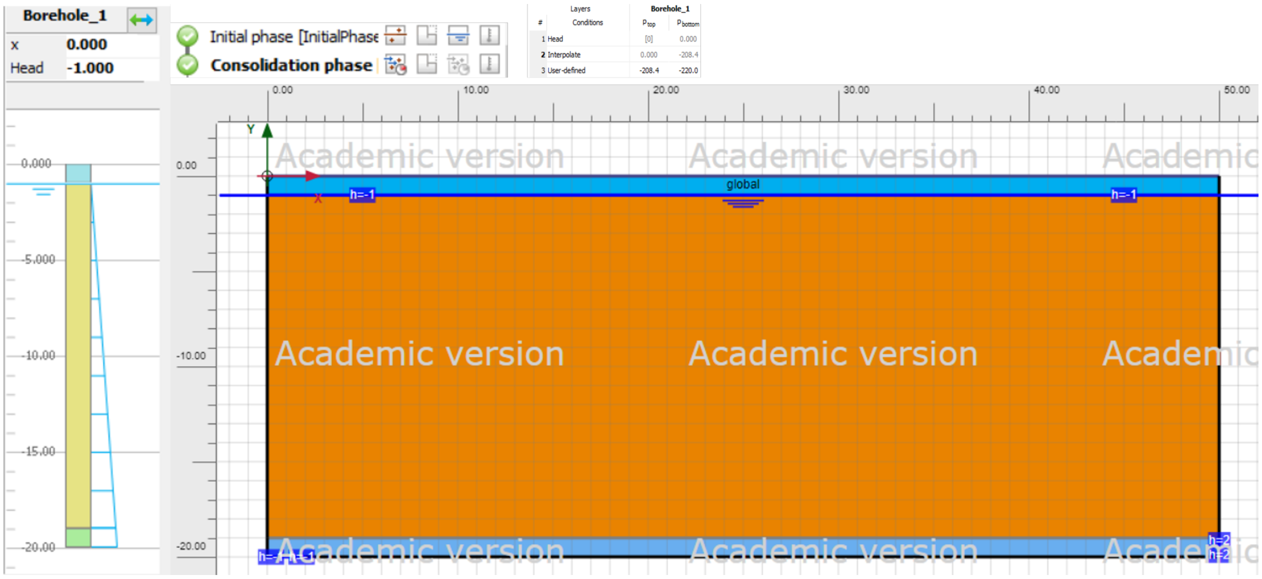


Figure 4.4: PLAXIS 2D Axisymmetrical model setup. Here shown 30 kPa overpressure to hydrostatic pore pressure with bottom left head-width of 2 m

right is also done for some models.

For the overpressure to hydrostatic model one has to keep the overpressure on the bottom right side, as the head=2 m extended at the whole right side makes the model incorrect due to the GWL/head at the depth, z=-1 m. Putting heads of different values next to each other gives a problematic model, therefore keeping the overpressure at 1m width for just the moraine layer was done. For an initially overpressurized system (e.g., artisian moraine aquifer) the overpressure can indicate a significant permeable layer resulting in that the right hand boundary is assumed more likely to have inflow such that a head or Q value can be assigned here. Whereas a moraine with initial hydrostatic pore pressure may be more smaller and more constricted by solid bedrock and the boundary condition for this layer can be closed. This is due to reasons discussed in section 2.5. On the other hand a pore pressure reduction due to drainage can be easier to do in a confined aquifer as denoted by the storage coefficients discussed in section 2.3. The bedrock-interface (r=2-50m) is modeled solid/impermeable (closed BC), however, there could be fractures or faults still causing some inflow/outflow of water.

A model with a permeable 2 cm layer on the outside of a casing of 12 cm radius is presented in Figure A.7. Note that this is twice as big a casing compared to the normal casing for a energy well in order to make the 2cm volumetric area bigger. This model is to see the hydraulic perforation of layers does, for overpressure leakage and GWL drawdown through the permeable layer with k=1m/day. There is no found case of overboring the whole soil profile, but there are cases of overboring in quick or soft clay and silt/moraine.

4.3 1D Analytical Solution

The hand calculations are done by using Microsoft Excel. This is based on 1-Dimensional consolidation theory equations from Janbu [1970], presented in section 2.1. The problem is, or is approximated into 1D drainage. Such approximations due to pore pressureloss can be done to an "accurate" level if parameters are known. The excel tool provided can handle more then used in this work, e.g., POP, strength increases with depth, GWL lowering and more. This is a tool the industry can put the put in the registered pore pressure reduction into and calculate the settlements at t_p. section 2.2 can be referenced to when changing the material parameters. The time-settlement curves needs user input for a manually check of the output, and may be too difficult for drillers or too simple for geotechnical engineers who would rather use geosuite or PLAXIS. Additionally the water recharge, heave and pore pressure history is something to consider when looking at the results. For the calculation of the results the soil profile and boundary conditions are the same as the 1D model in PLAXIS.

The material modulus m, see Table 4.2, for the soils other than clay are chosen are based on Figure 2.3 and section 2.2. With unit weights the same as in PLAXIS. Do note that the material modulus of moraine can be above 1E4 and that the a=0.5 is a simplification. Thus the moraine layer is conservatively chosen to have a much lower m-modulus than it perhaps should. On the other hand its main function is being a coarser grained permeable layer and since its settlement contribution is negligible already, increasing the m further has approximately no effect. Figure 4.1 shows the general soil and pore pressure profiles used, where the depth and Δu and thus pore pressure profile will change. The moraine layer is dropped in the calculation model as its contribution to settlements is negligible and is instead replaced with clay for the calculations, to be on the conservative side as mentioned in section 4.2. The $\Delta\sigma'$ is calculated by the final Δu loss based on the final pore pressure profile e.g as shown for 30kPa in Figure 4.1.

Table 4.2: Material modulus and unit weight for the soils in the hand calculation with 1D analytical solution

	Dry Crust	Clay	Loose Sand	Firm Sand / Moraine
m	20	12.8	120	220
γ [kPa]	18	20	20	21

Chapter 5

One Dimensional Drainage Results

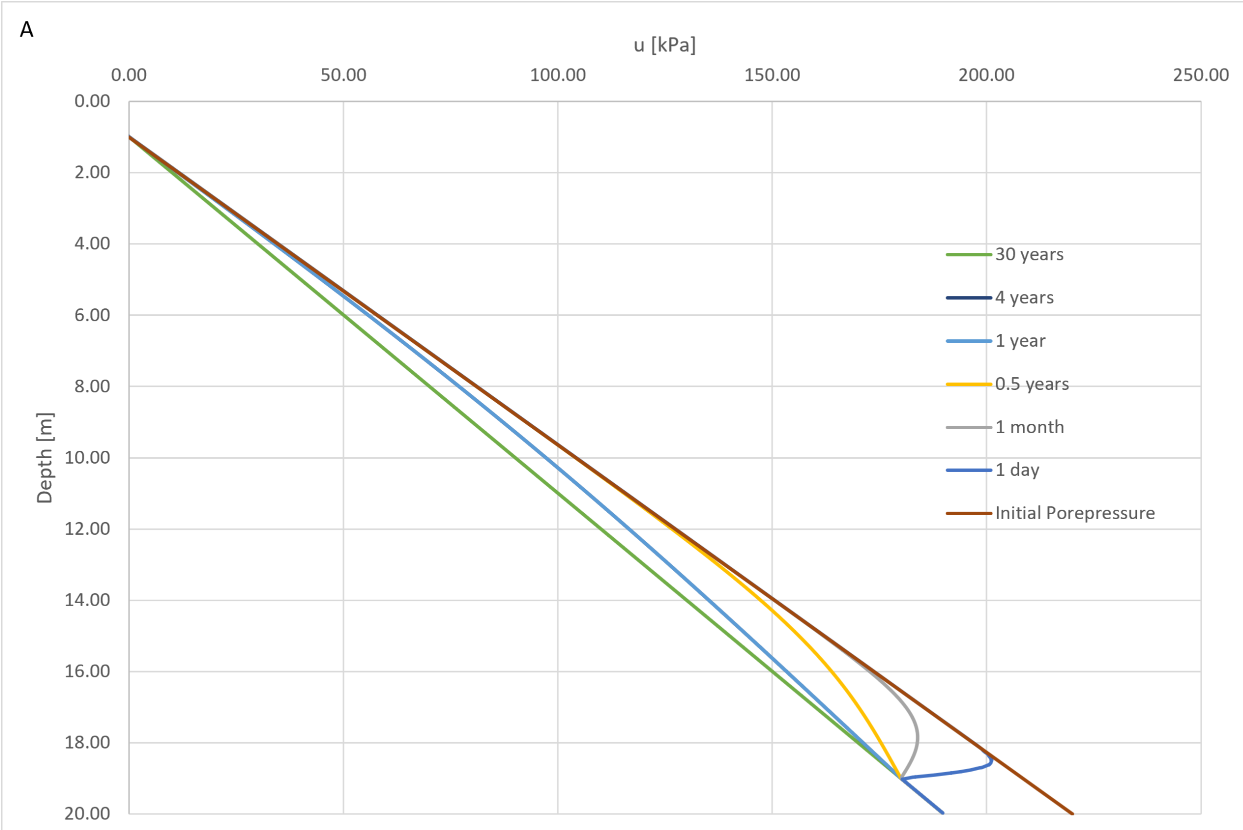
The results from the analytical solution and numerical analysis for 1 Dimensional drainage and consolidation are presented here.

5.1 1D PLAXIS and 1D Analytical Solution

Figure 5.1 a soil profile of 1 m dry crust, 18 m clay and 1 m moraine. The Δu of 30 kpa is immediate in the permeable coarse layer, here assumed stiff as firm sand/moraine, whereas the clay takes a significant time to reach its final linear position. In Figure 5.2 the increase in OCR=1.2 shows not much difference between 10 and 20 years, and using a softer $\kappa = \lambda/2$ with a tip (max point) at 0.004 showing also no significant difference between 10 and 20 years.

Since the ϵ_1 charts shown in Figure 5.3 are different for different pore pressure profiles the curves shown in Figure 5.4 will vary. They keep the same general shape, but having very low thickness of clay or a higher material strength like OCR will move the middle part especially towards the bottom left corner. Removing the forced 4 m hydrostatic head and replacing the dry crust gives the approx. same $T_p - U_p$ chart as the overpressure loss. Increase this depth at what the infiltration rate keeps the GWL fixed and the settlements and of the middle part of the time curve moves to lower left. Increasing the overpressure loss to 60 kpa or changing the k value does not change the overpressure curve, having more weathered clay or dry crust speeds up the settlements, putting it closer to the below hydrostatic line due to a lower h Equation 2.9. The Larsson et al. [1994] figure 26 shows more $T_p - U_p$ curves for comparison to other excess pore pressure drainage profiles. Figure 5.5 shows Figure 5.4 over Figure B.7, both being between curve A and B initially, before dipping sharply as the final 50-40% of the settlements are more long term. Replacing the dry crust with clay such that the 30 kpa below hydrostatic is linear to GWL instead of the bottom part of weathered clay/dry crust gives an approximate same $T_p - U_p$ graph.

These time-settlement curves are calculated in PLAXIS 2D. PLAXIS has solved it numerically, and the time to T_p may be an approximation. When calculating the T_p from PLAXIS results,



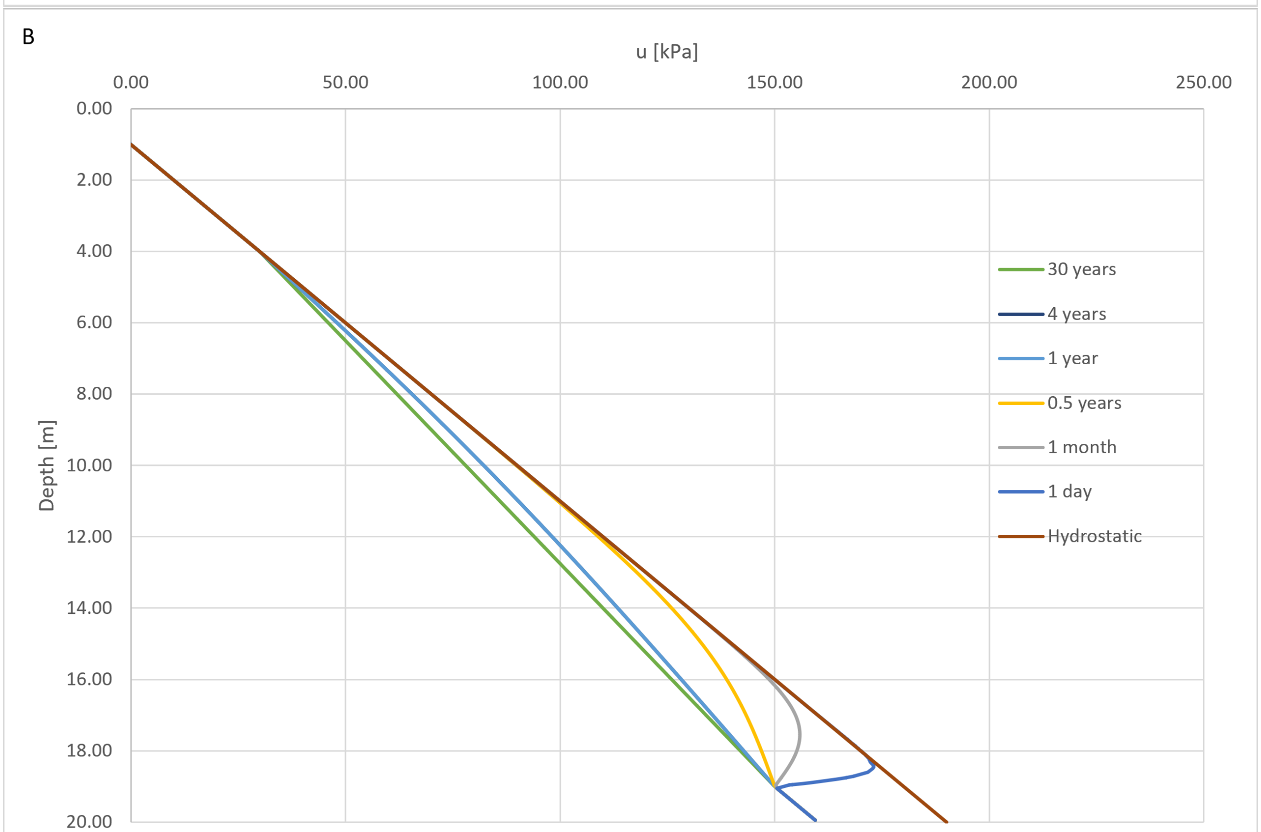


Figure 5.1: pore pressure vs. depth. 30 kpa overpressure at bedrock to hydrostatic (A) and 30 kPa below hydrostatic pore pressure at bedrock (B) for k=1.53E-4 m/day

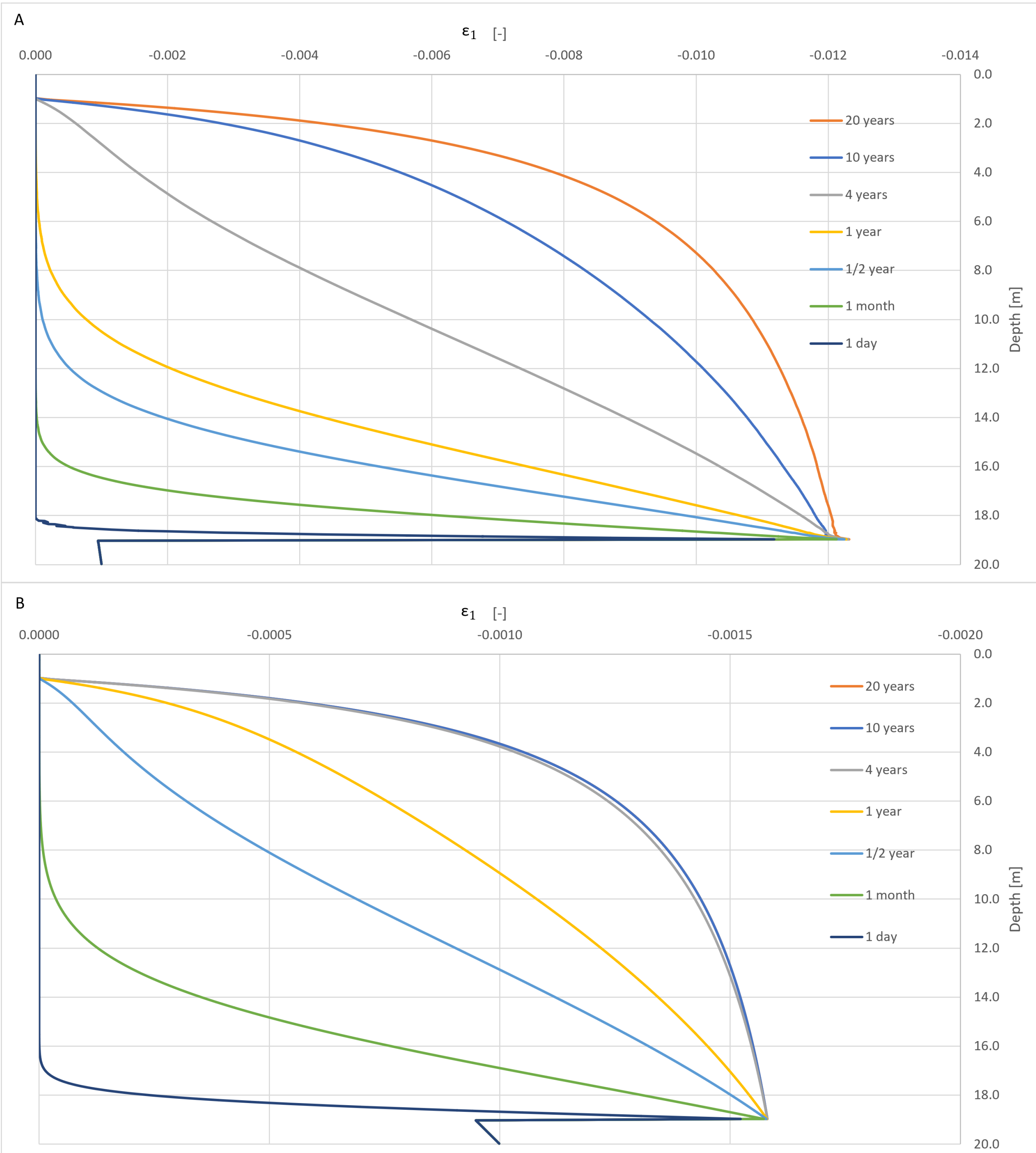


Figure 5.2: Primary (vertical) strain vs. depth. 30 kPa below hydrostatic pore pressure at bedrock. OCR=1 (A) and OCR=1.2 with $\kappa = \lambda/5$ (B) for k=1.53E-4 m/day

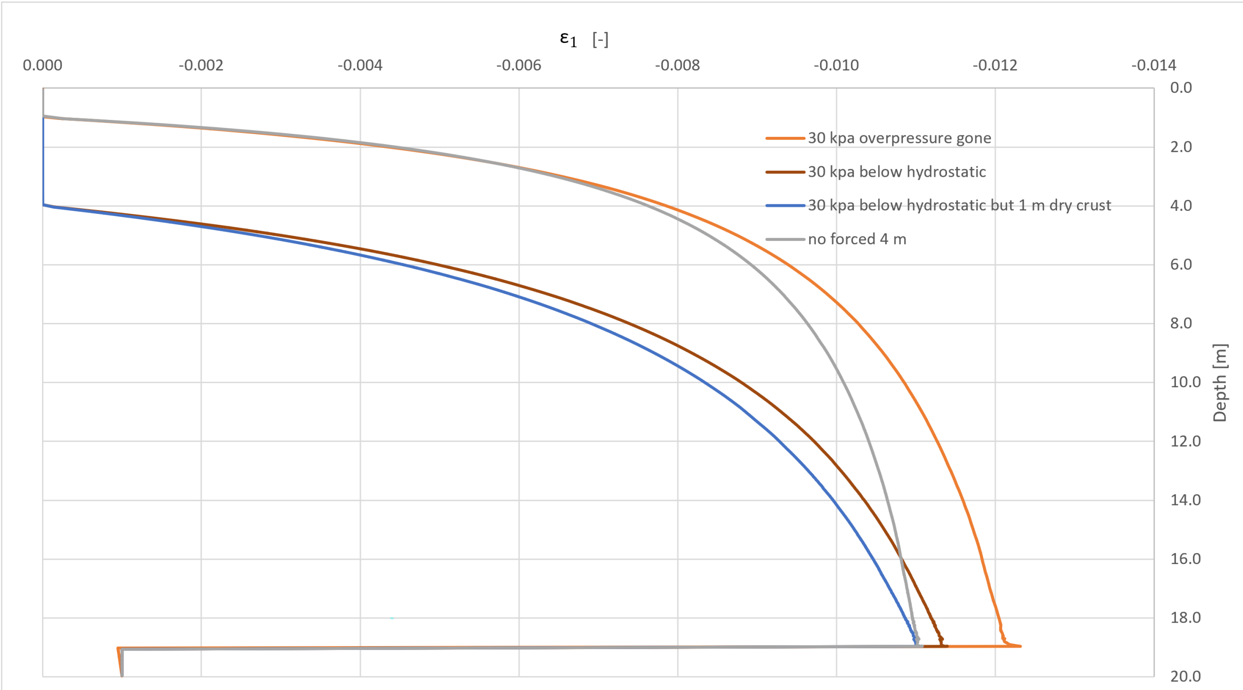


Figure 5.3: Primary (vertical) strain vs. depth. 30 kPa from hydrostatic pore pressure and overpressure with different soil thicknesses

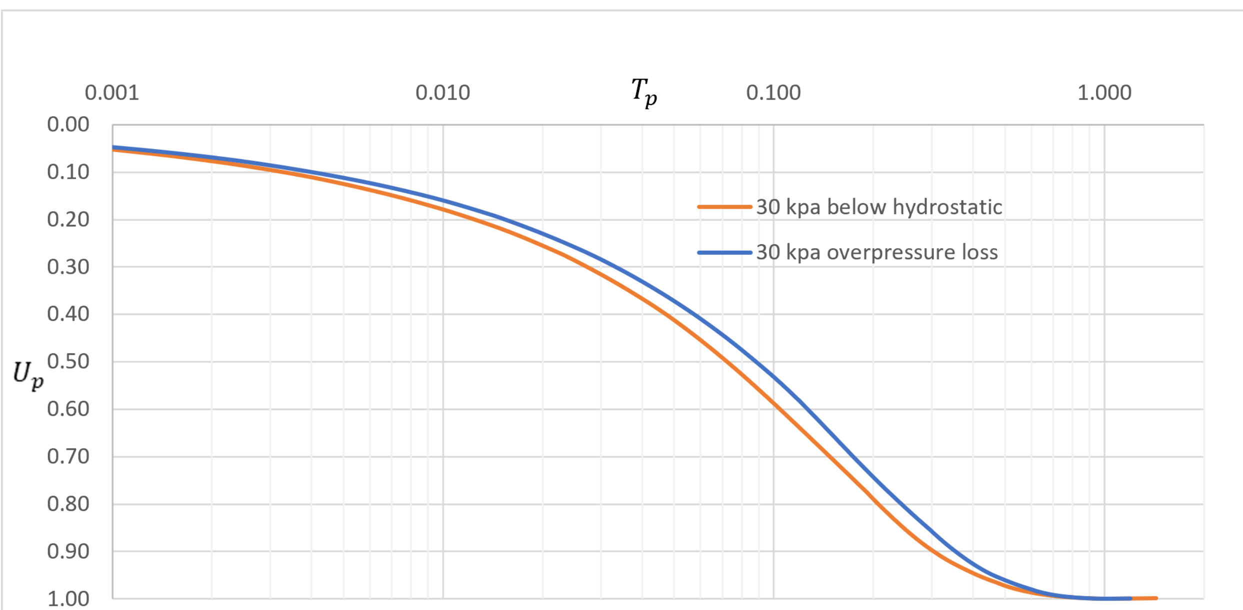


Figure 5.4: Primary time factor vs. Primary degree of consolidation. Settlements vs time in clay with a loss of 30 kPa from hydrostatic pore pressure 15 m thickness or overpressure to hydrostatic 18 m thickness.

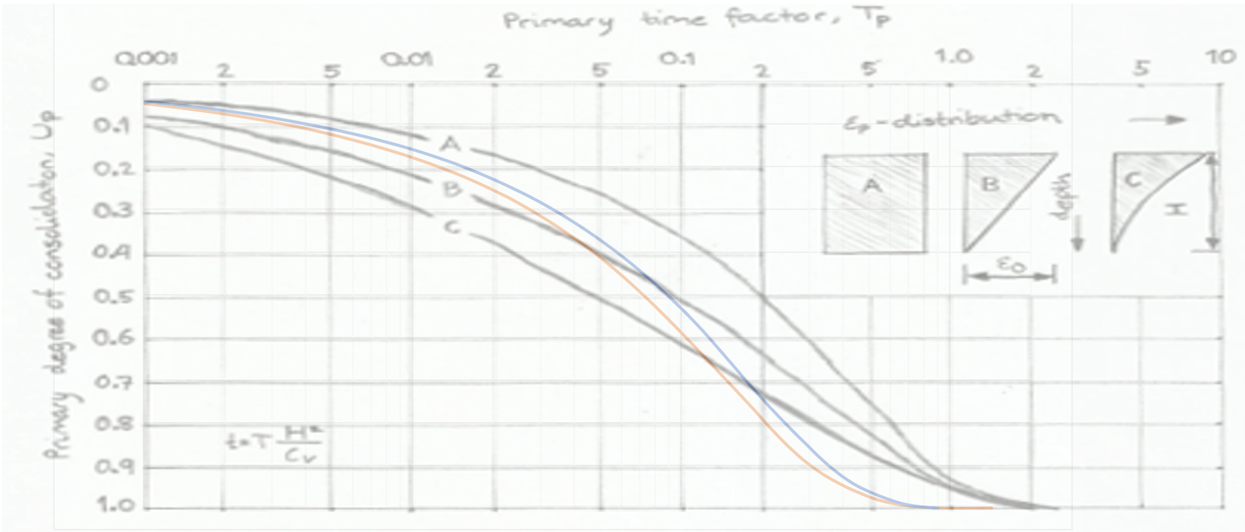


Figure 5.5: Primary time factor vs. Primary degree of consolidation. 30 kPa below hydrostatic pore pressure (orange) and 30 kpa overpressure to hydrostatic (blue) at bedrock.

the U_p and T_p must have several points before and after closeby, so that the inflection point is accurate. The settlement curve reaches a high point before going very very slightly down. When the settlement difference (between max and adjacent points) is ≥ 0.1%, when dividing t/T_p and U/U_p, the difference can offset the graph slightly and resulting in an erroneous result.

Settlements in Figure 5.6 are for the long term, when $t = T_p$. The settlements for overpressureloss are calculated in PLAXIS and checked by the 1D analytical solution. Figure 5.7 is calculated from 1D analytical solution and only checked in PLAXIS for 10,20,30 and 40m depths. The difference was 0.5-2% between the PLAXIS and 1D analytical solution. Notice how a hydrostatic pore pressure loss is much more dependent on depth than overpressure loss. However, a higher overpressure can be more likely with higher depths and in depressions. This is why the 0-5 m interval is not calculated Figure 5.6. For the hydrostatic Figure 5.7, the 0-5 m is omitted due to the fact that a high pore pressure reduction so close to terrain might lower the GWL and give higher settlements.

The drainage of a more permeable layer of sand or moraine, thicknesses of which are generally low, have higher m modulus such that the settlements are insignificant relative to clay, even though they are much more rapid in this layer. The stress can distribute gradually upwards and outwards, and thus a compaction of a deep layer of more permeable silt/moraine above bedrock might not be felt at terrain-height. Drainage in the moraine layer gives more immediate but much lower settlements due to high m-modulus, see Table 4.2. But since the Δu is highest at bedrock level, a thick sand or moraine layer can give significantly less settlements relative to if it were clay instead. For instance a 30 kpa drawdown below hydrostatic in a 10 m depth to bedrock (4 m dry crust, 6 m clay) will give 8 cm settlements, but if the bottom 2 m are moraine or loose, sand the settlements are 4.4 cm or 4.6 cm instead. In a 30kPa below hydrostatic for

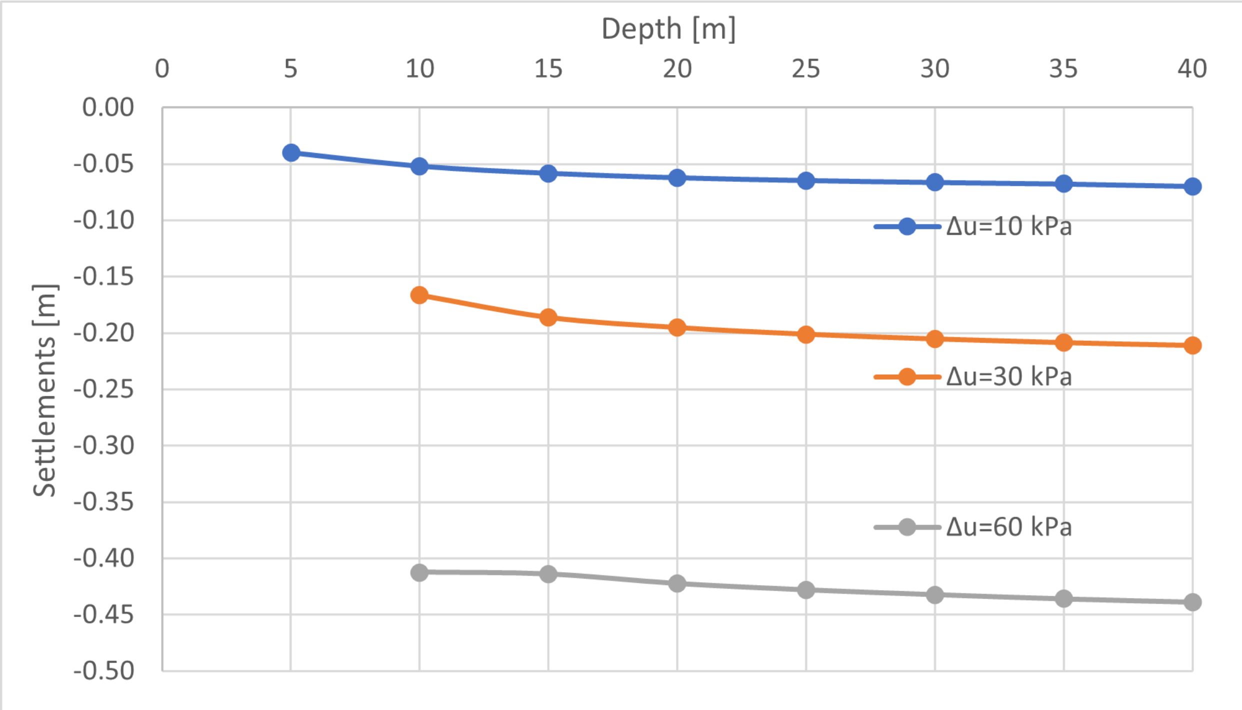


Figure 5.6: Vertical ground settlements u_y vs. depth. Overpressureloss to hydrostatic.

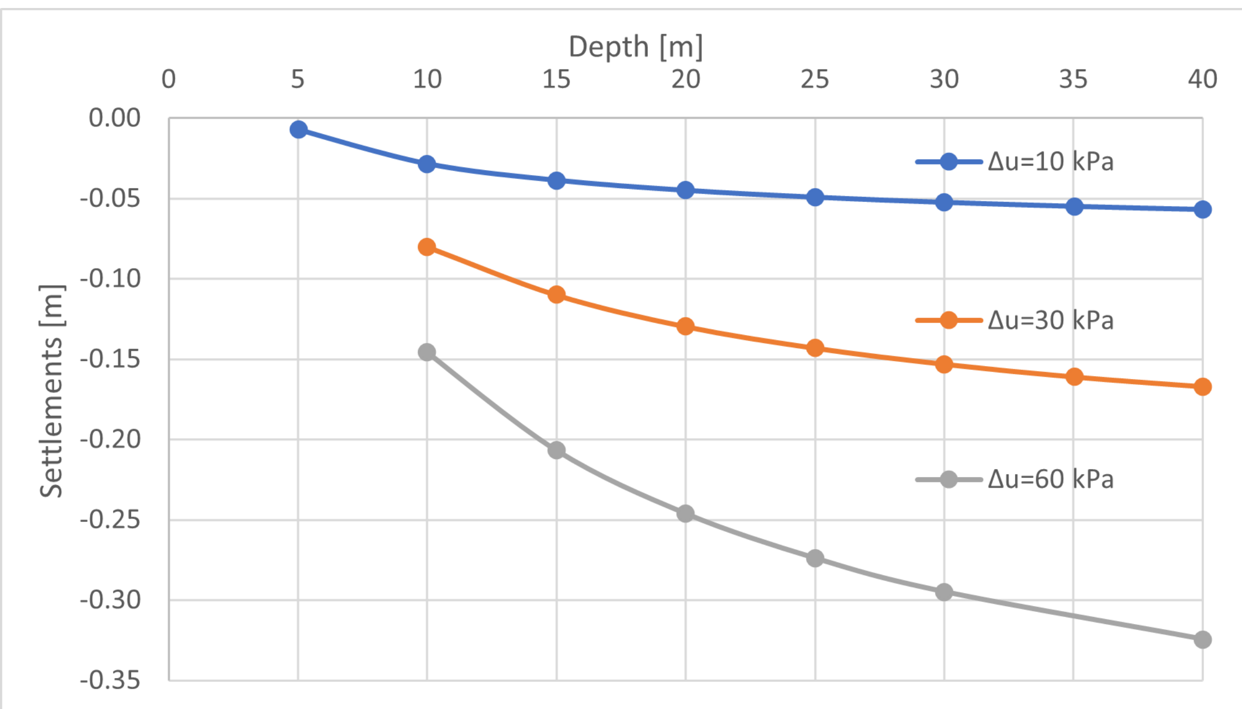


Figure 5.7: Settlements u_y vs. depth. pore pressureloss below hydrostatic. A 60 kpa loss is thought of as too extreme due to energywells, but is included for good measure

20 m to bedrock it is 12 cm with moraine and 13cm with clay all the way instead. Figure 5.6 and subsequent figures have this moraine/ more permeable layer omitted to be on the conservative end. Having more inflow or weathered clay moves the fixed GWL lower. By forcing the hydrostatic pore pressure (i.e adding weathered clay) to a depth of 6m or 8m gives 4.8 cm or 2.2 cm instead. See Figure 5.8 for a graphical 30 kpa pore pressure reduction at bedrock for a 10 m thick soil profile with variable GWL, dry crust and permeable weathered clay. Getting the right GWL and pore pressure reading is the most important variable for an overpressurized system, in order to get accurate calculations. A m=6 instead of 12.8 gives 26 cm instead of 13 cm for 30 kPa below hydrostatic. As the strain equations in section 2.1 show, a doubling of the material strength, results in cutting the settlements in half. It is recommended to use the digital excel product (1D analytical solution) to get the more precise calculation of soils or clay with soft soil layer(s).

For an overpressure to below hydrostatic pore pressure reduction, i.e. both at the same time the settlements can be added but the time-settlement curve is slightly different. Time to T_p is approx. the same as the overpressure case, the longest of the two. An example is presented by Figure A.3.

Building load applied at terrain in the calculations did not change much unless B and q is high and the thickness is low. The pore pressure change is usually uniform under a foundation unless its large [Karlsrud et al., 2003]. Transitions between buildings however give more cause for concern for differential settlements resulting in damage. E.g buildings in soft clay without foundation to bedrock and lots of transitions. If compensated foundation was not done, the stress history of soil should also be taken into account. Foundation to bedrock or piles to bedrock can make the building much more resistant to settlements. Further recording of differential settlements and their affects on buildings could be a topic for further work.

5.2 Settlements vs Time

Figure 5.9 (in Figure A.2 with logarithmic time), Figure 5.10 and Figure 5.11 show quicker settlement for lower depths, but in the long term the settlements are larger for deeper clays. Even more so for lower permeabilities. Interestingly the differences between the depths narrow more for overpressures than below hydrostatic with increasing overpressure at bedrock. This is also seen in Figure 5.6 and Figure 5.7. Thus the thickness of clay in a below hydrostatic scenario can be more important, than for the overpressure to hydrostatic assuming the same Δu loss at bedrock. The depth at what the weathered clay is at to has much to say in both below and to hydrostatic scenarios. The weathered clay in an overpressure scenario does not decrease settlements, but lowers T_p, shown in Figure 5.12 due to the drainage being expedited.

Even a lightly OC clay has significantly lower settlements as shown in Figure 5.12 and Figure 5.13. A OCR above 1.5 (given a $\kappa = \lambda/5$ or lower) will relative to NC clay have insignificant settlements. Due to reasons discussed in section 4.1 the moraine layer is gone after Figure 5.5. Using the

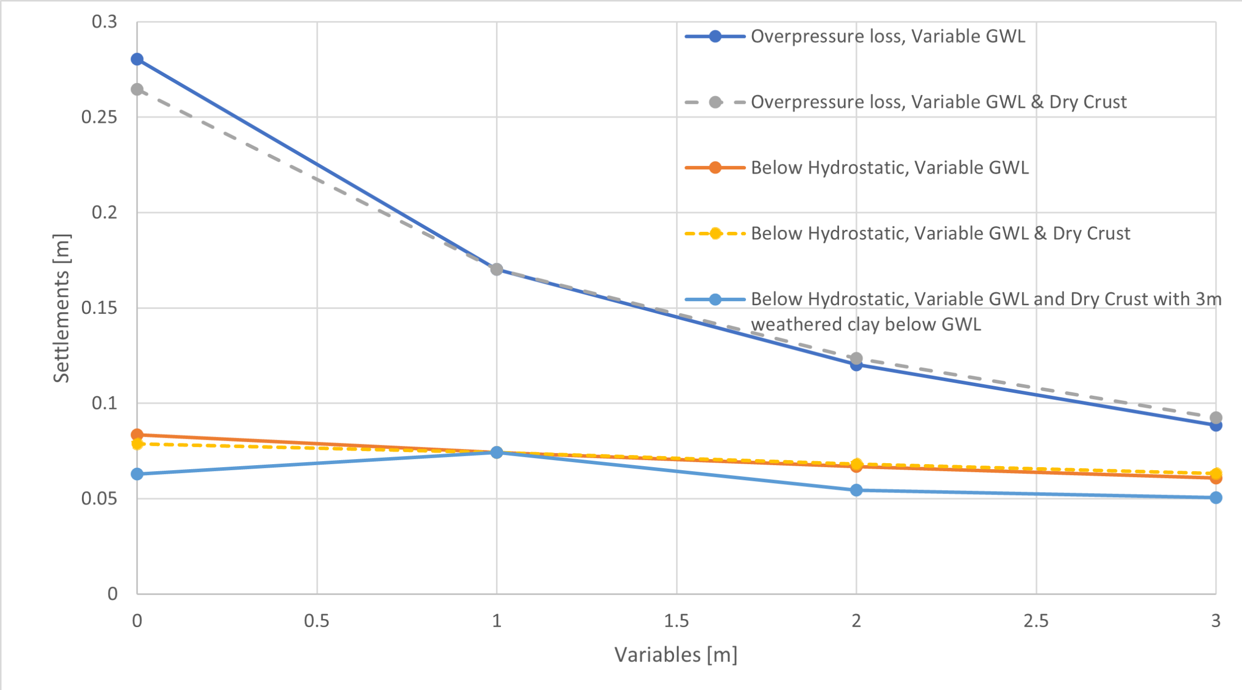


Figure 5.8: Variable GWL, dry crust and/or weathered clay in depths below terrain vs. Settlements u_y for 30 kPa pore pressure reduction at bedrock in a 10 m depth profile. Results from 1D analytical solution.

lower bound permeability in clay gets closer to Oslo clay $Cv = 4m^2/\text{year}$. A Cv table is shown in Figure A.1 for Cv by depths and pore pressure reductions.

Changing the m-modulus in PLAXIS such that $\lambda = 1/5 = 0.2$ (i.e a softer NC clay) results in an approx 12.8/5=2.56 increase in settlements and time as shown in Figure 5.14. As m is in the M in Equation 2.10 changes the time. Softer clays have generally lower k, further lowering the time, but Soft clays can differ or even have lower k and m than used here. Below hydrostatic act very akin to overpressure to hydrostatic pore pressure reductions, with regard to OCR increases. A very large long term pore pressure reduction is required in high OCR clays to give significant settlements, assuming medium to high M_{oc}, and that seems very rare in ground works other then tunnels, as no data of such can be found. But this can also be explained by the lack of data.

5.2.1 1.5 and 1 Way Drainage

1D in the overpressure dissipation scenario, the head=-1 m at GWL will drain the clay semi-two way, upwards and downwards due to the head at the bottom and top at -1 m. The overpressure is dissipating if not fed from the bottom. The head=-1m at GWL and head=2 m at bedrock in PLAXIS gives a linear above hydrostatic pore pressure profile long term same as the initial given linear line. This means that when comparing the settlement vs time the height affected, H in Equation 2.9, will fit for $H * 0.75$, i.e. halfway between two and one way drainage. An example with 20 m of clay, GWL at 1m and 30 kpa overpressure drawdown at bedrock, shown in Figure 5.15, requires $H * 0.75 = 14.25m$ to get the same result as PLAXIS. $M = \sigma'_{avg} * m =$

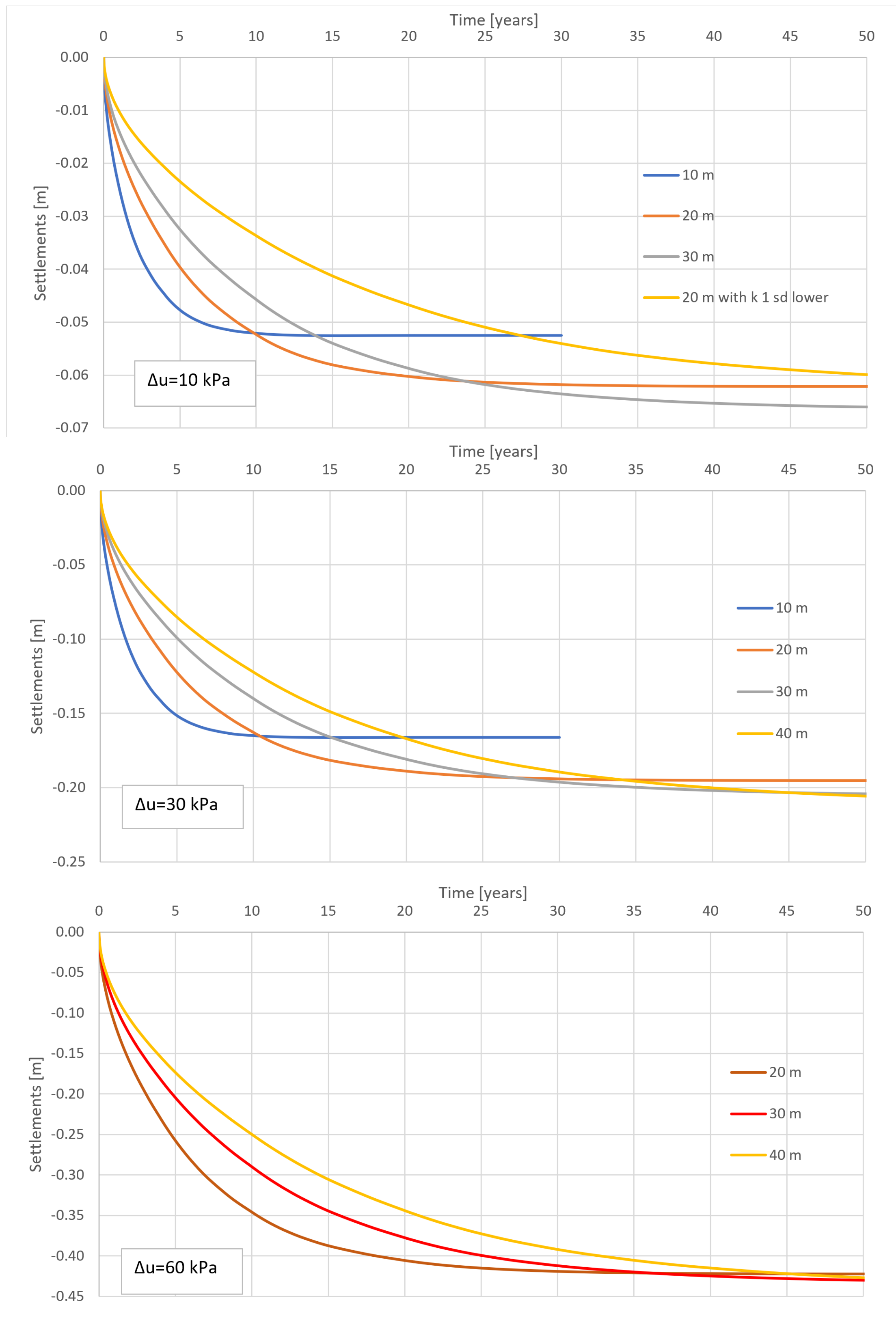


Figure 5.9: Time vs. settlements u_y. Overpressure to hydrostatic pore pressure loss for different depths. k=1.53E-4 m/day and k 1sd lower=5.5E-5 m/day

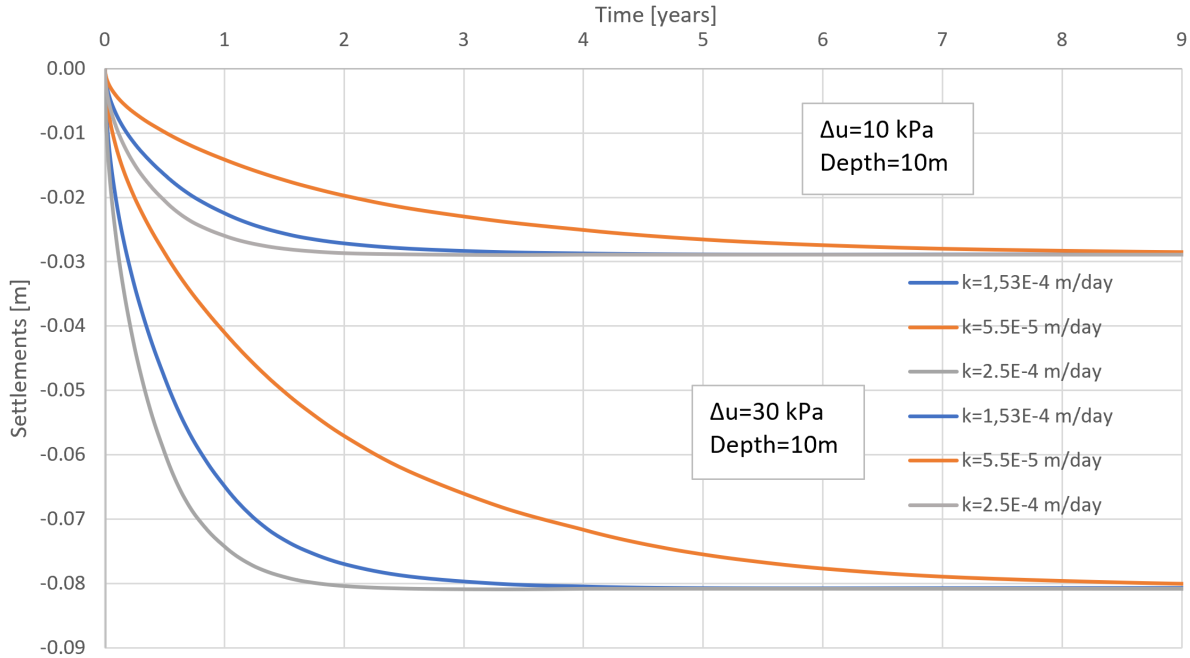


Figure 5.10: Time vs. settlements u_y for below hydrostatic and different permeability values $k = 1.77 \pm 1.14\text{E}{-9}m/s$.

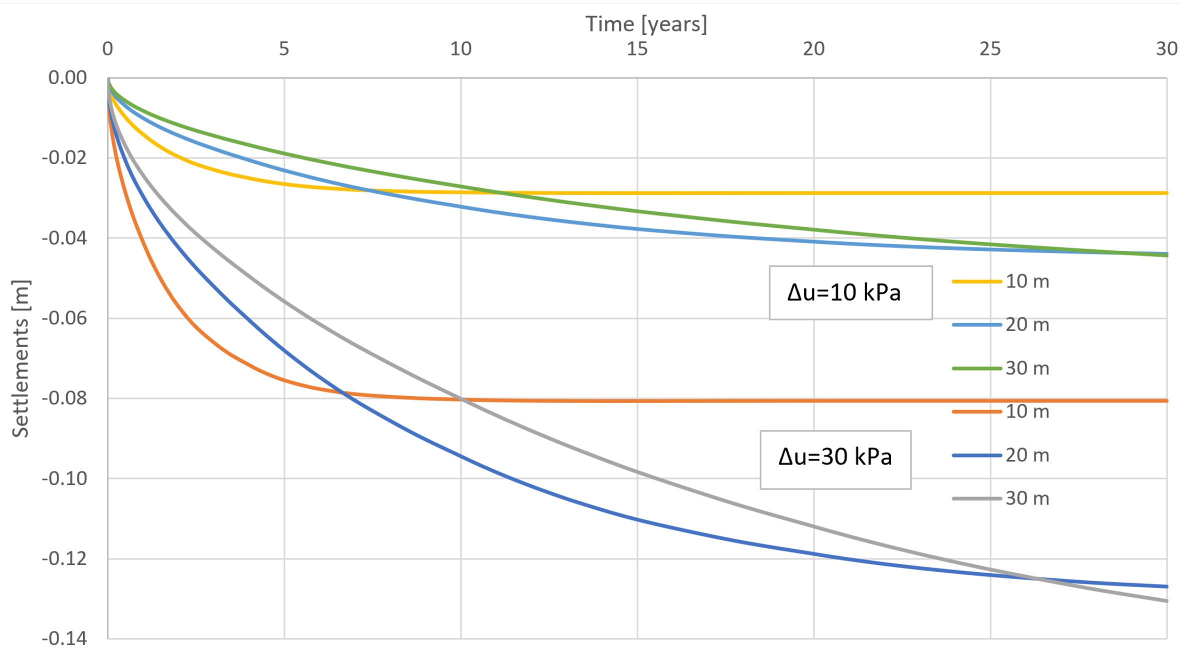


Figure 5.11: Time vs. settlements u_y for 30 kpa below hydrostatic at bedrock/bottom of clay layer with a lower k, k=5.5E-5 m/day.

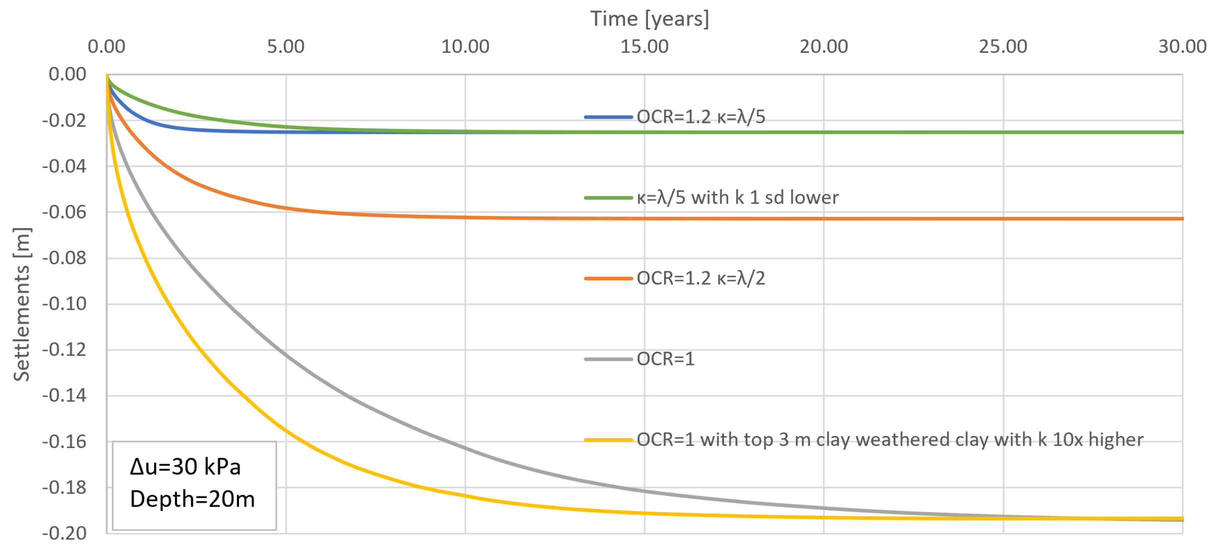


Figure 5.12: Time vs. settlements u_y for different OCR values. 30 kpa overpressure to hydrostatic at bedrock/bottom of clay layer.

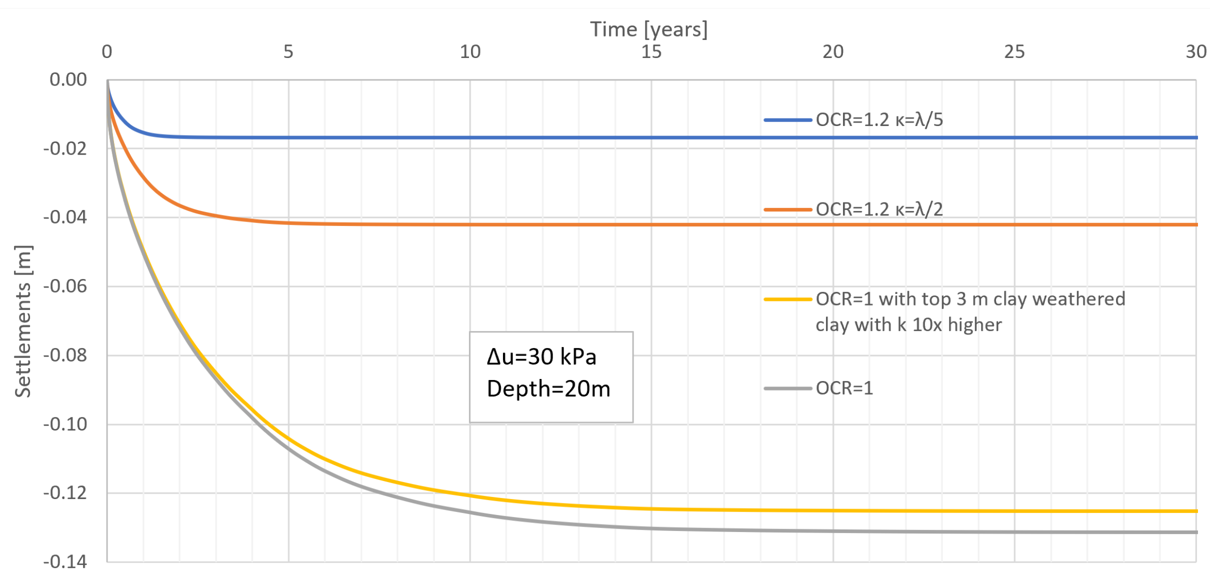


Figure 5.13: Time vs. settlements u_y for different OCR values. 30 kpa below hydrostatic at bedrock/bottom of clay layer. k=1.53E-4 m/day

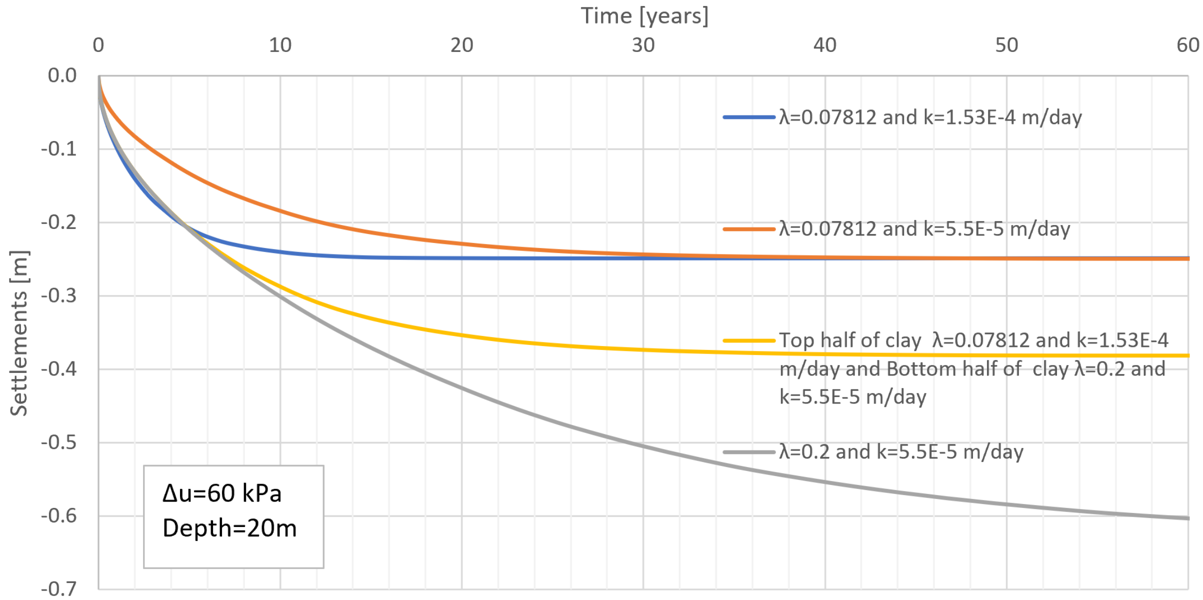


Figure 5.14: Time vs. settlements u_y for different compression index λ values. 60 kpa below hydrostatic at bedrock/bottom of clay layer. 4 m Dry crust and 16 m clay with GWL at z=1m.

$0.5(95+110)12.8 = 1.3$ MPa results in a Cv of 7.3 m^2/year by Equation 2.10. Adding more points and smoothing the curve would give an even better fit. Figure 5.5 is used to give the $U_p(T_p)$ values.

Since the overpressuized 10 kPa system has more pore pressure-area $\Delta u \times H$, where H is thickness affected (19 and 16 m for overpressure and hydrostatic respectivly) to drain, it produces higher settlements. In the hydrostatic scenario putting the hydrostatic forced depth to GWL (–> H=19) will increase the settlements and give the same pore pressure-area as the overpressure scenario, but the settlements will be less, due to higher σ'. Using section 2.1 theory, a OC clay the $\Delta\sigma'$ is same =30 kPa for both hydrostatic and overpressureized systems Equation 2.5, while NC clay Equation 2.6 the $\frac{\sigma'}{\sigma'_0}$ ratios are different. For OC into NC stress (i.e $\sigma'_0 < \sigma'_p < \sigma'$) these ratios are also different, see Equation 2.8. the pore pressure-area ratios are not the same and dependent on the thickness subjected to $\Delta u = \Delta\sigma'$. Increasing the δu lowers the ratios, but increases the difference between the hydrostatic and overpressure area ratios, while lowering the δu does the opposite. Changing instead the hydrostatic pore pressure forced to a height 4 to 2 m lowers this ratio, increasing the difference. Doing the opposite increases the difference also, in fact a forced depth/dry crust or weathered clay to 3.6 m has the same area ratio as the overpressure scenario. This is almost seen for the 10 m depth profile in Figure 5.8. Overpressure to hydrostatic is not quicker due to these increases in settlements, even with a 1.5 way drainage as opposed to 1-way.

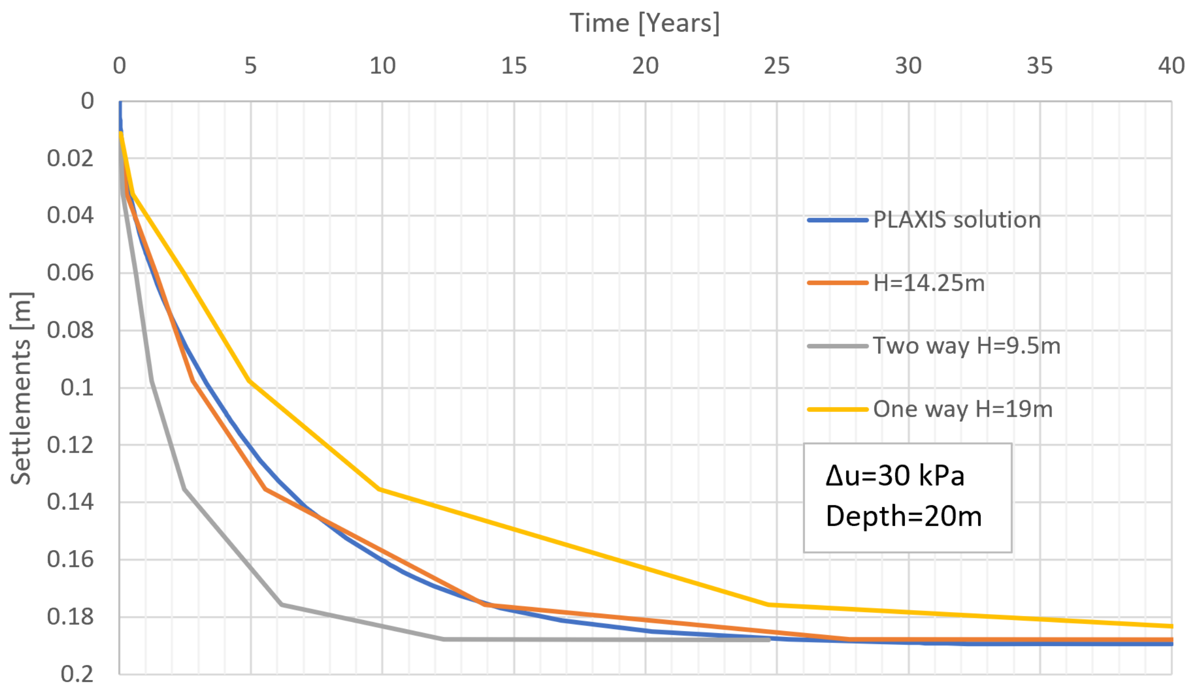


Figure 5.15: Time vs. settlements u_y for 30 kPa overpressure to hydrostatic 20 m clay, PLAXIS output compared to the 1D analytical solution with different drainage ways.

Chapter 6

Results of the 2D Axisymmetrical Numerical Models

The results of the numerical analysis for the 3D dimensional drainage models in PLAXIS 2D are presented here.

6.1 2D Axisymmetrical PLAXIS Results

In order to map the area of influence/effect and find out the parameters of note, an axisymmetrical model in PLAXIS 2D is used with a fine mesh. Using the x axis as radius from the left centre line, calculates a 3D cylindrical (360°) result.

The influence area or area of effect, depends greatly on the boundary conditions and the hydraulic conductivities. Water rechargement (how much water can inflow into the model), or if it is a perched groundwater is very difficult to ascertain on a general basis. Thus the differential settlements are highly dependent on the hydraulic boundaries, the radius out to them and presence and extent of a more permeable layer above bedrock (e.g., moraine).

$k_x = 2 \times k_y$ for the clay does not show much difference when there is a closed right boundary as expected, only in the very short term. A closed right boundary approximates this into 1D. In the overpressure model, the $k_x = 2 \times k_y$ shows a difference compared to $k_x = k_y$, but then the right hand boundary in the clay should perhaps be given or a very large model be used. $k_x = k_y$ is used for all following results shown.

Pore pressure reductions below hydrostatic there is no noticeable difference between dry crust or weathered clay in the top layer. If this top permeable layer is excluded the "circle" is longer (closer to terrain) in the depth axis due to the lower permeability. If the clay layer has lower permeability k, the "circle moves to the right as the right BC "feeds" the system more. Keeping the hydrostatic pore pressure constant at 4 m by 1 m dry crust above GWL and 3 m with weathered clay below GWL, 15 m clay and 1 m moraine resulting in Figure 6.1. Heads are here h=-1 m at top

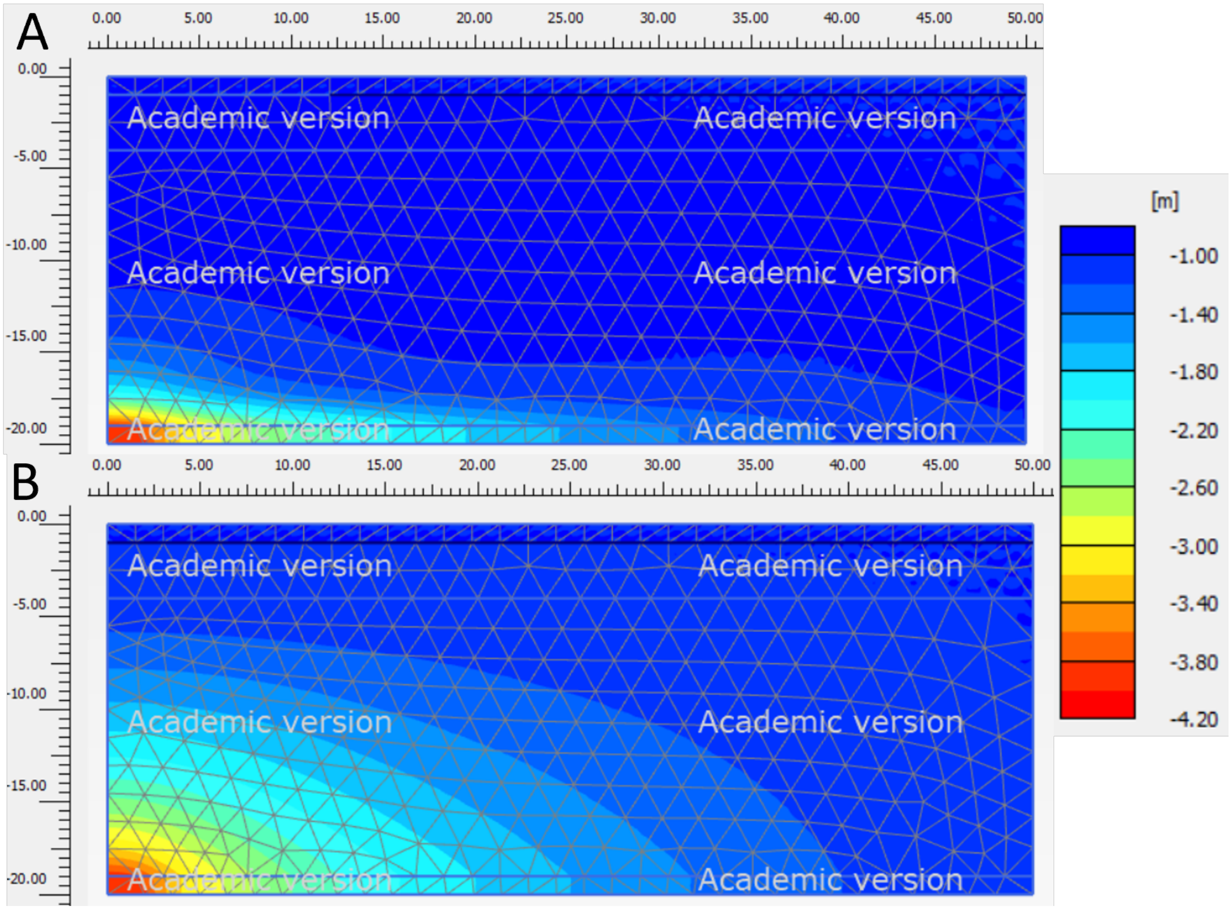


Figure 6.1: Groundwater head vs. depth. 30 kpa below hydrostatic at bedrock for (A) 31 days and (B) 10 years

and bottom as well as a h=-4 m with 2m width at the bottom left. Same as shown in Figure 4.4, except bottom right is h=-1 m. Keeping the same BC at right radius and changing the depth will change and deform this "circle" accordingly. Using weathered clay (kx10 or 1.53E-3m/day) and no forced head=-1 at -4 m depth sees approx. the same pore pressure in the weathered clay as if the dry crust is extended to -4m depth or a head=-1m is put on the topside of the normal clay layer. Changing the height to 30m but keeping the same boundary conditions (r/h)=(50/30) i.e. r/h ratio lowered, sees the pore pressure approx. constant at an increased depth. At r/h=50/10 the circle is flattened but still at approx. stable at 4m. The "circle" in r direction is the same for all three depths. The settlements are shown at terrain (z=0 m) in Figure 6.2, and here the settlements are approximately the same ≈ 4.5 cm for 20, 30 m total depths to bedrock and 3.5 cm for a 10 m depth. The other different soil layering scenarios (curves) are elaborated later below.

Changing the bottom layer to silt, or the clay's permeability to 5.5E-5 m/day will result in a more constrained circle, especially for the former, as shown in Figure A.4. Also the R/D ratio has much to say in the 3D models when operating with moraine at bottom. While large R/D (fixing the D) and having big different r can be done, in Norway a such big flat bedrock area is less

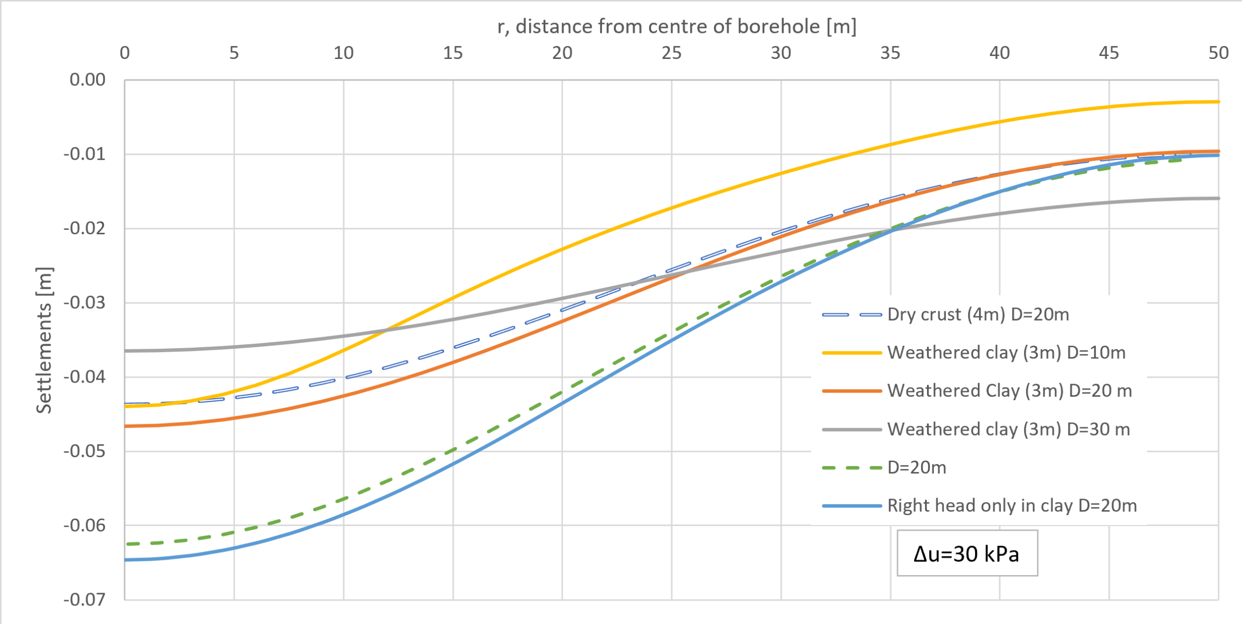


Figure 6.2: Distance from centre vs. settlements u_y at terrain. 30 kPa below hydrostatic at bedrock, whole right sidehead h=-1m save for the bottom curve (blue). With changing dry crust and/or weathered clay thichnesses in the top 4 m of the soil profile with 10, 20 or 30 m depths to bedrock or r/h=(50/10,50/20,50/30)

common while sloping terrain and bedrock are more so. Importantly, if the moraine does not follow terrain, but is horizontal or piecemeal present, the mapping of this layer determines the area of effect.

Figure 6.3 shows for 1 m dry crust, 18 m clay 1 m moraine and BCs of h=-1 m at top, h=-4 m at left bottom and h=2 m at bottom right, same as Figure 4.4. The 20 years is not far from final steady state. 10 years extra will increase the max settlement u_y from 0.0611 to 0.062 m. Therefore 20 years is the final curve shown. Having any permanent pore pressure reduction for such a long time might be very uncommon also, but lack of data makes it hard to say.

Figure 6.4, Figure 6.5 and Figure 6.6 show that a pore pressure reduction at bedrock is very alike even if it is overerpressure or hydrostatic at initial. While the settlements for the centre are the same, the addition to extend the right side head for the whole outer side affects the right side. While clay has little permeability, the moraine presence arguably requires extending the right side boundary, and consequentially all following below hydrostatic figures have the extended/whole right side boundary.

If there is a open outer/right boundary in clay, but not the moraine, h=-1 m and seepage respectively. These have 1 m dry crust and 18 m clay with 1m moraine. The seepage result is very alike to the 1D solution save for the far right long term, and taking away the right side head in the moraine, results in only 3% more settlements shown in Figure A.5 and Figure A.6. If we further reduce outer boundary length so it only encompasses a low ratio of the clay thickness we

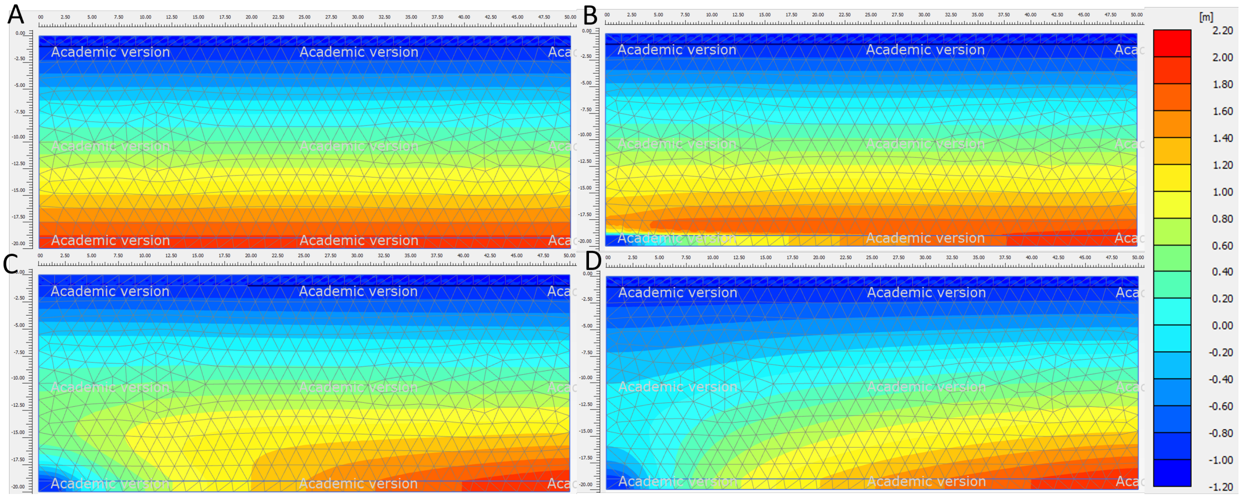


Figure 6.3: Groundwater head vs. depth. The pore pressure/GWL head in a 30kpa overpressure to hydrostatic changing with time. (A) Initial, (B) 1 day, (C) 0.5 year and (D) 10 years

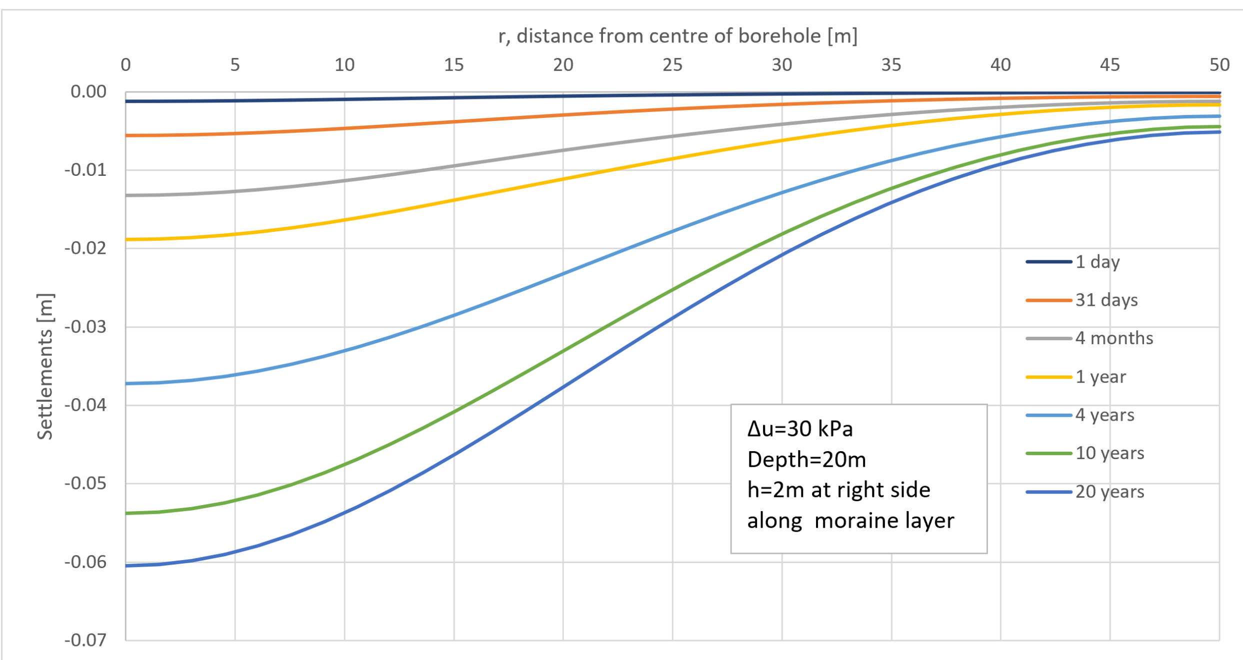


Figure 6.4: Distance from centre vs. settlements u_y. 30 kPa from Overpressure to hydrostatic.

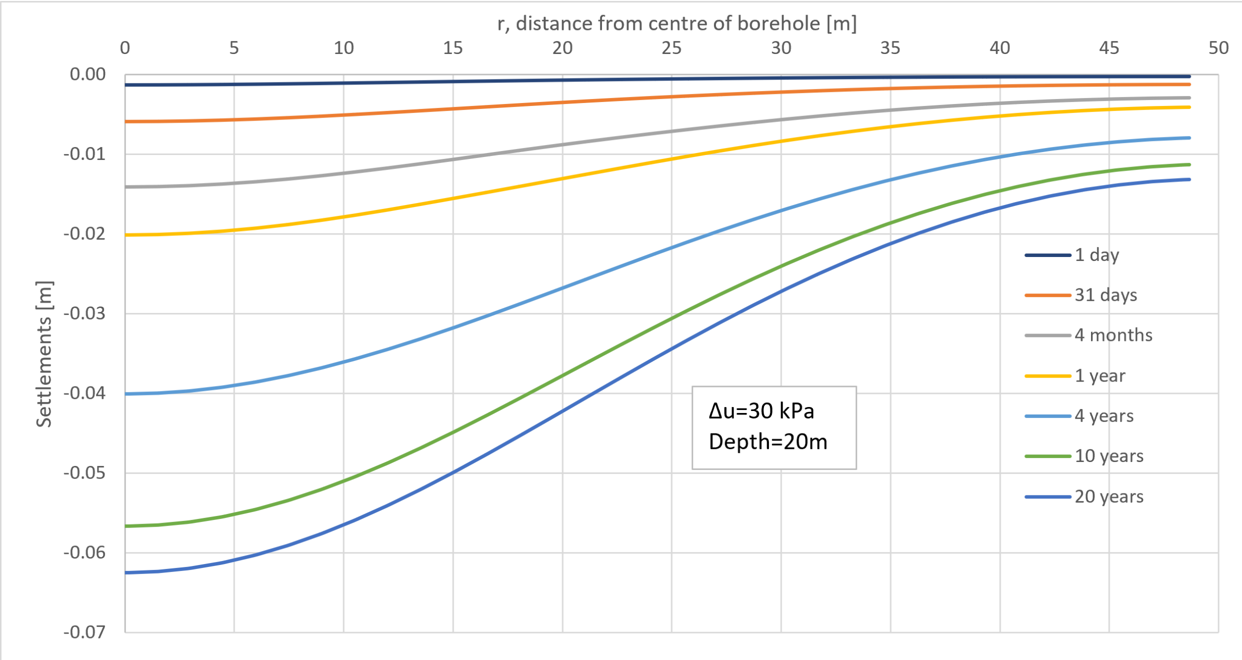


Figure 6.5: Distance from centre vs. settlements u_y. 30 kPa Below hydrostatic for right head=-1m at moraine layer only.

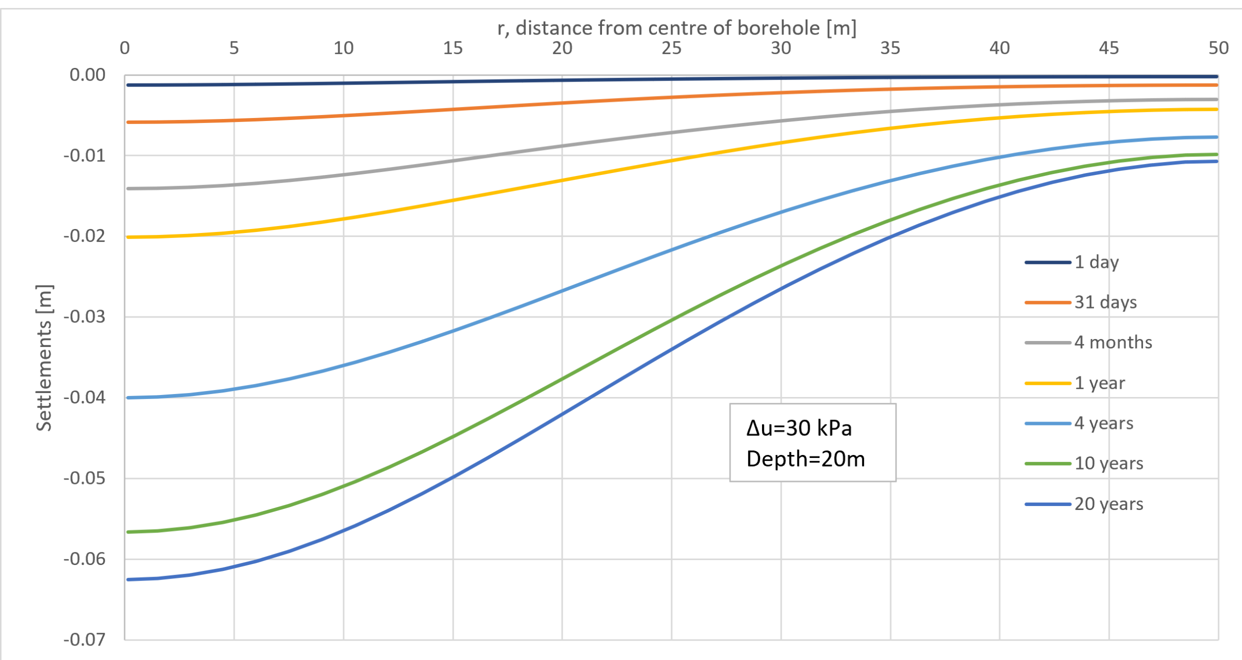


Figure 6.6: Distance from centre vs. settlements u_y. 30 kPa below hydrostatic whole right sidehead h=-1m

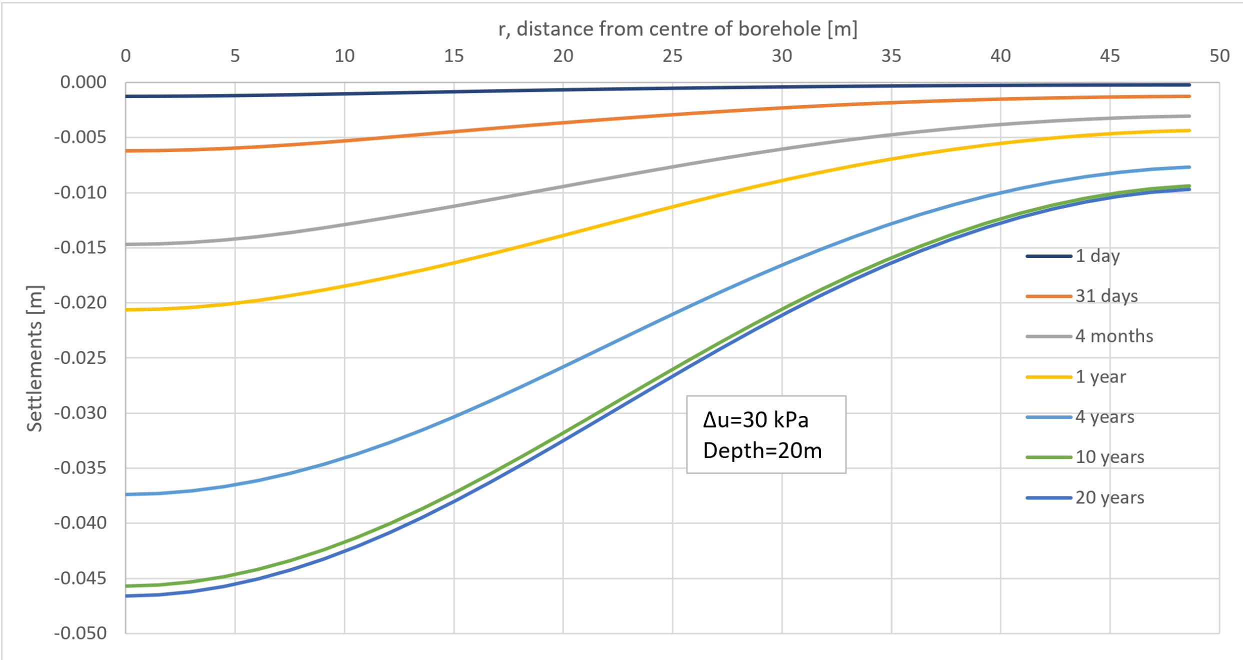


Figure 6.7: Distance from centre vs. settlements u_y. 30 kPa below hydrostatic at bedrock whole rightside head=-1m with top 3 m of clay as weathered clay $k = 10 \times k = 1.53E-3$ m/day.

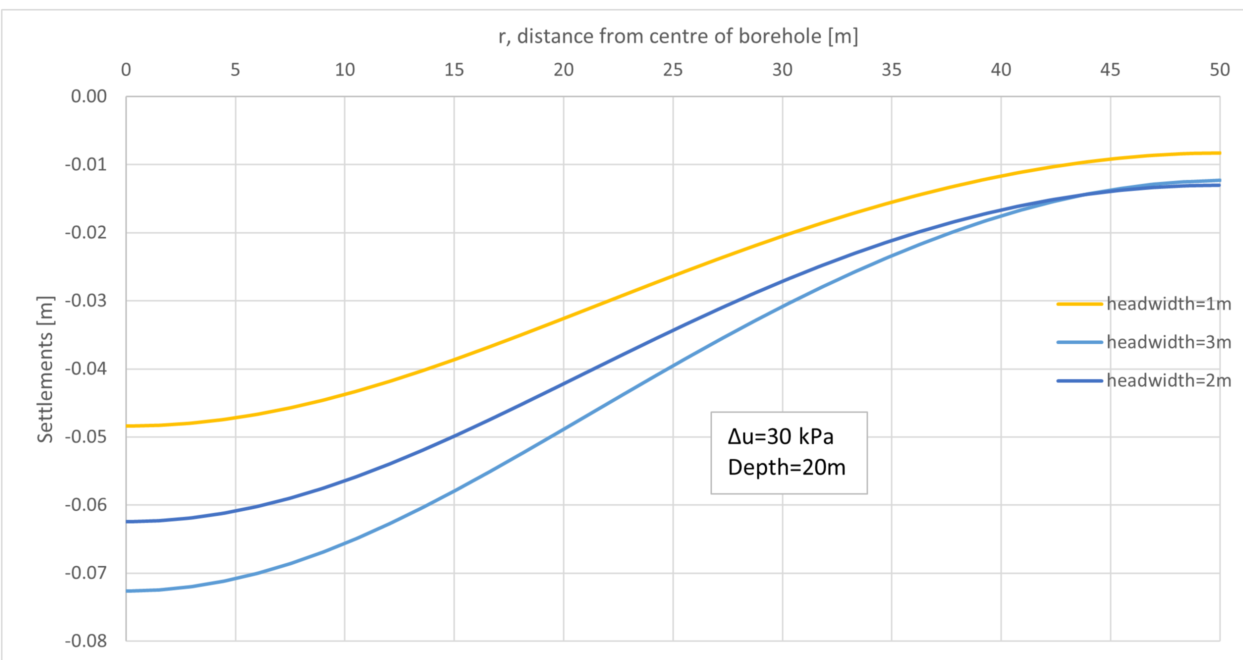


Figure 6.8: Distance from centre vs. settlements u_y. 30 kPa below hydrostatic for different head-widths for the h=-4m head at bottom left. 1 m dry crust and whole right side head=-1. At time 20 years.

get increased settlements as we transition more into a 1D drainage situation. This latter way of modeling the outer right BCs or assigning a low Q inflow, can be the more realistic approach, as a fixed head=1 m in clay is possibly excessive when having the moraine's right BC closed.

Figure 6.8 shows the impact of increasing or decreasing the width by 1 m for the head at bottom left. Figure 6.7 is for weathered clay in the top 3 m under GWL, resulting in lower settlements and quicker to reach steady state solution. Figure 6.2 shows that the difference between dry crust and weathered clay at 10,20 and 30 m depths to bedrock. The increase in depth decreases the R/D ratio, where R is the total radial distance to outer boundary (fixed at 50 m in this case) and D is the depth to bedrock. A lower R/D gives lower settlements throughout, while increasing the R/D ratio gives lower settlements for the first "half" (0< r < 0.5R) and higher settlements at the outer "half" (0.5R< r < 1R). This is due to the right hand BC /head dominating / feeding the most compared to when the top head was closer. Also by pore pressure at bedrock Figure 6.9 shows that different k values and distances to outer/right boundary give different waterflows, and thus settlements along r.

6.2 The problem with Unknown Boundary Conditions

The BCs in a 3D model are found to be problematic when using a fixed head value. And since the R/D ratios and BCs vary too much from case to case and affects the settlements to a varying degree, the figures in this chapter cannot be used on a general basis. I.e the 2D axisymmetrical model is not a general (if that even exists) model for 3D drainage.

It was hoped that one could use different pizeometer reading to interpolate what head-values and at what distance from the centre they should be placed at, but as Figure 6.9 show then the k values needs to be known. In other words, a pumping test is suggested to be done to figure out the infiltration rate/right hand side boundary, using Theis or Jacobs methods/equations [Brattli, 2018a]. An excessive pumping test might give settlements however.

The right hand boundary compared to the top boundary (GWL) dominates due to the permeable moraine layer as shown in Figure A.4(A). A low k or no moraine in the permeable layer above bedrock will mean that the right boundary is "far away" compared to the infiltration of water from above, such that the groundwater plot is dominated by the above head shown in Figure A.4(B). (B) has u_y at 20 years =0.011 m and 0.045 m for (A). The time to t_p in both are higher then the normal situation when the clay has k=5.5E-5 m/day resulting in t_p approx. 57 years compared to 17 years. This is for a 30kpa below hydrostatic pore pressure reduction with entire right side head=-1m.

6.3 Casing and Disturbed Clay Model

The casing is so small compared to the model that it is simplified out, but if included and the immediate vicinity is put as more permeable (k=1m/day), the different boundary conditions

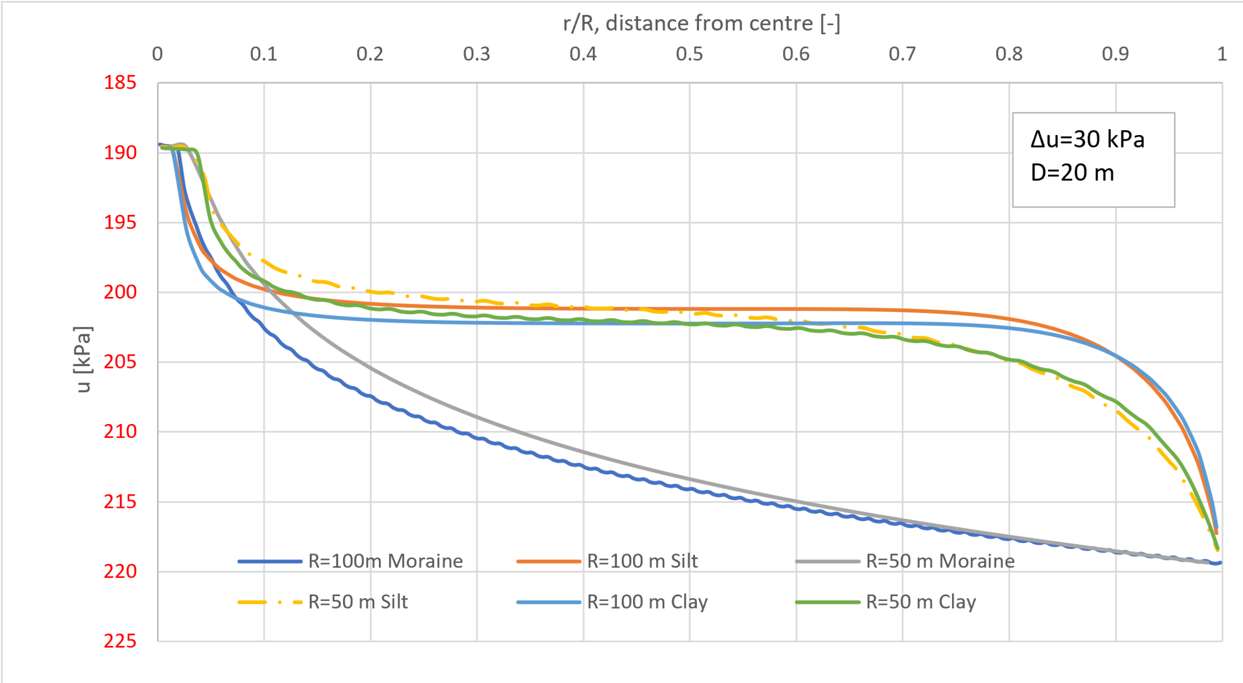


Figure 6.9: Distance from centre of the axsymmetrical model vs. pore pressure. 30 kPa from overpressure to hydrostatic at bedrockdepth 20m, with permeabilities and different distance from centre as a ratio r/R , where r is a function of distance from centreline and R is the total distance to the outer boundary. Silt/silty clay is here arbitrary chosen with k=2.51 E-4 m/day (2.9 E-9 m/s).

are changed. 12 cm casing radius and 12 cm more permeable soil with the same soil as its depth corresponds to. This gives long calculation time and for the overpressure system an additional jet of water is emitting up. Having the sole boundary condition as head=-1 m at the top right (r,y)=(50,0) to (50,-1) the drainage is too little and not much is learned from the model. The corner of the clay had high strain values, which is to be expected as massflow/erotion is expected at high hydraulic gradients. The 12 cm outside the casing is arbiterly chosen but a higher k value would give problems in PLAXIS, see E-6 difference section 4.1. A larger depth is more vulnerable to GWL drop than a large depth or not? For depth less than 5-8 m or a narrow trench the GWL might drop. Settlements vs time is faster and greater relative to if the GWL would not drop. Overpressure will change with depth and thus increasing depth might mean more infiltration into the model, and thus a stable pore pressure at a depth between GWL and bedrock. This infiltration again depends on if it is a perched groundwater or not and whether or not there is much infiltration into, or out of. A general assumption basis is not recommended as it will vary from case to case (bedrock, soil thickness, and terrain topography). Nevertheless, Figure A.7 and shows the PLAXIS model used.

The settlement in Figure A.9 long term is due to the overpressure going out of the model naturally and with the right side draining the water, this is where the settlements are the highest. While a head h=-1 m, could be placed at the whole bottom area of the dry crust, and a head=2 m at bottom to keep the overpressure, the results are in the same manner but with heave instead due to the waterpressure going upwards. Increasing the permeability from 1 to 10 in the more permeable 12 cm soil layer outside of the casing speeds up the settlements, but the accuracy is questionable section 4.1. A model that takes into the account for soil/mass loss is recommended.

Figure A.8 shows long time to drain the water if seepage only on the dry crust layer right additionally a jet upwards is seen in the output next to the centreline. A 30 kPa below hydrostatic and head=-1 at right side but not at the top gives higher settlements due to the GWL being lowered around the borehole as shown in Figure A.8. Again the are too many variables to assume anything in general, as these will vary from case to case. There seems to be a convergence error also, as when running for long time, the time interval chosen / max iteration steps gives different answers.

The clay layer should also be flush against the casing so assuming a gap here is guesswork as there is no good data to go by. There are cases of drilling in artesian aquifers and not grouting causing leaks along the casing, but the Q (water-flux) is not registered, and in practice hard to do. Therefore this is why not much is modeled with it, as there are too many unknowns for a general model/study.

Chapter 7

Discussion

A discussion of the results and comparison between the models and case study is presented below.

7.1 Discussion of the Results

Boundary conditions are hard to assume and can vary from the ones picked here. Even with such a general problem definition, are the chosen boundary conditions realistic or not? While data gathering was attempted there are no datasets for the whole soil profile pore pressurechange by depth. So the time variable, especially in 3D is very hard to predict.

The one dimensional drainage results and figures are accurate, with both PLAXIS and 1D analytical solution giving approx. the same results. However, when to generally assume 1-D drainage or an approx. 1-D drainage is difficult. If a moraine or permeable layer above bedrock has no hydraulic connections on its sides, e.g., is enclosed by solid bedrock on its sides and below, this layer will be more drained and thus the clay above will also be more easily be drained. And will be drained more 1 dimensionally. A perched groundwater is the most at risk of below hydrostatic drainage, see Figure 2.14. The presence of a drainage layer as moraine is deemed necessary for 1D drainage and its omission results in much lower settlements and influence area. While a variable topography of terrain and/or bedrock can indicate artesian conditions, a large variation in heights can decrease the risk of overpressure drainage as the artesian conditions can be sustained. However, this can lead to mass erosion. In other words the area of effect is lower due to more infiltration Q inflow. This is assuming that the closed aquifer thickness is following the terrain and continuous. A moraine layer that follows or is broken up by the bedrock topography gives significant different drainage and recharge paths and thus differential settlements. Paradoxically a moraine layer that follows the terrain results in higher inflow as its influence area is greater, but more inflow should give less settlements, provided that the drainage path is small and the same. In other words, while differential settlements are impacted by varying the depths to bedrock (bedrock topography), if the permeable layer is horizontal and thus more constricted, the settlements are bigger and closer, further increasing the risk of angular distortion.

If the pore pressure drawdown is substantial (60 kPa or more) such that the OC clay goes into the NC stress region, there is an increase in the settlements, but the magnitude of this increase depends on the swelling index κ or M_{oc} as well if it is POP or OCR. A POP profile where σ'_p is exceeded at the bottom of the clay layer would lead to more settlements, compared to OCR with $\sigma'_p > \sigma'$, since this is where $\Delta u = \Delta\sigma'$ is the largest. See figure Figure 5.2 and Figure 2.2.

A small pore pressure reduction (e.g., $\Delta u \leq$10 kPa) can have been experienced before and the GWL or overpressure can fluctuate with the season, so depending on the stress history a 10 kPa reduction might give no settlements. The results show that the drainage of overpressure to hydrostatic produces greater settlements and takes longer to fully develop, for the same Δu and depth, assuming that the overpressure is linearly above hydrostatic for the whole profile. 1D drainage from an overpressure to hydrostatic pore pressure in clay is 1.5 ways in PLAXIS, and despite being by Equation 2.6 quicker, overpressure drainage has higher σ' and larger pore pressure area to drain and is thus slower to reach U_p than the below hydrostatic scenario. This is not true for the 3D results where the same pore pressure loss in overpressure to hydrostatic or below hydrostatic for 20 m to bedrock, gives approx. the same settlements.

Additionally an overpressure drainage is more likely than below hydrostatic drainage to have mass erosion/flow further increasing the settlements. It should also be said that a certain/fixed Q escaping due to ground work, will give more settlements in a closed aquifer relative to if it was open by Equation 2.16. Since in PLAXIS the groundwater BCs had fixed heads, this difference between the overpressure and below hydrostatic scenarios are not magnified to the same degree.

In 3D the settlements are easily calculated if the drainage is approximately in 1D and the pore pressure is recorded. However the pore pressureloss may be limited to the bottom clay layer if the infiltration is high due to thick layers of dry crust or weathered clay compared to clay thickness, or that the moraine horizontal area is small compared to the clay under GWL.

Especially in overpressure systems a general basis cannot be assumed. What is the storage coefficient and what value should be assigned to the right boundary head? The overpressure is perhaps not all the way to GWL, but the overpressure is rather linearly to the bottom of the weathered clay layer or soft clay layer. As overpressure dissipation from an aquifer can last or dissipate quickly depending on its size and hydraulic connectivity. Overpressure as well as below hydrostatic areas of effect are dependent on infiltration rates, if there exists a permeable layer like moraine at bedrock and its extent, impermeable or fractured rock, and the thickness of the gap(s) or fracture(s) that enables waterflow, and the hydraulic connectivity of the soils. And that may be difficult to predict without proper soil mapping or long term recording, but then in the latter case the damage may already be too late to stop.

While bigger thicknesses of clay give higher settlements (especially for the below hydrostatic pore pressure scenarioes), the infiltration for big depths might fix the water level at a certain depth closer to the moraine layer, due to Q outflow being much less relative to the infiltration/ Q inflow. In a shallower area the infiltration is less, and thus the same Q outflow is now bigger in relation to the Qinflow. As mentioned previously and as Karlsrud et al. [2003] notes a shallow depth to bedrock or a confined "trench" will give higher settlements in 3D due to lowering the GWL into a "funnel shape". The permeability of the soils and the extent or absence of a permeable layer like moraine above bedrock, below clay, has also much to say. The groundwater head figures shows this. NC clay is more hydraulic dependent, than OC clay and OCR=1.2 shows not much change for 20 m depths, with regard to more dry crust or weathered clay. This is due to NC clay has higher settlement potential.

The 2D axisymmetric PLAXIS models are highly dependent on the hydraulic boundaries and moraine's area-extent to iterate generally without knowing them. These may be hard to obtain data on, and the former may also vary with time. Especially for the dissipation of pore pressure in 3D, the chosen boundary conditions might be very different from case to case. Time-settlement curves in 3D is therefore difficult to calculate, and the graphs shown must not be taken as accurate values for a general soil profile, but rather when the boundary conditions coincide. The 3D model results in less settlements when comparing it to the 1D models, due to a outer (right side) boundary head (especially when placed in the moraine layer), and stress redistribution in the non vertical dimensions. Using a low Q infiltration instead, would be more realistic in certain scenarios. Again, this depends on the extent of the moraine layer and its hydraulic contacts and conductivity. Choosing a correct Q inflow might however be impossible without pumping tests, and mapping the moraine layer in 3D and/or deciding if the bedrock is impermeable or has waterflow through fractures is difficult. Mapping the extent of the moraine and putting its contact with bedrock as impermeable will approximate the solution next to the leakage into 1D drainage if the moraine layer is big in comparison to the clay thickness and the leakage is substantial. In other words, having no outer right head or small Q, will give a more 1D drainage of the clay when the leakage is modeled as a fixed head. Also, a thick layer of clay or under a deep building may make the drainage 1D only in the long term, depending on the permeability of the soil and infiltration from pipes/terrain.

The differential settlements and the area of influence (upside down Bell / Gauss curve) have been attempted to be found out by 2D axisymmetrical PLAXIS. However, due to the problems with 2D axisymmetrical model, a very large 3D model, or just the 1D model/excel tool for different points is advised. A large 3D model will be more suited to deep-weathered zone with large depth to bedrock variation, as a axisymmetrical model is with soil layers perfectly horizontal and with the same thickness. The latter is a simplification, and especially in Norway with generally sloping terrain and bedrock, a large such model can be erroneous. A general 3D soil profile can be done, but it is rather advised to do so on a case by case basis. In practice one would need to get the permeability, k_y and also k_x, as well as knowing the stress history. If a building founded on clay has already settled such that the clay is slightly overconsolidated the settlements will be significantly less. A low permeability in the y/depth direction (soft clay can have 5.5E-4 m/day) makes the right BC more important when modeling moraine at bedrock in 3D.

However, based on a general basis the pore pressure reduction needs to be substantial and permanent/long term to give significant settlements, and this is perhaps why there exists so few cases of energywell or piling induced damage. Having a soft soil with low OC strength or NC clay, is present in some areas, but it could also be likely that a soft clay layer exists within the clay layer. Halving the material strength of clay M_{oc}, m-modulus, gives approx. twice the settlements by 1D analytical solution and PLAXIS. Bringing the average m-modulus in the clay down or modeling the clay into more layers, when a soft clay is present, gives higher settlements, and thus a lower pore pressure reduction is more severe than for the m chosen in the results.

The difficulty in calculating in 3D is therefore to a bigger degree, than anticipated at the start of this thesis, for the boundary conditions/infiltration rate than the right material parameters when in low OC to NC clay. A model is not recommended in 3D unless boundary conditions are known. Usage of the 2D axisymmetrical model to compare the measured pore pressures in a given case at different lengths from the centre is not advised as the head is fixed in PLAXIS and the permeabilities are kept constant at fixed values, which may not be the reality.

The PLAXIS 2D axisymmetrical model with gap is difficult to build, with no conclusions to be drawn from the results. Mostly pore pressure takes time to work itself upwards, or GWL downwards unless there is a big gap between the casing and the soils. The gap is more probable for the upwards seepage as the waterflow has a higher force and mass erosion, keeping this gap open. A different program is advised. Long time to calculate relative to omitting the casing. interface between the "gap" higher k layer and the rest of the clay? Having a gap in the model also interferes with section 4.1. Big deformation and stress difference between element nodes can give "bad" and inaccurate values [Sagmoen, 2017].

One possibility is that the artesian- /over-pressure is not high enough to be noticed as leakages at terrain, but go up and through the dry crust or weathered clay which is more permeable, taking the water seeping upwards away. I.e. if the hydraulic head potential is less then the thickness of dry soil above the GWL the leak may be difficult to see. Or if the GWL is far below the terrain the water leak can go unnoticed, although in Norway the GWL is generally close to terrain.

The moraine layers main function is its higher permeability, and could easily be rather named till, silt or a coarser material like moraine and sand. Also, if the settlements are limited to the bottom due to infiltration, stress redistribution may mean that the settlements are less above at terrain [Terzaghi, 1936].

Temperature and expansion and contraction due to the borehole heat exchanger (BHE) is not discussed here. There is also the case regarding u_x displacements in the x direction. In the calculations the were generally around a couple of cm long term or less. There could also be fractures in multiple areas and just the one next to the casing connecting multiple.

The figures based on the 2D axiysymmetrical models are unwise to use in general soil profiles.

It is rather recommended to use pizeometerdata / CPTU and other tests and use the 1D analytical solution to calculate the settlement potential. A 1D approximation may be enough to evaluate the potential of risks, but can be regarded as a worst case scenario, as when compared to the 2D axisymmentrical model the settlements were less. Selecting different points/depths to bedrock and use it to calculate differential settlements. Lowering the GWL will give higher effective stress and settlements, but it is uncertain to what degree this happens, but if measured this must be put in the input console. The GWL could become from horizontal or parallel to terrain to become nonlinear into a funnel around the borehole? While building pits/tunnels are not modeled, the 1D analytical solution can be utilized for measured pore pressure drawdowns here also. Also the differential settlements can vary with time if buildings have different depths to bedrock. Low depth quicker but lower settlements then higher depth which are slower but bigger long term. As the 3D results for a given infiltration / BCs (so this may vary from case to case) show that the differential settlements can increase or decrease with time. While the R/D ratios was known to be of importance in the model the effect was larger than anticipated. It is difficult to say what R/D ratio should be used, as its value determines greatly the outcome. Based on this, a large 3D model with varying depths of bedrock where the moraine is constricted into the depressions and to only have a BC fixed at the GWL or weathered clay is better.

For later work, the settlement hazard map or depth to bedrock map Oslo municipality is making can be utilized, especially the "ravines" big variation to bedrock will give a differential settlements potential. The 1D analytical solution can be used with Python to calculate 1D settlements with respect to GWL, bedrock depth and OCR. A more complex model is suggested with hydrostatic boundary conditions and building loads and more precise soil data especially stress history when the calculation needs to be more precise. While old housing complexes of brick can be more susceptible, only information about the foundation can verify if they are at bigger risk than other buildings. Mapping of deep weathered zones is recommended. Use Q, storage coefficients and infiltration data and calculate more properly, to avoid the head issues, especially for the right boundary head. Using a very large 3D model and have no fixed head values, save for perhaps GWL, would be better. MODFLOW with sloping terrain or large 3D models in PLAXIS 3D. Large scale tests and backcalculation would be best for a temporary pore pressure loss in moraine and stress redistribution in clay is proposed, however a small scale test model would be easier and cheaper. Settlements due to remoulding of quick clay due to porepressure reduction could be a topic for further work, as this may increase the settlements. Van Genuhten parameters and $kx \neq ky$ may be used if data on the Q inflow and outflow are more known / gathered for further work.

7.2 Comparison to the Case Study

Comparing to the case study in Appendix B.2 from Våpenstad [2021] is difficult due to lack of data, and another review after the court cases (case 1 and 2) are done is recommended. Especially on the magnitude and origin of the pore pressure loss.

Case 1 has some pore pressure loss at bedrock, 10-20 kPa and 110 mm over three years of recording, but much less recorded by InSAR data. The bedrock topography and soil under the foundation compounded the settlement damage. While "only" 7-10 m of clay indicated, the quick clay thickness can be enough to get the calculation of approx 10 cm when assuming 1D drainage. E.g., assuming a profile with 4 m dry crust and 6 m NC soft clay with average m=6.4 for a 20 kPa below hydrostatic pore pressure reduction resulting in 11 cm by the 1D solution.

Having a uniform NC clay was done in many of the calculations, but this simplification is not wise when calculating for case 2 as there are reports of soft and stirred quick clay below stiffer OC clay in the area. While case 1 had a quick clay layer as well, case 2 has it described as soft and stirred indicating a higher settlement potential. case 3 has the slowest and smallest in both affected area and settlement magnitude, and this can be due to a lower permeable clay and not so much difference in depths to bedrock (gentle bedrock slope). Rigid building transition is the main possible cause of damage.

They all have quick clay in the soil profiles nearby, and case 2 has it described as soft/stirred. Arguably downstream is affected in all cases, but this is based on terrain topography and InSAR data, and should be looked further into by pizeometers and mapping bedrockdepth and/or ravine filled with moraine. Moraine is found generally thicker in deep trenches and is vital for the area of influence mapping. A more permeable layer of moraine above bedrock is difficult to ascertain with certainty the presence of. Case 1 has total sounding tests that can indicate it, whereas most of the older geotechnical reports (e.g., the NADAG reports in Appendix C) in the vicinity of case 2 and 3 have soft clay indicated and in some reports clay with gravel/sand in the deeper layers. This may or may not be moraine, and some soil profiles do not show the deepest layers, further shrouding the matter. Soil profiling is outside the scope of this thesis, but Appendix C has some of the NADAG reports from Undergrundverket included.

Looking at the case study in Appendix B, the foundation and construction-transitions is the final "gatekeeper" for if the differential settlements will result in noticeable/significant damage. The degree of damage is also hard to predict. While case 1 in Appendix B has critical bearing failures, case 2 and 3 show facade damage in the pictures available. However the lack of data and that they have foundations on deeper clay thicknesses mean that the damage could be substantial long term, as the InSAR data show no flattening of the settlement-time curve for case 2 and 3. Karlsrud et al. [2003] denotes that pore pressureloss under building-foundations can generally be uniform, unless long/big buildings. Hans Amundsens gate 5 in case 1 might be an exception to this rule, due to large variation in depths to bedrock and/or perhaps the soil layering might not have been horizontal Figure 2.10. Additionally the risk/hazard is greater for the transitions between buildings as seen in case 3, Appendix B Figure.2.8.

Thermal expansion is not discussed, but Dalla Santa et al. [2019] shows that the hydraulic conductivity K, experiences an increase of about eight times in normal consolidated clayey sediments due to successive freeze thaw processes. One thing not considered in the models, due to the deemed low probability for a general case, is several layers of clay and sand/moraine soil aquifers perforated by the casing installation.

Having a leaking casing-bedrock transition is deemed seldom for energy wells as there is a leakage test required after plug installation described in Norsk Standard [2012], and while new fractures could appear or be introduced after further drilling, extending the casing 2 m Norsk Standard [2012] into solid bedrock and then grouting/injecting the plug should in most cases be enough to move past this fractured/brittle rock, however possibly not always due to improper determination of solid bedrock [Norwegian Geotechnical Society, 1994]. Bad estimation of the bedrock, e.g., misinterpreting moraine/boulders as bedrock, or a casing-weld very improperly done could be two possible causes for leakage. In addition to having a leaking passage through fractures or an insufficiently sealed plug, the water has to go somewhere by a hydraulic connection into the rock by fractured bedrock if below hydrostatic, see Figure 2.14. Or if the overpressure is high, on the outside of the casing to terrain or through the borehole itself, see Figure 2.8. And a permeable layer (e.g sand or moraine) is deemed necessary for facilitating enough waterflow. The low likelihood of having all these variables simultaneously and resulting in substantial differential settlements explains why there are so few recorded settlement damage-incidents related to energy wells.

7.2.1 Mitigation Measures

While mitigation measures are not looked at in detail in the calculations, a rough approximation would be to read the settlements of the time-settlement curves for a given duration of pore pressure loss. The pore pressure loss can be stopped with grouting of the casing-bedrock transition or along the whole casing depending on the leak.

Infiltration wells in moraine/bedrock can be done to speed up the pore pressure stabilization. For low depths to bedrock with soft clay the time to implement mitigation measures is more urgent. In PLAXIS as in the 1D analytical solution (excel) the moraine settlements happen rapidly for the moraine and slower for the clay, and if the pore pressure stabilizes back the settlements reset in the models due to heave. In reality this heave might be limited. Of note, even if the heave resets the settlements, the building damage inflicted is still the same and irreversible. It is advised to show caution and to check that the well is leak-free in soft soils over moraine above bedrock. Pizeometer installation at/next to bedrock in soft / NC clay areas is advised to see if the pore pressure stabilizes quickly after ground works, and the mapping of the extent of the permeable layer or lack of it above bedrock is important as its extent determines the area of effect.

Chapter 8

Conclusions

A permanent pore pressure loss at bedrock for bored ground works in soils was done in 1D and 2D axisymmetrical models by the 1D analytical solution and PLAXIS 2D. The results indicates that the general clays in Oslo can give substantial settlements if NC and the presence of a deeper soft soil above bedrock amplifies this. The depth to bedrock as well as extent or lack of a moraine/ permeable layer determines the area of influence. A softer soil above moraine results in more settlement and with lower permeability in clay the problem is more dependent on the outer boundaries/ infiltration into the moraine. While deeper thicknesses give higher settlements and take longer to fully develop, these can generally have more infiltration and thus limit more the extent of settlements.

For 1D drainage the results show that a pore pressure loss from an initial pore pressure above hydrostatic to hydrostatic compared to a initial hydrostatic pore pressure results in more settlements, longer time to fully develop, less dependency on depth and can generally be easier to drain. In clay a pore pressure loss at bedrock for an overpressure drained to hydrostatic, the results show a 1.5 way drainage in PLAXIS as opposed to a 1 way drainage for an initial hydrostatic pore pressure profile. Using the produced 1D analytical solution is recommended for a measured pore pressure reduction in a permeable soil above bedrock due to ground work, for quick assessments of 1 dimensional drainage. The presence of a drainage/permeable layer as moraine is deemed necessary for the 1D drainage and its omission results in smaller settlements and influence area in the models. However, an extensive moraine layer may have higher infiltration/waterflow lessening the pore pressure reduction or have an artisian pressure such that a leak is noticeable at terrain resulting in mitigation measures implemented.

The difficulty in 2D axisymmetrical modeling with unknown boundary conditions results in the mapping of these multidimensional drainage influence areas and figures non-general, as they are highly dependent on the hydraulic boundaries and moraine's area-extent to use generally without knowing them. A large 3D model where the soil layering (especially moraine) and bedrock topography is mapped will give better results, but should be done on a case by case basis, as the soil layering, boundary conditions and depth to bedrock varies too much to use a general model.

The unavailability of the technical reports regarding the energy well case study [Våpenstad, 2021], due to that the lawsuits are still ongoing or settled outside of court, hampers the research and must be considered the biggest limitation of this work.

Recommendations for further work:

The recommendations for further work are when/if the data is public after the court cases mentioned in Våpenstad [2021], large 3D models in MODFLOW and PLAXIS 3D with accurate boundary conditions, bedrock topography and soil layering for given infiltration/inflow and outflow amounts can be done. If the court cases outcomes are inconclusive, it is recommended to do a large scale test with pore pressure measurements long term and settlement readings close and far away from the borehole. This should be done in an area with fractures in bedrock, moraine and soft clay. Tests and backcalculation for the disturbed quick clay, permeability and pore pressure relationships is proposed.

Mitigation measures to further research are grouting on the inside of the borehole in the casing-bedrock transition and welded cap(s) where there are fracture zones or leaks to stop hydraulic connectivity. The variation of industry protocol in the use and effects of grouting/injection mass for energy wells can be looked further into. A mandatory use of grouting on the outside or inside of the casings in areas of soft soils, fault zones/bedrock depressions and sensitive built up areas could be enacted. A hazard map can also be mapped based on a depth to bedrock map and the 1D analytical solution run with Python for given pore pressure losses.

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