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Analysis of water ingress, grouting effort and pore pressure reduction caused by hard rock tunnels

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ABSTRACT

Norwegian ground conditions with hard bedrock underlying soft, marine clay deposits are challenging with respect to the risk of settlements caused by water ingress to tunnels. Settlements in clay-filled depressions and damage to nearby buildings are one of the main risks associated with future upgrading of infrastructure. This paper presents a database and findings from 44 tunnels in the Oslo-region, excavated between 1975 and 2020. One of the main findings is that few of the tunnels in the database meet the strict leakage limits necessary to avoid settlements for future tunnel projects. Previously, water ingress of typically 3-7 l/min/100 m has been allowed to limit pore pressure reduction to 10-30 kPa (1-3 m water head), limiting settlements and building damage. In areas where the pore pressures already have been affected by water ingress to an existing tunnel, any additional leakage and pore pressure decrease due to a new tunnel will cause additional settlements. Previous reduction in pore pressure results in an even stricter water ingress limits for new tunnels. Hence, water control through improved pre-excavation grouting (PEG) techniques and better monitoring of pore pressure development during excavation is needed. To ensure necessary data-collection for future research it is important to increase the quality of collected data.

KEYWORDS

Urban tunnelling; Water ingress; Pore pressure; Settlements; Pre-excavation grouting.

1. INTRODUCTION

Norwegian ground conditions with soft marine clay deposits overlying hard bedrock are challenging with respect to risk of groundwater leakage to rock tunnels and deep excavations, which can cause severe settlements. Even small water inflow can cause considerable pore pressure reduction at bedrock level in the confined aquifers between the bedrock and soft clay. This will initiate a consolidation process in the clay, which can cause large settlements damaging overlying structures, buildings, and infrastructure. The problem is illustrated in Figure 1, showing water ingress to a bedrock tunnel, causing pore pressure reduction in a water bearing layer of sand and gravel at the bedrock surface, Δu. With time pore pressures, u, in the clay layer are affected, resulting in settlements.

This paper presents and discuss monitoring data from 44 tunnels constructed over 45 years in Norway, mainly in the Oslo region. The original database from Karlsrud et al. (2003) contained 29 tunnels, recently 15 tunnels were added, mostly from tunnels constructed after 2000. An overview of the complete database was published in 2022 (Langford et al., 2022). Main findings from analyses of this database will be discussed and presented.

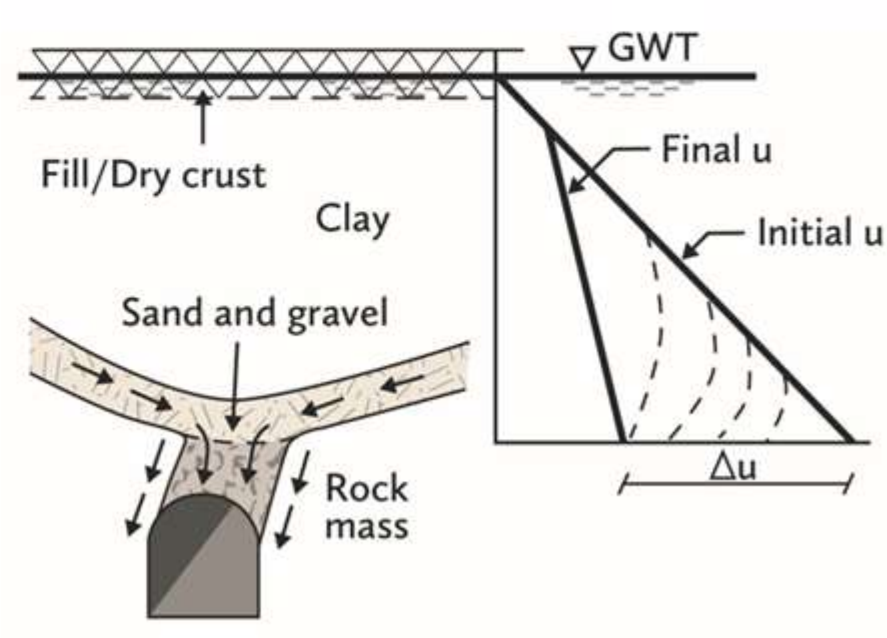

Figure 1 Illustration of water ingress to a bedrock tunnel causing pore pressure reduction in a water-bearing layer over the bedrock (revised from Karlsrud et al. 2003)

An increasing growth of population in the cities will require an increased need for upgrading of infrastructures, such as road-, railway-, and subway tunnels. These projects will pose challenges in avoiding consequences related to water inflow. In addition to underground rock tunnels, deep excavations will be necessary at stations and tunnel openings. The new tunnels will be built in areas already influenced by existing tunnels, where current drainage has caused settlements and thereby increased the risk associated with new projects. These conditions will result in even stricter requirements on inflow limits to avoid damage, which in turn will require improved performance pre-excavation grouting (PEG) procedures.

Pregrouting was the focus in the Norwegian research programmes “Tunnels for the citizens” (Lindstrøm and Kveen, 2005), and True Improvement in Grouting High pressure for Tunnelling (TIGHT). The latter project was completed in 2018 and had main goal to increase the understanding of rock mass grouting (Strømsvik, 2019). Another, R&D program, " REMEDY / BegrensSkade" has the aim to reduce the risk of damage to neighbouring buildings and structures caused by deep excavation and foundation works. In the projects, one of the main causes of damage is reported to be leakage to deep excavations, resulting in pore pressure reduction and settlements in soft clay (REMEDY, 2015). This fits well, since recent and ongoing projects also report difficulties with avoiding settlements and damage to nearby buildings.

2. TYPICAL ROCK AND SOIL CONDITIONS IN THE OSLO REGION, AND SENSITIVITY TO PORE PRESSURE REDUCTION

The geological conditions in the Oslo region consist of bedrock dating from the Precambrian, Cambro-Silurian, and Carboniferous and Permian periods. A graben structure was created by the Caledonian folding and Permian block faulting, resulting in the following main rock types:

- · Igneous rocks (including both plutonic and volcanic) – Carboniferous and Permian
- · Shale and limestone (sedimentary rocks) - Cambro-Silurian
- · Gneiss – Precambrian

A bedrock map of the Oslo region together with approximate locations of tunnels included in a big analysis presented in Langford et al. (2022) are shown in Figure 2. In the following sections analyses and main results will be presented.

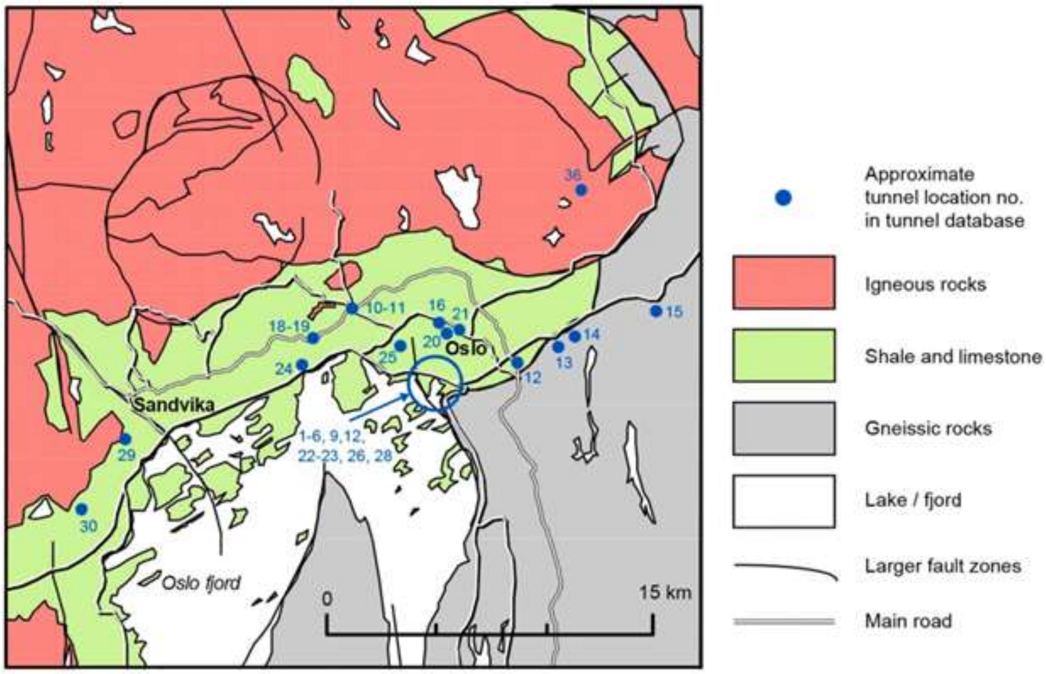

Figure 2 Location of the tunnels in the Oslo region in the database. Bedrock map modified from Bjørlykke (2004) and Langford et al. (2022).

The lower-lying areas of the city centre and nearby regions consist mainly of Cambro-Silurian sedimentary rocks such as shale and limestone. Igneous dikes (mostly syenite-porphyry and diabase) are common in these formations with thicknesses ranging from around half a meter to 50 m. The main igneous rocks in the Oslo region are syenite/monzonite, granite, and rhombus porphyry, which makes up the hills and ridges to the north and west of the city centre. Precambrian gneissic rocks dominate the areas south and southeast of the city centre.

Research has shown that igneous rocks are often more brittle and tend to have more open channels along the joints compared to other rock types (Klüver, 2000). As an example, in the Oslo region it has been documented that syenite (plutonic) and Permian dikes, such as syenite porphyry and diabase, have higher hydraulic conductivity than other rock types (Holmøy, 2008; and Lindstrøm & Kveen, 2005).

The soil deposits above the bedrock typically consist of soft marine clays (0–80 m thick) deposited at the end of the last glaciation, about 10,000 years ago. These clays have not been subjected to loads greater than the present overburden stress and are normally consolidated (Bjerrum, 1967), with an apparent over-consolidation ratio (OCR, the ratio between the pre-consolidation stress and the in-situ effective overburden stress) of 1.2–1.4 due to ageing (Bjerrum, 1973). Commonly there is a layer of glacial moraine between the bedrock surface and overlying marine clay. This layer exhibits significantly higher hydraulic conductivity than the clay deposit, and hence represents a permeable aquifer which extends along the bedrock surface. Pore pressure levels in such confined aquifers are sensitive to changes in water infiltration and extraction, such as water ingress to tunnels.

To illustrate the nature of time-dependant settlements, Figure 3 shows an example of calculated pore pressure reduction of 100 kPa at bedrock are performed using the Janbu modulus concept (Janbu, 1970), widely used in the Nordic countries to calculate consolidation settlements in clay (Andresen & Jostad, 2004). As the OCR is close to 1, minor changes in effective stresses, i.e., reduction in pore pressure, will cause significant consolidation settlements. The hydraulic conductivity of the clay is set to 5·10-10 m/s. The pore pressure reduction in the clay deposit is calculated for different time intervals. The low hydraulic conductivity of the clay layer results in a long consolidation process. From Figure *3* it is apparent that the pore pressures need to be monitored at the bedrock surface to detect any effects of water ingress to tunnels. In the example it will take more than 6 months for pore pressures at 10 m depth to be affected.

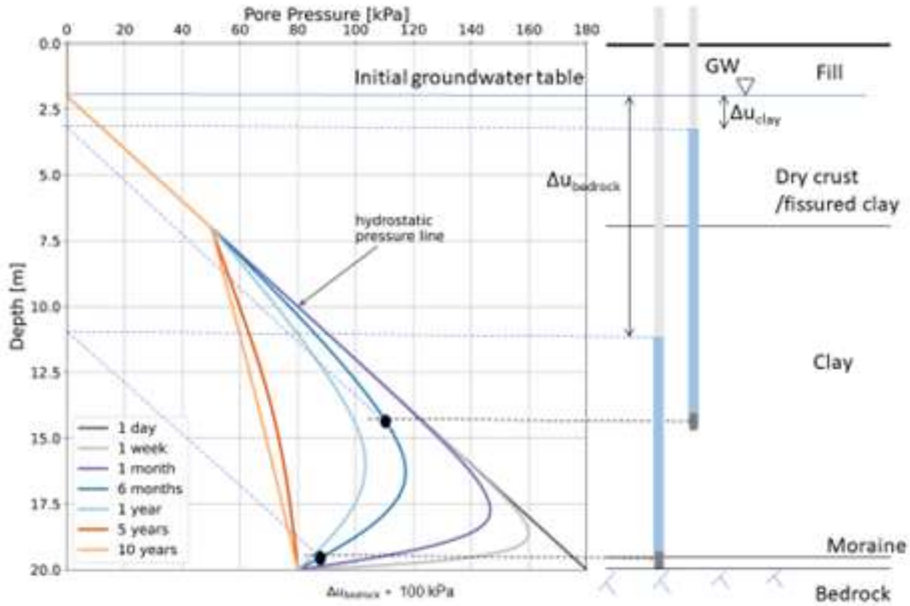

Figure 3 Example of calculated pore pressure profiles with time in a clay profile (Langford et al. 2022)

3. DATABASE

Data from 44 tunnel projects, mostly in the Oslo region, constructed between 1975 and 2020 have been collected (Langford et al., 2022). For some projects, data has been monitored in several sections along the tunnel, in other projects data has been monitored over the total tunnel length. In total, the database contains monitoring data from 56 sections. All tunnels have been excavated using drill and blast excavation technique. All sections have strict limits on water ingress, which has required extensive PEG. The locations of the tunnels in the Oslo region are shown in Figure 2, together with the three main bedrock types. The extent and quality of geological mapping varies across projects. In some cases, jointing, rock type and weakness- or fault zones were not described in detail. Hence, focus has been on assessing the data with respect to the main rock types in the Oslo region.

The data has been provided by the clients and consultants involved in the planning and execution of the projects. The extent of the monitoring and quality of the monitoring varies between projects. Since the data is collected from completed projects, it has not been possible to influence the extent of monitoring. The data collected is considered representative of the Norwegian state of-practice for monitoring of tunnelling projects in urban areas.

4. ANALYSES

Pore pressure reduction at bedrock level has been recorded in 14 tunnel projects. Figure 4 presents the measured decrease in pore pressure, in relation to horizontal distance from the tunnel centre. In many projects pore pressure reduction has been observed up to several hundred of metres from the tunnel centreline, typically up to 400 m. This large spatial influence is due to hydrogeological conditions with a confined aquifer over bedrock, which is sensitive to drainage. In the right-hand side of the figure the data is systemized with respect to the main bedrock type for the tunnels. There is no indication of a correlation between the magnitude of pore pressure reduction and bedrock type. One likely explanation for this is that the pore pressure response is primarily governed by the hydrogeological conditions of the confined aquifer underneath the clay, such as the orientation, areal extent, hydraulic conductivity and natural groundwater recharge. The scatter in data can also be explained by varying amounts of water ingress to the tunnels, as well as duration of the leakage to the tunnels with respect to time of monitoring. Despite the scatter, the data clearly highlight the potential for large reductions in pore pressure at significant distances from the tunnel centre line.

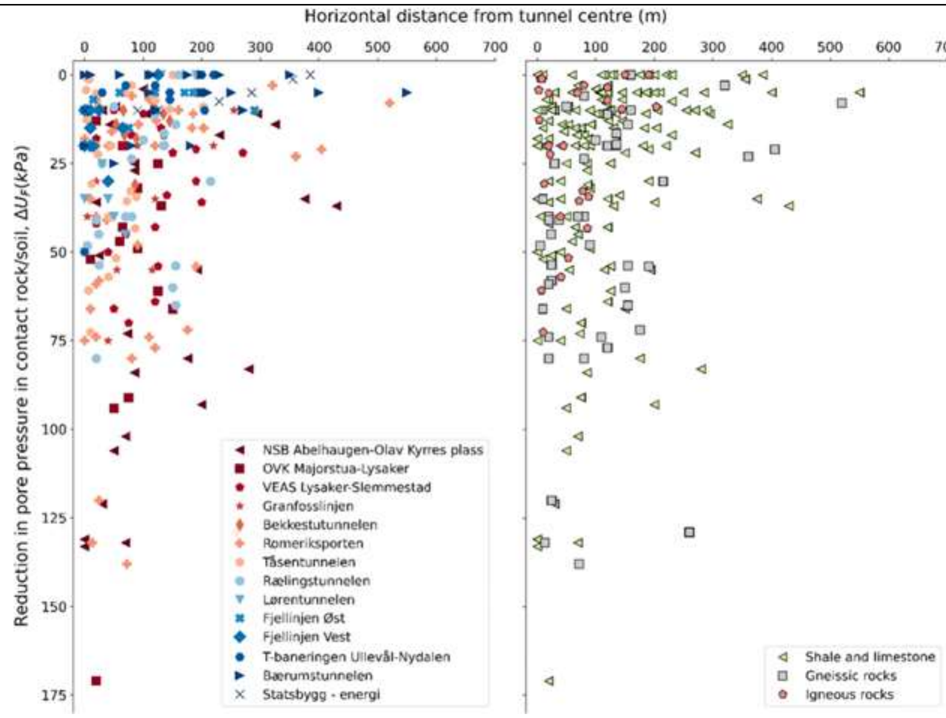

Figure 4 Relationship between measured pore pressure reduction at soil/bedrock interface in relation to horizontal distance from tunnel centre line. In the left-hand figure data is plotted for individual tunnels, in the right-hand figure data is systemized with respect to main rock type (Langford et al., 2022).

Figure 5 presents the measured decrease in pore pressure at bedrock along the centre line above the tunnel versus measured water ingress to the tunnel. This figure is used by the Norwegian tunnelling industry to determine water ingress limits in urban areas. Based on acceptance criteria for settlements, the corresponding pore pressure reduction for areas overlying the tunnel is determined. The figure is then used to choose the water ingress limit. Normally, a pore pressure reduction of maximum 10–30 kPa will result in small settlements in a clay deposit with an apparent OCR of 1.2 to 1.4 due to ageing. The shaded area indicates values "normally accepted" suggested by Karlsrud et al. (2003), with q in the order of 3 to 7 l/min/100 m, i.e. flow rates expected to result in ΔuF < 30 kPa and normally chosen as a water limit design value. It is important to note that in areas with existing tunnels or other underground structures, the pore pressures may already have been affected by leakage, causing an increase in effective stress level and a corresponding decrease in apparent OCR of the marine clay. Any additional pore pressure decrease will cause additional consolidation settlements. In these areas, limits on water ingress may be even stricter than indicated in Figure 5, down to 1–3 l/min/100 m.

The points indicate the average monitored values, the grey crosses show the range of measured pore pressure reduction and inflow rates for each tunnel. Red data points show projects without artificial water infiltration. Blue datapoints show projects with artificial groundwater infiltration in bedrock wells. For these cases, the water is infiltrated at a constant rate in drilled bedrock holes, with packers installed at approximately 5 m depth into the bedrock. The infiltration will recharge the confined aquifer through fractures the bedrock and contribute to maintaining pore pressure levels. For these projects, the decrease in pore pressure is expected to be lower for a given water ingress, compared to projects without water infiltration.

Figure 5 shows a considerable scatter in measured pore pressure reduction in relation to water ingress. This is likely caused by varying hydrogeological conditions previously described. In addition, there are uncertainties in the water ingress measurements, which are well-known to be challenging to perform. Nonetheless, there is a trend showing that pore pressure reduction increases with increasing water ingress. Based on the dataset a regression line has been derived for projects without water infiltration. This line is consistent with typical trendline previously suggested by Karlsrud et al. (2003). "Upper" and "lower" bound lines from Karlsrud et al. (2003) are shown in Figure 5. These indicate a characteristic area for the ΔuF to be expected, suggested in planning and design of tunnelling projects. The data from more recent tunnels (no. 19–26) are largely in agreement with previous data.

Projects with water infiltration were excluded when deriving these lines, as they are affected by artificial recharge.

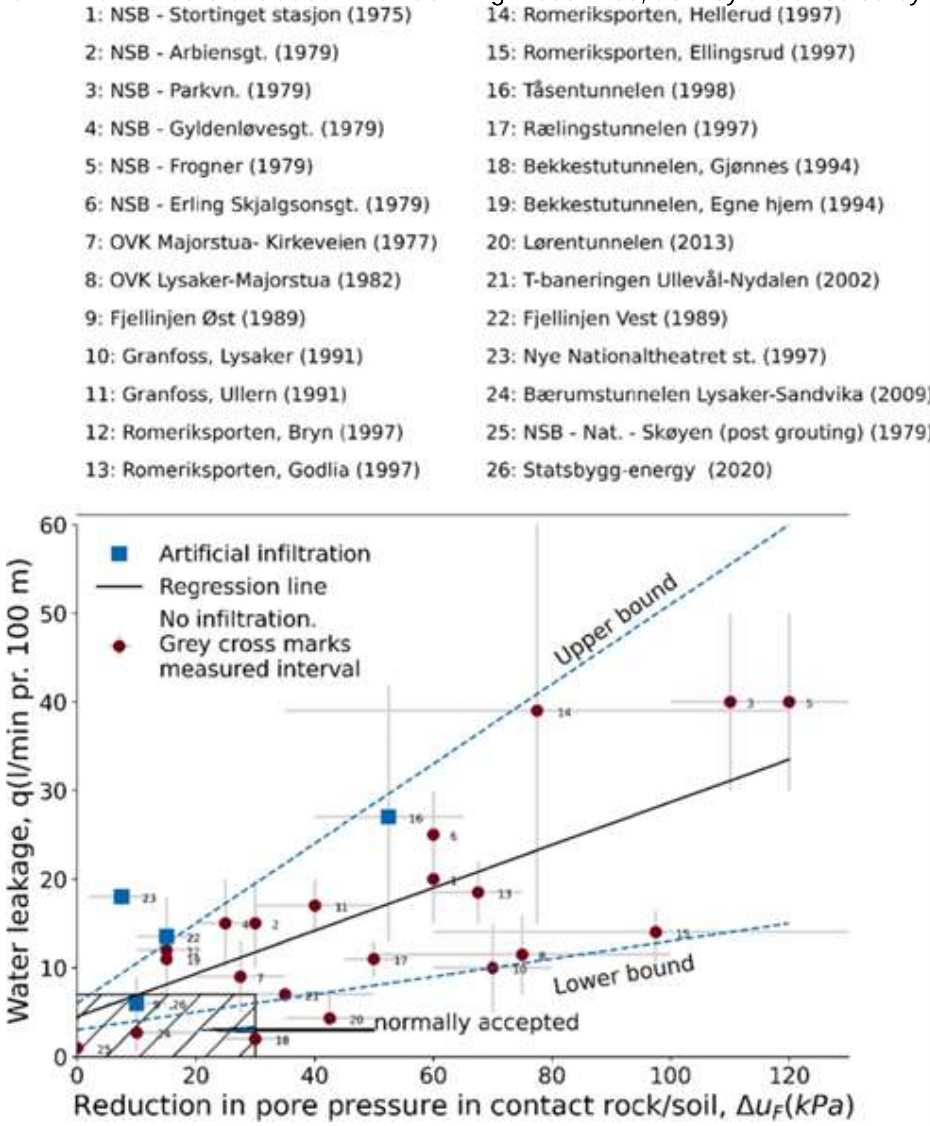

Figure 5 Relationship between reduction in pore pressure at bedrock level and water ingress. Normally accepted area is indicated as shaded area (Langford et al., 2022).

Several equations may be used for back calculating the hydraulic conductivity (El Tani, 2003, and Park et al., 2008). The equation given by Karlsrud et al. (2003) is used for back-calculation of the hydraulic conductivity in this paper, consistent with previous publications for tunnel projects in Norway. Karlsrud's equation has been shown to overestimate the water inflow rate for shallow tunnels with ratios of Re /h < 0.5 to 1.0 (El Tani, 2003). In the database, the tunnel depth below the groundwater table is more than twice the tunnel radius for all tunnel sections, with two exceptions. Hence, the approximation is considered sufficiently valid for this study.

$$Q = \pi k_i h \frac{2}{\ln\left(\frac{(R_e + t)}{R_e}\right)}$$

Where:
Q = water ingress to tunnel after PEG [m3/s/m].
ki = hydraulic conductivity of the grouted rock zone [m/s].
h = depth below the groundwater table [m].
Re equivalent radius of the tunnel [m].
t thickness of the grouted zone [m], assumed at 10 m for traffic tunnels and 5 m for sewage tunnels.

Langford et al. (2022) analysed how the grout effort (grout consumption and normalized drilling) influenced obtained K_i in different rock types. The data had significant scatter without clear correlations. A likely reason might be that K_i is back calculated from water ingress measurements over long sections, rather than measurements of leakage encountered in specific geological structures. Nevertheless, the figures imply that it is possible to obtain a hydraulic conductivity in the grouted zone of typically 4 to 6·10-9 m/s, when applying standard Norwegian practice. The most watertight tunnels after PEG has a K_i down to 1 to 2·10-9 m/s. Figure 6 indicates that the drilling effort has been larger in shale and limestone tunnels, without obtaining a more water tight grouted zone. Experience from execution indicates that the shale and limestone formations require more grouting effort in terms of drilling for PEG, to achieve a certain water tightness compared with igneous rock.

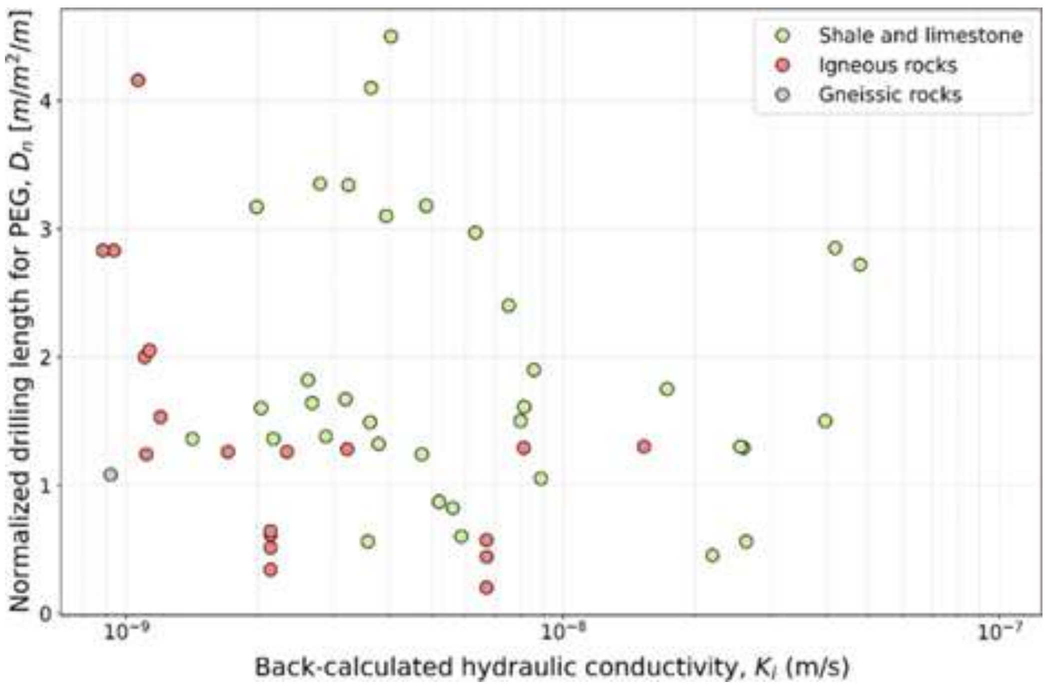

Figure 6 Plot of back-calculated hydraulic conductivity of grouted zone in relation to drilling length for grouting (Langford et al. 2022)

A timespan of 38 years gives a unique opportunity to check if it is possible to see any development for the grouting effort over time. Figure 7 shows normalized grouting consumption (kg/m2) versus back-calculated hydraulic conductivity over time. The colours gradually change from red in 1975 to blue in 2013.

The oldest projects (darkest red) have achieved hydraulic conductivity from 2 to 4 · 10-8 m/s with relatively low grouting effort (up to 65 kg/m2). While the newest projects (darkest blue) have achieved hydraulic conductivity from 1 to 4 · 10-9 m/s with grouting effort of ca. 25 kg/m2. The trend shows that almost all data from projects after 2003 have relatively low back-calculated hydraulic conductivity with mostly less than 50 kg/m2 in grouting effort. This is a good development and can be due to increased focus on grouting technique to reduce the water ingress and the risk of settlements in urban areas (Lindstrøm and Kveen, 2005; Strømsvik, 2019).

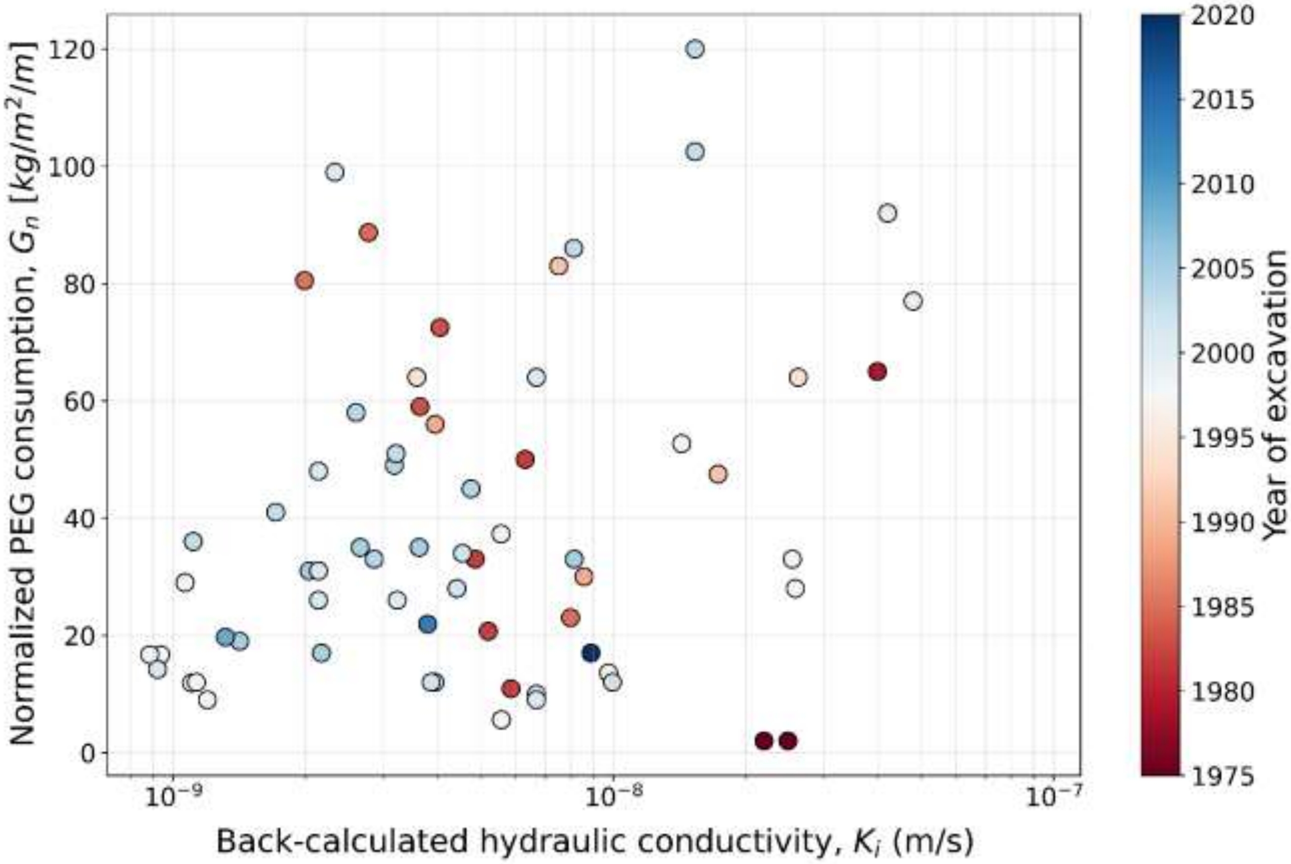

Figure 7 Relationship between achieved hydraulic conductivity and grouting effort with year of excavation shown in colours (Langford et al. 2022).

5. DISCUSSION

Much of the scatter in the database may be related to challenges in measuring water ingress, resulting in variations and uncertainty in the data itself. In addition, the measurements are average rates taken over distances ranging from hundreds of meters to several kilometres, whereas it is well known that the water ingress is often concentrated around local fracture zones. Also, the data is strongly influenced by varying geological and hydrogeological conditions for each project. Despite the scatter, the database provides unique insight into trends in terms of water ingress and pore pressure reduction and permits the suggestion of design limits for water ingress.

However, the data plotted in Figure 5 imply that estimating pore pressure reduction from water ingress levels is related to significant uncertainty. Hence, monitoring of water ingress is an insufficient measure to control pore pressure reduction. The resulting pore pressure reduction for a given leakage rate will vary depending on the hydrogeological conditions. An improved understanding of the sensitivity of pore pressure reduction in clay filled depressions could potentially be achieved with more detailed hydrogeological analysis. As an example, infiltration response tests (water loss measurements) can be performed during drilling of boreholes in bedrock for water infiltration wells. This requires installation of piezometers at bedrock level to monitor pressure levels during water infiltration. This type of test could be used to assess site-specific conditions, directly relating water infiltration rates to changes in pore pressure levels and allowing better understanding of the sensitivity of the areas to water ingress to tunnels.

6. CONCLUDING REMARKS

From the analysis it is apparent that focus should be aimed at monitoring pore pressure levels rather than water ingress, to reduce the risk of unacceptable pore pressure reduction and associated settlements. Furthermore, future projects will have stricter limits on water ingress and stricter requirements on water tightness. Real time monitoring of pore pressure during excavation and execution of PEG works, enable a more precise approach to PEG details, particularly the criteria for termination and decisions on further advance, having met the required result for maximum allowed water ingress. The following future developments are therefore proposed:

- Increased utilization of pore pressure measurements at bedrock level, to allow adjustments to PEG during construction.
- Continued research and development of PEG technology.
- Improved collaboration between geotechnical engineers, hydrogeologists, and engineering geologists. One common baseline report summarizing the hydrogeological situation, and all information related to risk of settlements (results from both geotechnical and geological investigations). In the same report mitigations such as grouting methods and pregrouting in both deep excavations and tunnels should be made.

7. ACKNOWLEDGEMENTS

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