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Doctoral thesis

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Thesis for the Degree of Philosophiae Doctor
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Abstract

The concept of unlined pressure tunnel design is well-tested and has a history of more than 100 years. In Norway, more than 95% of hydropower pressure tunnels are unlined and most of it was built between 1950 and 1990. It is also popular outside of Norway because of its cost-effectiveness and shorter construction time. The main design principle is to prevent hydraulic jacking, which is obtained by suitably aligning the tunnel such that the in-situ stresses are sufficient to withstand the internal water pressure, without the use of extensive rock support and lining. Minor rockfalls are accepted during operation as long as they do not develop significantly and increase the frictional loss or cause blockage in the tunnel.

It is seen that the operational regime of power plants in Norway has changed after the power market de-regulation in 1991. In the demand driven market, the power prices can vary on an hourly basis and the power plants can experience multiple load changes per day to benefit from the variable power prices, causing frequent pressure transients in the waterway. Further, an increasing share of unregulated energy from solar and wind power in the energy system as seen in the recent years will demand more operational changes from regulated hydropower systems which are used to maintain the balance between supply and demand. Such an operation will lead to frequent pressure pulsations and cyclic loading on the rock mass around unlined tunnels, and may contribute to increased instances of block falls as a result of rock mass fatigue.

This research is focused on understanding the effects of frequent pressure pulsations in the long-term stability of unlined water tunnels. The work is based on cases from Norway and includes observations from inspection of four dewatered tunnels, instrumentation, and monitoring of one tunnel, operational data of 10 hydropower plants and numerical modelling using the distinct element code 3DEC.

Results indicate that pressure transients can have significant influence on the pore pressure variation and joint displacement in the rock mass around unlined pressure tunnels as a result of the time-lag between the pressure transient in the tunnel and the rock mass pore pressure. It is the source of hydraulic stresses in the rock mass and is dependent on their hydro-mechanical properties. Results confirm the previous knowledge that mass oscillations cause larger hydraulic stresses in the rock mass as compared to water

hammer. However, exceptions are known and the effect of water hammer may not be completely ignored.

It is seen that 200-400 start/stops and more than 1000 load changes of varying magnitudes occur every year per generating unit in Norwegian power plants, causing frequent pressure transients. It is envisaged that this trend will further increase in the future due to addition of larger share of unregulated power from wind and solar energy. This implies that rock mass fatigue in unlined pressure tunnels may occur at an accelerated rate.

The results indicate that an increased conservatism may be needed in rock support decisions in critical areas where the rock mass permeability permits significant pore pressure changes in the rock mass during pressure transient, especially for tunnels excavated in schistose rock mass, and power plants with multiple load changes within a day.

Power plant operation is seen to have a significant influence on the amount of hydraulic stress acting on the rock mass during pressure transients. The shutdown/opening duration is usually dependent on the individual operator due to lack of standard guidelines for speed of load changes. Especially for large load changes, the power is usually changed in smaller steps, where the size and number of these steps are decided by the individual power plant operator. Results show that the shutdown/opening duration during load changes directly affects the time-lag between pressure in the tunnel water and in the rock mass. It is seen that shorter shutdown/opening duration i.e., faster speed, can cause significantly high hydraulic stresses on the rock mass. Thus, slowing down the load change operation can provide significant benefit in slowing down the fatigue process. Hence, it is recommended that more emphasis should be given towards keeping the speed of load changes consistently slow.

A new term called "Hydraulic impact" is proposed to quantify the hydraulic stress on the rock mass caused by pressure transients in unlined hydropower tunnels. It can also be used to define a suitable shutdown speed of the power plants in order to help slow down the fatigue process. It is recommended to instrument and monitor more tunnels in order to validate and expand the results.

Preface

This thesis is submitted to the Norwegian University of Science and Technology (NTNU) for partial fulfilment of the requirements for the degree of Philosophiae Doctor (PhD).

The work was conducted at the Department of Geoscience and petroleum, NTNU, Trondheim, with Professor Krishna Kanta Panthi as the main supervisor and Associate Professor Kaspar Vereide from the Department of Civil Engineering, NTNU, Trondheim, as the co-supervisor.

This PhD project is a part of the Norwegian Research Centre for Hydropower Technology (HydroCen). HydroCen is a Centre for Environment-friendly Energy Research (FME), which was established by the Norwegian Research Council in January 2017. This PhD research falls within the Work Package one (WP1) of HydroCen. The research emphasizes on the long-term stability of water tunnels, highlighting on the effect of operational regime.

The work in this thesis was financed through a 4-year PhD position at the Department of Geoscience and Petroleum. The PhD position was allocated to 25% teaching and 75% research. The teaching has included co-supervising nine master students and assisting with lecturing and assignments of the courses TGB 5110 Engineering Geology and tunneling, Basic course and TGB 4190 Engineering Geology and tunneling, Advanced course.

Funding for the procurement of instrumentation equipment was granted by HydroCen. Additional funding for field installation of the equipment at Roskrepp power plant was granted by Sira-Kvina kraftselskap. Funding for purchasing the distinct element code 3DEC was also granted by HydroCen. This thesis comprises an introduction and summary of the research that has resulted in four journal publications and three conference proceedings.

Acknowledgements

I express my sincere gratitude to my main supervisor, Professor Krishna Kanta Panthi, who has been a constant source of inspiration, not only during this PhD but also since the early days of my career as a Hydropower Engineer in Nepal. His knowledge, dedication and optimism towards work and life in general has helped me overcome many challenges during this doctoral work. I am very thankful to my co-supervisor, Associate Professor Kaspar Vereide for his scientific insight and guidance as well as the practical help during this work. I have enjoyed many fruitful discussions with him, which have helped shape the results of this work. My sincere appreciation goes to you both for your guidance, in-depth discussions and for always being available when I needed you. I express my sincere gratitude to HydroCen for providing the additional resources needed during the field instrumentation and purchase of necessary software and to Sira-Kvina Kraftselskap for allowing access to Roskrepp powerplant and funding the installation.

I am thankful to Chhatra Bahadur Basnet, who helped me immensely during the early days of my PhD by sharing his experience regarding both scientific and practical issues. It was an absolute pleasure to share an office with Henki, and I could not have asked for a better company. His good-natured presence and our many interesting scientific discussions (and not-so-scientific ones) were something I looked forward to everyday. It was a pleasant surprise that I met my old friend Trond as a fellow PhD candidate, with whom I share a common interest in bad (and dad) jokes alike.

Me and Shreejana will cherish the moments of warm welcome we received from Panthi family (Krishna, Laxmi, Kriti and Kritgya) and Basnet family (Chhatra, Sarbada, Sambodhi and Samyami) when we first arrived in Trondheim.

I am forever indebted to my dear parents for their constant encouragement. This work would not have been possible without their love and support. I think of my brother’s family (Bibhuti, Rama and little Beeraj) everyday even though we have been living far apart for many years now. Our son, Bishrut came into our lives only a few months ago and fills our hearts with joy every time we look at him. Finally, I am grateful to my loving wife, Shreejana, for supporting me in taking up this PhD and for her patience throughout these years. Thank you for your daily motivation, for listening to my rants and keeping me sane. Your calm and composed ways and high-spirited nature has affected me in many ways I cannot possibly explain. You are the best!

List of main papers and author contributions

Paper 1

Effect of power plant operation on pore pressure in jointed rock mass of an unlined hydropower tunnel: An experimental study

Authors: Bibek Neupane, Krishna Kanta Panthi and Kaspar Vereide

Published in Rock Mechanics and Rock Engineering 53: 3073–3092 (2020)

The instrumentation concept was developed by the main supervisor who is the second author of this manuscript. After this the overall instrumentation program was further developed jointly by the authors. The first author developed detailed plan in collaboration with the co-authors and conducted the installation of the equipment. The second author conceptualized the rock mass pore pressure measurement including location, orientation/length of boreholes and packer arrangement for the boreholes. The third author conceptualized choice of sensors and piping arrangement and data logging frequency for all sensors. Third author was also the contact person for gaining access to the power plant and managing installation help needed from the power company. The manuscript was written by the first author except for section 2.1 which was written by the third author. Both co-authors also reviewed and edited the manuscript.

Paper 2

Operation of Norwegian hydropower plants and its effect on block fall events in unlined pressure tunnels and shafts

Authors: Bibek Neupane, Kaspar Vereide and Krishna Kanta Panthi

Published in Water 13(11), 1567 (2021)

The first author conceptualized the paper, performed the analysis, and wrote the manuscript. Both co-authors reviewed and edited the manuscript.

Paper 3

Evaluation on the effect of pressure transients on rock joints in unlined hydropower water tunnel using numerical simulation

Authors: Bibek Neupane and Krishna Kanta Panthi

Published in Rock Mechanics and Rock Engineering 54: 2975–2994 (2021)

The first author conceptualized the paper, performed the numerical modelling and analysis, and wrote the manuscript. The co-author reviewed and edited the manuscript.

Paper 4

Cyclic fatigue in unlined hydro tunnels caused by pressure transients.

Authors: Bibek Neupane, Krishna Kanta Panthi and Kaspar Vereide

Published in Hydropower and Dams 5:46-54 (2021)

The first author conceptualized the paper and wrote the manuscript. The co-authors reviewed and edited the manuscript.

List of secondary papers

Neupane B and Panthi KK (2018) Effect of pressure fluctuations in long-term stability of unlined pressure shaft at Svandalsflona Hydropower project. In: Proceedings of 10th Asian rock mechanics symposium. ISRM international symposium. Singapore 29 Oct–3 Nov 2018.

Neupane B, Panthi KK and Vereide K (2019) Method for monitoring of pore pressure in jointed rock mass of an unlined headrace tunnel subjected to varying power plant operation: A case study. In: Proceedings of 14th ISRM Congress 2019, 13-18 September 2019, Brazil.

Neupane B and Panthi KK (2020) Numerical Simulation of Pore Pressure in Rock Joints During Pressure Transient in an Unlined Hydropower Tunnel. In: Proceedings of ISRM International Symposium - EUROCK 2020, June 2020.

Table of Contents

1. Introduction

1.1 Motivation

Unlined tunnels constitute more than 95% of the water conveyance system in Norwegian hydropower plants, with a total length of more than 4300 km and pressurized up to 1047 m of water pressure (Panthi and Basnet (2018), NFF (2013)). Most of the power plants that are now in operation in Norway were built before the 90's, with more than 3500 km of tunnels excavated between 1950 and 1990 (Broch 2016). About 1000 km tunnels have been built for hydropower since then. In these tunnels, the water is in direct contact with the rock mass around tunnel and support and lining is provided only in sections where the rock mass is deemed unable to withstand the internal water pressure. The design principle uses maximum water head as a design parameter and occasional pressure transients are not perceived to have significant impact on the long-term stability of tunnels.

However, the recent trend in operation of Norwegian hydropower plants shows that start/stop sequences and load changes have become more frequent as compared to what was envisaged during the design. The reason for this change in operational regime is the deregulation of power market implemented in 1991 (Bye and Hope, 2005), which caused the energy prices to vary every hour. Further, the inclusion of Variable Renewable Energy (VRE) or intermittent sources such and wind and solar energy in the grid causes price volatility due to an increased gap between supply and demand. According to Irena (2018), the share of renewable energy in the power sector would increase from 25% in 2017 to 85% by 2050, mostly through growth in solar and wind power generation. According to Eurostat (2021), the share of energy from renewable sources in gross final energy consumption has almost doubled in the last years in Europe, from around 8.5 % in 2004 up to 17.0 % in 2016. In Nordic and European power market, the share of solar and wind power is expected to increase from about 20% to over 55% between 2018 and 2040 (Stattnett, 2018).

Norway has almost half of the reservoir capacity in Europe (Energifaktanorge, 2021) and is exploring possibilities for providing the much-needed flexibility by acting as a battery for the future European power system (Graabak et al. 2017). However, the inclusion of larger amount of wind and solar power may increase the price volatility due to surplus or deficit of energy at any given time, depending upon the availability of wind and sun. This demands higher flexibility in operation of the existing power production and storage systems. Flexibility is the ability to make quick changes in operation at any time such that the balance between production and consumption can always be maintained, with lowest possible cost for carrying out such changes. The need for such flexibility can be both short-term where changes are needed to balance the system within hours, minutes and seconds, or long-term, in order to balance the system for days or weeks. There are various solutions that compete to provide the flexibility such as hydropower, hydrogen, and batteries. Among these solutions, regulated hydropower can provide both short and long-term flexibility. This implies that the future operation of hydropower plants will exhibit increased dynamic behaviour, with higher frequency of load changes of larger magnitudes. Such operation is of significance regarding long-term stability of unlined tunnels because every load change or start/stop causes sudden pressure transients in the system.

Hence, the motivation of this research comes from the fact that unlined tunnels now face increased vulnerability to block falls due to rock mass fatigue occurring as a result of increased dynamic operation. Thus, there is a need for a better understanding of the phenomenon that contributes to additional loading due to changed operational regime, which was not envisaged during the design of such tunnels. This research focuses on the effect of such dynamic operation, in the rock mass around the water tunnels of Norwegian hydropower plants. The analysis will focus on understanding the pore pressure response of the rock mass during pressure transients and the inter-relation between power plant operation and the resulting destabilizing forces under various rock mass conditions.

1.2 Research objectives

This research aims to contribute to fulfil the knowledge gap between the state-of-the art design principle of unlined pressure tunnels and problems of block falls associated with the current and future operational regime of Norwegian power plants. The main objectives of this research are listed as follows:

1. Investigate the basic mechanism of pore pressure changes which occurs in the rock mass during pressure transients.
2. Analysis of inter-relation among rock mass property, hydraulic/operational factors, and destabilizing forces along unlined pressure tunnels subjected to pressure transients.
3. Identify the most critical parameters that can contribute to rock mass fatigue and block falls due to frequent pressure transients.

The work is limited to unlined pressure tunnels where water is in direct contact with the rock mass in tunnel periphery such that flow and pore pressure in rock joints is directly affected by the change in tunnel water pressure during pressure transients.

1.3 Thesis structure

Chapter 1 has presented the motivation and research objectives. Chapter 2 presents overview of the state-of-the-art of the design of unlined pressure tunnels and recent developments in other relevant fields of interest regarding fluid flow in rock joints and rock mass fatigue due to cyclic loading. Chapter 3 outlines the research methodology. A summary of the results are presented in Chapter 4 and Chapter 5 where main conclusions, possible applications of the research, limitations and uncertainties are highlighted, and recommendations for further work are made. Appendix A consists of full text of the main papers. Appendix B contains the co-author statements.

2. Literature review

2.1 State-of-the-art for design of unlined pressure tunnels

The First World War led to a shortage of steel that affected the cost and construction time of hydropower plants in Norway. As a result, the construction of unlined tunnels started in the Herlandfoss power plant in 1919, with the motivation to reduce the use of steel lining in pressurized sections of the waterway (Vogt, 1922). The main objective of unlined pressure tunnel design is to avoid hydraulic jacking by providing sufficient confinement. Along with Herlandfoss, three other power plants with unlined waterways were commissioned between 1919 to 1921 with static heads ranging from 72 to 152 m. Over the years, the design criteria have been updated based on experience from its applications to higher static heads. The designs before 1968 were based on rule of thumbs connected with the general layout of the plant, in which pressure shafts with inclination of 45^0 were most common (Broch, 1984). The rule of thumb was revised by Selmer Olsen (1970) after the failure of pressure shaft in Byrte power plant, which has an inclination of 60^0. This revised rule would also be applicable for shafts steeper than 45^0, which were used until the failure of pressure shaft in Askora power plant in 1970. This failure led to further development of the rule of thumb by Bergh-Christensen and Dannevig (1971). The updated rule considers the shortest perpendicular distance from the valley slope, which is a significant development from the previous version which only considered the vertical rock cover. The parameters used in the rule of thumb are illustrated in Figure 1.

Even though the rules of thumb are still in practice today, they do not offer a complete solution because of unexpectedly low in-situ stresses due to local variation in geological and topographical conditions. This has been witnessed in the failure of tunnels of some power plants such as Bjerka and Fossmark power plants in 1971 and 1986 and recently in Bjørnstokk power plant in 2016 (Solli, 2018). Basnet and Panthi (2018) have carried out a detailed assessment of some unlined pressure tunnels and shaft failure cases mentioned above using 3D FEM numerical modelling. Hence, the rules of thumb should be used for preliminary design only and must be confirmed by in-situ stress measurements during construction. The construction contract should be flexible such that design changes can be made after the in-situ stress situation is properly investigated. Ødegaard and Nilsen (2021) propose a simplified method to increase the cost effectiveness of minimum principal stress measurement so that number of tests along the waterway can be increased in order to optimize the design during construction phase.

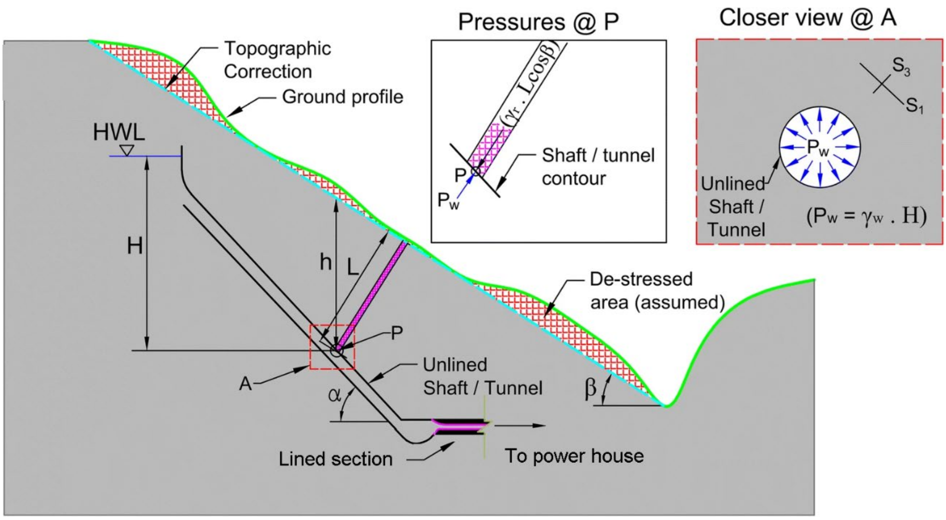

Figure 1: Different parameters used in different design criteria for unlined shaft/tunnel (Note: S1 is major principal stress, S3 is minimum principal stress, and HWL is head water level) (modified from Basnet and Panthi, 2020).

In addition, numerical modelling is a very important tool that is used to carry out a detail assessment of the in-situ stress condition in the area of question during design phases of the project. A validated rock stress model of such area can be made using the suggested method in Stephannson and Zang (2012) by using measured stress data which can be used for the prediction of in-situ stresses along the proposed unlined tunnel. Such a model has been prepared by Basnet and Panthi (2019) for the Upper Tamakoshi hydroelectric project in Nepal.

Unlined pressure tunnels have also been popular outside of Norway because of its cost-effectiveness. Some noted international applications of this design concept are in Colombia (Broch et al. 1984), Tanzania (Marwa, 2004), Portugal (Lamas et al. 2014), Chile (Norconsult, 2021), Albania (Assen et al. 2013) and Nepal (Panthi and Basnet, 2017). In addition to this, some studies have been conducted (Rancourt (2010), Meritt, (1999)) to enhance the knowledge on this topic and to extend its applicability of the design concept in the Himalayas (Basnet, 2018). The state-of-the art for design of unlined tunnels are explained in detail in Palmstrom and Broch (2017) and are summarized as follows:

1. Tunnels should be located in suitable rocks with sufficient confinement to avoid hydraulic jacking, which could lead to water leakage failures. This is ensured by locating the tunnels such that minor principal in-situ stress is always greater than the static water head, with recommended factor of safety for design.
2. Unstable rock mass in weakness zones, swelling and/or friable materials should be detected and provided with sufficient support.
3. During operation, minor rockfalls can be tolerated as long as they do not develop significantly and increase the frictional loss or cause blockage in the tunnel. Minor rockfalls spread out along the tunnel and are acceptable, which are trapped using a rock trap provided at the end of unlined section.
4. The tunnels should be filled and dewatered such that sudden changes in pore pressure in the rock mass around tunnel is avoided. First tunnel filling is always critical and should be done in a controlled manner. Monitoring of pore pressure and leakages should be done at critical locations during first infilling.
5. The frictional loss in unlined tunnel should be closely monitored during operation, especially during the first year of operation. Significant increase in frictional loss indicates stability problems.

The design factor of safety against hydraulic jacking currently in practice is 1.3 for static condition, 1.1 for surging (mass oscillation), as recommended by Benson (1989). No factor of safety is recommended for water hammer as the time of application of the hydraulic stress are deemed too short to cause hydraulic jacking. Hence, it can be seen that the recommended design method only addresses hydraulic jacking as the primary objective and does not address the issue of frequent dynamic loading and the resulting fatigue of rock mass during power plant operation.

Brekke and Ripley (1987) mentions that if operational requirements lead to frequent pressure pulsations, special efforts may be needed to ensure that blocky, unlined sections remain stable over the operational life of the powerplant. It further mentions that rates of recurrence of dynamic pressures are higher in pumped storage and peaking plants than in base load plants and such operation necessitates increased conservatism in the design due to fatigue of the natural geological materials. However, based on a literature review, it can be concluded that there is a knowledge gap regarding how pressure pulsations travel into the rock mass through joints and affect the tunnel stability in long-term.

2.2 Reported instabilities and block fall events

Various literature such as Brekke and Ripley (1987), Lu (1987), Palmstrom (2003), Brox (2017) and Palmstrom and Broch (2017) have documented a number of instabilities in unlined pressure tunnels around the world over its long history of design and operation. The reported cases date back to as long as 1911 and the instabilities in these have occurred just after tunnel filling to after more than 50 years of operation. This indicates that the tunnels and shafts have failed as a result of insufficient design/rock support measures as well as the effect of operation and/or a combination of both these factors. The main reasons of failure in these reported cases can be broadly categorized as presented in Table 1.

Table 1:Main reasons of reported unlined pressure tunnel/shaft failures

Reason of failure	Remarks
Hydraulic jacking	Design failure: Insufficient cover/confining stress Example cases: Byrte (1968), Askora (1970), Bjørnstokk, (2016) References: Basnet and Panthi (2018), Solli (2018).
Insufficient support in weak rock mass with or without swelling clay	Insufficient rock support: Short shotcrete/concrete lining length, missing concrete lining in invert and support with insufficient stiffness; leading to development of swelling pressure, erosion of gouge/infilling material Example cases: Lower Vinstra II (1991), Svandalflona (2008) Matre Haugsdal (2017) References: Palmstrom (2013), Panthi (2014), Author visited the Matre-Haugsdal site and tested the swelling clay sample
Support deterioration	Deterioration of poor-quality concrete Example case: Rendalen (1971) Reference: Kjølberg (1993)
Dynamic power plant operation	Hydraulic transients in tunnels: Frequent pore pressure fluctuation in rock joints leading to erosion of shear zone material resulting debris flow and block falls Example cases: Yuba New Colgate tunnel, (1970), Svandalsflona (2008) References: Lang et. al (1976), Panthi (2014)

Thidemann and Bruland (1991) inspected 330 km length of 35 unlined tunnel in Norway operating from 8-70 years. They found that majority of the tunnel length was stable if blocks less than 0.05 m³ are excluded. Major stability problems were mostly observed at local faults and weakness zones. An average 3.5 block falls of volume between 0.1 and 3.5 m³ were found per km. Similarly, block falls larger than 3.5 m³ were found to be 1 in every 5.5 km and two instances of block falls were found to block the whole tunnel profile.

Out of the reasons mentioned in Table 1, the one that is of interest for this study is the pressure fluctuation due to dynamic power plant operation. Bråtveit et. al (2016) conducted a similar study in a total tunnel length of 107 km and compared the results. The inspections were done 18 years apart but there is a major difference in operational regime of Norwegian power plants before and after 1991. The Norwegian power market was de-regulated, which led to more dynamic operation, causing a larger number of hydraulic transients in the tunnel. Such an operation is referred to as "Hydropeaking". Hydropeaking is defined as an operational mode in which the load change in power plants happen multiple times per day to benefit from variable power prices, causing frequent pressure transients in the waterway.

Bråtveit et. al (2016) concluded that compared with results from the previous study, the frequency of rock fall has increased by a factor of 3.4 in tunnels that have been subjected to hydropeaking but the average size of the blocks were reduced by 25%. They further concluded that that instability problems were still largely related to local faults or weakness zones, but the rock falls also occurred randomly in 2 out of 10 tunnel systems that were inspected. This suggests that changed operational regime in the recent years has not only increased number of instabilities in the weakness zones but also affected relatively competent rock mass. This indicates that tunnel sections with competent rock mass which are usually left unsupported have undergone fatigue as a result of frequent pulsations and cyclic loading.

2.3 Power plant operation

The Norwegian power market has undergone significant changes after the power market regulation in 1991. Bye and Hope (2005) explains how the Norwegian power market has evolved as a result of the market reform which intended to create a balance between demand and supply. Before the market deregulation, around 90% of the power was sold on long-term fixed contracts and thus the energy market was inflexible to address the changes in generation, resulting from the stochastic nature of inflow to hydropower systems. The power market was converted into a fully market-based system in 1991 following the new Energy Act in 1990. A common Norwegian-Swedish power market was established in 1996 as the first intercountry integrated power market in the world, which was joined by Finland and Denmark in 1998 and 2002, respectively. Today, this integrated market called the Nord Pool offers trading in both day-ahead and intraday markets across 16 European countries (Nordpoolgroup, 2021). Electricity is traded on a daily basis for delivery the following day (day-ahead market). Producers submit bids stating how much they are willing to produce at a specified price. End users submit bids indicating how much they wish to consume at different prices. The energy price thus determined is called the Nord Pool system price, which is the market equilibrium price for the aggregated supply and demand schedules for each hour (Energifaktanorge, 2021).

Since the energy prices can now vary from hour to hour, the operational regime of power plants has significantly changed as a result. This directly affects the number of times the power plant changes production over a period of time and the magnitude of such load changes. Currently, the power market is moving towards 15-minute resolution which will further impact the operational regime of the power plants.

As explained in Chapter 1, the global shift towards renewable energy has led to an ever-increasing proportion of intermittent energy sources in the energy grid. Hydropower is considered the most promising "battery" till date and consequently, has to be operated with increasing dynamics in order to maintain the balance between supply and demand. The load changes in power plants are done multiple times per day to benefit from variable power prices, causing frequent pressure transients in the waterway.

Frequent load changes cause rapid changes in the downstream water level of the power plants and thus affects the aquatic environment. To be able to mitigate this effect, licenses for Norwegian hydropower plants sometimes include restrictions against hydropeaking. Such restriction requires the hydropower plant to be run smoothly and that load changes occur gradually so that sudden changes in the outlet water level are avoided. Hence, operational regime of power plants is significantly depending upon whether or not such restrictions are imposed.

2.4 Hydraulic transients

Every start, stop and load change in a hydropower plant generates flow and pressure transients in the waterway. In the example of a shutdown, the result is a rapid decrease of the water flow in the waterway and the deceleration of the water causes a pressure increase on the upstream side of the turbine, and a pressure decrease on the downstream side. The elasticity of water will result in a pressure transient with a short time period, referred to as water hammer (Parmakian, 1963). The water hammer starts from turbine and progresses towards the nearest free water surface where the water hammer is reflected back towards the turbine. In this manner, the water hammer may travel back and forth many times until the energy is dissipated by friction.

To reduce and control the water hammer, many hydropower plants are constructed with a surge tank. The surge tank is constructed close to the turbines to reflect the water hammer as soon as possible, in order to reduce the amplitude and the affected tunnel length. However, in hydropower plants with a surge tank, a pressure transient with a long time period will occur, which is referred to as mass oscillations (Chaudhry, 1987). Mass oscillations are caused by the inertia of the water in the tunnel between the reservoir and the surge tank. When the turbines close, water cannot flow through the turbine and will instead flow into the surge tank, causing the water level to rise. In the opposite case of a power plant startup, the water level in the surge tank will drop as it takes time to accelerate the water in the rest of the headrace tunnel to the reservoir. The rise or drop of the water level in the surge tank will reverse once the water in the main tunnel is accelerated or decelerated. This mass oscillation will oscillate back and forth until the energy is dissipated by friction. Mass oscillations have larger time period than water hammer and thus their impact on flow through rock joints around unlined tunnels will be different. A typical layout of a hydropower plant along with the waterway lengths where water hammer and mass oscillations occur is shown in Figure 2a. Figure2b shows a typical pressure measured during a shutdown event, recording both water hammer and mass oscillations. This is because the measurement location is between the turbine and the surge tank. A measurement done between the surge tank and reservoir will only record mass oscillation.

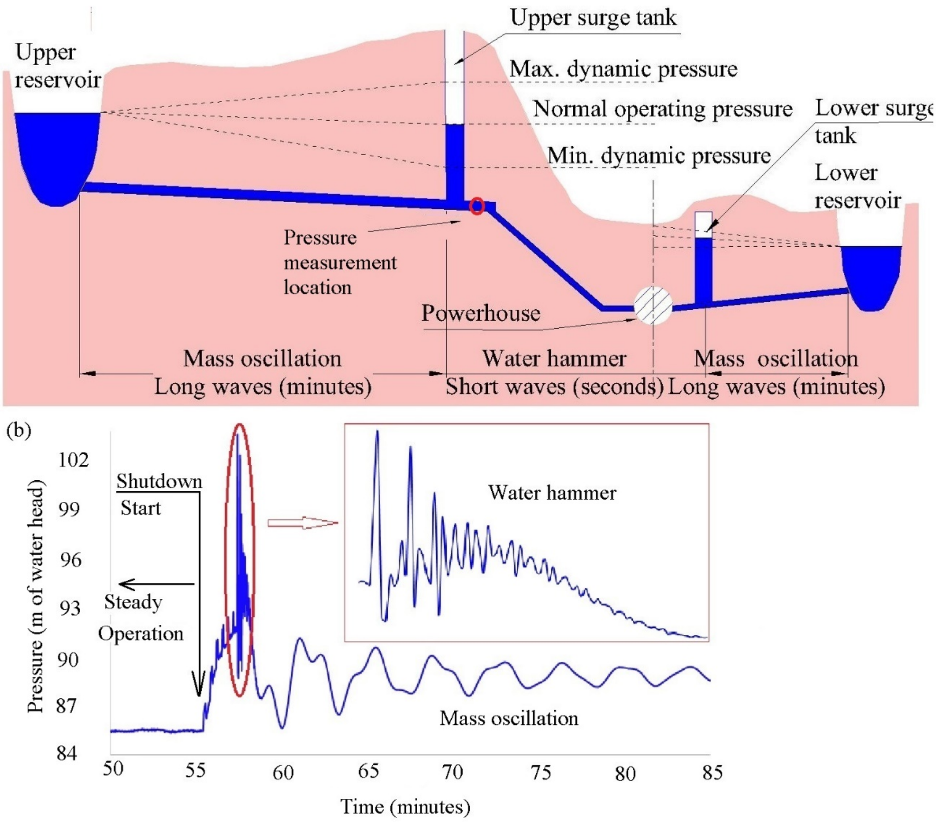

Figure 2: Typical layout of a hydropower plant (a) and pressure signal measured during a shutdown event showing water hammer and mass oscillation.

2.5 Flow processes in rock mass

A rock mass consists of solid intact rock material of varying strength and permeability, which is divided into blocks by a network of structural discontinuities or joints. The mechanical process that involves stresses and deformations in a rock mass governs the stability of any civil engineering structure that is built over or inside it. In addition, the presence of fluid or groundwater in the fractured rock mass has a significant role to play in this context. As shown in Figure 3, a fluid saturated porous medium or rock fracture can deform because of either change in the external load (or stress) or of change in the internal pore-fluid pressure. Fluid flow in a fractured rock mass mainly occurs through two different ways. (1) seepage through the solid intact rock blocks, and (2) flow though interconnected network of fractures. For all practical purposes, the rock mass is assumed impermeable and thus the flow through fractures dominates the fluid flow through a rock mass. Hence, physical characteristics of the joint or fracture plays a crucial role in this phenomenon.

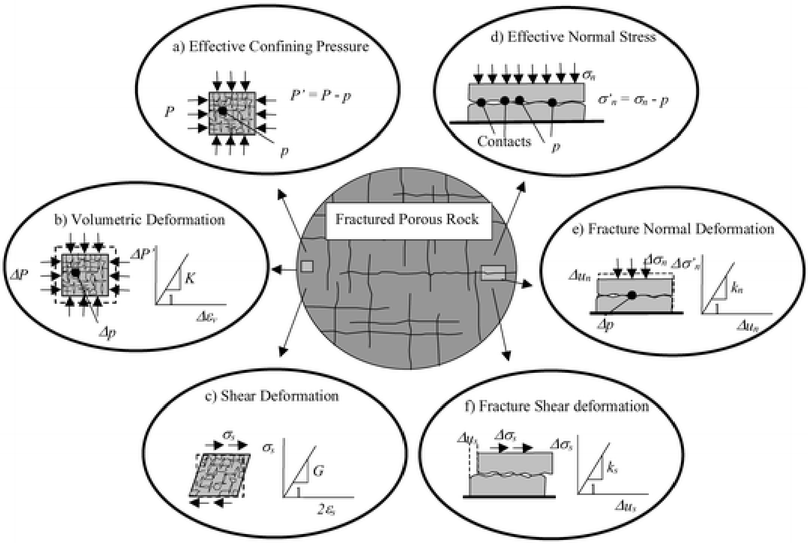

Figure 3: Schematic overview of a fractured geological medium composed of an intact porous rock matrix and macro-fractures. (Rutqvist and Stephansson, 2003).

Hydro-mechanical behaviour of rock joints

The mechanical and hydraulic processes in rock joints are interlinked with each other such that each process is affecting or being affected by the other one. Such physical interaction between these processes is referred as ¨hydro-mechanical (HM) coupling¨. The change of fluid pressures (and hence the change of effective confining stress) on the rock joint affects the deformation of fractures, which causes the aperture to change. This change of hydraulic apertures affects its flow rate and fluid pressure distribution along the fracture surface, which in turn affects the deformation. Hence, the mechanical and hydraulic behavior of a rock joint are not independent but are interacting with each other.

Significant research has been carried out regarding the hydro-mechanical behavior of fractures under normal stress and are summarized in Rutqvist and Stephansson (2003).

Hydro-mechanical coupling is divided into two types, i.e., direct and indirect couplings. As described by Wang (2000), direct coupling includes two basic phenomena: I. Solid-to-fluid coupling that occurs when a change in applied stress produces a change in fluid pressure or fluid mass. The applied stress produces displacement in the rock joints. This deformation generates surface stress on the fluid domain boundary, which deforms accordingly. A reduction in channel volume induces fluid outflow. II. Fluid-to-solid coupling. It occurs when a change in fluid pressure or fluid mass produces a change in volume of the porous medium. A fluid inflow induces fluid pressure along the flow channels, which act on the channel boundaries and deforms the surrounding rock material. As a result of deformation, the rock counteracts the fluid pressure with surface stress at the fluid–rock boundary, which affects the fluid pressure and volume of fluid domain.

Mechanical and hydraulic processes can also affect each other through change in material properties, which are considered as indirect coupling. For example, the reduction in channel volume may increase contact area between the joint surfaces resulting a stiffer material. Indirect HM couplings tend to be most important in fractured rock mass or intact rock with fat inter-grain micropores, where changes in permeability caused by fracture or pore dilation can be dramatic (Rutqvist and Stephansson 2003). Indirect coupling composes of two basic phenomena: a solid-to-fluid coupling that occurs when an applied stress produces change in hydraulic properties; and a fluid-to-solid coupling that occurs when a change in fluid pressure produces a change in mechanical properties.

The coupling between fracture flow and deformation under normal stress is described using the parallel-plate flow concept as is referred to as the ''modified cubic law'' (Witherspoon et al. 1980).

$$q = \frac{(bh_i + f\Delta U_n)^3 w\rho g}{12\mu} \frac{\Delta p}{l} \tag{1}$$

$$bh = bh_i + f\Delta U_n \tag{2}$$

Where, where q is the flow rate per unit width (w), ρ is the fluid density, g is the gravitational acceleration, μ is the fluid dynamic viscosity, Δp is the pressure difference; and l is the length of joint, bh_i is initial hydraulic aperture, bh is the hydraulic aperture, U_n is the fracture normal displacement and f is a factor reflecting the influence of roughness on the tortuosity of flow. As seen in the above expression, hydraulic aperture is the most important parameter that governs the flow through a joint. Therefore, it is necessary to understand the mechanics behind the deformation of the joints under varying amount of normal stress acting on it.

Fracture displacements are induced by a change in the effective stress field acting on the fracture. The mechanical behaviour of rock joints has been studied extensively over the years, Various non-linear models (Goodman (1976), Barton et al. (1985), Evans et al. (1992) have been proposed to explain the deformation of joint due to normal loading. The fundamental relation between a change in fracture normal displacement (U_n) and shear displacement (U_s) caused by a change in effective normal (σ_n) and shear stresses (σ_s) can be explained using the linear equations of Goodman et al. (1968)

$$\Delta U_n = \frac{\Delta \sigma_n}{k_n} \tag{3}$$

$$\Delta U_s = \frac{\Delta \sigma_s}{k_s} \tag{4}$$

Goodman et al. (1968) first introduced the terms ¨normal stiffness (k_n)¨ and ¨shear stiffness (k_s)¨ to describe the rate of change of normal stress with normal displacement and shear stress with shear displacement respectively. The idea behind introducing these terms was that the rock mass classification systems until then took joints into consideration, mainly to refer to the pattern of joint sets, with only minor attention to the ¨character¨ of such joints. Further, the approach of idealizing the rock mass as a continuum by increasing the deformability and decreasing the strength of the rock mass to account for the effect of joints was not considered representative of the scenario, where most failures in civil engineering construction occur on defects in the material, i.e, joints in the case of rock mass. The introduction of joints with relevant properties was proposed to be used in finite element analyses to have a more realistic representation of the rock mass.

k_n depends on the contact area ratio between two joint walls, the perpendicular aperture distribution and amplitude/aspect ratio. k_s depends on the roughness of the joint walls determined by tangential distribution, amplitude and inclination of asperities. Both k_n and k_s also depend on the relevant properties of joint filling material. In addition, shear strength, S, along the joint (described by c and $\emptyset$) is an important parameter. It depends on friction along the joint, cohesion due to interlocking of asperities and the strength of the filling material. Further, water content in a joint will influence all three parameters indirectly through the influence on filling material properties and may also directly influence the frictional strength of an unfilled joint.

Application in hydropower tunnel design

For hydropower plants in the construction phase, inflow into excavations plays a major role in stability of the working face and progress of construction. Considerable research has been caried out for predicting inflow into underground excavations [Goodman et al. (1964), El Tani (2003), Moon and Fernandez (2010), Holmøy and Nilsen (2014), Panthi and Basnet (2021)] During the operation phase, leakage through rock joints is of primary importance which can cause severe economic losses, especially for unlined pressure tunnels. Hence, the leakage is monitored during first water filling to ensure that the minimum leakage criteria is met (Palmstrom, 1987). Schleiss (1986) mentions that water infiltrates the cracks and fissures in permeable medium and exerts surface pressure, which is not loaded purely by the mechanical effect of water pressure (as boundary loading) but by hydraulic body forces. Fractures are deformed by these forces and therefore, permeability in rock mass around the pressure tunnel will be increased. This change in permeability that in turn affects the seepage flow and, therefore the seepage forces. Hence the consideration of hydro-mechanical coupling is relevant for the design and operation of such tunnels.

Water hammer and mass oscillation may not cause significant increase in seepage flow through joints and out of the tunnel system but the pore pressure response of rock mass during such dynamic events is important. This is because they can induce additional pressure on rock joints, which is of interest regarding tunnel stability in long-term operation. Helwig (1987) conducted a theoretical study to estimate the depth to which significant transient pressures are transferred to the rock mass. It concluded that the effect of water hammer is limited to a relatively shallow depth around the tunnel walls and the pore pressure changes are not enough to cause instabilities. It further mentioned that mass oscillations, because of their large time period, travel considerably deeper into the rock mass and thus the design should be based on maximum surge pressure (mass oscillation) rather than static pressure. Hence, from the available literature, it is evident that the application of fluid flow theory and hydro-mechanical coupling is limited to maintaining equilibrium at static conditions with acceptable leakage values. However, a detailed study of long-term effects of pressure transients and additional loading due to pore pressure variation is yet to be conducted.

2.6 Cyclic fatigue in rock mass

Failure in any material can occur as a result of a monotonic load that exceeds the strength the material, or as a result of a cyclic load acting for longer time with cyclic stresses smaller than the monotonic strength, referred to as cyclic fatigue. Further, fatigue can also occur as a result of a sustained load or residual stress acting for a long time, referred to as stress corrosion as described by Schijve (2009). Cyclic fatigue is the phenomenon in which accumulation of plastic deformation and damage of a material occurs as a result of a number of cycles of load which is lower than its monotonic strength. The most important parameters for cyclic fatigue are the magnitude of cyclic stress, frequency, and the number of cycles. Extensive research has been done carried out in the field of cyclic fatigue of both intact rock and rock joints.

Burdine (1963) conducted one of the first studies regarding the cumulative damage of intact rock under cyclic stress. Costin and Holcomb (1981) presents a model which describes the failure of brittle rock under cyclic compressive loading. Cyclic fatigue behaviour in natural rock material has been studied by various researchers in the past and is reviewed comprehensively by Cerfontaine and Collin (2018). The main conclusion of these studies is that stress corrosion and fatigue mechanisms are responsible for the subcritical crack on rock specimens, such that stress corrosion dominates at high mean stress while fatigue mechanism is dominated by high-cycle amplitude. Fatigue in intact rocks is a result of progressive decohesion and loosening of material caused by microcracks initiating and propagating to form a macroscale crack (Cerfontaine and Collin, 2018). It is seen that the results of cyclic loading are different in terms of crack growth process as compared to monotonic loading, which is shown in Figure 4.

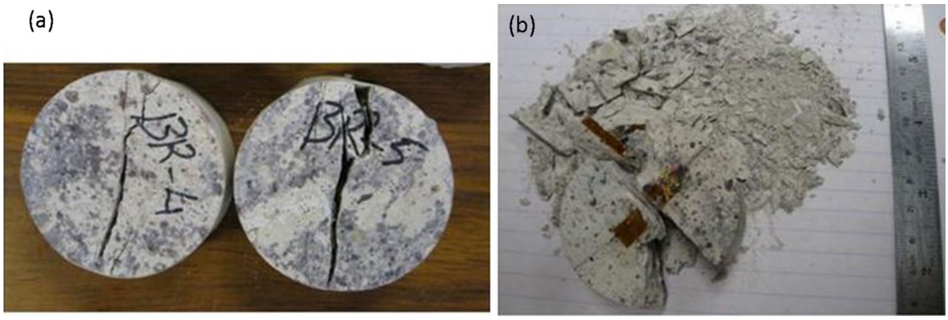

Figure 4: Brazilian disc specimens of Brisbane tuff after failure by monotonic loading (a) and cyclic loading (b) from Erarslan et al. (2014).

A monotonic load results in a definite crack while cyclic load involves a wider fracture zone which creates significant crack and dust (Erarslan 2016, Erarslan et al. 2014). This is because for monotonic loading, the failure mode is brittle and the rock grains along the failure surface are highly cracked. Whereas for cyclic loading, failure occur along grain boundaries and inter-granular cracks are the primary failure mechanism. Also, the wear and shearing between rock grains starting at the boundaries further leads to intragranular cracks. The failure finally results from the coalescence of many microcracks rather than the growth of a single macrocrack as discussed by Cerfontaine and Collin (2018) and Erarslan (2016).

The effect of cyclic loads on rock joints is also important when assessing the fatigue of a rock mass in general. The strength reduction occurs because of shearing of asperities and surface degradation of rock joint wall a result of shear stress. Some constitutive models have been proposed by Belem et al. (2007) and Nemcik et. al (2014). Experimental results [(Liu et al. (2018), Tsubota et al. (2013), Ferraro et al. (2010) Jaferi et al. (2004)] show that number, frequency, and stress amplitude of the cycles reduce the resulting peak and residual shear strength of the joints subjected to cyclic loading.

Patton (1966) classified the asperity of rough joints into first and second order, which represent the waviness and unevenness of the surfaces, respectively. A typical example of how these asperities are damaged as a result of cyclic shear loading is shown in Figure 5.

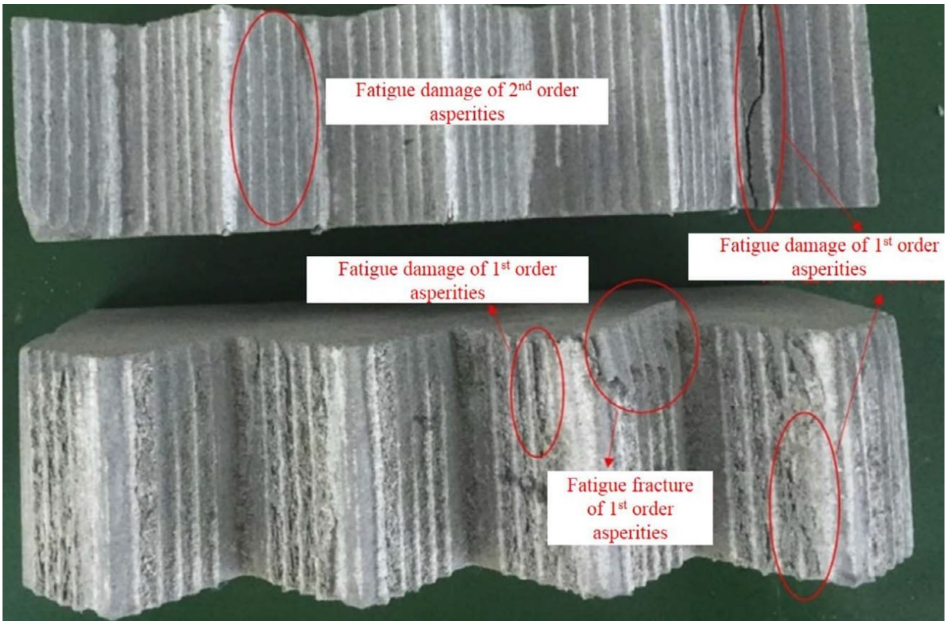

Figure 5: Final fatigue damage mode of the rock joint surface at the end of cyclic loading (Liu et al. 2018).

According to Fathi et al. (2016) and Liu et al. (2018), during cyclic loading the contact area between joint surfaces increases for the first few cycles, which is named as the contraction effect. On further cycles, this effect decreases and damage of the second order asperities starts which is named degradation. On further cycles, fatigue cracks initiate in the first order asperities and then coalesce with each other and the rock joint which eventually leads to failure.

3. Methodology

3.1 Overview

This research is primarily based on measured rock mass pore pressure and tunnel water pressure obtained from a tunnel instrumentation carried out during this research. Prior to this work, only a theoretical understanding of the pore pressure response of the rock mass during pressure transients existed. Hence, a full-scale monitoring of an operational unlined pressure tunnel was crucial to understand both the rock mass response during pressure transients. The results from field instrumentation are further enhanced using numerical simulation.

In addition to this, supplementary information was collected from surface geological mapping and laboratory testing for acquiring relevant geological and rock-mechanical properties. Further, operational data from various power plants was collected to enhance the understanding of operational trend of Norwegian power plants in the recent years. Hence, field experiment, analysis of production data of various power plants and numerical simulation are the main methods used in this research. The general methodology of this PhD study is illustrated in Figure 6.

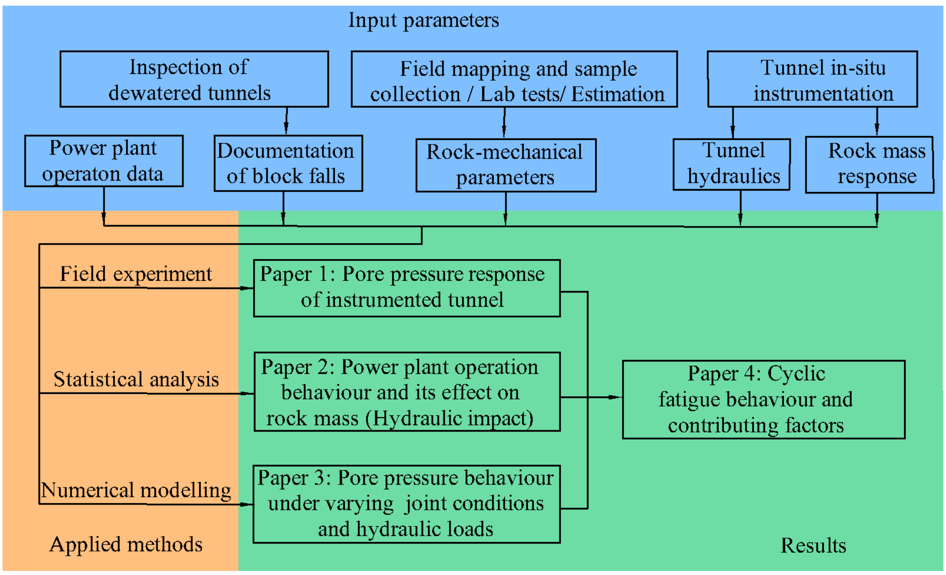

Figure 6: General methodology

The candidate inspected four headrace tunnels, namely Ulset, Roskrepp, Matre Haugsdal, and Suldal I. In addition, information was gathered from the headrace tunnels of Brattset and Svandalsflona power plants, which were inspected and documented by the main supervisor prior to this study. All of these tunnels have been in operation for more than 30 years except Matre-Haugsdal, where a section of the tunnel collapsed within 3 months of operation as a result of swelling clay in a weakness zone. The main objective of these inspections was to make visual observations of their present condition and document block falls or instabilities, which are of interest in relation to the effect of long-term operation. In addition to tunnel inspections, the candidate conducted surface geological mapping, rock sample collection and laboratory tests of Ulset, Brattset, Svandalsflona and Roskrepp power plants to collect the necessary input parameters. This work was done with the help of four master students, whom the candidate co-supervised during their master theses [Døvle (2019), Thorbergsen (2019) Halseth (2018), Urdal (2018)].

3.2 Tunnel instrumentation

The instrumentation was carried out in the unlined headrace tunnel of Roskrepp hydropower plant in southern Norway. Five boreholes were drilled in the tunnel walls such that they intersect a particular joint set almost perpendicularly. Stainless steel pipes were fixed in the boreholes using packers at different lengths in the borehole and the pipes were taken out of the tunnel to a dry area in the access tunnel, where they are fitted with pressure sensors and a datalogger. The length of the boreholes ahead of the packers collect water from the rock joints and are connected to the pressure sensors through the steel pipes and thus record the rock mass pore pressure. Simultaneous readings of the tunnel water pressure are also recorded from a pipe installed at the junction between the headrace tunnel and access tunnel. The instrumentation location and setup are presented in Figure 7a and b and c, d and e present the photos of setup inside the tunnel, borehole arrangement and pressure sensor and datalogger in the dry area respectively.

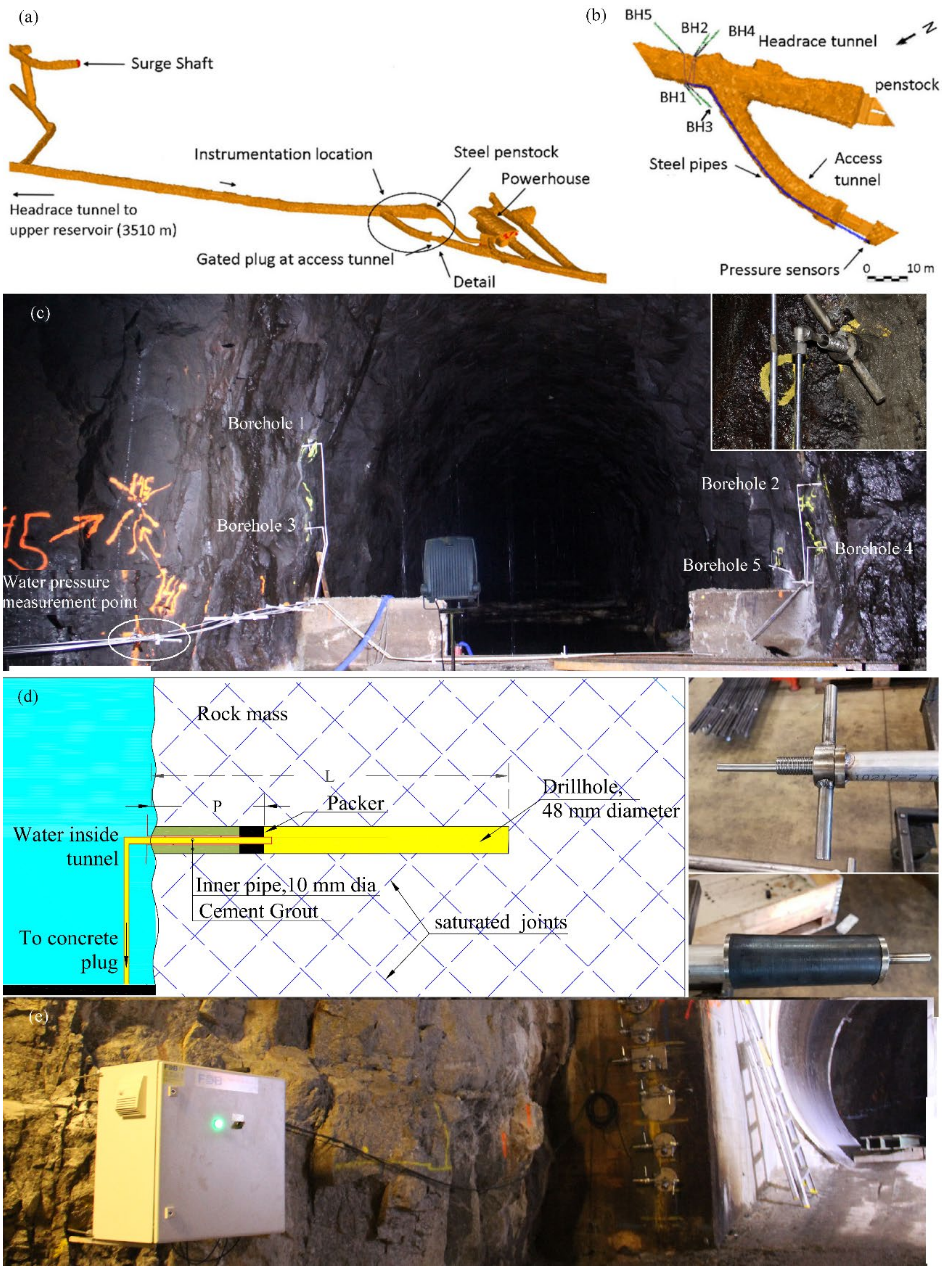

Figure 7. Instrumentation location (a) and detailed view of the setup (b) setup inside the tunnel (c), borehole arrangement (d) and pressure sensors and datalogger in the access tunnel (e) respectively.

Monitoring of the pore pressure increase in the rock mass during first infilling of unlined tunnels after construction is a usual practice [Lamas et al. (2014), Halvorsen and Roti, (2013)] to monitor the changes in the hydrogeological conditions. Such monitoring is usually done close to the transition zone between lined and unlined sections and close to the powerhouse cavern, adit tunnels, galleries and even to the surface, where leakages and displacements due to high static pressure is of primary concern. However, the instrumentation carried out in Roskrepp power plant differs from the conventional monitoring due to the following reasons:

1. It aims to monitor the pore pressure changes close to the tunnel wall, with maximum distance up to one tunnel diameter. This region is of most interest regarding block falls due to power plant operation.
2. Orientation of the boreholes is given special attention during planning because only pore pressure changes in specific joints which may contribute to block falls are of interest.
3. The measurement location needs to be selected so that the effect of both water hammer and mass oscillations can be recorded. A measurement location upstream of the surge shaft will not be able to record the effect of water hammer.
4. The measurement frequency needs to be much higher than conventional monitoring as it needs to monitor the pressure changes within minutes (for mass oscillation) and within seconds (for water hammer).
5. The monitoring needs to continue for years to record the long-term operation of the power plant and not just after tunnel filling as in conventional monitoring.

During transients, the additional loading on the joint surfaces occurs when the rock mass pore pressure is higher than the tunnel water pressure. For comparing the results from a number of transient events, the area enclosed between the pressure signals when rock mass pore pressure is higher is defined as “Hydraulic Impact” and has a unit of MPa.sec and is the force acting on the joint surfaces per unit area over time. The hydraulic impacts caused by water hammer and mass oscillation are also calculated separately by the use of a Butterworth filter (NI, 2021) with suitable low pass frequency. The effect of three different parameters i.e., the shutdown duration, magnitude of load change and the static head before transients, on the hydraulic impact have been studied to determine the most dominant parameter among them. Shutdown duration is the time between start of shutdown event and the peak mass oscillation amplitude is considered to be a relative measure of how fast the shutdown was carried out. The magnitude of load change is indicated by the difference in water pressure before and after the transient or the headloss before transient.

3.3 Statistical analysis of power plant operation

Since power plant operation is envisaged to be the main contributor of additional loading and eventual fatigue of the rock mass, it is important to investigate the nature of load changes over the years and to gain knowledge about the magnitude and frequency of such load changes. Production data per hour in MWh were available for 10 hydropower plants of different installed capacities ranging from 35 to 960 MW. A total of 21 generating units ranging from 35 to 320 MW were analyzed, which includes power plants with and without hydropeaking restrictions and one pumped storage plant. The length of data ranged from 6 to 19 years. The data was analyzed by categorizing the production values in five types, namely LC1 to LC5. The first type LC1 counts the number of start/stops and provides an insight into how frequently the load changes are occurring and its overall trend over the years. The remaining types LC2 to LC5 indicate the magnitude of load changes occurring in the power plants. For these types, the production values are counted when they fall within a range of values as a percentage of the production capacity of each generating unit. This information has been used to study the development of hydropower plant operation over time.

3.4 Numerical modelling

The numerical modelling is carried out using three-dimensional distinct element code 3DEC version 5.2 (Itasca, 2018). 3DEC can calculate fluid flow and effect of fluid pressures on rock/soil, based on specified material properties and fluid/mechanical boundary conditions using coupled hydro-mechanical calculations through a network of fractures between deformable rock blocks. The methodology adopted for numerical modelling is shown in Figure 8.

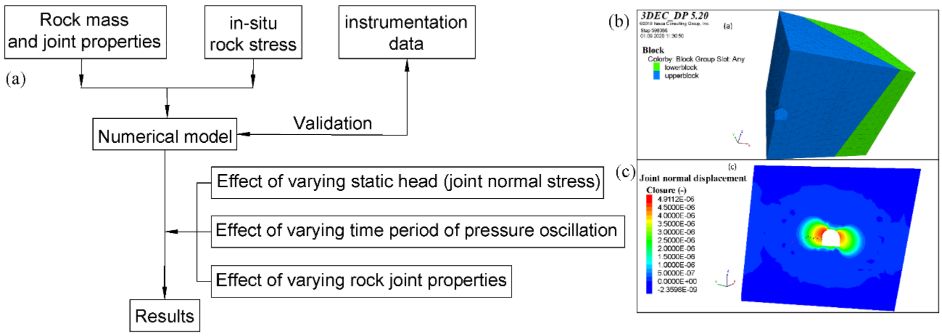

Figure 8: Methodology of numerical modelling (a) 3DEC model geometry (60 x 60 x 60 m) with joint plane (b) and joint normal displacement during tunnel filling (c)

The main objective of numerical modelling is to understand the behaviour of fluid flow through a single rock joint and its deformation under varying conditions of pressure scenarios and rock-mechanical properties. It uses the tunnel and joint geometry from the instrumentation location at Roskrepp and the measurements are used to validate the model. The basic workflow of the model is as follows:

1. Solving for mechanical equilibrium and steady state fluid pressures of model after tunnel excavation.
2. Application of water pressure in the tunnel and run to steady state (full hydromechanical coupling) to simulate tunnel filling.
3. Validate the model using measured pore pressure readings.
4. Use the validated model to investigate the effect of varying input parameters such as static pressure, time period of oscillation and rock joint properties.

4. Results and discussion

4.1 Summary of main papers

Paper 1

Neupane B, Panthi KK and Vereide K (2020). Effect of power plant operation on pore pressure in jointed rock mass of an unlined hydropower tunnel: An experimental study. Rock Mech Rock Eng 53: 3073–3092

This paper presents a detailed method of instrumentation conducted in the 3.5 km long unlined headrace tunnel of 50 MW Roskrepp hydropower plant in southern Norway, along with some observations and findings. The main objective of the instrumentation and monitoring is to measure the changes in pore pressure in the rock mass near the tunnel walls during start-stop operation and load changes and to simultaneously measure the water pressure fluctuations in the headrace tunnel. This monitoring program is the first one known to the authors which monitors the changes in rock mass pore pressure during long-term operation of the power plant.

The result reveals that pore pressure changes in rock mass due to tunnel pressure fluctuations are localized in nature and is dominated by one or more conductive joints in the vicinity. Two of five boreholes showed a fast pore pressure response to the tunnel pressure fluctuations because of their direct contact with the tunnel through conductive joints. The other three remained unaffected since the rock joints connecting the tunnel and these boreholes were tightly closed joints and thus did not have sufficient conductivity for the pressure pulses to travel into the rock mass.

For conductive joints, it was seen that not only mass oscillations but also water hammer travelled as far as 8 m deep into the rock mass along the joint length. The amplitude of such water hammer, however, was reduced significantly, most likely because of the length of the flow path and the void geometry of the joints. This observation shows that the joint geometry has a larger effect on the pore pressure response as compared to the time period of pressure transients.

A delayed response from the rock mass or time-lag between the pressure peaks is observed during pressure transients in some of the boreholes. Such data is crucial to understand the effect of power plant operation in the rock mass since the time-lag causes additional loading on the rock blocks over the lifetime of the tunnel. Such additional loading occurs when the rock mass pore pressure becomes higher that the tunnel water pressure, during the period with falling pressure. Mass oscillations are seen to induce a higher hydraulic stress between the rock mass and the tunnel as compared to water hammer, because the time of application of such stress is longer. The result of 356 days of monitoring demonstrates that power plant operation causes small but frequent pore pressure changes in the rock mass, which can produce destabilizing forces in the rock joints and cause rock mass fatigue over long-term operation of power plants.

Paper 2

Neupane B, Vereide K and. Panthi KK (2021). Operation of Norwegian hydropower plants and its effect on block fall events in unlined pressure tunnels and shafts. Water, 13 (11), 1567.

This paper is divided into two parts. The first part analyses the production data of some Norwegian power plants to understand their current operational trend. The second part further elaborates on the results of monitoring presented in paper 1, with additional data from one more year of power plant operation and links the results with shutdown behaviour during load reduction and resulting tunnel hydraulics.

The analysis of production data shows that on average 200 to 400 start/stop events (LC1) occur per generating unit per year for power plants without operational restrictions. This number varies significantly among different powerplants and between different years even within a single power plant. The data shows that it is an increasing trend of start/stop events per year, particularly among smaller power plants. The analysis of magnitude of load changes shows that smaller load changes (LC2) are more frequent than large load changes (LC5). However, larger load changes (LC5) occur in higher proportion for smaller power plants as compared to larger hydropower plants. These results provide an insight on the extent of dynamic operation of the powerplants which can lead to larger destabilizing forces in the rock joints and accelerated fatigue of the rock mass in the future.

A new method is proposed to quantify the effect of hydraulic transients on rock joints, referred to as the hydraulic impact (HI). The HI is a destabilizing load that is regarded to be the main driver for instability, rock falls, and potential tunnel collapses caused by hydraulic transients. It was found that shutdown duration is the most dominant parameter contributing to increase the hydraulic impact due to mass oscillations i.e., shorter the shutdown duration, higher the hydraulic impact.

The monitoring at Roskrepp also revealed that there is a large variation in shutdown duration, even for similar magnitude of load changes, because there is no standard procedure for normal load change operations and is entirely based on the power plant operator. Based on these findings, the authors recommend that larger load changes take longer durations such that the hydraulic impact can be reduced which would help slow down the cyclic fatigue process.

Paper 3

Neupane B and Panthi KK (2021). Evaluation on the effect of pressure transients on rock joints in unlined hydropower tunnels using numerical simulation. Rock Mech Rock Eng 54: 2975–2994.

This paper presents effect the pressure transients on the pore pressure response and deformation behaviour of rock joints, under varying joint normal stresses, time period of pressure transient using numerical modelling. The effect of varying mechanical properties of rock joints such as joint normal and shear stiffness, joint friction angle and dilation angle is also studied.

The pore pressure distribution along a joint length further confirms the importance of delayed response of the rock mass or the time-lag between pressure peaks. It is seen that the highest impact occurs where there is sufficient pore pressure buildup and also enough flow resistance to cause a significant time-lag. Such locations are few meters into the tunnel wall, depending upon joint properties.

This ratio between joint normal stress and static water pressure in tunnel is the factor of safety (FoS) used for design of unlined tunnels. The results indicate that relative joint deformation due to pressure transients are the highest when this ratio is between 1.5 to 2.5. Critical locations along unlined tunnels usually have FoS lying within this range since larger FoS are usually undesirable due to economic reasons, whereas smaller FoS are uncommon since they would require specific stress measurements during construction. During pressure transients, larger relative displacement in this FoS range will probably have more “loosening effect” on the rock mass as a result of cyclic loading over the years of operation. Hence the result implies that tunnels designed within this FoS range could be the most impacted by cyclic fatigue.

The simulation results also confirm that mass oscillation cause larger hydraulic impact as compared to water hammer. Further, mass oscillations usually apply to longer stretches as compared to the water hammer. However, the authors conclude that the effect of water hammer, wherever applicable, cannot be neglected because it is seen to travel deep into the rock mass even in stiff joint conditions and sufficiently high normal stresses (FoS 1.5 to 2.5), which are considered safe as per the conventional design practice. Further result shows that reduced friction angle causes larger joint deformations already during tunnel water filling or steady state itself and pressure transients cause larger deformation when joint stiffness is reduced. Hence, a possible scenario is a reduction of these two parameters due to fatigue causing the joints to deteriorate and eventually leading to macroscopic movement during filling/dewatering and/or mass oscillations.

Paper 4

Neupane B, Panthi KK and Vereide K (2021). Cyclic fatigue in unlined hydro tunnels caused by pressure transients. Hydropower and Dams 5:46-54.

This paper presents a qualitative description of fatigue phenomenon that occurs in the rock mass around unlined pressure tunnels when they are subjected to frequent pressure transients due to power plant operational changes. This description links the pore pressure changes observed during such transients with the resulting additional hydraulic stress that acts on the rock mass. Further, it explains how rock mass fatigue could occur as result of cyclic nature of the hydraulic stresses over the long-term operation of power plants. The factors contributing to this mechanism are then presented with recommendations to slow down the fatigue process. The conclusions are based on observations from dewatered unlined hydropower tunnels and the experimental and numerical studies mentioned above. Possible application and limitations of these findings are discussed and recommendations are given.

The results indicate that an increased conservatism may be needed in rock support decisions in potential failure zones due to rock mass fatigue. It may also be considered to treat permeable zones in critical locations such that pressure transients cannot travel deep into the rock mass and cause additional stresses during load changes. The described contributing factors can be used as a preliminary guide to identify critical locations where additional support measures are needed. Further, it is recommended that load changes in hydropower plants with a vulnerable rock mass should be carried out less frequently and at slow rates so that the rock mass pore pressure can closely follow the mass oscillation pressure. Dewatering and filling of the hydropower tunnels should be reduced to a minimum. This will reduce the long-term aggregated hydraulic stresses acting on the rock mass and thus slow down rock mass fatigue.

4.2 Possible application and limitations

Realtime pore pressure monitoring

The instrumentation method described in the first paper has proven to be useful in monitoring pore pressure changes over the real-time operation of the power plant. Readings from similar monitoring programs along with the proposed method of calculating hydraulic impact can thus be used to define the most suitable shutdown duration to delay the rock mass fatigue and prolong the serviceable lifetime of unlined pressure tunnels with serious block fall issues. The following issues are of primary importance which can affect the results.

1. Identification of open conductive joints is critical for the success of monitoring program, which also decides the location and orientation of the boreholes. Ideally, a location between the surge shaft and steel lined section is desirable in order to measure both water hammer and mass oscillation. Proximity of the possible location to a construction adit is preferable to keep the pipe length as short as possible.
2. The paper outlines possible sources of error such as choking of pipes, water tightness of the system, entrapped air, and pipe vibration, along with their remedies. Experience from Roskrepp shows that pipe vibration is one of the major issues, which can produce noise in the collected data and damage to pipes if they are not secured properly.

Operational changes

The recommendation to carry out slower load changes can be readily implemented in power plants to reduce the hydraulic impact caused by pressure transients. However, flexibility needs in the future may demand faster load changes which may be in contradiction to the recommendation to show down the load changes. Since, it is difficult to predict, there may be some limitations to follow this recommendation. A compromise between these two contradictory requirements may be needed in order to cater to the market needs as well as ensure the long-term stability of unlined pressure tunnels.

Assessment of block fall possibility

The knowledge of the observed mechanism and contributing factors can be used to assess the potential of block falls and decide necessary support recommendations. This should be considered in power plants with significant block falls and concerns about the tunnel stability. However, its application in quantitative terms demands more data from similar instrumentation in different power plants such that a larger database is created in order to correlate the observed block fall events and the contributing parameters.

4.3 Uncertainties

1. The experiment is carried out in only one tunnel and hence more data is needed to validate and expand the results. In addition, more tunnels need to be inspected to document block fall events and their operational data need to be studied.
2. The finest resolution of production data available is hourly which creates some limitations to do precise calculations of magnitude and frequency of load changes in MW. Hence, it is assumed that every change in production per hour is a result of a single load-change event. It results in a conservative number of load-change events if more than one event occurs within an hour.
3. The analyses and results presented above are based on numerical models assuming the rock joint as the interface between two parallel plates. Limitation of these models is that it does not consider the effect of flow tortuosity in rock joints caused by joint roughness, which is still an outstanding issue in numerical simulation of flow processes in rock.

5. Conclusion and recommendations

This research contributes to a deeper understanding of the effect of frequent pressure transients on the long-term stability of unlined tunnels. A description of the forces and mechanism involved is presented based on a first-of-its-kind data from a full-scale monitoring program, with the help of numerical modelling and analysis of power plant operation data.

5.1 Main conclusions

The main conclusions are as follows:

1. Experimental and numerical simulation results indicate that pressure transients can have significant influence on the pore pressure variation and joint displacement in the rock mass around unlined pressure tunnels of hydropower plants. This influence is dependent on the hydro-mechanical properties of the rock joint contributing to the transmission delay or time-lag between the pressure transient and rock mass pore pressure.
2. Results confirm the previous knowledge that mass oscillations cause larger hydraulic stresses in the rock mass as compared to water hammer since they have larger time of application of hydraulic stresses and are also applicable to longer stretches of tunnels. However, the effect of water hammer may for some exceptions have a higher effect than mass oscillations and cannot be ignored.
3. The analysis of operational data of some power plants for the last 19 years shows that already large number of start/stops and load changes occur every year in Norwegian power plants, causing frequent pressure transients. It is envisaged that this trend will further increase in the future due to addition of larger share of unregulated power from wind and solar energy. This implies that rock mass fatigue in unlined pressure tunnels can occur at an accelerated rate.
4. The result indicates that additional rock support may be needed in critical areas where the rock mass permeability permits significant pore pressure changes in the rock mass during pressure transient, especially for tunnels excavated in schistose rock mass, and power plants with large number of load changes (multiple load changes within a day).

5. Power plant operation is seen to have a significant influence on the amount of hydraulic stress acting on the rock mass during pressure transient. It is seen that the shutdown/opening duration during load changes directly affects the time-lag between pressures pulses. It is seen that shorter shutdown/opening duration can cause significantly high hydraulic stresses on the rock mass.
6. It is seen that slowing down the load change operation can provide significant benefit in slowing down the fatigue process. However, it is as a challenge that the shutdown/opening duration is usually dependent on the individual operator and lack of standard guidelines for speed of load changes. Hence, more emphasis should be given towards keeping the speed of load changes consistently slow.
7. A new method called “Hydraulic impact” has been developed to quantity the hydraulic stress on the rock mass caused by pressure transients in unlined hydropower tunnels. It can also be used to define a suitable shutdown duration of the power plant in order to help slow down the fatigue process.

5.2 Recommendations for further work

1. It is recommended to instrument and monitor more tunnels with different characteristics such as static head, design discharge, waterway length and in different rock mass conditions so that the results can be expanded. A specific target should be to make a generalized recommendation for the optimum shutdown durations.
2. Expansion of the results to shotcrete lined tunnels and casted concrete lined tunnels may increase the applicability outside of Norway, in regions where such tunnels are more common than unlined tunnels.
3. Possibilities of simultaneously measuring joint deformation along with pore pressure in a single borehole needs to be explored.
4. Improvements of the measurement setup such as using fibre optic sensors and cables for data acquisition/transmission from inside the tunnel to the data logger in the dry area should be explored. This will potentially reduce the noise in the data

due to pipe vibration and also make it possible to carry out the measurements at larger distances from construction adits.

5. Further study with experimental and numerical simulation to quantify the fatigue process after various number of cycles of pressure transients is recommended. Numerical back analysis of recorded block fall cases using advanced numerical modelling with known parameters of load changes and rock mass properties could be the next step.
6. Further study of the hydraulic impact including other potential damage processes during water filling and emptying should be done to better understand the damage potential of such events compared with the cyclic loading.

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Appendix A: Main papers

Paper 1	Effect of power plant operation on pore pressure in jointed rock mass of an unlined hydropower tunnel: An experimental study
Paper 2	Operation of Norwegian hydropower plants and its effect on block fall events in unlined pressure tunnels and shafts
Paper 3	Evaluation on the effect of pressure transients on rock joints in unlined hydropower water tunnel using numerical simulation
Paper 4	Cyclic fatigue in unlined hydro tunnels caused by pressure transients

Paper 1

Effect of power plant operation on pore pressure in jointed rock mass of an unlined hydropower tunnel: An experimental study

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ORIGINAL PAPER

Effect of Power Plant Operation on Pore Pressure in Jointed Rock Mass of an Unlined Hydropower Tunnel: An Experimental Study

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Abstract

Load changes in hydropower plants result in significant pressure transients and unsteady flow in the waterway. It has been observed that instances of block falls in tunnels have increased in unlined pressure tunnels subjected to frequent load changes. To examine this problem, field instrumentation was conducted in the 3.5 km long unlined headrace tunnel of 50 MW Roskrepp hydropower plant in southern Norway. This article describes the methodology of instrumentation, presents the observations and findings. The monitoring clearly demonstrates that frequent load changes have a considerable effect in the rock mass consisting of system of joints. The observations show that pressure transients can travel deep into the rock mass irrespective of their time period. Moreover, pressure transients with longer time periods, i.e. mass oscillations, are seen to induce a higher hydraulic gradient between the rock mass and the tunnel itself. A delayed response from the rock mass is observed during pressure transients, which is the main cause of development of hydraulic gradient and additional pore pressure acting on the rock blocks. Hence, it is evident that the cumulative impact of small but frequent pressure gradients is significant and is responsible for increased instances of block falls over a long period of operation of the unlined tunnels of hydropower plants with frequent start–stop sequences. The overall impact is governed by pore pressure response of the jointed rock mass which depends on the conditions of joint geometry and joint wall properties.

Keywords Hydropower · Unlined pressure tunnels · Hydropeaking · Pressure transients · Pore pressure · Rock joints

1 Introduction

The main design principle of unlined tunnels and shafts for hydropower plants is to place them in suitable rock mass with sufficient confinement. The placement is selected to avoid hydraulic jacking, which could lead to water leakages. Local sections with unstable rock mass, including swelling and/or friable materials are mapped and provided with sufficient support (Palmstrøm and Broch 2017). In such tunnels, hydraulic jacking needs to be prevented by ensuring that the maximum water pressure in the tunnel does not exceed the minimum principal in-situ stress in the rock mass. Design requirements and failure mechanisms of unlined pressure tunnels under static loading conditions are a well-studied issue described by several authors including Bergh-Christensen (1975), Buen (1984), Garshol (1988) Benson (1989), Panthi (2014), and Basnet and Panthi (2018). However, long-term instabilities caused by dynamic pressure transients over years of operation are not covered in existing design practices and literatures.

Lang et al. (1976) reported a case study of a hydropower tunnel failure as a result of rapid pressure transients caused by fluctuating power plant operation. A more recent failure of similar nature occurred in the Svandalsflona hydropower plant in southern Norway, where a rockslide occurred in an unlined shaft in 2008 and caused significant damage to the power plant (Panthi 2012; Neupane and Panthi 2018). Further, a list of several tunnel failures that occurred after years of operation is compiled by Brox (2018). Bråtveit et al. (2016) has concluded that compared with results from a previous study (Bruland and Thidemann 1991), the occurrence of rock falls has increased by a factor of 3–4 times in tunnels that have been subjected to frequent start/stop sequences (also referred to as hydropeaking).

The total length of hydropower tunnels and shafts in Norway is close to 4300 km and there are more than 100 unlined tunnels and shafts with maximum static head of up

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to 1047 m (Panthi and Basnet 2016). Most of Norwegian hydropower plants constructed during the 60s and 70s were designed to be operated as base load plants, providing a continuous supply of electricity throughout the year. Deregulation of the power market in Norway in 1991 has changed the operational regime of hydropower plants significantly. At present, there are far more frequent start–stop sequences and load changes compared to when they were designed and constructed.

In the future, the trend of fluctuating operation is expected to rise because of the increasing share of electricity generation from variable renewable energy (VRE) sources such as solar and wind power in the energy market. Electricity generation from these sources is dependent on weather conditions and most often does not comply with the system demand. Hence, hydropower plants will have more frequent load changes and start–stop sequence of operations in the future, to balance the production and load as the VRE share of production increases.

EPRI (1987) mentioned that the rate of recurrence of dynamic pressure is higher in peaking plants and pumped storage plants than in base load plants. According to Benson (1989), a factor of safety for static and dynamic pressures during normal operation must be used during unlined pressure tunnel design. However, pressure transients with a short time period, i.e. water hammer, does not require a factor of safety since the time of application is too short to cause hydraulic jacking. Helwig (1987) presented a theoretical study to estimate the depth to which significant pressure transients are transferred into the rock mass. It concludes that the effect of water hammer is limited to a relatively shallow depth around the tunnel walls because of the short time period. It also concludes that pressure penetration into the rock mass could be considerably larger during pressure transients with a longer time period, i.e. mass oscillations.

Frequent load changes in hydropower plants result in significant pressure transients and unsteady flow in the waterway. In an unlined pressure tunnel, pore pressure in the surrounding rock mass is dependent on water pressure in the tunnel itself. During long periods of standstill or steady operation, an equilibrium is reached. However, the authors' hypothesis is that when load changes are occurring, delayed response of rock mass may cause significant pressure gradients between the tunnel and rock mass. This situation may be the leading cause of instabilities in the unlined pressure tunnels that have been occurring more frequently.

The main aim of this article is to describe a full-scale experimental study conducted on the unlined medium pressure headrace tunnel of the 50 MW Roskrepp hydropower plant in southern Norway. The major objective of the instrumentation and monitoring is to measure the changes in pore pressure in the rock mass near the tunnel walls during load changes and start–stop sequence of operation and to simultaneously measure the pressure fluctuations in the headrace tunnel. The methodology of instrumentation and recorded measurements for a period of 356 days are presented in this article. The obtained data provide valuable insights and are comprehensively analyzed. The monitoring will continue for a number of years to come so that possible changes during long-term operation are investigated.

2 Tunnel Hydraulics and Fluid Flow in the Rock Mass

During steady operation of a hydropower plant, a constant pressure is maintained in the tunnel which tends to push the water out of the tunnel to low-pressure areas such as nearby access tunnels/construction adits and out to the surface. Since hard rocks have relatively low porosity (mostly less than 2%), a system of rock joints act as flow paths governing the overall permeability of the rock mass. The design and construction of an unlined pressure tunnel are carried out such that leakage out of the tunnel is under acceptable limits (Panthi 2013). An equilibrium is reached for the given pressure and flow situation when the power plant is operated at constant load for a long time. This equilibrium is changed when there is a change of load in the power plant. Load changes cause pressure transients in the tunnel, which changes the pore pressure and the flow through system of joints. Hence, the issue under consideration is an interdisciplinary topic which involves two complex and dynamic mechanisms that are occurring simultaneously during the operation of an unlined pressure tunnel of a hydropower plant. They are described in brief in the following sections.

2.1 Flow and Pressure Transients in a Hydropower Tunnel

Every start, stop and load change in hydropower plants generate flow and pressure transients in the waterway. In the example of an emergency shutdown at Roskrepp power plant, the turbines close rapidly within 10 s and result in a rapid decrease of the water flow. The resulting deceleration of the water in the waterway causes a pressure increase on the upstream side of the turbine, and a pressure decrease on the downstream side. This pressure transient cannot be calculated simply with Newton's second law, as water is an elastic medium that can be compressed and decompressed.

The elasticity of water will result in a pressure transient with a short time period, referred to as water hammer (Parmakian 1963). Water is barely elastic and requires a large pressure increase to cause a small compression, resulting in the maximum compression being reached in a short time, which leads to a fast-traveling water hammer. The water hammer starts from turbine and progresses towards the nearest free water surface where the water hammer is reflected back towards the turbine with the opposite pressure, a positive pressure becoming a negative one. In this manner, the water hammer may travel back and forth many times until the energy is dissipated by friction.

To reduce and control the water hammer, hydropower plants are constructed with a surge tank. The surge tank is constructed close to the turbines to reflect the water hammer as soon as possible, reducing the amplitude and the affected tunnel length. However, in hydropower plants with a surge tank, a pressure transient with a long time period will occur, which is referred to as mass oscillations (Chaudhry 1987). Mass oscillations are caused by the inertia of the water in the tunnel between the reservoir and the surge tank. When the turbines close, water cannot flow through the turbine and will instead flow into the surge tank, causing water level to rise. In the opposite case of a power plant startup, the water level in the surge tank will drop as it takes time to accelerate the water in the rest of the headrace tunnel to the reservoir. The rise or drop of the water level in the surge tank will reverse once the water in the main tunnel is accelerated or decelerated. This mass oscillation will oscillate back and forth until the energy is dissipated by friction. Figure 1 presents a principle drawing of a hydropower plant and the effect on water pressure from water hammer and mass oscillations.

The water hammer has a higher potential for pressure increase compared with the mass oscillations and may result in bursting pipes and structural damage if uncontrolled. The purpose of the surge tank is thus to reduce the water hammer, even though the surge tank causes mass oscillations to occur. Some measures other than surge tanks are available, such as pressure relief valves, but are often insufficient in hydropower plants with long waterways. The surge tank may thus be the only viable option, as is the case for Roskrepp power plant. The maximum water hammer pressure peak may be estimated with the following simplified equation: (Joukowsky 1889):

$$\Delta p = \frac{c\Delta v}{g}, \tag{1}$$

where Δp is the pressure increase by water hammer, c is the speed of sound in water, Δv is the change in water velocity in the waterway. When there is a surge tank in the system, the rule of thumb is that water hammer is reduced with the factor T_r/T_l, where $T_r = 2L/c$ is the return time of the water hammer and T_l is the closing time of the turbine. The maximum water level rise in surge tank during mass oscillation is calculated by the following equation:

$$z = \frac{LA_t v^2}{2gh_f A_s}, \tag{2}$$

where L is the length of the tunnel between the reservoir and surge tank, A_t is the tunnel cross-section, v is the water velocity in the tunnel, g is the gravitational acceleration, h_f is the headloss in the tunnel between the reservoir and the surge tank, and A_s is the area of the water surface in the surge tank (the horizontal plane cross-section). For more accurate estimation of the water hammer and mass oscillations, numerical simulations are applied (Chaudhry 1987).

2.2 Fluid Flow Through Rock Fractures

Fluid flow through rock mass consists of two components: (1) flow through interconnected network of fractures, and (2) seepage through the pores of the rock itself, which is insignificant for crystalline rocks. Therefore, only the first component is considered here since the permeability in the rock mass is mainly controlled by the system of joints and

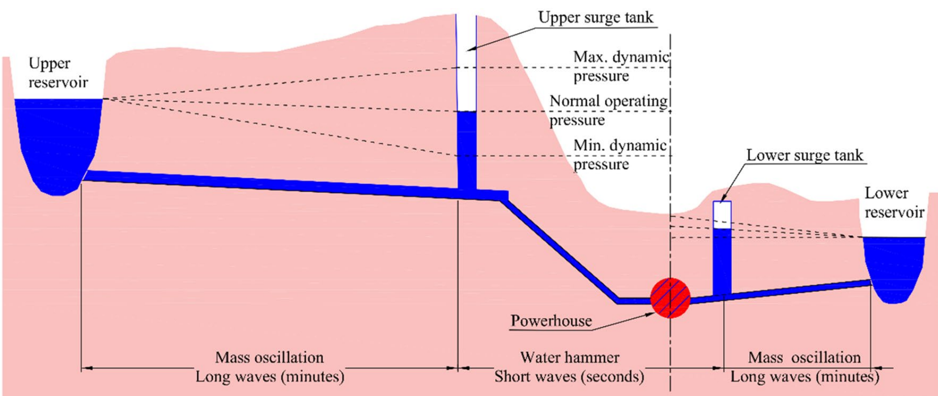


Fig. 1 Flow and pressure transients in hydropower plants

not by the intact rock. According to Jing and Stephansson (2007), the fluid flow and block deformation are coupled through a two-way interaction. The change of fluid pressures on the rock joint surface affects the deformation on the joint wall, which causes the joint aperture to change. This change of hydraulic aperture affects its transmissivity, flow rate and fluid pressure distribution along the fracture surface, which in turn affects the deformation. Hence, the mechanical and hydraulic behaviors of a rock joint are interacting with each other. This hydro-mechanical coupling is shown in Fig. 2.

Significant research has been carried out regarding the hydro-mechanical behavior of single fractures in rocks since the 1960s. Rutqvist and Stephansson (2003) provide a summary of major research carried out in this area. It is an established fact that the void space geometry between fracture surfaces has a major influence on the flow through them. Hakami (1995) highlights the fracture properties affecting the void space geometry consisting of: (1) aperture, (2) roughness, (3) contact area, (4) matedness, (5) spatial correlation, (6) tortuosity, (7) channeling, and (8) stiffness of the fracture. A simplified form of the Navier–Stokes equation of viscous fluid flow through two parallel surfaces having a narrow aperture is used as a conceptual model to define the flow between two planar fractures. This is the most commonly used model in rock mechanics and is referred to as the ¨cubic law¨. The flow rate per unit width of fracture is given by the following expression:

$$q = \frac{a^3}{12\mu}\frac{\Delta p}{l}, \tag{3}$$

where a is the aperture, μ is the kinematic viscosity of the fluid, Δp is the pressure difference; and l is the length of joint. Witherspoon et al. (1980) modified the cubic law using lab experiments on artificial tension fractures in granite and marble and introduced the terms ¨apparent aperture¨ and friction factor into the equation which accounts for the roughness of fractures. Barton et al. (1985) propose the use of hydraulic aperture in Eq. (3) instead of the mechanical aperture and relate these two parameters with the following expression:

$$e = \frac{\mathrm{JRC}^{2.5}}{(E/e)^2}\mu\mathrm{m} \quad (\text{valid for } E \geq e), \tag{4}$$

where JRC is the joint roughness coefficient, E is the real physical aperture which can be calculated as the arithmetic mean of the separation between two fracture surfaces which can be measured with a feeler gauge and e is the hydraulic aperture.

3 Brief on Roskrepp Hydropower Plant

Roskrepp hydropower plant is located in Sirdal municipality in southern Norway and is operated by Sira-Kvina Kraftselskap. The power plant was commissioned in 1979 and has a design discharge of 70 m^3/s with operating head between 52 and 109 m depending on the reservoir level. The installed capacity is 50 MW in one Francis turbine unit. The headrace tunnel is an inverted D shape, 6.5 m high, 7.5 m wide and approximately 3500 m long from the intake to the start of steel-lined pressure shaft (Fig. 3).

3.1 Geological Setting of the Project

The rock mass in the project area consists of a mixture of coarse-grained granite and weakly schistose granitic gneiss. At regional scale, the geological structures are characterized by coarse-grained granite pluton which have intruded the gneiss and caused the foliation of granitic gneiss to follow the boundary of granite plutons (Sira-Kvina kraftselskap, 1977). Surface mapping of the project area revealed that the general orientation of foliation joints vary in the range N 135°–150° E/40°–60° NE (Jf). In addition, two prominent cross-joint sets having strike/dip as N 80°–100° E/70°–80° N (J1) and N 0°–20° E/40°–50° SE (J2) are present in the rock mass. During walkover survey along the dewatered

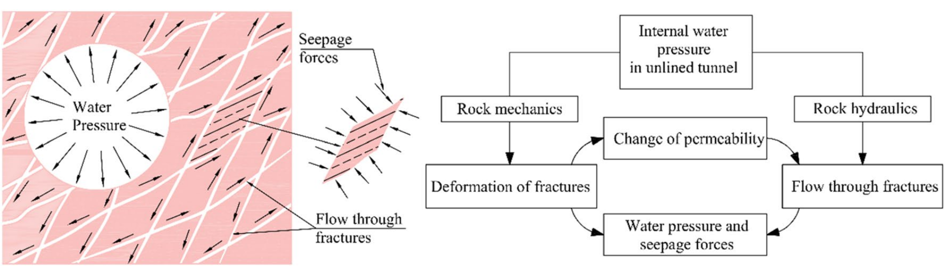


Fig. 2 Hydro-mechanical coupling in rock mass in an unlined tunnel (modified from Schleiss 1986)

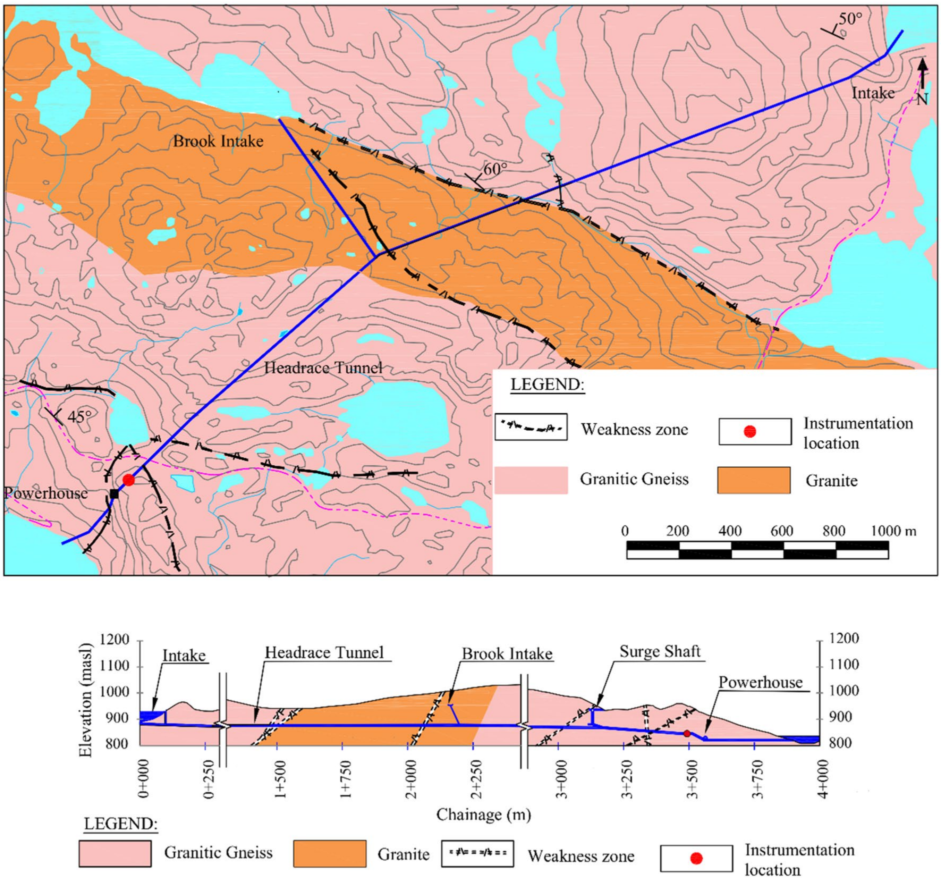


Fig. 3 Geological plan and profile of Roskrepp hydropower plant

tunnel, it was noted that rock mass along the tunnel is of good quality and very little rock support is provided except few meters of concrete lining when crossing through weakness zones. Most sections of the tunnel have tight-jointed walls and were observed to be dry during the survey. Weakness zones are mostly concentrated along the lower reaches of tunnel. Weakness zones crossing the tunnel are shown in the geological plan and profile of the power plant (Fig. 3).

3.2 Joint Conditions at the Instrumentation Location

The summary of comprehensive joint mapping carried out at the instrumentation location is presented in Table 1 following ISRM's (1978) suggested method for quantitative description of discontinuities in rock mass. A major weakness zone crosses the tunnel approximately 150 m upstream of the instrumentation location (Fig. 3). Dripping flow was observed at several locations downstream of this weakness zone, which indicated relatively open joints in this section as compared to rest of the headrace tunnel. Joint mapping at the instrumentation location in the tunnel (Fig. 3) shows slightly different orientation of the joints as compared to surface mapping. Figure 4 shows typical joint orientation at the surface and the rosette of joints measured at the instrumentation location.

A 3D model of the tunnel (Fig. 5) generated using laser-scanned data shows the plan view of the jointing conditions at instrumentation location, along with the orientation of boreholes drilled for pore pressure measurement in the rock mass. The details of boreholes are presented in Sect. 4.2. On the right tunnel wall, BH1 and BH3 are aligned such that they intersect foliation joint set (Jf) and are roughly parallel to cross-joint set (J1). On the left wall,

Table 1 Engineering geological properties of joint sets at the instrumentation location

Joint set	Jf	$Jf_{conductive}$	J1	J2
Strike	N 140°–160° E	N 150° E	N 80°–100° E	N 60°–75° E
Dip	75°–90° SW	80° SW	70°–85° SW	20°–40° SE
Persistence (m)	3–10	More than 10 m	3–10	3–10
Joint wall weathering	Fresh (W1)	Slightly weathered (W2)	Fresh (W1)	Slightly weathered (W2)
Joint roughness	Rough planar JRC 4-6	Rough undulating JRC 14-18	Rough planar JRC 4-6	Smooth undulating JRC 10-14
Joint aperture (mm)	Tight (0.1–0.25 mm)	Partly open (0.25–1 mm)	Tight (0.1–0.25 mm)	Partly open (0.25–1 mm)
Joint infilling condition	Clay	Washed out	Clay	Washed out
Seepage	Damp but no dripping or following water present	Continuous flow	Wet with occasional drops of water	Continuous flow
Typical spacing (m)	1–2 m	More than 10 m	1–2 m	More than 10 m

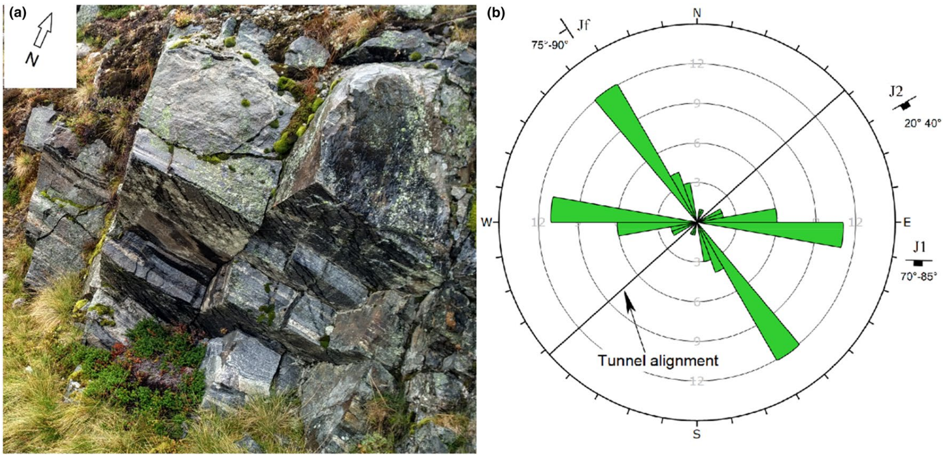


Fig. 4 Orientation of joints **a** at surface near the instrumentation location and **b** joint rosette at the instrumentation location in tunnel

BH2 and BH4 are aligned such that they intersect cross-joint set (J1) and are roughly parallel to foliation joint set (Jf). BH5 intersects through foliation joint set (Jf) on the left wall. The cross-joint (J2) is clearly visible at the tunnel crown and is contributing to most of the inflow into the tunnel. In addition, a conductive single joint is present at the right wall (indicated by plane marked $Jf_{conductive}$). It follows the same orientation as foliation joint set (Jf) but has different joint aperture and infilling conditions, especially near the crown where it is in proximity to cross-joint (J2). On the right wall, both boreholes BH1 and BH3 intersect $Jf_{conductive}$ (Fig. 6b) in addition to foliation joints (Jf). It is noted here that in BH3, the packer is located 4 m from the tunnel wall and the conductive single joint hits this borehole in the grout-filled impermeable section behind the packer. Hence, there is no direct hydraulic connection between the borehole and this single joint. On the left wall, BH4 intersects the cross-joint (J2), while BH2 does not intersect it (Fig. 5). Hence, only BH4 has a direct hydraulic connection to cross-joint (J2).

Figure 6 show the joint conditions at the instrumentation location. Tunnel contour on the right wall is relatively less undulated as compared to left wall where undulations and formation of wedges are more prominent. The formation of wedge was due to blasting through unfavorably orientated joints in the tunnel wall during excavation, which exposed the joints on the left wall. Hence, a relatively larger number of joints are exposed to the tunnel contour on the left wall near BH2 and BH4 as compared to the right wall.

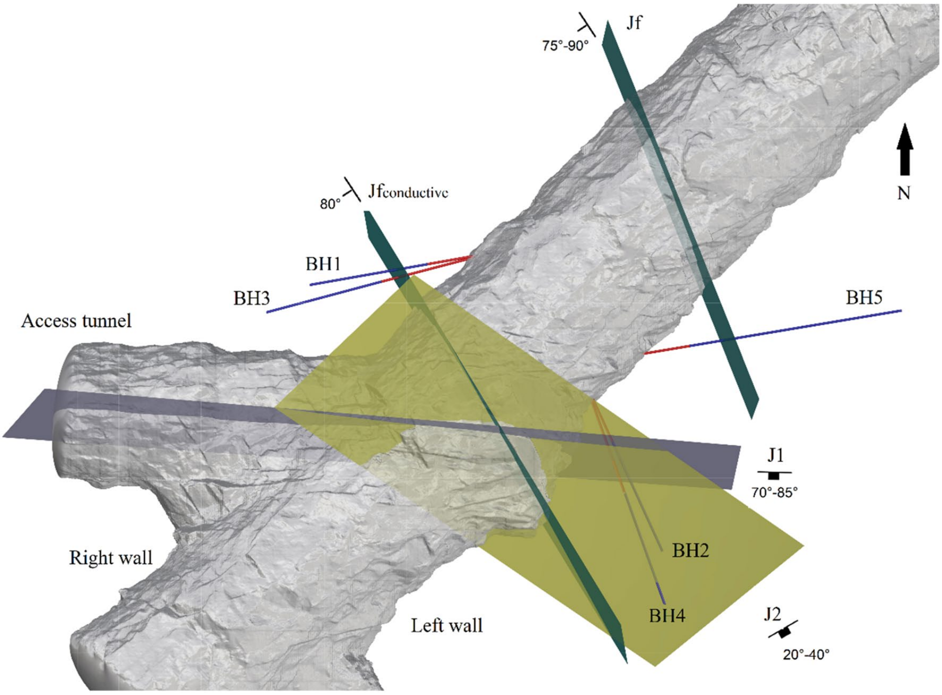


Fig. 5 Plan view of jointing conditions and orientation of boreholes at instrumentation location

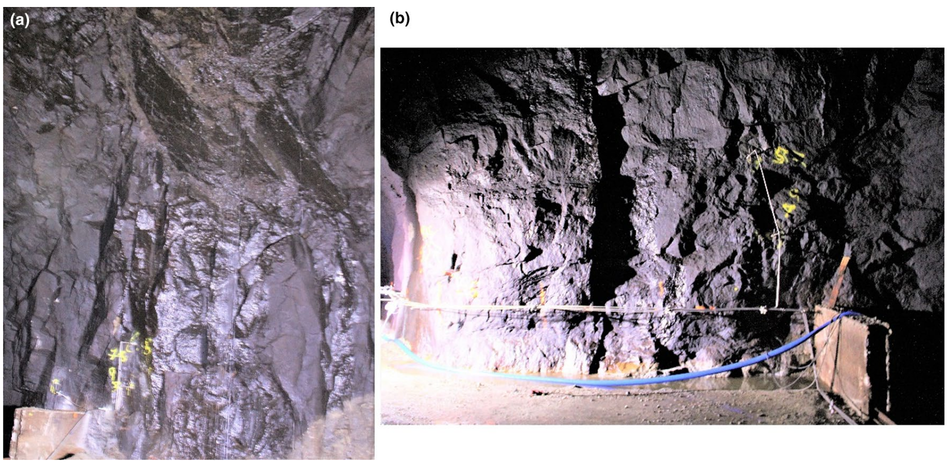


Fig. 6 Joint conditions at **a** left wall (cross-joint J2 is visible) and **b** right wall showing $Jf_{conductive}$

4 Instrumentation Setup

4.1 Selection of Instrumentation Location

The instrumentation is carried out just upstream of the junction between the construction adit and headrace tunnel at the downstream end as shown in Fig. 3. This location has been chosen because of the following reasons:

1. Measurement in headrace tunnel section between surge shaft and the steel-lined section will record the maximum pore pressure transient in the rock mass due to both water hammer and mass oscillation.
2. This location is nearby a construction adit, making it close to a dry area where pressure transducers and datalogger can be placed safely.
3. A detailed survey carried out along the headrace tunnel after 72 h of dewatering revealed that the rock mass at this location is suitable for the instrumentation where

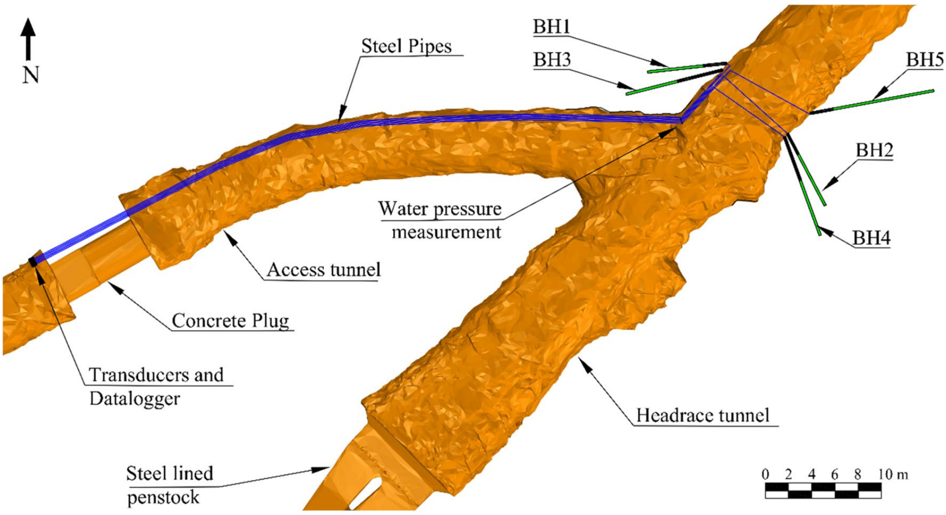


Fig. 7 Layout of the instrumentation setup and location of boreholes

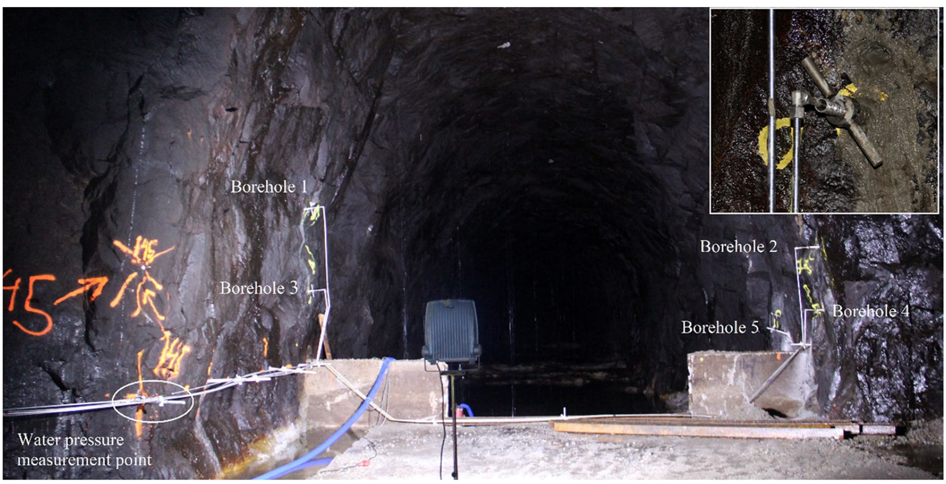


Fig. 8 Location of boreholes in the tunnel (looking upstream) and detail (top right corner)

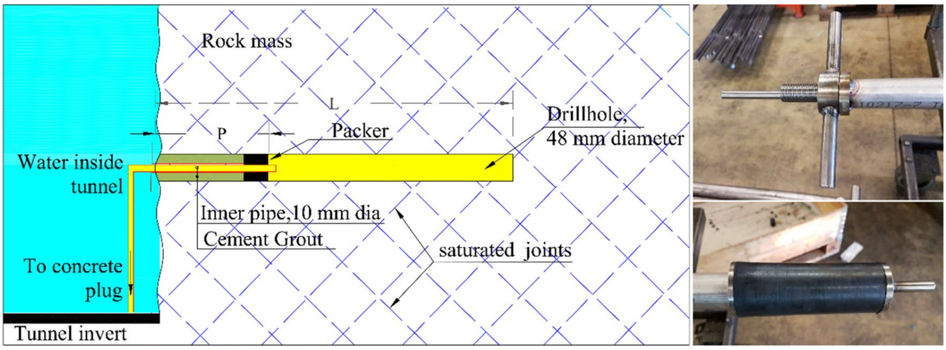


Fig. 9 Schematic detail of borehole showing location of packer (left) and details of packer assembly (right)

water inflow in the tunnel was distinct with a clear hydraulic connection between different joint systems and the tunnel walls.

4.2 Instrumentation Layout and Details

In total, five boreholes of 48 mm diameter are drilled at selected locations with varying length and orientation (Figs. 7 and 8). Stainless steel pipe of 10 mm internal diameter is fixed inside each borehole using rubber packers as shown in Fig. 9. The empty length of borehole inside the packer of each hole is used to collect water from the rock mass and to convey through installed stainless steel pipe to the pressure transducer located outside the concrete plug (dry area). As seen in Fig. 9, the packer when tightened fixes the steel pipe tightly in the borehole and creates a barrier between the pore pressure in rock mass and water pressure in the tunnel. In addition, grouting is carried out to achieve complete isolation between water flowing in the tunnel and the borehole beyond the packer. The packer is placed at different distances from the tunnel wall in the boreholes to study the pore pressure variation at various distances from the tunnel wall (Table 2). In addition, one steel pipe is installed in the tunnel to measure water pressure. The open end of this pipe is placed at the junction between headrace tunnel and construction adit as shown in Fig. 7.

The steel pipes are laid out along the construction adit, through the concrete plug out to a dry area (Fig. 7). GE Unik 5000 absolute pressure transducers with an accuracy of 0.2% of full scale are connected to each pipe. The frequency of data logging for each sensor is 10 Hz, which is sufficient to record the pressure changes due to both water hammer and mass oscillations. Signals from the transducers are transmitted to an automatic datalogger shown in Fig. 10.

4.3 Possible Error Sources and Remedies

Possible sources of error were identified during the planning process and measures were taken during installation to increase the accuracy of the measurements. They are as follows:

Choking of pipes The boreholes were thoroughly cleaned with a mixture of pressurized air and water to avoid the possibility of choking of pipes from the debris material which may come from the empty borehole. In addition, the boreholes are drilled in downward inclination of about 10° so that larger particles which can clog the pipes can settle towards the far end of the borehole, away from the pipe opening when there is no fluctuation of pressure in the borehole during standstill or steady power plant operation.

Water tightness of packers After the packer was fully tightened, the tightening handle was left in place and welded with the outer pipe (Fig. 9, top right). To ensure that no leakage occurs through the packer, the length of borehole outside the packer is filled with non-shrinking cement grout mix. This ensures that hydraulic connection between tunnel and

Table 2 Borehole details

Borehole	BH 1	BH 2	BH 3	BH 4	BH 5
Trend/plunge	255°/10°	155°/10°	260°/10°	160°/10°	80°/10°
Location	Right wall	Left wall	Right wall	Left wall	Left wall
Borehole length (L), m	7	7	9	9	11
Depth of packer from tunnel wall (P), m	2	2	4	4	2

Fig. 10 Location of datalogger in the construction adit. Arrangement of pressure transducers and deaeration valve is shown in top right corner

boreholes is only through the joint systems in the rock mass. The setting of grout was checked with a geological hammer.

Leakage from pipes After installation, all six pipes were pressure tested with a maximum pressure of 30 bars (300 m) to prevent possible leakage from the connections. Detected leakages from some connections were rectified and tested again. This was done for the whole stretch of pipe except the last connection near the boreholes (Fig. 8, top right). This is to make sure that the test pressure does not affect the joints in the rock mass. After the pressure testing was completed, this last connection was installed with extreme caution to avoid leakage.

Removal of entrapped air in pipes Each steel pipe outside of the plug (air side) is equipped with a deaeration valve so that air entrapped in the pipe can be expelled out (Fig. 10, top right). This is to ensure that the readings are not affected by the air present in the pipes.

Pipe vibration due to flowing water It is possible that the pipes could move and vibrate due to flowing water during operation and affect the measurements. To avoid this, all six pipes are fixed rigidly to the tunnel wall/floor using grouted rock dowels and metal clamps at 1 m interval. The data shows that minor vibration still occurred in the pipes during operation and it increases slightly with increasing flow in the tunnel. However, during shutdowns the velocity of water in this tunnel section is zero as it lies downstream of the surge shaft and hence will not affect the quality of the readings.

Condition of setup after 1 year The tunnel was dewatered and inspected after 1 year of installation of pipes to confirm that the setup is still in good condition and that the data produced within this period are free of errors due to any possible damage of the setup. The pipe connected to BH2 was found broken which created a direct hydraulic connection between the sensor and the tunnel. This explains its sudden behavior change in day 168 (Sect. 5.2) and the readings after this being identical to tunnel water pressure. All the other pipes were found to be intact and in good condition. Also, the pipe joints were in good condition and the cement grout filled in boreholes was observed to be free of cracks and in good condition.

5 Observed Behavior

Continuous monitoring is carried out after the installation. The pressure readings during tunnel filling, plant operation for a period of about 1 year and dewatering are illustrated in Figs. 11 and 12. Although the frequency of data acquisition is 10 Hz, the figures presented in Sects. 5.1 and 5.2 are based on pressure values averaged to a frequency of 1/60 Hz (one data point per minute) for clarity of the figures which represent the behavior over a larger duration. The short-term changes occurring during pressure transients are presented at a frequency of 10 Hz in Sect. 5.3.

5.1 Tunnel Filling and Dewatering

According to Palmstrøm and Broch (2017), the rate of controlled tunnel filling or dewatering in Norwegian unlined

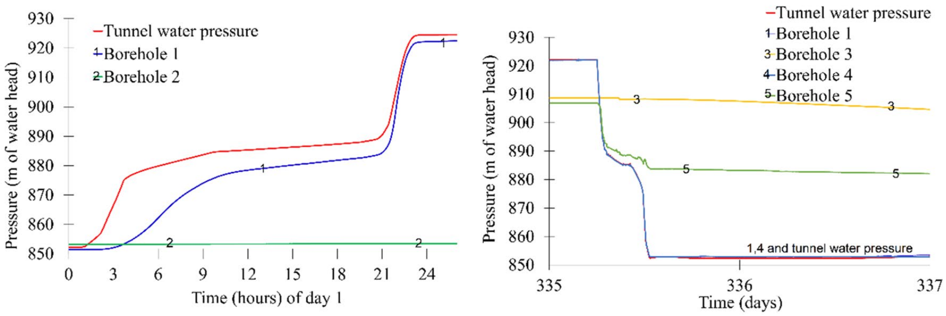


Fig. 11 Pressure readings **a** during tunnel infilling (day 1) and **b** tunnel dewatering (day 335–337)

headrace tunnel and shafts is generally carried out at a rate of 15 m head increase/decrease per hour with a stop for minimum 2 h per 150 m head change and maximum head of 300 m per day. The tunnel filling and dewatering rates at Roskrepp are shown in Fig. 11a, b, respectively.

During filling, the maximum water pressure of 86 m at the instrumentation location was reached in about 24 h. The deaeration valves for all pipes were opened after 1 week of tunnel filling to release any air entrapped in the pipes, which is indicated by a sharp pressure drop in all pipes (Fig. 12) and plant operation was started afterwards. Simultaneous readings of the pore pressure in the rock mass are also shown and are discussed below.

5.2 Pore Pressure Response

At the time of installation, only three sensors were available at site. Therefore, only the pipe directly connected to the tunnel and pipes connected to BH1 and BH2 were equipped with sensors immediately after the installation work was completed. Three remaining sensors were added to the pipes connected to boreholes BH3, BH4 and BH5 on day 63. Hence, the initial pore pressure build-up during tunnel filling could not be recorded in these three holes. Therefore, the general response of BH3, BH4 and BH5, respectively, can be inferred only after day 63.

Borehole 1 (BH1) The pore pressure build-up in BH1 almost follows the same rate as water pressure increase in the tunnel. It regained pressure rapidly (within 5 min) after deaeration. This borehole registered a pressure drop between deaeration and start of operation when the tunnel water pressure was constant which indicates water seepage from the rock mass to the daylight area at the construction adit (i.e. area where the joint is exposed in the tunnel wall). A similar phenomenon is noticed during some intermittent shutdowns (Fig. 12). Also, the pore pressure variations between day 63 and day 166 seem random and did not follow any pattern. During this time, the power plant was shut down and the tunnel water pressure gradually increased due to increased water level in the reservoir. Except for this period, the pressure variations in BH1 during power productions usually follow the same pattern as the tunnel water pressure. Similar behavior is seen during tunnel dewatering. This borehole is responsive to pressure transients in the tunnel such that pore pressure closely follows the pattern of mass oscillation (discussed in Sect. 5.3).

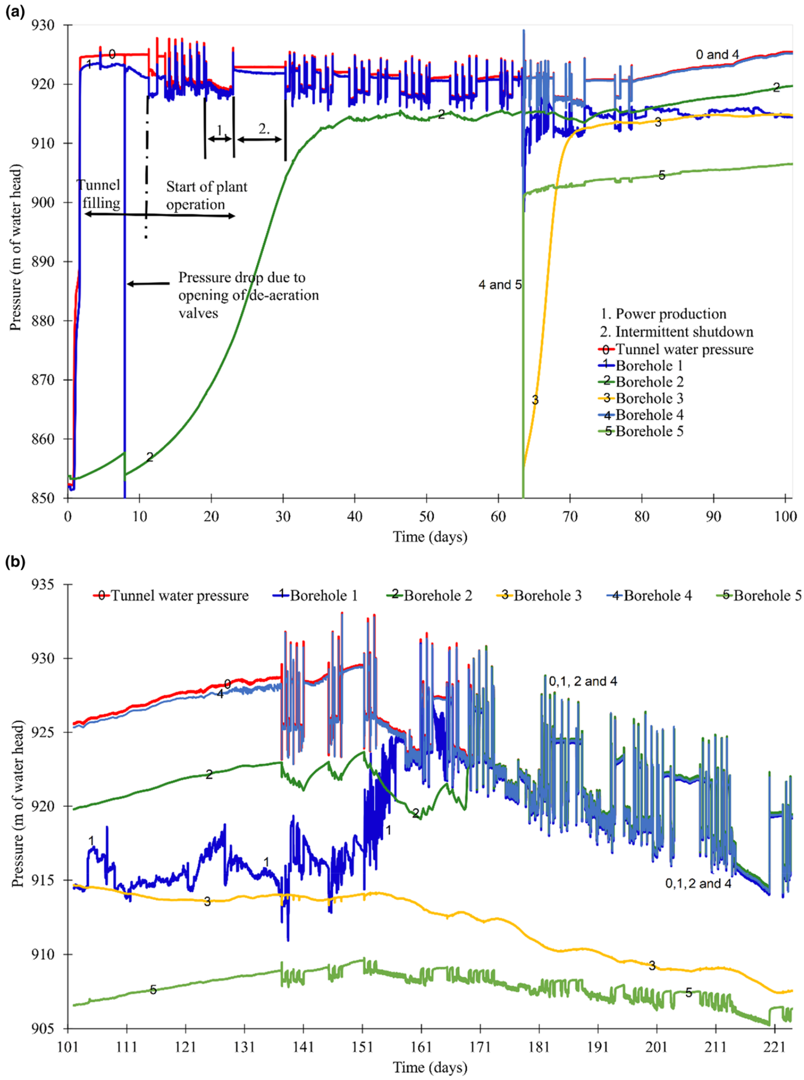
Borehole 2 (BH2) The pore pressure build-up in BH2 is very slow as compared to BH1. It reached a pressure of about 18 m in 7 days. After deaeration, it took about 5 days to regain the same pressure (Fig. 12). This borehole is found to be non-responsive to the pressure transients in the tunnel (discussed in Sect. 5.3). The pore pressure in this borehole is found to be continuously rising as one can see until around day 135 (Fig. 12). This is happening even during intermittent shutdowns (days 72–76) and when water pressure in the tunnel is in static condition.

However, as one can see in Fig. 12, a drastic change in pressure behavior in this borehole occurred after around day 135. At first the water pressure was fluctuating and afterwards a sudden increase in pore pressure was observed making this borehole responsive to plant shutdown and start of operation and also started responding rapidly to the pressure transients.

Borehole 3 (BH3) BH3 registered most of the pressure build-up in 7 days after deaeration. Since the rock mass is already saturated by the time the sensor was installed, pressure build-up in the pipe after deaeration occurred relatively faster as compared to pressure build-up in BH2. It has the lowest magnitude of pore pressure as compared to other boreholes except BH5, which does not respond to pressure transients in the tunnel (discussed in Sect. 5.3).

Between days 78 and 136, the production is stopped and the tunnel water pressure is increasing due to rising water level in the reservoir. During this period the pore pressures in BH2, BH4 and BH5 are also increasing at the same rate. However, pore pressure in BH3 is increasing at a slower rate

Fig. 12 Pressure readings during plant operation



between days 78 and 102 and further decreasing between days 102 and 136 similar to the behavior seen in BH1, which also indicates water seepage through the rock mass from the daylight area of the construction adit. The rate of pore pressure drop during dewatering is the slowest in this borehole (Fig. 12).

Borehole 4 (BH4) BH4 registered rapid pressure build-up just after deaeration. This borehole has high pore pressure magnitude and is responsive to pressure transients (discussed in Sect. 5.3).

Borehole 5 (BH5) The pressure build-up in BH5 is faster as compared to BH3 but it has the lowest magnitude of pore pressure among all the boreholes. This borehole is non-responsive to pressure transients. However, the pore pressure changes occur much faster during transients as compared to other non-responsive boreholes (Fig. 12). During dewatering, the rate of pressure drop is almost as steep as responsive boreholes to a certain level and then becomes slower, which is similar to the non-responsive boreholes.

5.3 Response to Pressure Transients

Figures 13 and 14 show typical events of a normal shutdown and an emergency shutdown, respectively. These events are

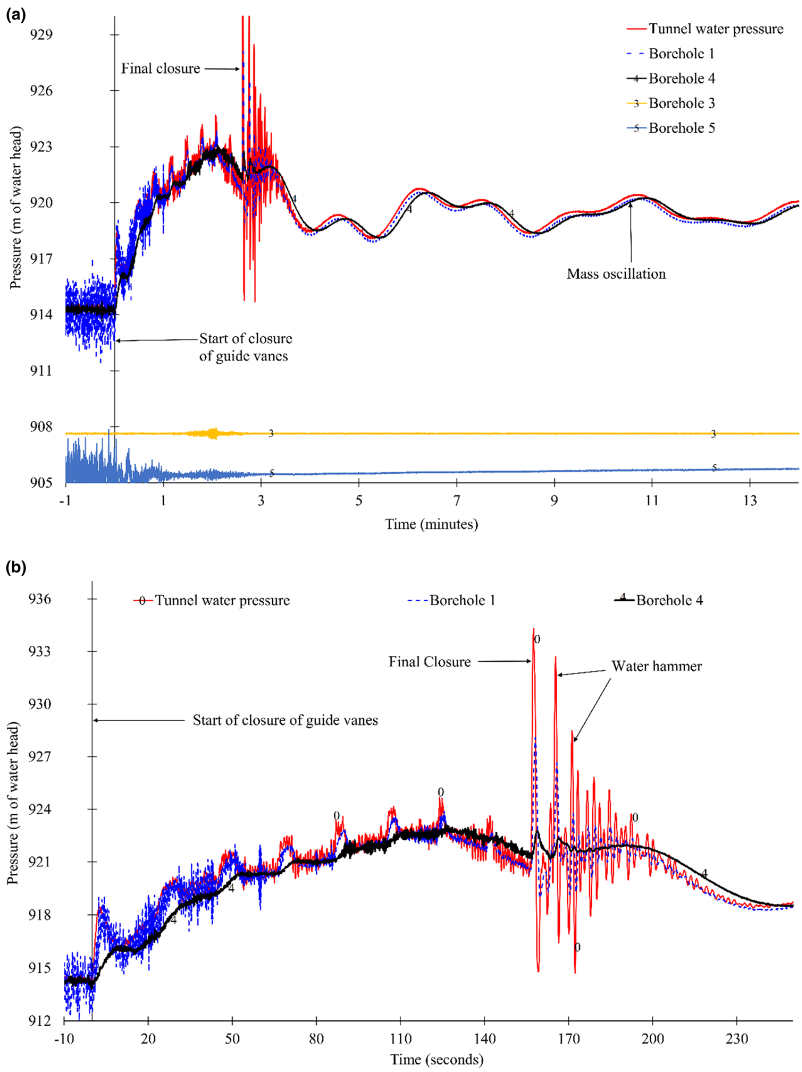


Fig. 13 Response of boreholes during normal shutdown (day 221) showing **a** complete transient event and **b** water hammer

representative of the load changes and shutdowns that are occurring very frequently in recent years. The difference between these two shutdowns is the speed of closure of the turbine guide vanes. In this case of normal shutdown, complete closure of guide vanes takes place within 160 s. On the other hand, during an emergency shutdown, complete closure of guide vanes takes place within 10 s. The time period of water hammer and mass oscillation is about 2 s and 220 s, respectively.

Two distinct behaviors of rock mass response from different boreholes in terms of pore pressure change can be seen in both shutdown cases (Figs. 13 and 14). The behavior of BH1 and BH4 is responsive with pressure variation inside the borehole almost identical to the pressure variation in the tunnel, but with reduced amplitude of pressure. The second behavior shown by BH2, BH3 and BH5 is non-responsive and shows very little or no variation in pore pressure inside the rock mass during load changes.

For the responsive case, the effect of mass oscillation can be clearly seen in both shutdown cases (Figs. 13a and 14a). It is observed that the mass oscillations cause significant changes in rock mass pore pressure. However, the effect of water hammer is different between normal and emergency shutdowns even though they have similar frequencies. It

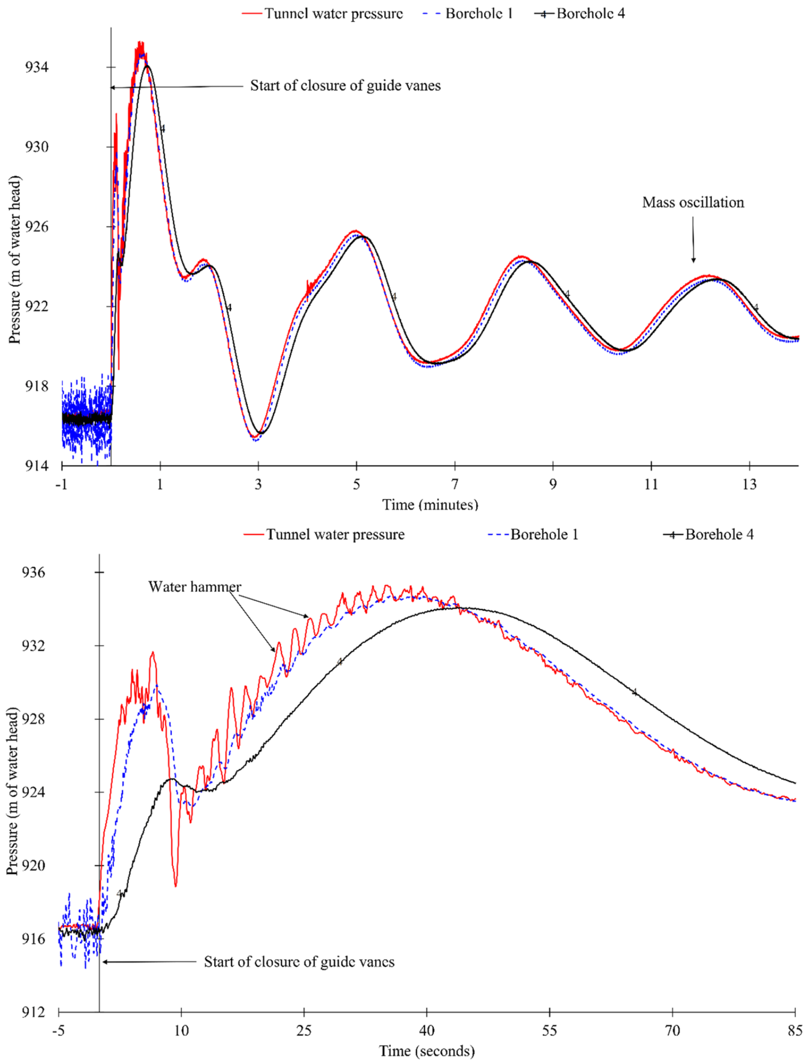


Fig. 14 Response of boreholes during emergency shutdown (day 334) showing **a** complete transient event and **b** water hammer

is observed that during normal shutdown the water hammer travels into the rock mass (Fig. 13b) causing the pore pressure to vary with almost the same frequency but with reduced amplitude. This effect is more pronounced in BH1 than BH4. On the other hand, during emergency shutdown, water hammer shows very little impact on the rock mass pore pressure in both BH1 and BH4 (Fig. 14b).

Normally, the magnitude of water hammer should be higher during an emergency shutdown as compared to a normal shutdown but is just the opposite as seen above. This could be explained by the fact that during emergency shutdown at Roskrepp, the guide vanes are closed first, then reopened slightly to reduce water hammer, and then fully

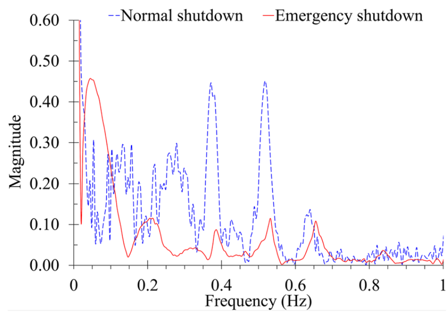


Fig. 15 Result of FFT analysis on water pressure signal during normal and emergency shutdowns

closed. This reopening dampens the water hammer but it is triggered two times (Fig. 14b). Further, to confirm that pressure peaks seen during normal shutdown is due to water hammer, a fast Fourier transform (FFT) analysis of the pressure signals after final closure was carried out. The result is shown in Fig. 15.

This figure shows five different peaks of different frequencies during normal shutdown. In a simple system, there should only be one peak, the water hammer traveling from turbine to reservoir and back to the turbine. However, at Roskrepp power plant there is a waterway system consisting of penstock shaft, transition from penstock shaft to unlined tunnel, sand trap, surge shaft and the unlined headrace tunnel. These structures act as obstacles for the pressure waves and reflect parts of the water hammer, resulting in multiple peaks at different frequencies.

It is seen that the two largest peaks during shutdown events have frequencies of 0.38 Hz and 0.52 Hz, which is equivalent to the time periods of 2.6 s and 1.9 s, respectively. The distance between turbine and free water surface at the surge shaft is 560 m and thus the wave propagation speed for these frequencies is 830 m/s and 1166 m/s, respectively. Both these velocities are within the normal range of water hammer propagation speed (800–1200 m/s). However, the water hammer speed is dependent on the stiffness of water and the conduit wall. In stiffer material such as steel pipe, the velocity will be higher as compared to a relatively flexible material such as the rock mass in an unlined tunnel wall. Hence, out of these two pressure waves, it is likely that the pressure wave with the higher velocity could be due to reflection from the cone area at transition between steel-lined and unlined section of the waterway. The one with lower speed could be the water hammer wave traveling through both steel-lined and unlined section between the turbine and free water surface in the surge tank.

5.4 Delayed Pore Pressure Response

There is a delayed pore pressure response in the rock mass, which can be observed during pressure transients in one of the responsive borehole BH4 as an example (Fig. 16). As one can see, during negative pressure transients, the drop in rock mass pore pressure is slower than the tunnel water pressure, which causes the rock mass pore pressure to be higher for some time. This situation occurs for the first few cycles of the pressure transient and then the pressure gradient gradually decreases as the mass oscillation attenuates.

BH1 also shows such time delay but for a shorter time period than BH4 (Fig. 14b). It is interesting to note here that the pore pressure in BH1 becomes equal to the tunnel water pressure faster than in BH4 indicating joint roughness, joint opening and infilling condition have an important role

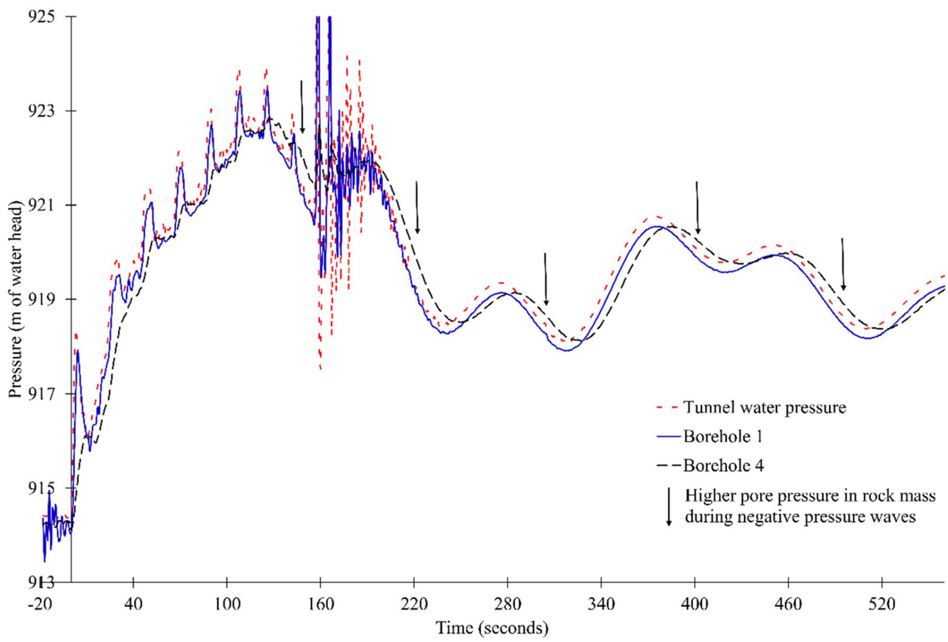


Fig. 16 Delayed pore pressure response of BH4

Table 3 Behavioral summary of the boreholes

Borehole	Pore pressure response
BH1[A]	*Responsive to transients* In general, fast build-up of pressure. Pore pressure drops when tunnel pressure is static (days 8–11 and days 23–30), indicating possible seepage to the daylight area in the construction adit. Erratic behavior between days 63 and 166. Relatively fast pressure drop during dewatering. Pore pressure magnitude is high, almost equal to tunnel water pressure
BH2[B]	*Non-responsive to transients* In general, slow build-up of pressure. Pore pressure increasing slowly even when tunnel pressure is static (days 44–46 and 72–76). Pore pressure magnitude is highest among all non-responsive boreholes
BH3[B]	*Non-responsive to transients* In general, slow build-up of pressure. Pore pressure drops when tunnel pressure is rising between days 102 and 137, indicating possible seepage from the daylight area in the construction adit. Slowest pressure drop during dewatering. Pore pressure magnitude is second lowest
BH4[A]	*Responsive to transients* In general, delayed pore pressure response observed during pressure transients in the tunnel. Fast build-up and drop of pressure during filling and dewatering. Pore pressure magnitude is high, almost equal to the tunnel water pressure in the tunnel
BH5[B]	*Non-responsive to transients* In general, slow build-up of pressure. Pore pressure increasing slowly when tunnel pressure is static (days 72–76). Fast pressure drop during dewatering up to a certain level and then becomes slower. Pore pressure magnitude is lowest

Superscript A or B on borehole names refers to their behavior as shown in Fig. 17

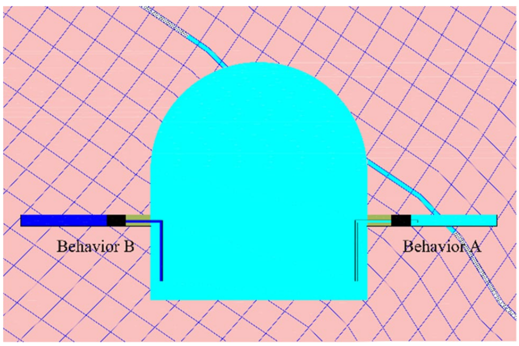


Fig. 17 Idealized sketch of the pore pressure behavior at the instrumentation location

in the pore pressure behavior. After this, the pore pressure changes in BH1 are almost as fast as the tunnel water pressure during the entire period of mass oscillation and hence the pressure gradient does not change during transients. As a result, the rock mass pore pressure is always less than tunnel water pressure.

6 Interpretation and Discussion

A summary of the most important observed behaviors of all boreholes is presented in Table 3 and logical interpretation and discussions are made in this section.

6.1 General Behavior

Two main behaviors (A and B shown in Fig. 17) are observed. Behavior A where a fast pore pressure build-up with high magnitude and responsive to pressure transients (BH1 and BH4) is recorded. Behavior B where slow pore pressure build-up with lower magnitude and non-responsive to pressure transients (BH2, BH3 and BH5) is observed. An idealized sketch for these behaviors is shown in Fig. 17.

The boreholes with behavior A intersect an open joint in addition to other relatively tight joints. For example, BH1 intersects the single conductive joint (Fig. 5) in addition to foliation joints (Jf) and BH4 intersects the conductive joint set (J2) in addition to joint set (J1). The boreholes with behavior B intersect relatively tighter joints of both foliation joints (Jf) and cross-joints (J1) and have no direct hydraulic connection to the conductive joints. These observations show that pore pressure response of the rock mass is generally dominated by joint systems that are conductive and communicative, which is quite logical.

6.2 Pore Pressure Magnitude

It is observed through this instrumentation that the boreholes with behavior A have almost the same pore pressure magnitude. On the other hand, the boreholes with behavior B show varying pore pressure magnitudes. BH2, BH3 and BH5 have pressure magnitudes from highest to lowest (Fig. 12), respectively. This may be explained due to the fact that either the length of flow paths from the tunnel through joint systems to each boreholes are different or the joint systems in the rock mass have varying roughness and infilling conditions, which pose different levels of resistance to flow through the joint system.

Conductivity of rock mass is a phenomenon that describes the movement of a fluid fracture openings and is a function of condition of the discontinuities present in the rock mass, viscosity of the fluid (in this case water) and degree of porosity of the intact rock. Since the viscosity of water

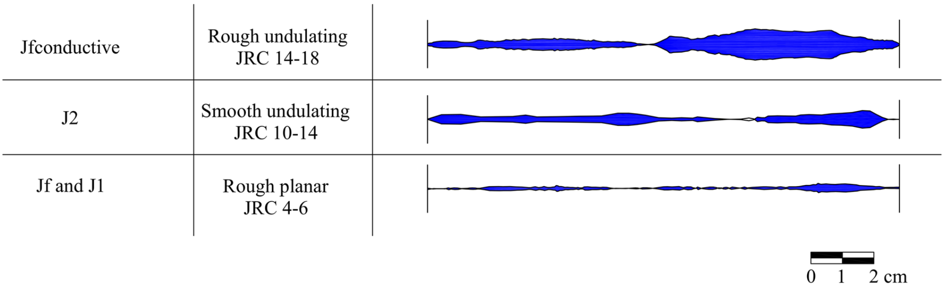


Fig. 18 Roughness profiles and resulting aperture of major joint system contributing to the pore pressure response at the instrumentation area of the tunnel at Roskrepp

and intact rock porosity are constant in this experiment, the conductivity of the joint system mainly governs the extent of borehole response.

The roughness profiles of major joint sets that contribute to the observed behavior were measured using Barton's comb. Typical roughness profiles from the measurements are shown in Fig. 18 in such a way that it is possible to visualize possible jointing conditions prevailing in the rock mass at the instrumentation area. Panthi (2006) presents a modified description of roughness profiles along with corresponding joint roughness coefficient (JRC) described by Barton and Bandis (1990). The measured roughness profiles are compared to this description to qualitatively describe the roughness conditions and assess the ranges of JRC values.

Figure 18 is a simplified version of the jointing condition that reflects the response phenomenon prevalent at the instrumentation area where similar joint stiffness and stress levels exist. In general, the higher undulation in the rock joint walls is linked to larger hydraulic aperture. The undulating joints with higher aperture have relatively less contact between the joint surfaces and thus will cause less tortuosity of flow through them resulting in higher conductivity as well as higher local pore pressure in the joint wall as seen in boreholes with behavior A. Brown (1987) conducted computer simulation to study the effect of surface roughness on flow through joints and concluded that at large separations (aperture) the surface topography has little effect on flow. At small separations, the flow is tortuous, tending to be channeled through high-aperture regions.

It is evident that longer flow paths will pose higher resistance to flow causing higher tortuosity and friction loss which reduces pore pressure in the rock mass. This type of behavior is more pronounced in case of tighter joints. BH2 showed the highest pressure magnitude (almost the same pressure as in the tunnel) among the non-responsive boreholes, which may be linked with the fact that it is close to cross-joint set (J2), which reduces the length of flow path through other joint sets. Moreover, the left wall has more joints exposed at the tunnel wall (Sect. 3.2), which also contributes to more net flow from the tunnel towards the borehole and hence higher pore pressure. In BH3, the nearest high-pressure zone to the borehole is $Jf_{conductive}$ on the right wall. All other joints intersecting with this joint contribute to the pore pressure registered in this borehole. However, unlike the left wall, the right wall has a smaller number of joints exposed to the tunnel, which may explain its pressure being lower than BH2. BH5 has the least magnitude of pore pressure, which may be linked to the fact that it only intersects tight joints with longer flow paths and have no connection to other conductive joints. This shows that pore pressure is a highly localized phenomenon in unlined pressure tunnels and varies at different tunnel locations depending upon the jointing intensity and jointing conditions present in the rock mass.

Further, both BH1 and BH3 have registered pressure drops while the power plant is not in operation and the tunnel water pressure is either static or increasing. Such pressure drops can be attributed to the seepage that is occurring through interconnected joint network in the rock mass that daylight to the existing construction adit. The adit tunnel wall near the datalogger is seen to be wet due to such seepage from the headrace tunnel (Fig. 10).

6.3 Response to Pressure Transients

Assuming very tight joint walls (73 μm) in the rock mass, Helwig (1987) concluded that the changes in rock mass pore pressure due to fast-acting water hammer are limited to a relatively shallow zone of rock mass wall (1.5 m) around the unlined tunnel perimeter, while the slow-acting mass oscillations may cause pore pressure changes deeper into the rock mass.

The observations of this instrumentation show that in case of open joints the water hammer can also travel deep into the rock mass as seen in BH4 (Fig. 13b). However, the amplitude is greatly reduced as compared to BH1. This difference in amplitude is most likely attributed to the difference in length of flow paths along the joints from the tunnel to the respective boreholes. The instrumentation has clearly demonstrated that if the joints are tight, the water hammer has almost no effect on the rock mass pore pressure (BH2, BH3 and BH5).

The current design practice in general, ignores the effect of water hammer in unlined pressure tunnel because the hydraulic stress on the rock block surfaces is acting only for a very short period of time. Dynamic phenomena with very short time period such as seismic waves can have the highest effect on structures and cause failure when they have eigenfrequencies close to the seismic wave frequency. It can be neglected in case of water hammer because the eigenfrequency of rock mass is much slower than water hammer. However, it may still have a significant impact for a hydropower plant where the frequency of water hammer is slower and comparatively closer to the seismic wave frequency. Further, this instrumentation indicates that it is worthwhile to investigate the effect of higher occurrence of such events over a long period of time since the cumulative dynamic impact of frequent pressure pulses may be relevant regarding long-term stability of rock blocks in tunnel periphery.

It is learned from this instrumentation that even slow-acting mass oscillations do not travel into the rock mass if the joints in the rock mass are very tight (BH2, BH3 and BH5). On the contrary, if the joints are open, the effect of mass oscillations is considerable deep into the rock mass as seen in BH4. The BH4 intersects conductive joint J2 at least 8 m away from the tunnel wall. The effect of pressure transients can be seen clearly in this borehole. The length of flow path along an open joint does not seem to significantly affect the pore pressure magnitude in case of open joints, since BH1 and BH4 have almost similar pore pressure magnitude during mass oscillations as summarized in Table 4.

From these observations, it can be concluded that joint geometry plays a dominant role regarding the effect of pressure transients as compared to the time period of pressure transients.

6.4 Delayed Pore Pressure Response

Out of two responsive boreholes BH1 and BH4, the noticeable delay in pore pressure response has been seen only in BH4. In the case of BH1, the delay occurs for a very short period and later the pore pressure changes almost as fast as the tunnel water pressure and hence does not cause destabilizing pressure gradients in the rock mass during transients. However, this will not be true in case of delayed response as observed in BH4.

Both BH1 and BH4 are located in similar rock mass conditions, intersecting the open joints with similar roughness and aperture but at different lengths from the tunnel contour. Hence, the only parameter that is different between BH1 and BH4 is the distance between tunnel contour and the boreholes along the length of the conductive joint. The borehole intersects the respective conductive joints approximately 1.5 m, 8 m along the length of these joints in case of BH1 and BH4, respectively. Therefore, it can be concluded that the delay in pore pressure response is mainly due to the distance that needs to be traveled by the pressure transient along the joints in the rock mass. This in authors' opinion is quite logical since the pressure wave requires more time to travel through a longer distance in the joint with a similar void geometry. It is noted here that the pressure gradient during a transient event is only about 0.1 MPa (Fig. 14) which is relatively small compared to the strength of rock joint walls with no clay infilling. However, regular occurrence of such events may ultimately lead to block failure even in relatively fresh joint wall condition as a result of cyclic fatigue over long-term operation of unlined pressure tunnels with frequent start–stop sequences as being experienced in Norwegian hydropower plants.

The observed phenomenon of delayed response during a pressure transient principally has the same effect on rock blocks in the rock mass as in case of a tunnel dewatering. Both events induce seepage forces on block surfaces as a result of hydraulic gradient developed between the rock mass and tunnel. Nevertheless, transients occur much faster and the pressure gradient and its time of application are much less as compared to a dewatering event. Even though they have smaller amplitude acting for a small period of time, they occur more frequently as load changes are occurring almost every day or even every few hours as seen in Fig. 12.

Table 4 Summary of behaviors during pressure transients

	Water hammer	Mass oscillation
Open joints	Water hammer travels deep into the rock mass but with reduced pore pressure amplitude (BH4). Higher effect is seen in shallow zones around the tunnel (BH1)	Mass oscillation travels into the rock mass. Pore pressure magnitude is not significantly affected by the length of flow path. But longer flow path causes delayed pore pressure response (described in Sect. 6.4)
Tight joints	No effect	No effect

7 Conclusion

The effect of power plant operation on pore pressure in jointed rock mass of an unlined hydropower tunnel has been studied experimentally in an operating power plant for the first time. Monitoring of leakage from the tunnel during first water filling in unlined pressure tunnels is a standard procedure in Norway and worldwide. However, to the best of authors' knowledge, field monitoring of pore pressure variation in an unlined headrace tunnel during power plant operation has not been conducted in the past.

The observations confirm the understanding that joint properties dominantly affect the fluid flow through the rock mass. More importantly, the experiment clearly demonstrates that the dominance of one or more sets conductive joints decides the pore pressure response of the rock mass. It is also concluded that around unlined hydropower tunnels, rock mass pore pressure is a highly localized phenomenon and varies in different locations in rock mass surrounding the tunnel depending upon the length of flow path and void geometry (aperture) of the joint systems.

Two main behaviors in pore pressure response against pressure transients have been clearly visualized: (A) responsive and (B) non-responsive condition. It is also concluded that the open joints show similar responsive behaviors, where roughness of joint wall seems less relevant in relation to flow and pore pressure. On the other hand, in the case of tighter joints, roughness influences the flow tortuosity, which results in different pore pressure magnitudes. It is seen through the observation that joint geometry plays a more dominant role regarding the effect of pressure transients as compared to the time period of pressure transients.

A delayed pore pressure response of the rock mass was observed during load changes, which causes a pressure gradient in the rock mass and in the tunnel. As instrumentation results suggest, even though the pressure gradient during a transient event is only about 0.1 MPa which is relatively small compared to the strength of rock joint walls, the regular occurrence of such events may cause block failure as a result of cyclic fatigue over long-term operation of unlined pressure tunnels with frequent start–stop sequences.

Hence, this experiment demonstrates that frequent load changes can affect tunnel stability. The overall effect depends on the pore pressure response in the rock mass, which is mainly governed by the conditions of joint geometry and their properties. The cumulative impact of small but frequent pressure gradients over many years of operation can result in increased instances of block falls and tunnel collapses. The findings of the instrumentation are valuable and will be helpful in developing a methodology using numerical modeling so that it is possible to predict extent of block fall events in unlined water tunnels experiencing different hydrostatic head with frequent start–stop sequences, which the authors are presently focused on.

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Compliance with Ethical Standards

Conflict of interest The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no financial support for this work that could have influenced its outcome.

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Paper 2

Operation of Norwegian hydropower plants and its effect on block fall events in unlined pressure tunnels and shafts

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Article

Operation of Norwegian Hydropower Plants and Its Effect on Block Fall Events in Unlined Pressure Tunnels and Shafts

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Abstract: The main objective of this study is to investigate the effect of hydropower plant operation on the long-term stability of unlined pressure tunnels of hydropower plants in Norway. The authors analyzed the past production data of some hydropower plants to find out the number of starts/stops and the frequency and magnitude of load changes. The study demonstrates that an average of 200–400 start/stop events are occurring per turbine per year for the analyzed period, with an increasing trend. Currently, 150–200 large load changes per turbine smaller than 50 MW are occurring every year, and this is expected to increase by 30–45% between 2025 and 2040 for one of the studied power plants. Most importantly, the monitored pressure transients and pore pressure response in the rock mass during real-time operation at Roskrepp power plant are presented. A new method is proposed to calculate and quantify the hydraulic impact (HI) of pressure transients on rock joints and the effect of duration of shutdown/opening, which is found to be the most dominant parameter affecting the magnitude. The results show that faster shutdown sequences cause unnecessary stress in rock mass surrounding pressure tunnel. The hydraulic impact (HI) can be more than 10 times higher when the shutdown duration is reduced by 50 percent. The study recommends that duration of normal shutdowns/openings in hydropower plants should be slower so that hydraulic impacts on the rock joints are reduced and cyclic hydraulic fatigue is delayed, prolonging the lifetime of unlined pressure tunnels and shafts.

Keywords: hydropower; unlined pressure tunnels; hydraulic transients; hydraulic impact; long-term stability; cyclic fatigue; block falls

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1. Introduction

The world energy market is going through a monumental shift from a fossil-fuel-based system to a renewable energy-based system, following the Paris agreement signed in 2015. According to [1], the share of renewable energy in the power sector would increase from 25% in 2017 to 85% by 2050, mostly through growth in solar and wind power generation. In the Nordic and European power market, the share of solar and wind power is expected to increase from about 20% to over 55% between 2018 and 2040 [2].

Technological advances in the field of wind turbines, solar panels, and batteries are increasing the efficiency of these systems and thus reducing their levelized unit cost of energy (LCOE). According to [1], it will be possible to build wind farms at 25 euros/MWh within 10 years. The inclusion of larger amount of wind and solar power causes price volatility due to surplus or deficit of energy at any given time. This also applies for Norway because the share of its regulated production is falling, as the growth in consumption is primarily covered by wind power [3]. This calls for the need to increase flexibility in operation of the existing power production and storage systems. Flexibility is the ability to make quick changes in operation at any time such that the balance between production and consumption can always be maintained, with lowest possible cost for carrying out such changes. The need for such flexibility can be both short-term, where changes are needed to balance the system within hours, minutes, and seconds, or long-term, in order to balance the system for days or weeks. There are various solutions that compete to provide flexibility, such as hydropower, hydrogen, and batteries. Among these solutions, regulated hydropower can provide both short- and long-term flexibility.

Norway has almost half of the reservoir capacity in Europe [4], and thus has a great potential for providing the much-needed flexibility for the European power market in the future. Various studies are being conducted to explore this possibility [5–8]. The results indicate that a high proportion of hydropower with large reservoirs has so far resulted in relatively low short-term price volatility in the Nordic region. However, the price volatility is currently increasing, and the operation of the hydropower plants is becoming more dynamic. Operating the existing and new power plants with dynamic operational regime comes with various technical challenges and operational risks. The Norwegian Research Centre for Hydropower Technology (HydroCen) [9] is conducting research in a number of areas to assess such technical difficulties and provide sustainable solutions to meet the future flexibility requirements in Norwegian hydropower system. Its scope of work ranges from long-term stability of structures, electrical and mechanical systems, environmental impacts, and market conditions.

In this article, the authors analyze the production data of some Norwegian power plants to establish some operational trends in order to explore how the operational regimes may influence the long-term stability of unlined hydropower pressure tunnels. Such tunnels constitute the majority of water conveyance system in Norwegian hydropower plants, with a total length of more than 4300 km and pressurized up to 1047 m of water pressure [10,11]. Since tunnels are unlined, water is in direct contact with the rock mass and the pressure transients resulting from operational changes directly affect the discontinuities in the rock mass, which in the long-term, cause block falls as a result of cyclic fatigue due to frequent pressure pulsations. Further, the authors analyze a two-year long real-time monitoring data [12] consisting of tunnel water pressure and rock mass pore pressure from the unlined headrace tunnel of Roskrepp hydropower plant in Southern Norway. The results are discussed, and the implications of powerplant operation regarding block instabilities in the pressure tunnels are discussed. A recommendation to reconsider the shutdown/opening duration is made based on the findings.

2. Operation of Hydropower Plants

2.1. Data Set

The data set consists of hourly production data for 10 hydropower plants of different installed capacities ranging from 35 to 960 MW. A total of 21 units ranging from 35 to 320 MW were analyzed, the details of which are shown in Figure 1. The number of starts/stops for all power plants were available for 19 years. However, the amount of production data per hour (MWh) available for each hydropower plant is different. In total, 19 years of data series were available for Roskrepp, Ulset, Brattset, and Driva; 16 years of data were available for Duge; and 6 years of data were available for all the other power plants. Driva, Brattset, and Ulset are located in central Norway in Orkla and Driva rivers and all other power plants are located in Sira and Kvina river valleys in southern Norway, which are connected through a series of reservoirs. The data set for Ulset, Brattset, and Driva was provided by Trønder Energi, and the data set for other power plants was provided by Sira-Kvina Kraftselskap.

In the Norwegian hydropower industry, one of the conditions for issuing a license may be that the hydropower plant is restricted from carrying out sudden and frequent start/stop production, commonly referred to as hydropeaking. This condition specifically applies to hydropower plants with a tailrace directly discharging into a river reach with vulnerable fish species or downstream water use. This requires the hydropower plant to be run smoothly and that load changes occur gradually so that sudden changes in the outlet water level are avoided. Evidently, such conditions affect the operational regime of the hydropower plant, which is also of interest in relation to long-term stability of unlined pressure tunnels used to convey the discharge. Out of the hydropower plants that have been used in this analysis, Driva and Brattset have such restrictions against hydropeaking. All the other power plants discharge into downstream reservoirs and have no operational restrictions.

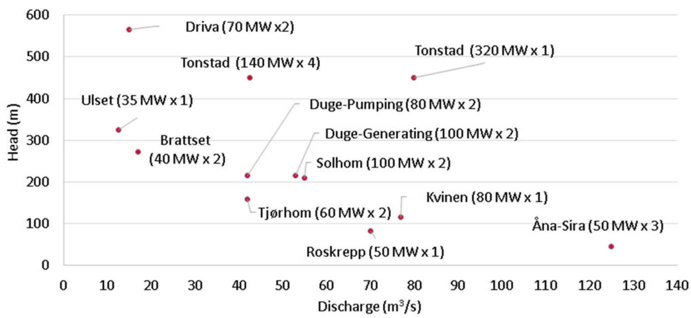

Figure 1. Details of power plant units in the data set.

2.2. Analysis Method

The main objective of analysis is to investigate the nature of load changes over the years and to gain knowledge about the magnitude and frequency of such load changes. The finest resolution of production data available is hourly resolution which creates some limitations to do precise calculations of magnitude and frequency of load changes in MW. For example, a production value of 40 MWh could be the result of the power plant being operated at 40 MW for an hour (no load change) or operated at 30 MW and 50 MW for 30 min each (one load change). Hence, it is assumed that every change in production per hour, larger than the minimum threshold, is a result of a single load-change event. It results in a conservative number of load-change events if more than one event occurs within an hour. However, the results still provide an insight into the nature of load-change events. Further, continuous measurement of water pressure with 10 Hz frequency at the Roskrepp headrace tunnel for two years (Section 3) shows that multiple load changes within an hour are not frequent; such events occurred seven times during the measurement duration. Hence, the production data were analyzed to count the number of events which matched the load-change types presented in Table 1.

For example, for a 100 MW unit, the maximum possible production per hour is 100 MWh. If the change in production between two consecutive hours is higher than 75 MWh, it is counted as one event larger than 75%. Depending upon unit capacity, changes in production smaller than a minimum threshold of 2–5 MWh are neglected because small changes may simply occur as a result of change of flow in the brook intake or changing reservoir levels and not necessarily due to turbine operation. The first type, LC1 provides an insight into how frequently the load changes are occurring and its overall trend over the years. The other types provide information about the magnitude of such load changes. Figure 2 illustrates these criteria with some examples.

Table 1. Load-change types considered for analysis of production data.

Load-Change Type		Description
LC1	Start/Stop	Number of starts/stops regardless of the production value when the hydropower plant is in operation. For example, a load-change event from 0 to any value higher than 0 is counted as one start event, regardless of the load in the second hour, and vice versa for a stop event.
		Number of load changes between consecutive hours when the magnitude of change is
LC2	LC < 25%	smaller than 25%
LC3	25% < LC < 50%	between 25% and 50%,
LC4	50% < LC < 75%	between 50% and 75%,
LC5	LC > 75%	larger than 75%

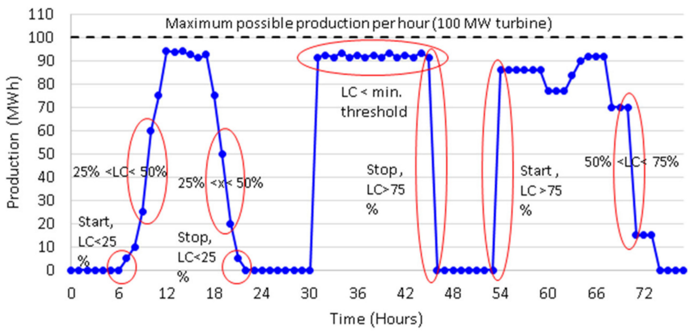

Figure 2. Illustration of the analysis method.

2.3. Results

2.3.1. Starts/Stops

Figure 3 shows the number of starts/stops for all hydropower plants over the years. Figure 3c shows the results for Tonstad units, which have significantly large capacities as compared to other units in Figure 3a,b,d. Figure 3d shows the results for units with operational restrictions, except for Ulset. It can be seen that there are large variations in the start/stop numbers over the years, especially for the hydropower plants without operational restrictions. The main reason for such variations is the power market [13], where production is directly connected to the power prices and financial benefit is the dominant factor. The results also show a general increasing trend in the number of starts/stops after 2009 for all the hydropower plants except Driva, Brattset, and Tonstad. This trend is stronger for smaller hydropower plants. Even with large turbine units, all units of Tonstad still undergo an average of 300–400 starts/stops every year. For power plants with operational restrictions, i.e., Driva and Brattset, the variations are much smaller, and the numbers of total events are also much smaller. This is in agreement with the conditions imposed by environmental regulations as explained before. A similar study carried out by [14] for 256 power plants in Norway with plant capacities up to 15 MW revealed that the number of starts varied from just 1 start to more than 250 starts. The study also concluded that the restrictions on hydropeaking did not really affect the number of starts.

Figure 4 presents statistical values of starts/stops (LC1) events in the power plants. A clear distinction in results is seen between hydropower plants with or without operational restrictions. Both average values and standard deviation are much smaller for hydropower plants with operational restrictions. The lowest number of starts/stops among all power plants is 65 per year per unit for Brattset. All other units experience an average of 200–400 starts/stops every year on average, and there is a large standard deviation in the data set.

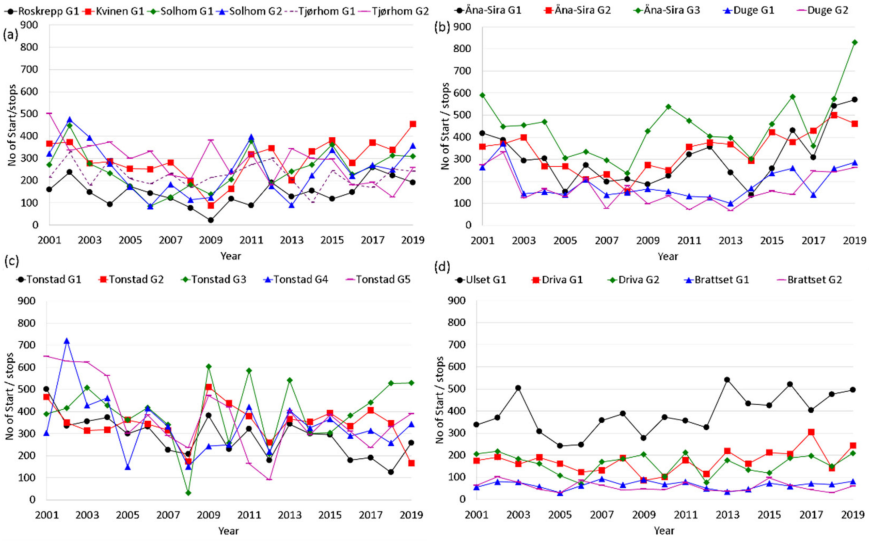

Figure 3. Number of start/stops of various powerplants between 2001 and 2019.

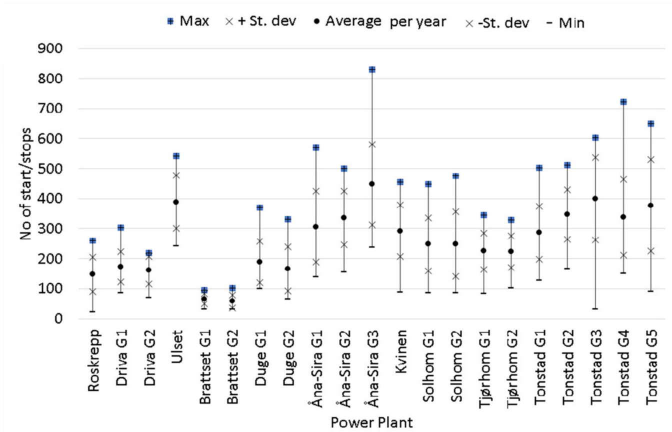

Figure 4. Statistical values of start/stops for various power plants.

Further, it should be noted that for hydropower plants with multiple units, the total number of pressure transients in the waterway caused by starts/stops is the sum of the number of events from all the units. This is because station-wide shutdowns are extremely rare emergency situations. For example, Åna-Sira and Tonstad waterways experience an average of 1100 and 1750 starts/stop events from single-unit operations every year, respectively. This also applies for load changes of various magnitudes, i.e., LC2 to LC5. It is difficult to ascertain whether any station-wide shutdowns had occurred during the period of analysis because of the 1 hr resolution data. For example, in Åna-Sira, one can see that all three units have been shut down from 42 MWh to 0 within an hour, but there are no sufficient data to decide whether all three units were shutdown simultaneously or with some time interval between them.

2.3.2. Frequency and Magnitude of Load Changes

Figure 5 presents some statistical values of load-change events that meet the criteria presented in Table 1. Results for only the first unit of all power plants are presented for the sake of clarity, because the units have the same capacity (except Tonstad G5) and the results for such units are almost similar.

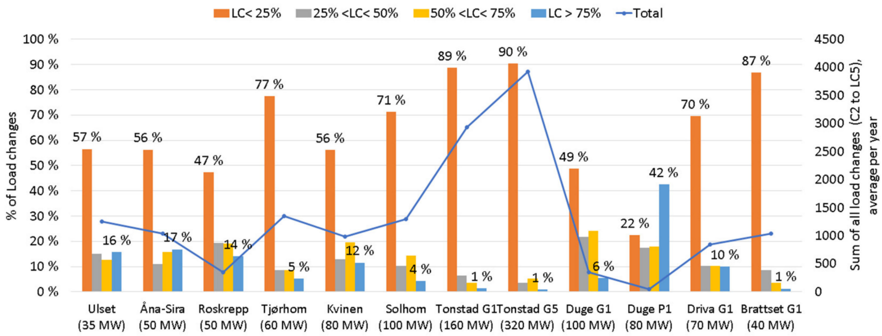

Figure 5. Percentage of load changes of different magnitudes.

For all power plants, the general trend is that there are higher numbers of smaller load changes (LC2) and relatively smaller numbers of larger load changes (LC5). It can be seen that the total number of load changes increases with increasing unit capacity, the highest number being close to 4000 for Tonstad G5. Smaller load changes (LC2) amount from 50% to 90% of the total and generally increase with unit capacity. On the contrary, larger load changes (LC5) decrease with the unit capacity. For smaller units such as Ulset and Åna-Sira, the number of LC5 events is more than 15% of the total, while for large units, it is as low as 5%, with the lowest being 1%. In all cases, the number of medium load changes (LC3 and LC4) are relatively smaller than small load changes (LC1), but they are also in higher proportion for smaller power plants. The results seem logical because for smaller turbines, a relatively smaller change in power price requires a load change which can be relatively larger compared to its capacity. For a larger turbine, a similar change can be handled by making a minor load change. For example, a load change of 30 MW is a large load change for Ulset, medium load change for Roskrepp/Åna-Sira, but a small load change for Tonstad G5. Power plants with operational restrictions, especially Brattset, show a different trend even though they have a small unit capacity; the share of large load changes is significantly smaller (1% of total) than other small units. This also applies for generating mode of Duge because it is primarily operated to serve the purpose of pumping. For Duge pumping mode, the share of large load change is the highest, which is logical because pumping is only possible at full capacity (synchronous generator-motor).

Hence, it can be concluded that power plants with larger units experience more load changes every year but of small magnitudes. Power plants with smaller units, however, experience fewer load changes annually, but the percentage of large load changes is much higher.

2.3.3. Future Trend in Power Plant Operation

As discussed in the introduction section, predictions made regarding future energy mix indicate a greater variability in operating condition. With regards to the long-term stability of pressure tunnels and shafts, it is of interest to see how the power prices affect the number of start/stops and frequency of load changes of various magnitudes as described in Table 1. This section presents the results of a production simulation for Roskrepp hydropower plant carried out by Statkraft Energy for different price levels representing the years 2025, 2030, and 2040. Statkraft Energy is also the source of data for the analysis presented in this section. The analysis is carried out using an hourly resolution with hydrological data for a period of 88 years and included all the power plants in the Sira–Kvina scheme. Four different alternatives were analyzed, including possible upgrading of the power plant to pumped storage plant with different capacities as shown in Table 2, and the results are presented in Figure 6.

Table 2. Alternatives for production simulation of Roskrepp hydropower plant.

Case	Description
A0	Existing situation (50 MW generation only)
A1	Pumped storage plant with 50 MW generation and 50 MW pumping capacity
A2	Pumped storage plant with 50 MW generation and 30 MW pumping capacity
A3	Pumped storage plant with 50 MW generation and 10 MW installed capacity

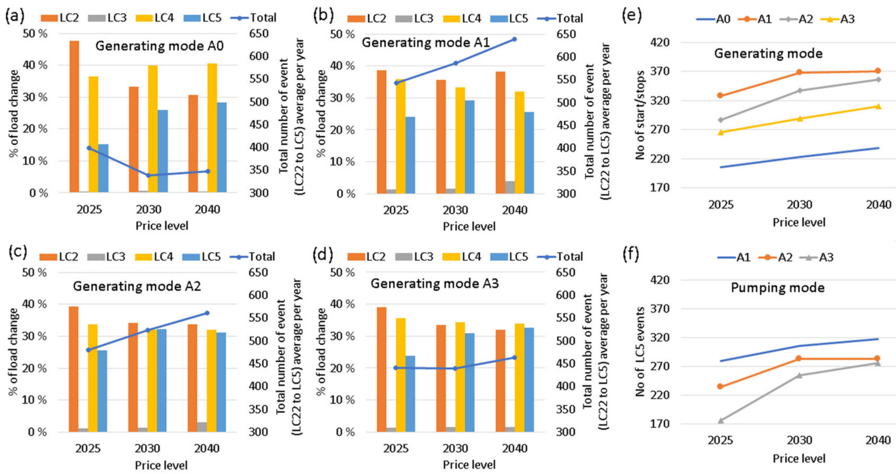

Figure 6. Trend of load changes of different magnitudes with future price levels.

The results show that for all cases of generating mode, the total number of load-change events increases, except for case A0. However, it can be seen that the increase in number of large load changes is the highest between 2025 and 2040 for this case. It can also be seen that the number of large changes (LC5) for all cases increases by 30–45% between 2025 and 2030 but slightly decreases or remains almost constant between 2030 and 2040. The number of starts/stops for all cases of generating mode is increasing with time. For all cases of pumping mode, almost all load changes are LC5 since pumping is done with full capacity (synchronous unit). Hence, the number of LC5 events is also equal to the number of starts/stops. It can be seen that the number of LC5 load changes for pumping mode also increases with time, but the rate of increase flattens between 2030 and 2040. A general conclusion that can be drawn from this analysis is that both the number of load changes and their magnitude are likely to increase significantly over the years. This further emphasizes the fact that tunnels will experience stronger transients with increased frequency in the future.

3. Tunnel Hydraulics and Hydraulic Impact

3.1. Data Set

The analysis in this section is conducted on the data set acquired from real-time monitoring of unlined headrace tunnel of Roskrepp hydropower plant in southern Norway for a period of two years. The experimental setup is designed by the authors and is described in detail in [12]. The waterway longitudinal section, instrumentation location, and setup are presented in Figure 7a–c, respectively. Figure 7d,e show the instrumented tunnel section and the pressure sensors and datalogger.

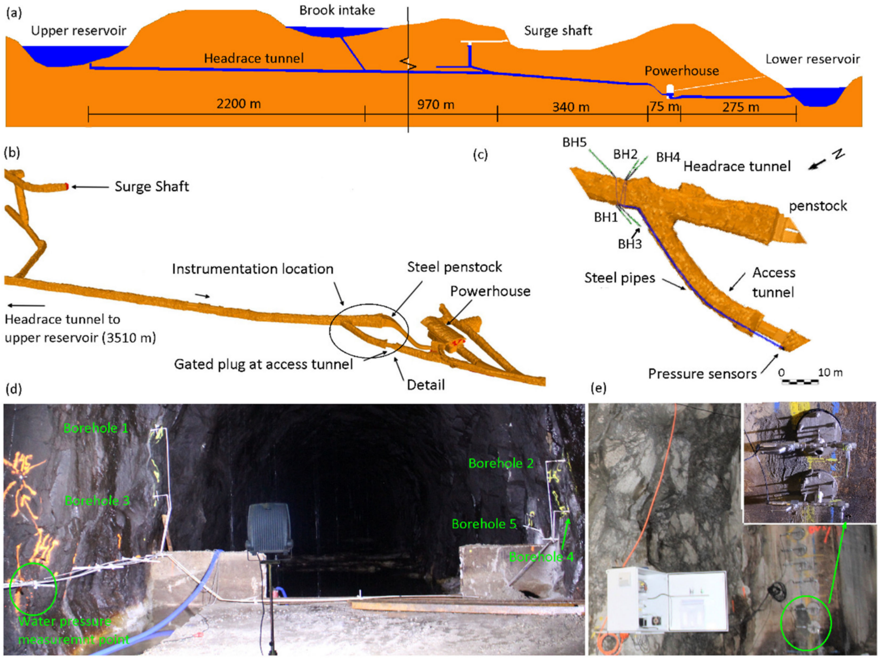

Figure 7. Longitudinal section of Roskrepp waterway (**a**), instrumentation location (**b**), and detailed view of the setup with boreholes BH1–BH5 (**c**), view of the instrumented tunnel section, looking upstream (**d**), and view of pressure sensors and datalogger in the access tunnel (**e**).

The rock mass at this location consists of weakly schistose granitic gneiss with three major joint sets. Five boreholes (BH1–BH5) were drilled in the tunnel walls such that they intersect a particular joint set almost perpendicularly. Stainless steel pipes were fixed in the boreholes using packers at different lengths in the borehole and the pipes were taken out of the tunnel to a dry area in the access tunnel, where they were fitted with pressure sensors and datalogger. The section of boreholes ahead of the packers collects water from the rock joints and are connected to the pressure sensors through the steel pipes, thus recording the rock mass pore pressure. Simultaneous readings of the tunnel water pressure are also recorded from a pipe installed at the junction between the headrace tunnel and access tunnel.

3.2. Shutdown Procedure

Figure 8 shows a pressure transient in the tunnel and the rock mass pore pressures during a typical shutdown event at Roskrepp hydropower plant. The rock mass pore pressure measured in three boreholes, along with the hydraulic impact during the transient (explained in following section) are shown in the figure. Both water hammer and mass oscillation are recorded by the pressure sensor because the measurement is done at a location between the turbine and the surge shaft.

Shutdowns in hydropower plants can be done in two different ways. An emergency shutdown is the fastest possible closing time for a particular unit and usually only occurs when the unit is stopped by the protection systems or if the unit is disconnected from the power grid during operation. Emergency shutdowns cause strong transients with large pressure amplitudes but are rare. During a normal shutdown, the unit is de-loaded manually in steps by the power plant operator. The purpose in this category of shutdown is to make soft shutdowns in several steps to reduce stresses on the electromechanical components. Both the emergency and normal shutdowns can be divided into two parts:

(1) the guide vanes closing to reduce the flow through the turbines and (2) the final closing of the slower moving main inlet valve (MIV) to completely stop the flow. In the case of a normal shutdown, the operator decides the number of steps to reduce the power output before the final shutdown signal is given. For example, the operator could run the unit down in 5 MW steps over several minutes or complete the shutdown in a single operation within a few minutes. When the final shutdown signal is given, the guide vanes are fully closed, the unit is disconnected from the grid and starts to decelerate and finally the brakes are activated to bring the unit to a standstill. The MIV normally starts to close after a fixed amount of time after the shutdown signal is given. Usually, this is in between the time when the unit is disconnected from the grid and the time when the unit has fully stopped rotating.

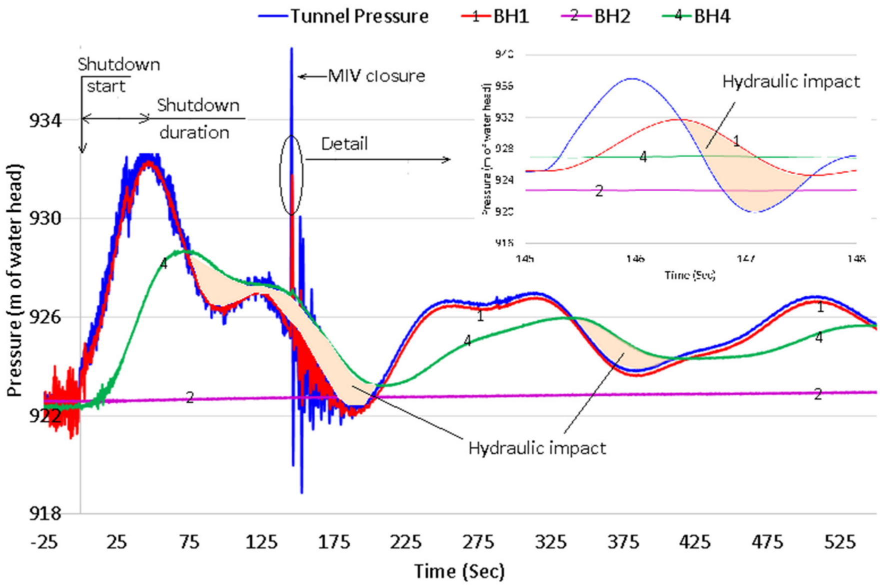

Figure 8. Typical pressure transient with pore pressure responses from boreholes BH1, BH2, and BH4.

It is difficult to ascertain the exact load steps taken for every shutdown event presented here since the data availability is with 1 h data resolution and this is a manual operation without any fixed standards. Hence, for this analysis, the time between start of shutdown event and the peak mass oscillation amplitude (as shown in Figure 8) is considered to be a relative measure of how fast the shutdown was carried out and is referred to hereafter as "shutdown duration". The measurements show that this parameter significantly affects the pressure readings, which are discussed in the results section.

3.3. Rock Mass Response and Hydraulic Impact

Fluid flow in the rock mass mainly occurs through interconnected network of joints and discontinuities since the permeability of intact rocks of igneous and metamorphic origin is negligible. When a pressure transient occurs in an unlined pressure tunnel, the fluid flow and pressure in the rock joints are changed. The change of fluid pressure on the rock joint surfaces causes additional seepage forces to act on these joint surfaces. This is mainly due to water transmission delay or time-lag between pressure peaks in the pressure tunnel and in the joint surfaces of the rock mass. This force can be calculated based on the area bounded between the pressure curves of the tunnel and pressure curves developed in the rock mass when rock mass pore pressure is higher than the tunnel pressure (shaded areas in Figure 8). It can be observed that this situation occurs for some cycles of the pressure transient and the number of such cycles depends on the magnitude of time-lag. It gradually decreases as the transient attenuates and both pressures become almost equal or the pressure in the tunnel is higher. This additional force is hereafter named as "hydraulic impact" (HI) and has a unit of MPa.s and is similar to dynamic viscosity or the force acting on the joint surfaces per unit area over time. The hydraulic impact (HI) is a destabilizing force, and the authors regard it to be the main driver for rock block destabilization in the tunnel periphery due to hydraulic transients and may cause rock falls and potentially tunnel collapse.

It can be seen in Figure 8 that the boreholes which intersect the conductive joints in the rock mass, i.e., BH1 and BH4, strongly respond to pressure transients, whereas the others are nonresponsive, since they do not have direct hydraulic contact with the conductive joints. BH1 registers a stronger response to pressure transients, but there is very little time-lag during mass oscillation, resulting in very little to zero hydraulic impact during mass oscillation and significant hydraulic impact during water hammer. For BH4 on the other hand, a clear time-lag is registered during both mass oscillation and water hammer. However, the amplitude of pore pressure in BH4 in response to the water hammer is smaller as compared to BH1. This difference in the response is due to different resistance to the flow through joints in the rock mass, which is a function of void geometry of joints and the length of flow path, i.e., joint length between tunnel wall and its intersection points with individual boreholes. The distance between tunnel wall and boreholes (length for flow path) BH1 and BH4 are 2.3 and 8 m, respectively.

From a theoretical standpoint, it can be deduced that the hydraulic impact on rock joints is dependent on the magnitude of change of discharge during shutdown and the duration of shutdown event. These two parameters govern the nature of transient pressure pulses which travel inside the rock joints causing additional forces on the joint surfaces. Another important parameter is the static pressure before transient which governs the resistance to flow through joints during transients. The joint hydraulic aperture is influenced by the effective stress across joints. During the operation of a power plant, the effective stress across the joints can vary depending on reservoir levels, which may change the initial hydraulic aperture before transients. Hence, the effect of these three parameters on the hydraulic impact is further analyzed.

3.4. Method for Calculating Hydraulic Impact

The total hydraulic impact due to a pressure transient is divided into two parts: due to water hammer and mass oscillation. The hydraulic impact due to water hammer is the sum of the pressure pulses that start when the guide vanes start closing and ends after the MIV has closed. The largest pressure pulse occurs when the MIV is finally closed, and pulses continue to travel between the turbine and surge shaft, until they dissipate after some time (Figure 8). The pressure sensors also register some noise introduced by the vibration of steel pipe caused by the deceleration of the water and turbulence during the shutdown until the final closure of MIV.

3.4.1. Total Hydraulic Impact

The total hydraulic impact for each borehole in the rock mass is then calculated as the pressure difference integrated over time between pressure signals at the tunnel and in the boreholes, after removing the noise due to pipe vibration in the pressure signals (the area indicated in Figure 8). A Butterworth filter with low-pass frequency [15] has been used to filter the noise which occurs due to sudden deceleration of water during the guide vane closing phase. A low-pass cutoff frequency of 3 Hz has been selected. This cutoff frequency is decided using theoretically calculated water-hammer frequency, assuming the wave speed from 800 to 1200 m/s and a distance of 87 m between the turbine and the sensor. The reason for selecting this cutoff frequency is that any pressure wave originating from the turbine which has a frequency higher than this would not be picked up by the sensor because they would not reach the sensor location. Such signals are the result of parts of water hammer being reflected by various physical structures and transitions between sensor location and turbine. They would only be recorded if the measurements were done close to the turbine. Hence, the major source of such high-frequency signals in the measurements is mostly pipe vibration. This argument is supported by the fact that signals with frequency higher than 3 Hz are drastically reduced when the MIV is completely closed. After MIV closure, the net water flow toward the turbines is zero, and hence, there is limited vibration of the pipes. A comparison of HI values due to water hammer computed with and without using the noise filter resulted in a difference of 11% for BH1 and 2% for BH4.

3.4.2. Hydraulic Impact Due to Water Hammer and Mass Oscillation

The water-hammer pulses are superimposed with the mass oscillation, and thus, they need to be isolated in order to calculate their hydraulic impacts separately. Fast Fourier transformation (FFT) of the pressure signal (Figure 9) shows that there are three mass oscillation frequencies corresponding to time periods of 3.4, 1.6, and 1.1 min, moving between reservoir and the surge shaft, between reservoir and brook intake, and between brook intake and surge shaft, respectively. The strongest frequency corresponds to the one that occurs between the reservoir and surge tank, which is also the slowest of the three pulses. The water-hammer pulses were filtered from the pressure signals using the Butterworth filter of cutoff frequency 0.0167 Hz, i.e., time period of 60 s, which is close to the fastest mass oscillation frequency of 0.015 Hz. The area between pressure pulses calculated after using this filter gives the HI value for mass oscillation. The HI value for the water hammer is then calculated by subtracting the mass oscillation HI from total HI.

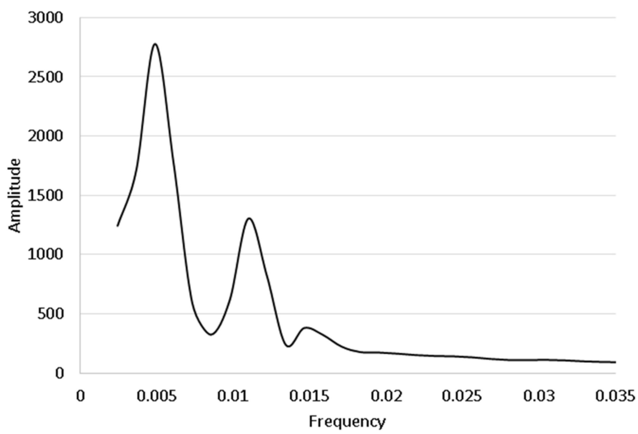

Figure 9. Result of FFT analysis of mass oscillation.

3.5. Results

3.5.1. Effect of Shutdown Duration

As a result of the differences in the shutdown procedures, normal shutdowns are rarely similar with each other since it is a manual operation and depends on the operator. An example of the effect of such difference is illustrated in Figure 10, which shows pressure signals from two shutdown events with similar durations until MIV closure and similar head loss before transients but producing different pressure signals.

It can be seen that Transient 1 is a result of a gradual de-loading and shutdown, whereas Transient 2 is a faster shutdown, before the MIV closure which occurs almost at similar durations indicated by the largest water-hammer pulses. The mass oscillation amplitude for Transient 2 is larger than Transient 1, and the peak is reached much faster than Transient 1, which is the result of a faster shutdown. Shorter shutdown duration

(Transient 2) results in a steeper mass oscillation curve which causes a larger time-lag between the pressure peaks in the tunnel and in the rock mass (Figure 10b), since it allows shorter time for the rock mass pore pressure to increase. The time-lag for the first pressure peaks for Transient 1 and Transient 2 are 8.5 and 26 s, respectively. A higher time-lag combined with a larger oscillation magnitude during short shutdown duration results in a larger HI.

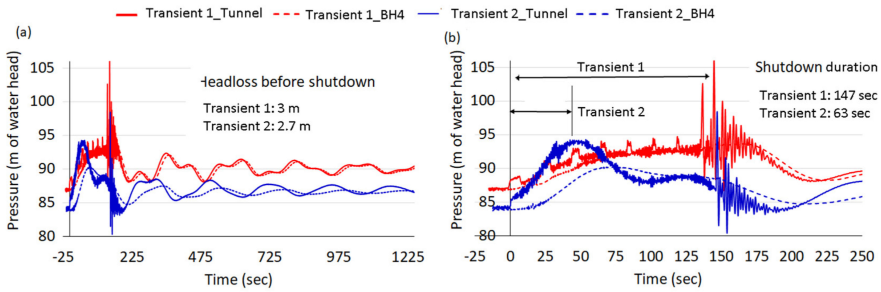

Figure 10. Comparison of pressure signals of two normal shutdowns (**a**) and enlarged view until 250 sec (**b**).

Figure 11 shows the hydraulic impact of 161 recorded shutdown events, in relation to their shutdown durations in chronological order. It presents the HI caused by water hammer and mass oscillation for BH4 along with the shutdown durations and the time-lag of the first mass oscillation pulse for each transient. It can be seen that the time-lag increases significantly with shorter shutdown duration, which consequently increases the HI due to mass oscillation. It is interesting to note the sudden decrease in shutdown duration between event 118 and 119. These two events have occurred within 24 h, and thus the head loss and static pressure before transient for these events are only marginally different. This has caused a sharp increase in time-lag from 6 to 10 s, and in addition, the HI value has increased from 0.8 to 1.9 MPa.s. An even higher increase in time-lag and HI is seen after event 129. The time-lag and HI after event 129 are larger than 20 s and 3 MPa.s, respectively.

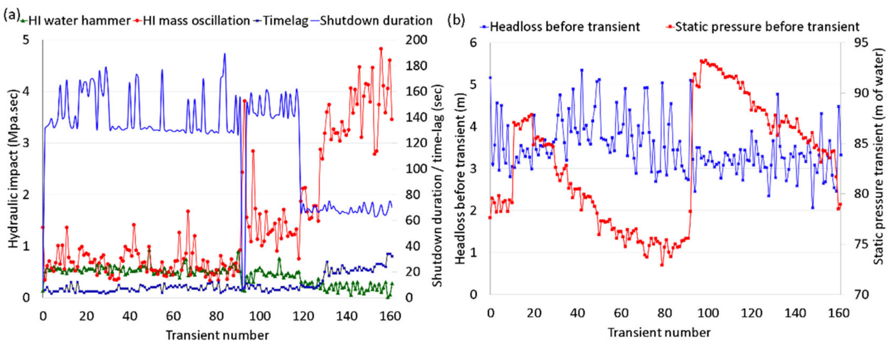

Figure 11. HI for BH4 during various transients in chronological order (**a**) and corresponding discharge change magnitude and static pressure before transient (**b**).

The hydraulic impact due to water-hammer increases with longer shutdown duration (Figure 12) because longer shutdown duration allows more water-hammer pulses to occur before the MIV closure. Further, the water-hammer pulses after MIV closure also have some time-lag which contributes to additional HI. It can be seen that mass oscillation causes almost no hydraulic impact for BH1.This is because the time period of mass oscillation is much slower, and thus, it allows pore pressure build-up in the borehole almost with the same rate as the pressure pulse, thus preventing any time-tag. This shows that the rock mass can respond differently for varying time periods even with same joint conditions or flow resistance. The effect of the time period on HI and joint deformation has been analyzed in detail in [16] using numerical simulation.

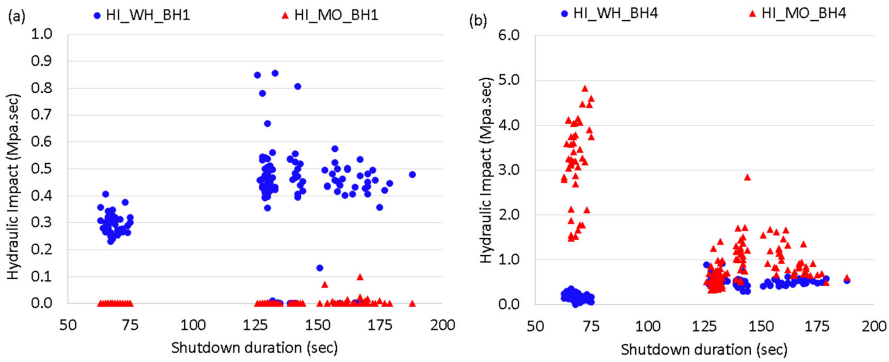

Figure 12. Effect of shutdown duration on hydraulic impact (**a**) for BH1 and for BH4 (**b**).

For BH4, HI caused by water hammer is in a similar range as for BH1. The HI caused by mass oscillation is significant for BH4 because the flow resistance is higher, as the length of flow path is larger than for BH1 and thus causes significant time-lag. It can be seen that faster shutdowns cause larger HI, and it is more than 10 times higher when the shutdown duration decreases from 130 to 65 s. Further, it is interesting to note the contribution of water hammer in the total hydraulic impact during pressure transients. For longer shutdown durations (125 to 200 s), water hammer contributes 40% of the total HI on average, the minimum and maximum values being 3% and 65%, respectively. However, for faster shutdowns (below 75 s) the average, minimum, and maximum values are 7%, 1%, and 20%, respectively.

3.5.2. Effect of the Discharge Change Magnitude

The head loss between the reservoir and the measurement location before the shutdown event is a measure of the magnitude of discharge change in the waterway. Larger head loss means larger change in discharge and large load change during a shutdown event. Calculation of the discharge during each transient is not attempted because the discharge added in the tunnel from the unregulated brook intake introduces uncertainties in the calculation. In general, a shutdown from a larger discharge should result in a larger mass oscillation. However, a distinct relation between the discharge change magnitude and HI due to mass oscillation was not observed as was the case for shutdown duration. Figure 13 shows two different clusters of HI values for mass oscillation which are strongly influenced by shutdown duration.

Most of the HI values above 2 MPa.s (Figure 13a) correspond to shutdown durations smaller than 75 s and show a somewhat increasing trend with larger head loss before a transient. The majority of values below 2 MPa.s correspond to shutdown durations longer than 125 s and tend to show a decreasing trend with increase in head loss. However, a shutdown from a larger discharge should result in a larger mass oscillation and hence larger HI. This opposite trend can be explained by the fact that there is a greater variation in shutdown duration larger than 125 s (Figure 13b). Hence, it is concluded that shutdown duration is more dominant parameter as compared to the discharge change magnitude. This is because the gradient of pressure change is more important than magnitude, as seen from the measurements.

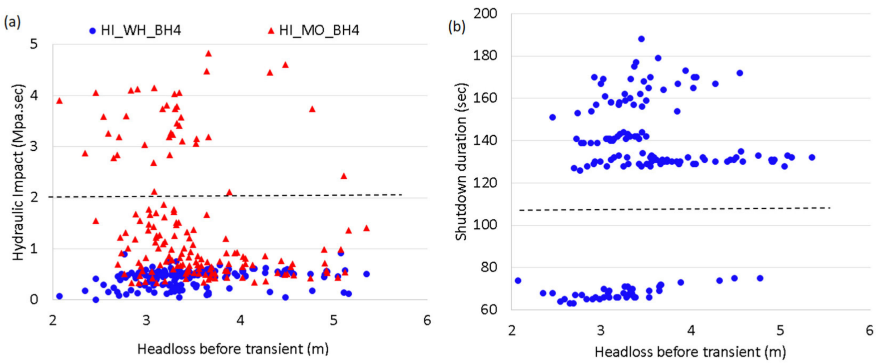

Figure 13. Hydraulic impact in relation to the magnitude of discharge change (**a**) and magnitude of discharge change vs. shutdown duration (**b**).

3.5.3. Effect of Static Pressure before Transient

The static pressure before transient can have some influence in the shape of the mass oscillation curves because of two reasons: (1) it decides from which water level in the surge tank and brook intake that the oscillation starts, and (2) different static heads may mean different water levels in the reservoir. This could affect the HI due to mass oscillation to some extent. However, Figure 14 shows that the HI values are unaffected by the increase in static pressure. The largest values inside the oval are from the events with the shortest shutdown duration (event 118 to 161).

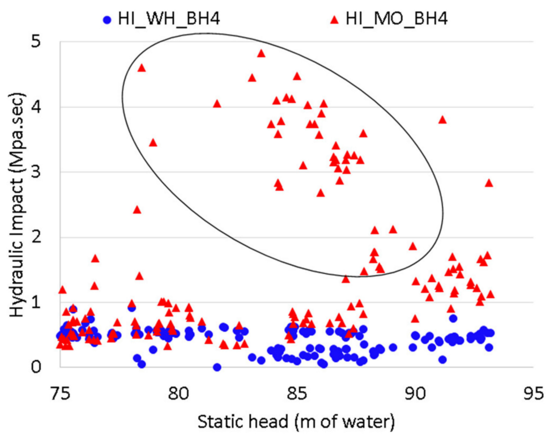

Figure 14. Effect of static head before transient on hydraulic impact.

The explanation to why the result is almost unaffected by the static pressure is most likely because effective stresses are much larger as compared to the change in static head and static pressure and are not large enough to cause significant changes in the joint aperture. Further, the high stiffness of the joint surface for hard rocks can prevent noticeable changes in the joint aperture.

3.5.4. Effect of Tunnel Dewatering

As shown in Figure 15a, the tunnel was dewatered after event 126 in spring, and then the refilling was done after two months during summer and event 127 occurred in autumn. According to [17], the controlled tunnel filling or dewatering in Norwegian unlined pressure tunnels and shafts is generally carried out at a rate of 15 m head increase/decrease per hour with a stop for minimum 2 h per 150 m head change and maximum head of 300 m per day. This is done in order to reduce excess hydraulic impact on rock mass around tunnel during filling and dewatering. The static head at the measurement location in the Roskrepp tunnel before dewatering was 80 m, and the dewatering was conducted in two stages within a duration of 7 h. The pore pressure sensors connected to BH1 and BH4 record HI values of 3 and 20 MPa.s, respectively, over the dewatering duration.

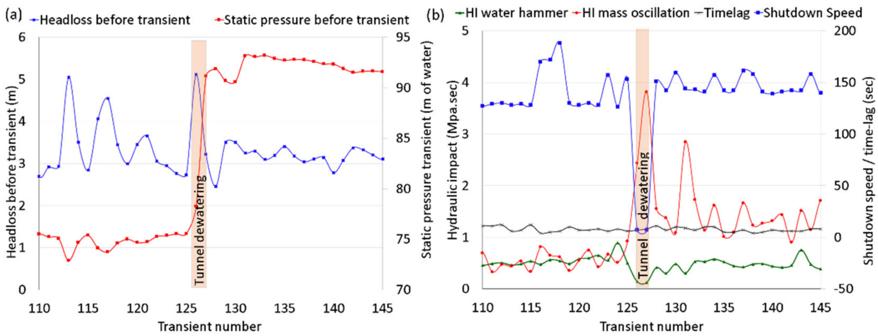

Figure 15. Discharge change magnitude and static pressure before transient for events 110–145 in chronological order (**a**) and corresponding HI for various transients (**b**).

It is observed that normal shutdowns after event 127 show increased HI values which are greater than 1.5 MPa.s, whereas the values before this event were in the range of 0.5–1 MPa.s. (Figure 15b). The static head before transient has significantly increased because event 127 occurred in autumn, after the reservoir had been filled by snowmelt during the spring and summer. However, as discussed in the previous section, this has no significant influence on the HI. Hence, it shows that dewatering is the reason for increase in HI values after event 127. Sudden increase in hydraulic impact (HI) after dewatering suggests that it has caused irreversible changes in the joint void geometry possibly caused by shear displacement, increasing the joint permeability, and thus allowing more flow into the joint during transients. This is a reasonable explanation because tunnel dewatering is known to cause macroscopic joint displacements and block falls, because the draining of rock joints takes time and rock mass pore pressure exceeds the tunnel pressure during such events. This occurs until the rock mass is fully drained and pressures are equalized.

Further, it should be noted the events 126 and 127 are emergency shutdowns with shutdown durations of 7 s, which caused significantly high hydraulic impacts (2.5 and 3.8 MPa.s) as compared to previous events. A similar phenomenon was also reported by [18] in the air cushion surge chamber of Osa power plant where significant increase in air loss was experienced after a few turbine rejections with pressure rise as high as 20% of static pressure Hence, the difference in HI values could thus be linked to both dewatering as well as contribution from the emergency shutdowns. Nevertheless, more measurements are needed to investigate whether emergency shutdowns alone can cause a noticeable change in behavior.

4. Discussion

The underlying axiom of block theory is that the failure of an excavation begins at the boundary with the movement of a block into the excavated space [19]. This implies that the orientation of joints should be conducive to create a wedge in order to cause a block fall. Further, removal of key blocks could result in extended fallouts. The existing block theory is still applicable for assessing the block falls due to tunnel operation, except for the fact that additional destabilizing forces are created during hydropower plant operation. This is not a one-time event but a cumulative effect of many small events, referred to as cyclic fatigue, which occurs over the course of hydropower plant operation. During tunnel excavation, potentially unstable wedges are identified, and adequate support is provided. However, relatively stable and unsupported blocks during construction may be destabilized as a result of weakened joints due to long-term fatigue over the years of hydropower plant operation. In some occasion, the potential rock blocks are not detected during construction. More importantly, some blocks may be held together by an intact rock bridge posing no threat of block falls during construction. Such intact rock bridges can gradually weaken due to the cumulative effect of HI and eventual rupture causing block falls. Similar findings have been highlighted by Preisig et al. [20] where the rupture of intact rock bridges due to seasonal pore pressure changes are attributed as the cause of progressive failure and fatigue in deep-seated landslides.

Figure 16 shows an example of a block fall witnessed in the TBM section of the 4.5 m diameter unlined headrace tunnel of Ulset power plant, which has been in operation since 1985. Unlined tunnels are generally dewatered for inspection after one year of operation to investigate if there are large block falls which could pose major stability issues in the future. The first inspection of this tunnel was conducted in 1986 after one year of operation, which revealed no major instabilities but showed some minor block falls [21]. For unlined pressure tunnels, minor block falls are expected during the first inspection because the rock mass is subjected to a new ground water regime and reduced effective stresses after tunnel filling. The washing of joint infilling material also reduces joint stiffness, which further contributes to block falls. The situation stabilizes after a new equilibrium is reached. The second inspection report carried out in 1992 [22,23] also does not show any serious instability issues. The last inspection was conducted by the first and third author of this article in 2017. During this inspection, several block falls similar to the one shown in Figure 16 were observed along the whole TBM tunnel length. From the Roskrepp measurements, it is now evident that additional load or hydraulic impact occurs during load changes. Hence, it is postulated herein that such block falls are the result of rock mass fatigue caused by cumulative HI due to transient events with load changes over the years. It is noted that the operational regime of Norwegian hydropower plants has changed after the deregulation of power market in 1991 [13]. This is most likely a contributing factor for the observed block falls because the deregulation has resulted in more start/stops and load-change events.

The results presented in previous sections clearly indicate that additional load on rock joints or HI due to hydropower plant operation depends on the shutdown duration and the magnitude of load changes. With the increase in intermittent energy in the future power system, it is very likely that both these parameters will be affected. Results from the analysis of production data show that both the number of starts/stops and frequency and magnitude of load changes will increase in the future. This means that the tunnels will experience stronger transients with increased frequency, causing more transient cycles with higher HI and accelerated fatigue.

The analyses show that hydropower plants with smaller units experience smaller number of load changes every year, but the proportion of large load changes relative to their plant capacities are much higher. This means that their waterways experience more transients with larger amplitudes relative to the static pressure. The minimum design factor of safety against mass oscillations in unlined pressure tunnels is 1.3 [24], meaning the normal stress acting against a critical joint must be at least 1.3 MPa if the design tunnel static pressure is 1 MPa. A larger load change causes bigger mass oscillation amplitudes which are closer to critical normal stress (or reduced factor of safety). Frequent events with such oscillation pressures close to critical normal stress contribute to accelerating the cyclic fatigue, especially if the transients occur when the tunnel static pressure is close to design static pressure. On the other hand, small transients even though in larger numbers may not necessarily have a higher impact on the rock mass. This could suggest that the rock mass around the tunnel in smaller hydropower plants may experience fatigue at a faster rate, assuming similar rock mass and effective stress conditions. However, it is difficult to ascertain in absolute terms whether a relatively strong transient in hydropower plants with low static head is more damaging than relatively weaker transient in hydropower plant with high static head. It is highlighted that the HI is dependent on the resistance to flow through joints, which is a function of void geometry of joints in the rock mass and the length of flow path, i.e., length between tunnel wall and any particular point in the rock joint inside the tunnel wall. Hence, the HI values presented in the results are specific for the length of joint between tunnel wall and BH1/BH4. The variation of HI along joint length with different rock–mechanical properties such as joint stiffness, friction angle and dilation, and effective stresses is studied in detail by [16] using numerical simulation.

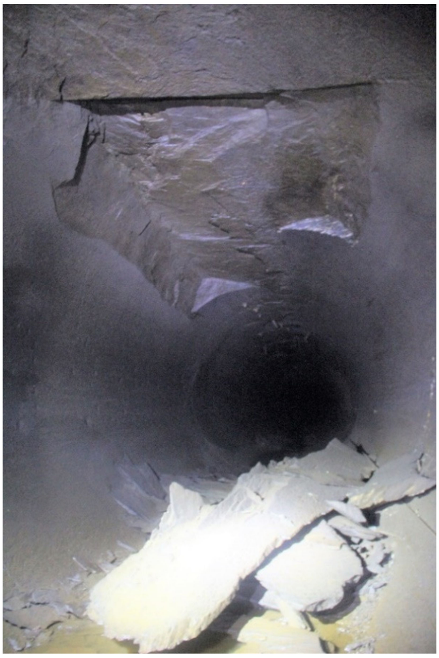

Figure 16. Example of block fall witnessed in Ulset headrace tunnel in 2017.

The analysis shows that shutdown duration is the most dominant parameter affecting hydraulic impact (HI) on rock joints. The results presented in this work are only from shutdown or reduction of load in the hydropower plant. Pressure transients also occur in the system when increasing load or opening the turbine valves, which also cause significant hydraulic impact. It can be inferred that similar to shutdown duration, the duration of opening has a significant impact since it also affects how the mass oscillation pressure develops during the transient.

A larger share of intermittent energy demands for increased flexibility in operation, which may mean that power plants need to change load faster, hence affecting the shutdown duration. However, it is uncertain to what extent the shutdown duration is affected owing to higher flexibility needs. As seen from the measurement and analysis, the current trend of shutdown duration as seen in Roskrepp is still contributing to accelerate the rock mass fatigue, which is explained below.

Based on the results, it would be logical that for larger load changes, both shutdown and opening should be carried out slowly to avoid stronger transients. However, it is dependent on the individual power plant operators due to lack of standard procedure for normal load-change operations. In Figure 13b, we can see that the shutdown duration is irrespective of head loss before the transient and three distinct clusters of shutdown durations are seen for similar head loss values. As seen in the results, faster shutdown causes significantly larger HI, which could be reduced by a slower shutdown. For example, for head loss of 3 m, the shutdown durations vary from 170 to 65 s, and the HI due to mass oscillation are 3–9 times higher (Figure 12). This could be avoided by having a slower normal shutdown. The idea is to carry out normal shutdowns/openings based on the magnitude of load changes such that larger load changes take longer shutdown/opening durations. These durations should be long enough for the rock mass pore pressure to closely follow the change in pressure in the tunnel during a transient. The optimum shutdown duration must be decided individually for each hydropower plant since the hydropower plants are unique in terms of parameters such as rock joint conditions in the tunnel contour, design head and discharge, length of the waterway, and number of brook intakes, contributing to different nature of the mass oscillations. As seen from the Roskrepp measurements, shutdown durations larger than 200 s seem to give the lowest possible impact with respect to shutdown from full load and is the recommended shutdown duration from full load for this power plant to reduce the hydraulic impact.

For hydropower plants to be constructed in the future or upgraded, a larger surge tank can be designed to reduce the pressure rise and also the time between start of shutdown and maximum mass oscillation amplitude, i.e., the shutdown duration becomes longer. In the case of pre-existing hydropower plants, slower operation of the units may be the most reasonable solution to reduce the hydraulic impact on rock mass around unlined pressure tunnels.

Slower shutdowns/openings cause lower hydraulic impact on the rock mass, which would help slow down the fatigue process. It is envisaged that such slow shutdowns/openings could be done in two different ways: (1) by standardizing a slow manual loading/de-loading of the units or (2) by using a slow preprogrammed and automated governor operation routine. Such measures may help to reduce the number of block falls and prolong the serviceable lifetime of unlined pressure tunnels and shafts.

Prediction of block falls due to transient events is a challenging issue because of the lack of a governing equation that defines the process of cyclic fatigue due to external (hydraulic) and internal (gravitational, friction and shear) forces that cause the failure of rock joints and intact rock bridges. Large variation in the hydromechanical properties of rock joints and in situ rock stresses add challenges in quantifying the hydraulic impact and the eventual fatigue. Further, the cumulative effect of HI is difficult to quantify in real cases since no monitoring systems are installed to record the pore pressure and long-term deformation in the rock mass. However, back analysis of particular block fall cases may be carried out using advanced numerical modelling with specific input parameters such as joint orientations, hydromechanical properties, in situ stress conditions, and pressure oscillations, which would help to gain more knowledge to address the aforementioned challenges.

5. Conclusions

The production data of some Norwegian hydropower plants shows that there is a large variation of start/stop events for each hydropower plant every year and also between different hydropower plants. The hydropower plants without operational restrictions have average annual start/stop events between 200 and 400 per unit, with a standard deviation up to 150. It is also seen that the number of start/stop events are in increasing order after the year 2009, and this increasing trend is significant for smaller hydropower plants. Further, the study of the magnitude of load changes in these hydropower plants suggests that there are higher numbers of smaller load changes (smaller than 25% of full capacity per hour) and smaller numbers of larger load changes (larger than 75% of full capacity per hour). More importantly, larger load changes are in higher proportion (more than 15% of total load-change events) for smaller power plants as compared to larger hydropower plants. This amounts to about 150–200 large load changes per turbine with installed capacity smaller than 50 MW. The production forecast for Roskrepp hydropower plant suggests that start/stop events and large load changes will increase by 30–45% between 2025 and 2040. From these observations, the authors conclude that dynamic operation of hydropower plants shows an increasing trend which can lead to larger destabilizing forces in the rock joints and accelerated fatigue of the rock mass in the future.

The monitored pressure transients and the pore pressure response in the rock mass during real-time operation at Roskrepp power plant have been used to develop a new method to quantify the effect of hydraulic transients on rock joints, referred to as the hydraulic impact (HI). The hydraulic impact is a destabilizing force that is regarded to be the main driver for instability, rock blocks fall, and potential tunnel collapses caused by hydraulic transients. The authors conclude that the duration of shutdown during a load reduction event is the most dominant parameter regarding the hydraulic impact, followed by the magnitude of load change. The faster the shutdown event, the higher the hydraulic impact, and it is more than 10 times higher when the shutdown duration is halved (i.e., from 130 to 65 s). The measurements show that tunnel dewatering has also caused significant increase in hydraulic impact, indicating irreversible changes in the joint void geometry and increase in the joint permeability, which can contribute to block falls over long-term operation.

It is observed that there is a large variation in shutdown duration, even for similar magnitude of load changes, ranging from 60 s to 200 s, because there is no standard procedure for shutdown duration and is entirely up to the operators to decide. Based on the results in this work, the authors recommend that durations of normal shutdowns/openings should be longer than current practice so that changes in pore pressure in the rock mass are more gradual. Normal shutdowns/openings should be carried out based on the magnitude of load changes such that larger load changes take longer shutdown/opening durations. Slower shutdowns/openings cause a slower pressure increase in tunnels and shafts and, thus, a lower hydraulic impact on the rock mass, which would help slow down the cyclic fatigue process.

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Paper 3

Evaluation on the effect of pressure transients on rock joints in unlined hydropower water tunnel using numerical simulation

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ORIGINAL PAPER

Evaluation on the Effect of Pressure Transients on Rock Joints in Unlined Hydropower Tunnels Using Numerical Simulation

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Abstract

Frequent pressure transients are identified as the cause of block failures in many unlined hydropower tunnels. The primary design objective of such tunnels is to prevent hydraulic jacking at design static pressure and mass oscillation but neglects the effect of short transients, i.e., water hammer. The issue has not been studied from the perspective of hydro-mechanical interactions due to frequent pore pressure changes in the rock mass. This article mainly focuses on the effect of pressure transients at different static heads, or different effective normal stresses across the joints and the effect of time period of pressure transient. Further, the change in such behaviour due to different mechanical properties of rock joints, such as stiffness, friction angle and dilation, is investigated. Numerical simulations of observed pore pressure response in the rock mass during a pressure transient are carried out using distinct element code 3DEC. The results show that relative joint deformation due to short pressure transients are the highest when joint normal stresses are 1.5–2.5 times higher than static water pressure in the tunnel and thus the vulnerability to weakening of such joints by hydraulic fatigue is higher. Further, results show that water hammers can travel up to 4 m into the rock mass even in stiff joint conditions and sufficiently high normal stresses. Results further indicate that the hydraulic impact due to water hammer is smaller as compared to mass oscillation. It is concluded that water hammers, wherever applicable along the waterway, can still contribute to hydraulic fatigue of rock joints in addition to the effect of mass oscillation and cannot be neglected when pressure transients occur frequently. Tunnel filling/dewatering and mass oscillations cause macroscopic joint displacements or block movements over long-term operation which is the major cause of block falls in unlined pressure tunnels.

Keywords Unlined hydropower tunnels · Pressure transients · Block falls · Numerical modelling

1 Introduction

Hydromechanical coupling in fractured media is a major field of scientific research in rock engineering and has applications in engineering projects that involve fluid flow in rock mass, such as reservoir engineering, nuclear waste disposal, inflow during underground excavations, design of dam foundations, etc. A previously unexplored application is in the field of design of unlined pressure tunnels functioning as waterways for hydropower projects, where there is direct contact between flowing water and rock joints exposed to tunnel wall. Any change in water pressure in tunnel causes the rock mass pore pressure to change and thus involves the process of hydro-mechanical coupling. It should be noted that the term “Pore pressure” in this manuscript solely refers to water pressure in the rock joints and the pressure in the rock stratum is not considered.

The state-of-the-art design principle of unlined pressure tunnels (Palmstrøm and Broch 2017; Panthi and Basnet 2018) considers static tunnel pressure as the primary design load and the major design objective is to prevent hydraulic jacking. Most of the tunnel length is placed in the rock mass where in situ minor principle stress is higher than the static tunnel water pressure. Steel or concrete lining is carried out in places where the minor principal stress is smaller. This is a well-tested design criterion with applications all over the world (Panthi 2014; Broch 2010; Marwa 2004) and especially in Norway, where more than 95% of hydropower pressure tunnels are unlined, with a history of over 100 years.

However, the operation regime of hydropower power plants in Norway has changed drastically over the years,

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especially since 1990s. The share of intermittent energy sources, such as solar and wind power, in the energy market is increasing every year (Charmasson et al. 2018). In EU member states, the share of renewable energy has almost doubled between 2004 and 2018 and wind and solar power share a large portion of this energy mix (Eurostat 2020). Hydropower plants must act as battery to supply energy when these intermittent sources are not producing. Thus, the operation of hydropower plants is becoming more dynamic, with load changes occurring on daily and even hourly basis. This induces large number of pressure transients, and frequent changes in rock mass pore pressure around the tunnel. It has been documented by Bråtveit et al. (2016) that block falls in tunnels have increased by a factor of 3–4 times as a result of dynamic operation. Such block falls are attributed to the cyclic fatigue of rock joints caused due to additional seepage forces acting on joint walls during pressure transients. Hence, in addition to hydraulic jacking, block falls during long-term operation of power plants are also becoming an important design issue which needs to be addressed.

The studies conducted so far regarding unlined pressure tunnel design and their failures (Brox 2019; Basnet and Panthi 2018; Rancourt 2010; Helwig 1987; Lang et al. 1976) point out the need for a detailed investigation in this topic. In Norway, there is now a renewed interest in understanding the consequences of hydromechanical effects regarding potential block falls in unlined pressure tunnels during long-term operation. The main reason being, majority of the 4300 km long waterway system in Norwegian hydropower plants consisting of unlined tunnels subjected to high static pressures maximum up to 1047 m.

A full-scale field instrumentation and monitoring is conducted in a headrace tunnel of a Norwegian hydropower plant to measure changes in rock mass pore pressure near tunnel walls during plant operation (Neupane et al. 2020). In this article, a brief summary of the instrumentation and some results relevant to the numerical simulation are presented. Numerical simulation of the observed pore pressure response in the rock mass during a pressure transient event is carried out using 3D distinct element code 3DEC. The model is then used to evaluate the impact at various static water heads and time periods of pressure oscillation in the tunnel, which are the key parameters for unlined pressure tunnel design. A parametric study has also been conducted to observe how the behaviour changes with some relevant rock mechanical parameters. Finally, implication of the observed results is linked to the actual block fall events.

2 Background

2.1 Physical Behaviour During Tunnel Filling and Pressure Transients

An unlined water tunnel is filled at a slow pace so that there are no sudden changes in pore pressure and effective stresses in the rock mass. When a tunnel is being filled, the effective stresses are reduced, which causes the rock joints to deform due to normal opening and shear dilation. When it is ensured that the minor principal stress is higher than the maximum tunnel water pressure, the displacements are negligible, and joint apertures do not change significantly, thus limiting the leakage to an acceptable range. When the tunnel pressure is steady (during steady plant operation), the displacements remain constant.

When turbine valves are closed to shut down or change the production, a pressure transient is induced in the tunnel. Pressure transient in a typical hydropower waterway consists of water hammer and mass oscillation. Water hammer is a pressure oscillation that occur in the conduit between the turbine and the surge shaft and have a short time period similar to sound waves in air or water. The water hammer is superimposed with the other, much slower but long-lasting oscillation (up to hours), i.e., mass oscillation, which gradually builds up when the water hammer has died down (Jaeger 1977). The time period of water hammer and mass oscillation for Roskrepp waterway are about 3.5 and 200 s, respectively. Mass oscillation can continue for hours after a load change event in the power plant, whereas water hammer dies down relatively faster.

During transient upsurge (increasing pressure), higher pressure is acting on the joint wall surfaces, further reducing the effective normal stress. A factor of safety is provided for static water head and for mass oscillation. Hence, the effective stress is still high enough to limit the joint deformations. On the other hand, during transient downsurge (decreasing pressure), higher pore pressure will be ¨trapped¨ in the rock joints due to pressure transmission delay between the tunnel and joint surfaces. This tends to push the rock blocks out of the tunnel wall. This phenomenon with higher rock mass pore pressure has been observed in the field experiment and is explained in Sect. 3 (also referenced to Neupane et al. 2020).

It is considered that pressure transients with a short time period, i.e. water hammer, does not require a factor of safety since the time of application is too short to cause hydraulic jacking (Benson 1989). Still, water hammer has a higher potential for pressure increase as compared to the mass oscillation, even though its zone of influence is lesser as compared to mass oscillation, due to smaller time period. Thus, it is believed that water hammer may also contribute to block falls, even though the magnitude is small as compared to mass oscillation. During first few years of operation, the joint stiffness in areas closer to the tunnel wall is usually reduced because the infilling material is washed away as a result of pore pressure changes. Yet, block falls in tunnels which are under operation for over 30 years, indicate that the joint properties continue to change as a result of prolonged cyclic loading and fatigue.

2.2 Theoretical Aspect

The mechanical and hydraulic processes that govern fluid flow in rock joints are interdependent with each other, are called hydromechanical couplings and are divided into two types, i.e., direct and indirect couplings. As described by Wang (2000), direct coupling includes two basic phenomena:

I. Solid-to-fluid coupling

   It occurs when a change in applied stress produces a change in fluid pressure or fluid mass. The applied stress produces displacement in the rock joints. This deformation generates surface stress on the fluid domain boundary, which deforms accordingly. A reduction in channel volume induces fluid outflow.

II. Fluid-to-solid coupling

It occurs when a change in fluid pressure or fluid mass produces a change in volume of the porous medium. A fluid inflow induces fluid pressure along the flow channels, which act on the channel boundaries and deforms the surrounding rock material. As a result of deformation, the rock counteracts the fluid pressure with surface stress at the fluid–rock boundary, which affects the fluid pressure and volume of fluid domain.

Mechanical and hydraulic processes can also affect each other through change in material properties, which are considered as indirect coupling. For example, the reduction in channel volume may increase contact area between the joint surfaces resulting a stiffer material. Indirect HM couplings tend to be most important in fractured rock mass or intact rock with flat inter-grain micropores, where changes in permeability caused by fracture or pore dilation can be dramatic (Rutqvist and Stephansson 2003). Indirect coupling composes of two basic phenomena: a solid-to-fluid coupling that occurs when an applied stress produces change in hydraulic properties; and a fluid-to-solid coupling that occurs when a change in fluid pressure produces a change in mechanical properties.

3 Field Experiment

3.1 Instrumentation Site

The field instrumentation is carried out in the unlined headrace tunnel of Roskrepp power plant in southern Norway, which has been in operation since 1980. The instrumentation setup is presented in Fig. 1a; readers are referred to Neupane et al. (2020) for details. The rock mass at this location consists of weakly schistose granitic gneiss with three major joint sets. The fracture network at this location consists of two sets of low permeability joints, i.e., foliation joints and joint set J1. A relatively flat dipping joint set J2 is exposed in the tunnel crown and has high permeability. One single joint ($Jf_{conductive}$) of the same orientation as the foliation joints also has relatively larger permeability (Fig. 1b). Among these joint sets, Jf and J1 have a spacing 1–2 m whereas $Jf_{conductive}$ has spacing of about 10 m and has persistence of more than 10 m. The intact rock is of crystalline nature and thus can be

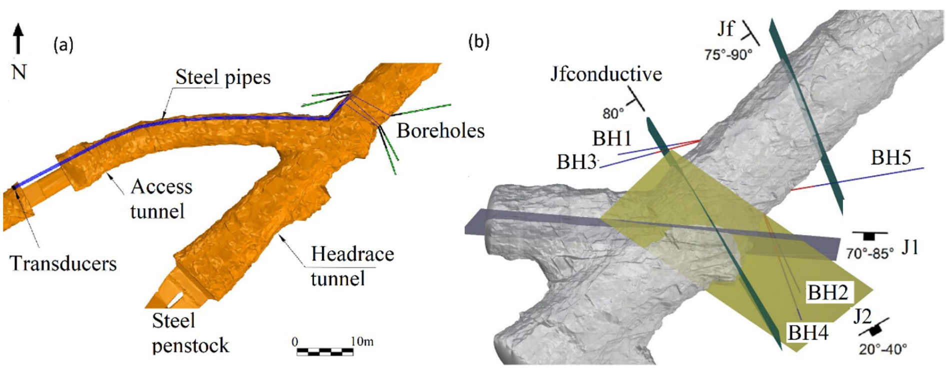

Fig. 1 **a** Layout of the instrumentation setup and **b** detailed view of instrumentation location showing orientation of boreholes with respect to major joint sets

considered impermeable such that the pore pressure in the rock mass is only governed by fluid flow through rock joints.

3.1.1 Instrumentation Setup

Five boreholes are drilled in the tunnel walls such that they intersect a particular joint set almost perpendicularly (Fig. 1b). Stainless steel pipes are fixed in the boreholes using packers at different lengths in the borehole and the pipes are taken out of the tunnel to a dry area in the access tunnel, where they are fitted with pressure sensors and data logger (Fig. 1a). The length of boreholes ahead of the packers (marked in blue in Fig. 1b) collects water from the rock joints and is connected to the pressure sensors through the steel pipes. Simultaneous readings of the tunnel water pressure are also taken from a pipe installed at the junction between the headrace tunnel and access tunnel.

3.1.2 Test Results

The pressure readings from boreholes BH1, BH2 and BH4 during a typical pressure transient event are shown in Fig. 2 along with the water pressure in the tunnel. It is seen that the boreholes which intersect the conductive joints, i.e., BH1 and BH4 strongly respond to pressure transients whereas BH2 is non-responsive, since it does not have direct hydraulic contact with the conductive joints. Even though BH3 intersects $Jf_{conductive}$, it does not have a direct hydraulic connection because the intersection point is behind the packer. (Fig. 1b).

The delayed pore pressure response or timelag between pressure peaks in the two responsive boreholes is of relevance regarding potential effect on rock block stability. The shaded area in Fig. 2b shows the condition when the rock mass pore pressure is higher than the tunnel pore pressure during a negative pressure wave. It is observed that such situation occurs for the first few cycles of the pressure transient and then gradually decreases as the transient attenuates and both pressures become almost equal or the tunnel pressure starts to become higher (Neupane et al. 2020). Therefore, the first few cycles are of interest regarding block displacement due to pressure transient and will be simulated in the numerical models.

It is observed that BH1 registers a stronger response during water hammer (Fig. 2b) but there is very little timelag during mass oscillation. For BH4, clear timelag is registered during both mass oscillation (Fig. 2a) and water hammer (Fig. 2b). But the amplitude of pore pressure in BH4 in response to the water hammer is smaller as compared to BH1. The difference in response is due to different resistance to flow through joints, which is a function of void geometry of joints and the length of flow path, i.e., joint length between tunnel wall and its intersection point with individual borehole.

4 Rock Mass Properties

4.1 Intact Rock Properties

Laboratory tests were carried out on rock samples of granitic gneiss collected during field mapping showed mean values of uniaxial compressive strength and modulus of elasticity of 148 MPa and 65 GPa, respectively. A summary of lab test results on intact rock samples and other estimated properties are presented in Table 1. The intact rock is modelled as an isotropic, homogeneous, linearly elastic material.

Normal and shear stiffness of joints are among the key parameters needed for numerical simulation. However, there is a great deal of uncertainty regarding these parameters since these are not easily measurable in the field and very limited amount of data are available. Zangerl et al. (2008) concludes that laboratory and in situ normal closure experiments in granitic rock showed a very

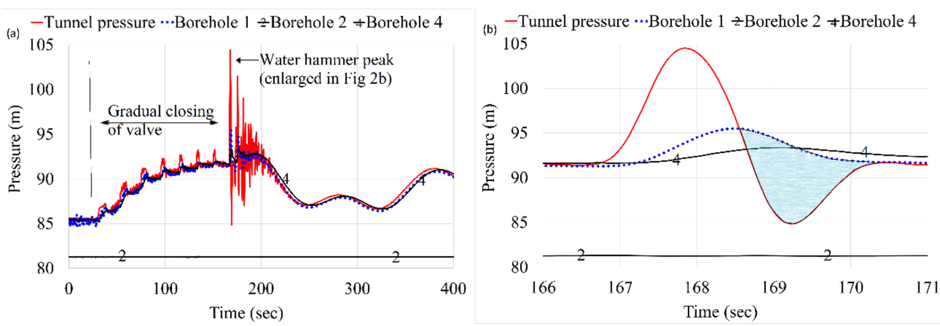

Fig. 2 **a** Pore pressure response of BH1, BH2 and BH4 during a typical pressure transient event in the headrace tunnel and **b** response during water hammer peak

Table 1 Summary of lab and estimated intact rock properties

Parameter	Min. value	Max. value	Remarks
Intact rock UCS (σ_i), MPa	137	156	Lab test results
Young's modulus (E_i), GPa	61	69	
Density (γ), kg/m^3	2741		
Poisson's ratio (ϑ)	0.27	0.31	
Basic friction angle ($\varnothing_b$), °	33.8	34.5	
Intact rock bulk modulus, (K_i), GPa	44	61	$K_i = \frac{E_i}{3(1-2\gamma)}$
Intact rock shear modulus, (G_i), GPa	24	26	$G_i = \frac{E_i}{2(1+\gamma)}$

large range of stiffness characteristic values, even for well-defined laboratory tests within the same rock type. This is because, fracture normal stiffness is highly affected by several extremely complex interacting factors, such as fracture surface geometry, asperity deformability, fracture interlocking, testing condition, etc., which are difficult to determine, if not entirely impossible. In addition, joint properties measured in the laboratory typically do not represent field joint conditions because of scale effects. For this analysis, the joint stiffness is calculated assuming that the jointed rock mass has same deformational response as an equivalent elastic continuum, for uniaxial loading of rock containing a single set of uniformly spaced joints oriented normal to the direction of loading (Barton 1972). To account for the uncertainty, a range of rock mass deformation modulus has been estimated using three different methods, i.e., Barton (2002), Hoek and Diederichs (2006) and Panthi (2006) and are presented in Table 2. Minimum and maximum values of rock mass deformation modulus have been calculated using each of these methods.

4.2 Joint Constitutive Model and Properties

An elasto-perfectly plastic Mohr–Coulomb model is chosen for simulating the fracture behavior. During simulation with this joint model, the fracture stiffness is kept constant, assuming it to be independent of normal stress, and friction and dilation angle are also kept constant. This is a reasonable assumption since the joint deformation during one pressure transient event can be very small. In this model, the fracture normal stiffness controls the normal deformation due to change in normal effective stress and shear stiffness controls the elastic shear behaviour of the fracture. The plastic shear behaviour is governed by fracture shear strength, which is a function of friction angle, cohesion and effective normal stress. The aperture changes as a result of joint displacement/dilation, which governs the flow rate and pore pressure along the joint. The maximum and minimum values of joint stiffness have been calculated based on rock mass modulus (Table 2), and presented in Table 3, along with other parameters used in the numerical simulation.

Table 2 Summary of mapped and estimated rock mass properties

Parameter	Min. value	Max. value	Remarks	
Q	10	14	Mapped	
Q_c	14	22	$Q_c = Q \times \frac{\sigma_i}{100}$	
GSI	60	70	Estimated	
D	0			
mi	28		Estimated	
	Panthi (2006)	Barton (2002)	Hoek and Diederichs (2006)	Remarks
Deformation modulus, GPa	19/24[a]	24/28[b]	32/51[c]	Min/max values
Rock mass bulk modulus[d], Km, (GPa)	14/21	18/25	23/44	
Rock mass shear modulus[e], Gm, (GPa)	8/9	10/11	13/19	

[a] $E_m = \frac{1}{60} E_i \times \sigma_i^{0.6}$
[b] $E_m = 10 \times Q_c^{\frac{1}{3}}$
[c] $E_m = E_i \left(0.02 + \frac{1-\frac{D}{2}}{1+e^{\frac{60+15D-GSI}{11}}} \right)$
[d] $K_m = \frac{E_m}{3(1-2\gamma)}$
[e] $G_m = \frac{E_m}{2(1+\gamma)}$

Table 3 Estimated joint properties for $Jf_{conductive}$

	Min. value	Max. value	
Normal stiffness (K_n) (GPa/m)	4	25	$K_n = \frac{E_i \times E_m}{S(E_i - E_m)}$
Shear stiffness (K_s) (GPa/m)	1.5	9	$K_s = \frac{G_i \times G_m}{S(G_i - G_m)}$
Average joint spacing (S), m	8	8	
Friction angle ($\emptyset$), °	25	40	
Dilation angle, °	0	5	
Cohesion (c), MPa		0	
Initial hydraulic aperture (μm)		5	
Residual hydraulic aperture (μm)		1	

Table 4 Estimated in-situ stress at instrumentation location

Description	Magnitude		Direction
	Min	Max	
Max. tectonic stress (σ_{tmax}) MPa	9.1	20.1	E–W
Min. tectonic stress, (σ_{tmin}) MPa	2.1	2.1	N–S
Rock cover, m	113	113	
Vertical stress, MPa ($\sigma_v = \sigma_2$)	3.1	3.1	
Max. horizontal stress ($\sigma_{hmax} = \sigma_1$) MPa	10.3	21.3	E–W
Min. horizontal stress ($\sigma_{hmin} = \sigma_3$) MPa	3.3	3.3	N–S

4.3 Joint Hydraulic Properties

Water inflow into the tunnel was noted until a month after tunnel dewatering through the joints $Jf_{conductive}$ and J2, which shows that these joints are highly permeable and act as direct hydraulic connection between natural ground water table and the tunnel. Other joints are tight and do not indicate any seepage through them, indicating low permeability. This is also established by the fact that BH1 (intersecting $Jf_{conductive}$) responds quickly to the pressure transients. Further, outflow was measured at the pipe outlet of all boreholes to have an idea of the relative difference in permeability between the joint sets. The de-aeration valve connected at the pipe outlets was opened and flow was measured for about 5 min when the tunnel registered a static water pressure of 9.45 bars. The steady flow rate observed during such event gives the flow occurring through the length of joints between the tunnel and the boreholes at a known tunnel water pressure. The outflow from BH1 was measured to be 0.0012 L/s. The outflow from BH2, BH3 and BH5 was negligible and was difficult to be measured with relative accuracy. This shows that the differences in permeability between the conductive $Jf_{conductive}$ and other joints are in several orders of magnitude.

4.4 In Situ Rock Stresses

An estimate of the in-situ rock stress at the instrumentation location is made based on 3D stress measurement carried out at Holen power plant in 1980 (Myrvang 2019), located about 30 km from Roskrepp power plant. Both these locations are in the same geological and geo-tectonic setting consisting of massive Precambrian gneisses. Hence, the tectonic stresses are assumed to be similar in these locations. At Holen, the maximum principal stress is approximately horizontal and oriented towards east–west direction and magnitude vary between 12 and 23 MPa. The minimum principal stress is approximately horizontal also and oriented towards north–south direction and has magnitude of 5 MPa. Intermediate principal stress is approximately vertical and has magnitude of 7.5 MPa in accordance with the gravity induced vertical stress (σ_v). Since major (σ_1) and minor (σ_3) principal stresses are horizontal, they are equal to maximum (σ_{hmax}) and minimum (σ_{hmin}) horizontal stresses at this location. Hence, the maximum (σ_{tmax}) and minimum (σ_{tmin}) tectonic stresses are calculated using the following equation (Panthi 2012):

$$\sigma_{horizontal} = \sigma_{tectonic} + \frac{\vartheta}{1 - \vartheta}\sigma_v. \qquad (1)$$

These tectonic stress values are then used to calculate maximum and minimum total horizontal stresses at the instrumentation location using Eq. (1) and are presented in Table 4.

5 Numerical Simulation

The numerical simulation is carried out using three-dimensional distinct element code 3DEC (Itasca 2018). 3DEC can calculate fluid flow and effect of fluid pressures on rock/soil, based on specified material properties and fluid/mechanical boundary conditions using coupled hydro-mechanical calculations through a network of fractures between deformable rock blocks. The flow rate through contacts is calculated by assuming interface as two parallel plates with a defined aperture width, where flow is laminar and is governed by the modified form of cubic law (Witherspoon et al. 1980) given by the following expression:

$$q = \frac{(b_{hi} + f\Delta U_n)^3 w\rho g}{12\mu}\frac{\Delta p}{l}, \qquad (2)$$

where q is the flow rate, b_{hi} is the initial hydraulic aperture at initial effective stress, f is a factor reflecting the influence of roughness on the flow tortuosity, ΔU_n is the change in fracture normal displacement, w is the fracture width, ρ is the fluid density, μ is the fluid dynamic viscosity, Δp is the pressure difference; and l is the length of joint. Fracture deformation is calculated as a function of effective normal stress assuming a constant fracture stiffness. The domain pressure is then updated, considering the net flow into the domain and possible changes in domain volume due to incremental motion of surrounding blocks. The new domain pressure is computed using following equation:

$$p = p_0 + k_w q \frac{\Delta t}{V} - k_w \frac{\Delta V}{V}, \tag{3}$$

where p_0 is the initial pressure, q is the flow rate defined from Eq. (2), k_w is the bulk modulus of the fluid, V is the initial fracture volume and ΔV is the change of fracture volume due to deformation. As a result of new domain fluid pressure, the forces exerted by the fluid on the edges of surrounding blocks are obtained. New mechanical calculations are carried out to update the geometry and the process is continued until equilibrium is reached.

5.1 Model Geometry and Boundary Conditions

The model geometry consists of a rock mass block of $60 \times 60 \times 60$ m with an inverted D-shaped tunnel section with height and width of 8 m. The rock mass block consists of a single fracture resembling $Jf_{conductive}$. This model size was found to be sufficient to prevent boundary effects since the radius of influence around tunnel periphery regarding pore pressure changes during tunnel infilling is smaller than the joint area formed by this model size. It is also seen that the radius of influence during a pressure transient is even smaller as compared to the radius affected during tunnel infilling. Figure 3 shows the model block and the joint plane showing pore pressure buildup and joint displacements during tunnel filling.

The model consists of a graded mesh with the finest discretization of 1 m zone size around an area equal to one tunnel diameter in all directions from the tunnel boundary and direction perpendicular to the rock joint. Rest of the model is discretized with 2 and 4 m zone size with the coarsest discretization along outer boundaries.

Fixed boundaries are applied on all faces of the rock block. The initial stresses are set as depth-dependent with the minimum in situ stress values from Table 4 acting at the centre of the model. The boundary pore pressure is set in all faces according to the hydrostatic pressure gradient corresponding to water pressure of 78 m (0.76 MPa) at the tunnel centre, with a linearly varying pressure gradient. The model allows outflow from the boundary. The same magnitude of initial pore pressure is applied on the joint with a linearly varying pressure gradient. This value is estimated based on the elevation of water surface in small lakes above the instrumentation location, which are hydraulically connected to the tunnel through conductive joints.

5.2 Calculation Sequence

The calculation sequence for each simulation consists of the following steps:

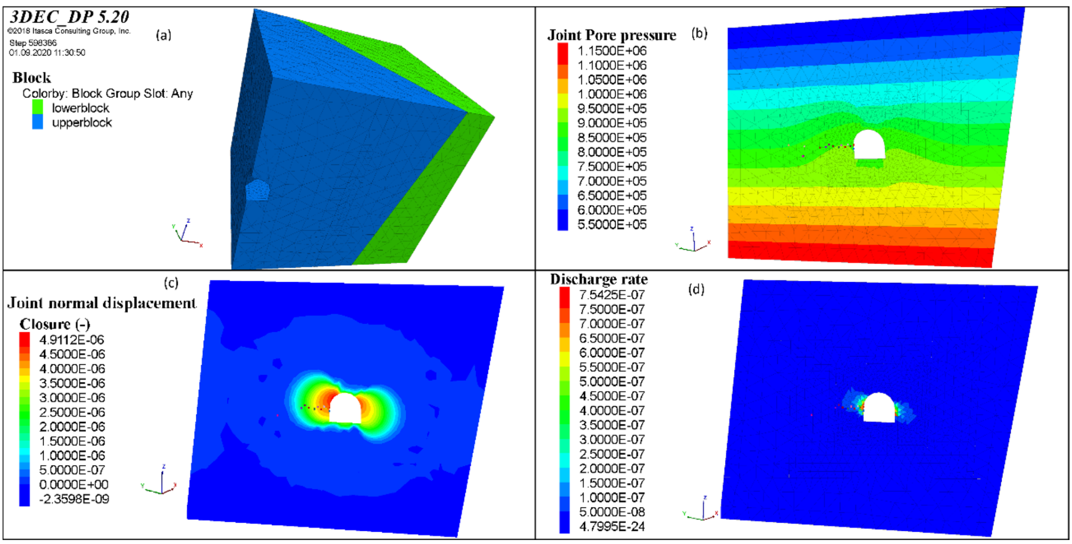

Fig. 3 **a** Model geometry ($60 \times 60 \times 60$ m) and **b–d** fracture plane with pore pressure, normal displacement and discharge rate contours during tunnel filling

1. Solving for initial state to achieve mechanical equilibrium of the rock block.
2. Mechanical equilibrium after tunnel excavation.
3. Flow only calculations to achieve steady state fluid pressures.
4. Application of water pressure in the tunnel and run to steady state (hydromechanical coupling) to simulate tunnel filling.
5. Time-dependent pressure variation in the tunnel boundary (hydromechanical coupling) to simulate pressure transient (Fig. 4).

The comparison between results of various simulations is done using three output parameters, i.e., maximum normal and shear displacements, and the area bounded between the pressure curves of the tunnel and the rock mass, when rock mass pore pressure is higher than tunnel pressure (shaded area in Fig. 2b). This area is the result of transmission delay between two pressure peaks, during the transient event which is the additional seepage force acting on the joint surfaces during pressure transients. This term hereafter named as ¨hydraulic impact¨ has a unit of pa s and is similar to dynamic viscosity or the force acting on the joint surface per unit area over a certain time.

5.3 Fluid Bulk Modulus

The bulk modulus of water at 20 °C is 2.2 GPa, which if used in the calculations, results in very small time steps, making the calculations time-consuming. To mitigate this problem, Itasca (2018) suggests that fluid bulk modulus be reduced such that the apparent stiffness of fluid-filled joint (fluid bulk modulus divided by hydraulic aperture) is approximately equal to the equivalent stiffness of the adjacent zone in the block representing the rock. The average length of zone in the model is 1 m, which gives equivalent stiffness of the zone adjacent to the joint to be in the range of 8.4–16.7 GPa. The apparent stiffness of joint, on the other hand, is at least three orders of magnitude higher than these values. Comparative simulations were run using bulk modulus values of 0.02 and 0.2 GPa, which resulted time steps of 9.5e−4 and 9.5e−5 s, respectively. The difference in calculated timelag, maximum pore pressure magnitude, maximum normal and shear displacements between these zone sizes was less than 10% whereas the runtime was almost 10 times higher for 0.2 GPa. Ivars (2006) mentions that there was no noticeable effect on the equilibrium results when conducting simulations with reduced fluid bulk modulus and that it only exhibits transient differences. Hence, fluid bulk modulus of 0.02 GPa was deemed to be sufficient and has been used for all calculations.

6 Results

6.1 Validation of BH1 Pore Pressure

The conductive joint $Jf_{conductive}$, was modelled for the largest water hammer peak (Fig. 2b) that occurred after complete closure of the valve. The water hammer peak has a cycle time of about 3.5 s, which travels through joints into the borehole, which registers a pore pressure response with a timelag of 0.75 s between the pressure peaks.

In the simulation, the total normal stress acting across the joint before tunnel filling is 6.25 MPa and effective normal stress after tunnel filling is 5.25 MPa. The result shows that exact simulation of both pore pressure magnitude and timelag was not possible with a single set of parameters. Hence, two different values of joint normal stiffness were used to simulate these two parameters separately (Fig. 5). It

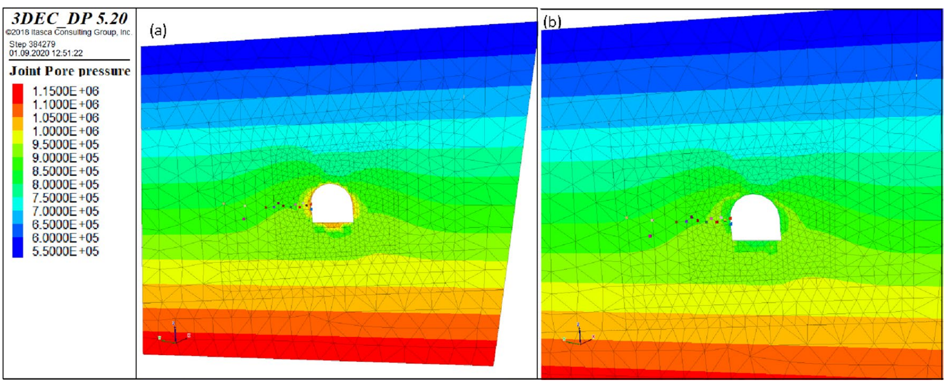

Fig. 4 Joint pore pressure contours **a** when water hammer is at peak in the tunnel, **b** when water hammer is at negative peak

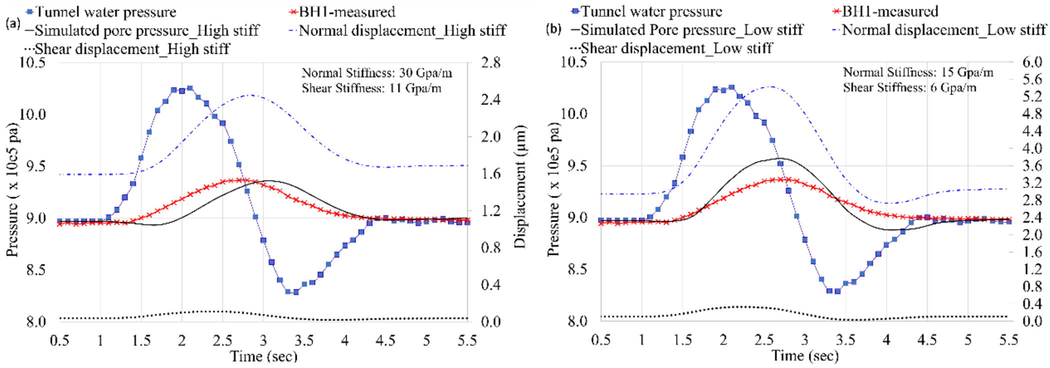

Fig. 5 Simulation results for BH1 during pressure pulse for **a** high normal/shear stiffness and **b** low normal/shear stiffness

was found that low normal stiffness resulted a better simulation of the timelag but the pressure magnitude was overestimated. Higher normal stiffness resulted a better match for pore pressure magnitude but increased the timelag. Cappa et al. (2008) reports similar pore pressure behavior as a result of change in aperture during HM coupling when modelling pressure pulse in rock fractures. This is because low normal stiffness results in larger hydraulic aperture during tunnel filling and thus the response is faster during a pressure transient and vice versa. At steady state during filling, the maximum hydraulic aperture around the tunnel contour is 99 µm for high normal stiffness and 170 µm for low normal stiffness cases, respectively. The joint displacements can also be seen in Fig. 5.

Since timelag and pore pressure magnitudes were separately simulated, the hydraulic impacts for these two simulations are also different. At joint length of 2.3 m, the measured hydraulic impact is 8.1e4 pa s while the simulated hydraulic impact for low and high normal stiffness cases are 8.9e4 and 7.8e4 pa s, respectively. This shows that the simulation with high stiffness is a better simulation in terms of total hydraulic impact per cycle.

6.2 Hydraulic Impact Along Joint Length

The effect of pressure pulse at different lengths along the joint was also analyzed with the same model (Fig. 6). During the pressure pulse, pressure magnitude is higher at locations near the tunnel wall with the smallest timelag. As we go deeper, the pressure magnitude decreases as a result of frictional loss due to increased resistance to flow. The timelag increases as the pressure pulse travels deeper into the rock mass. Since the hydraulic impact is a combination of these two parameters, it is not the highest closest to the tunnel wall since the timelag is too small. On the other hand, deeper into the rock mass, it is again small because the pressure

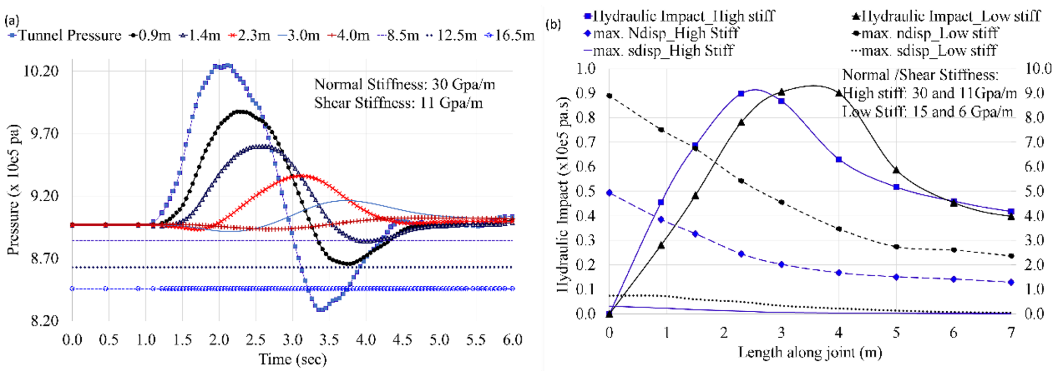

Fig. 6 **a** Effect of pressure pulse at different lengths of $Jf_{conductive}$, and **b** hydraulic impact and maximum joint displacements during the pulse along joint length

magnitude is too small even though the timelag is larger. In this case, the highest hydraulic impact is between 2 to 4 m from the tunnel wall, along the joint length.

It is also seen that, the maximum hydraulic impact for both stiffness values is almost equal but the curve for high stiffness joint is much steeper and the zone of influence is much smaller as compared to the less stiff joint. This means that the pressure pulse will impact a larger area of the rock mass when the joint has less stiffness. This seems logical since for a smaller stiffness, larger hydraulic aperture is formed at the end of tunnel filling. Hence, the flow is larger during a transient, and the pressure can travel deeper into the rock mass and thus the total impact will be distributed over a deeper area as compared to the stiffer joint.

A detailed analysis of such impact is carried out in the following sections. The area of interest with respect to block stability is the joint length of less than one tunnel diameter from the tunnel wall, which is the most vulnerable area in terms of block stability and also the starting point for a larger cave-in as a result of consecutive block failures over a period of power plant operation. To gain a better understanding of the behaviour under varying conditions of power plant operation and joint properties, several similar simulations are carried out by varying the parameters which are of specific interest, particularly the tunnel static head and time period of pressure wave oscillation.

6.3 Effect of Varying Static Tunnel Water Pressure

More than 200 unlined pressure shafts and tunnels with a combined length of over 4300 km are currently in operation in Norwegian hydropower plants with a maximum static head of up to 1047 m. Palmstrøm and Broch (2017) presents a graph which shows that more than 95% of these tunnels/shafts in Norway have static heads below 600 m. In this analysis, simulations are carried out to see the effect of increasing static head up to 600 m. The joint properties are the same as for high stiffness case for the model used for validation in Sect. 6.1, and the normal and shear joint stiffness values of 30 and 11 GPa/m, respectively, are referred to as stiffness ST1 hereafter. The time period of pressure pulse is kept constant at 3.5 s and the peak magnitude is increased linearly, relative to the static head increase (14.5% of the static head) in all simulations. The normal stress acting across the joint before tunnel filling is 6.25 MPa and effective normal stress reduces during tunnel filling according to the static head for each case. In terms of conventional design approach, the Factor of Safety (FoS) of design is the ratio between the minimum principal in situ stress and the static water pressure in the tunnel. Hence, the FoS gradually decreases as the design static water pressure increases.

It is important to study the changes in joint aperture during tunnel filling to assess the impact of pressure transients. This is because, the final hydraulic aperture after tunnel filling governs the flow, pore pressure and the transmission delay (timelag) during a transient. Hydro-mechanical behaviour during tunnel filling and pressure transients affects rock mass around the tunnel in different ways. Tunnel filling event impacts a larger area of rock mass around the tunnel and even affects the local groundwater level in the vicinity (Vigl and Gerstner 2010). A very small but steady outflow is maintained through the joint network at the end of filling up to a pre-designed static head. However, transients have short period of application and thus the effect travels only within shallow depths as compared to filling and causes joint deformations in a localized area. Hence, the results are explained by differentiating the effect of these two phases of tunnel operation.

Figure 7a, b and Table 5 present normal and shear displacements occurred during tunnel filling for various static heads and maximum normal and shear displacements during respective transient events. The solid lines in Fig. 7a, b show joint normal and shear displacements at different static heads along the length of the joint. During filling, it is observed that, both normal and shear displacements are relatively small for up to 400 m head but then increase significantly for higher heads. As per Benson (1989), the required FoS for static (steady) condition is 1.3 and no FoS is required for water hammer. The FoS for steady state at 400 m head is 1.6 and then reduces to 1.27 and 1.06 for 500 and 600 m, respectively. Hence, according to current design practice, 500 m case is at the boundary of allowable limit whereas 600 m case is an unsafe design. Therefore, the observed joint deformations are as expected since there is large joint shear and dilation when the FoS is equal to higher than allowable limit (Table 5).

During respective transient events, the joint deformations seem to be gradually increasing with static head. However, it is seen that the percentage increase in joint deformations at different static heads Fig. 7c, d is non-linear. It is seen that, they are the highest at 300 m static head, when the FoS is close to 2. The change in normal deformation is the lowest at 500 m, (FoS 1.3) and then again begins to increase at 600 m static head, which indicates hydraulic jacking of the joint since the factor of safety is less than 1 for transient in this case. This jacking, however, occurs for a very short time since the time of application of this pressure transient is small and hence does not travel deeper into the rock mass.

The reason for this behavior where the impact of pressure transients is higher in the mid ranges of static head is intuitively linked to the joint aperture at the end of tunnel filling. At smaller static heads, the aperture after tunnel filling is still small, causing high flow resistance during a transient event, which then causes joint deformation as a result of hydraulic–mechanical interaction. For larger static heads, the joint aperture after tunnel filling is larger, which

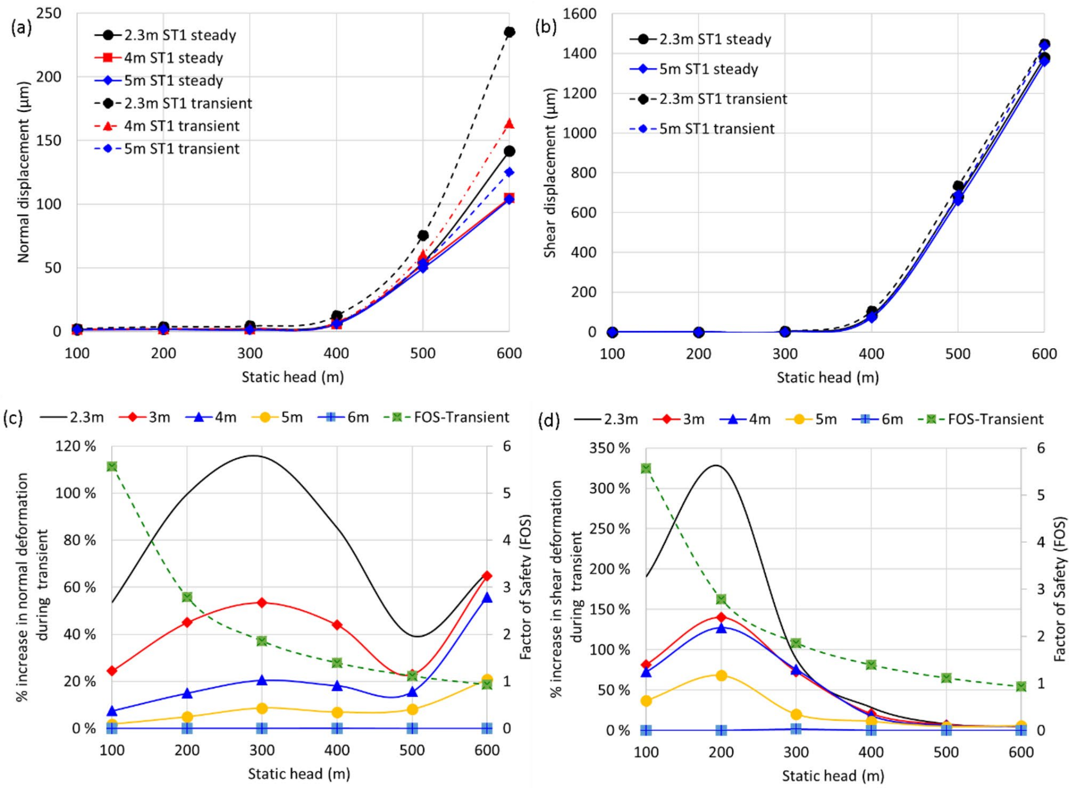

Fig. 7 **a** Normal displacement and **b** shear displacement as result of increasing static head and percentage increase in **c** normal displacement and **d** shear displacement during respective transient events for high stiffness case (ST1)

Table 5 Simulation results for high stiffness case with varying static heads at 2.3 m joint length

Static head (m)	Eff. normal stress (MPa)	Factor safety (FoS)		Joint normal deformation (µm)			Joint shear deformation (µm)		
		Steady state	Transient	Steady state	Transient	% change	Steady state	Transient	% change
100	5.27	6.37	5.57	1.59	2.45	54	0.04	0.11	191
200	4.29	3.19	2.78	1.99	3.97	100	0.05	0.22	327
300	3.31	2.12	1.86	2.09	4.51	115	1.69	3.17	88
400	2.33	1.59	1.39	6.75	12.50	85	82.57	105.94	28
500	1.35	1.27	1.11	54.26	75.62	39	682.12	735.43	8
600	0.37	1.06	0.93	141.8	235.5	66	1380.20	1447.81	5

allows a relatively higher flow out through the joint and thus causing less seepage forces acting on the joint surface and hence smaller joint deformation during transient. Further, it is observed that the effect of pressure transients reduces as we go deeper into the rock mass but there were no joint deformations at areas deeper than 6 m.

Hydraulic impact along the joint during pressure transients at varying static heads is presented in Fig. 8. It is seen that the hydraulic impact increases almost linearly until 400 m along the whole joint length. The highest impact is between 2 to 4 m and the curve is flat at locations deeper than 5 m because the transient does not travel beyond this point. At 500 m, the increase in hydraulic impact is smaller at locations close to the tunnel wall. This is because, large joint aperture after filling causes less flow resistance, leading to smaller timelag as compared to

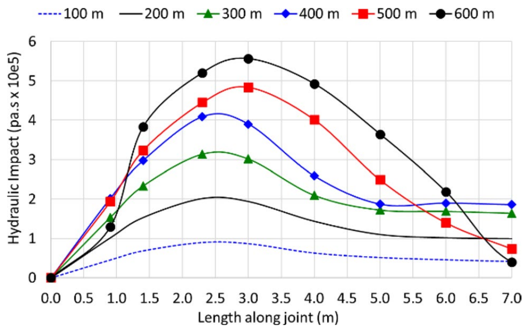

Fig. 8 Hydraulic impact due to pressure transient at various static heads for high stiffness (ST1)

smaller static heads. At locations deeper than 5 m, there is a sharper drop in hydraulic impact, again because of less flow resistance, causing smaller pressure buildup during filling phase itself. Similar behaviour is seen in 600 m case but is more pronounced due to even higher joint aperture.

Joint deformation magnitudes during pressure transients are very small when the FoS is at a safe limit (Table 5) and are thought to be insignificant in relation to tunnel stability as per current design practices. However, these observations may have some implications in relation to design consideration regarding long-term operation scenario, when considering the effect of pressure transients, which will be presented in the discussion section.

6.4 Effect of Joint Stiffness

Similar simulations as in the preceding sections were carried out using the lower range of stiffness values presented in Table 3 and the normal and shear joint stiffness values of 4 and 1.5 GPa/m, respectively, are referred to as stiffness ST2 hereafter. The results are presented in Fig. 9 and Table 6 and show similar behaviour but with different magnitudes. For static heads up to 400 m, the normal

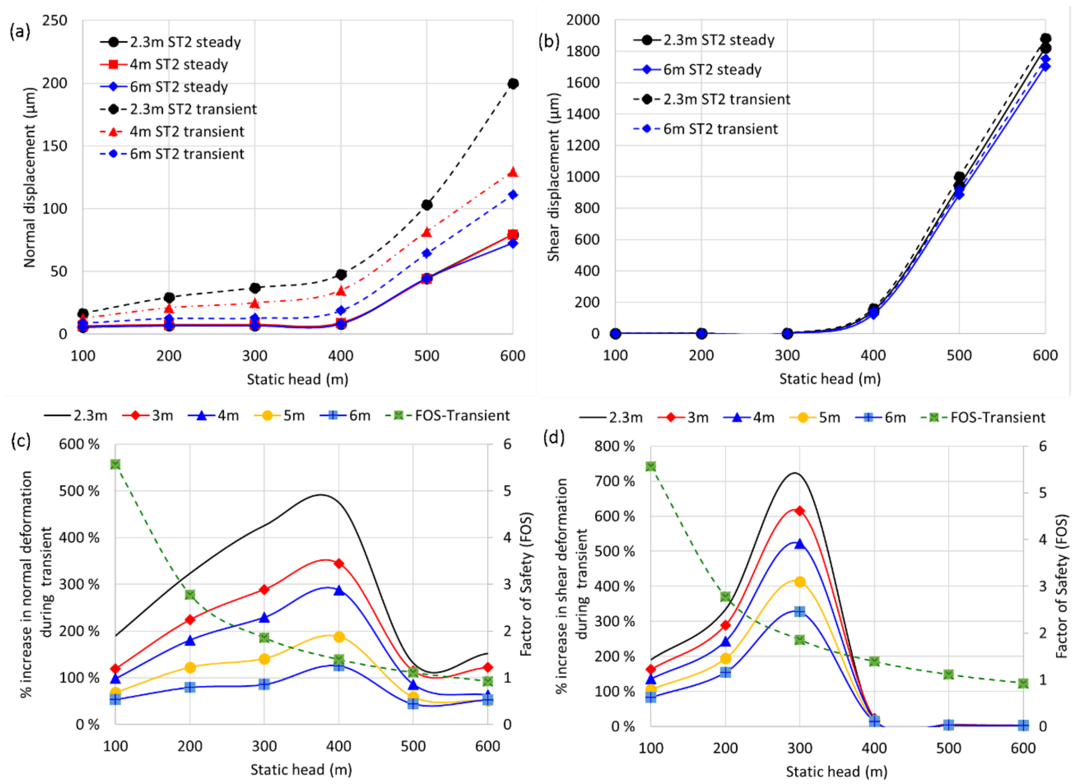

Fig. 9 **a** Normal displacement and **b** shear displacement as result of increasing static head and percentage increase in **c** normal displacement and **d** shear displacement during respective transient events for low stiffness case (ST2)

Table 6 Simulation results for low stiffness case with varying static heads at 2.3 m joint length

Static head (m)	Eff. normal stress (MPa)	Factor safety (FoS)		Joint normal deformation (μm)			Joint shear deformation (μm)		
		Steady state	Transient	Steady state	Transient	% change	Steady state	Transient	% change
100	5.27	6.37	5.57	5.70	16.52	190	0.55	1.59	191
200	4.29	3.19	2.78	6.90	29.23	324	0.70	3.02	334
300	3.31	2.12	1.86	7.00	36.82	426	0.49	4.00	717
400	2.33	1.59	1.39	8.27	47.55	475	146.10	162.19	11
500	1.35	1.27	1.11	44.6	89.14	100	943.71	1000.75	6
600	0.37	1.06	0.93	79.4	133.92	69	1822.68	1881.48	3

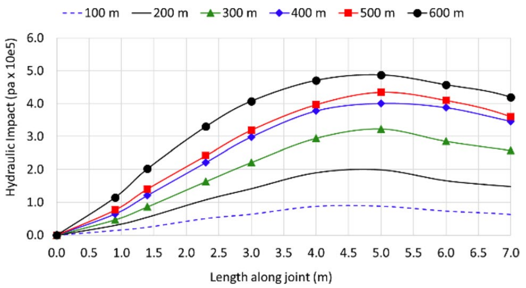

Fig. 10 Hydraulic impact due to pressure transient at various static heads for low stiffness (ST2)

deformations during filling are higher than the high stiffness case, which is logical (Tables 5, 6). However, for 500 and 600 m, it is seen that the deformations are smaller than the high stiffness case. This seems like an anomaly but can be explained by the fact that in low stiffness case, the flow can travel deeper into the rock mass, causing higher flow out of the system. It is seen for 500 m, the maximum flow out of the tunnel for low stiffness case is almost ten times higher than the high stiffness case (0.005 and 0.04 L per second for ST1 and ST2, respectively). Such higher flow out of the system causes pressure release during filling, thus causing smaller deformation as compared to high stiffness case.

Similar to high stiffness case, the percentage change in deformation due to transient is higher in the mid ranges within safe range of FoS but the magnitude of such change is higher because of low joint stiffness. Figure 10 shows hydraulic impact along the joint at different static heads for low stiffness case. It is seen that the pressure transient travels deeper into the rock mass than the high stiffness case and thus the hydraulic impact is more evenly distributed over a larger area. The peak hydraulic impact is at a deeper location as compared to the high stiffness case. Up to 400 m static head, the maximum value of hydraulic impact is almost equal for both cases. But for 500 and 600 m cases, this value is slightly smaller. This is because of the high joint aperture after filling which allows, more outflow in these static heads and lesser pressure buildup during transients.

Three major conclusions can be drawn from the simulations conducted at varying static heads with different stiffness values.

1. The change in joint normal displacement during transient is higher in the mid ranges of static head within acceptable FoS. Hence, pressure transients affect the rock joints more when the FoS within 1.5–2.5.
2. For low stiffness cases, the transient travels deeper into the rock mass and thus affects a larger area of the rock mass as compared to a high stiffness case. The magnitude of peak hydraulic impact is almost the same as high stiffness case but is distributed over a large area. For high stiffness case, the effect of pressure transient is confined to a smaller area.
3. Results indicate that hydraulic impact caused due to short pressure transients are between 2 to 6 m depth. This area seems of greater significance in relation to contribution of fast transients to potential block falls. However, the depth of impact is greatly affected by the time of application of pressure transient. A larger area of rock mass will be affected by a slower pressure pulse; i.e. mass oscillation. The next section presents results of simulations with various time periods of oscillation.

6.5 Effect of Time Period of Pressure Transient

Simulations were run for four different time periods; i.e. 3.5, 60, 120 and 200 s at a static head of 100 m for both stiffness cases, to analyze the effect of time period of oscillation. The peak amplitude of the pressure pulse is kept constant at 114.5% of the static head for all time periods. The main objective of these simulations is to observe the changes in zone of influence, joint displacements and hydraulic impact with different time periods. The results are presented in Figs. 11 and 12.

In Fig. 11, it is seen that increase in time period from seconds to minutes causes a significant increase in both normal and shear displacements. The effect of time period increase

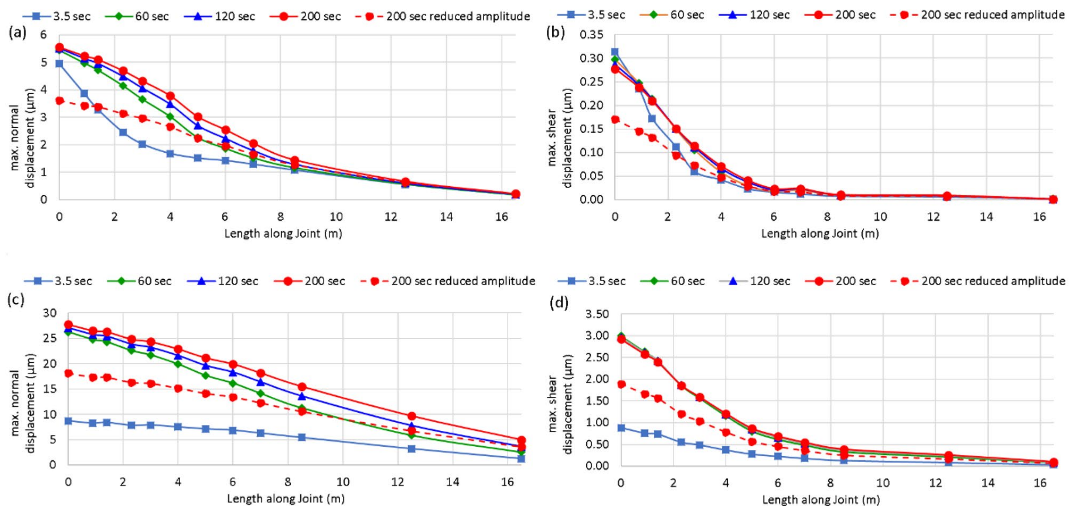

Fig. 11 **a** Normal displacement and **b** shear displacement for high stiffness case and **c** normal displacement and **d** shear displacement for low stiffness case during transients with different time periods

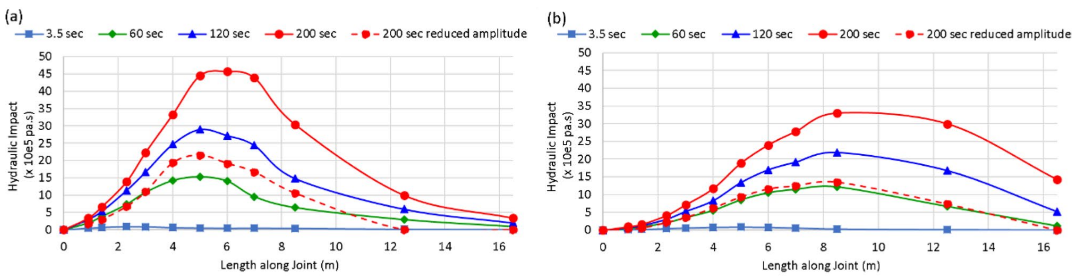

Fig. 12 Hydraulic impact along the joint due to pressure transients with different time periods for **a** high stiffness (ST1) and **b** low stiffness (ST2)

from 60 to 200 s, however, is relatively lesser as compared to the same between 3.5 to 60 s. For high stiffness case, the impact is concentrated within 8 m depth and maximum displacement is seen between 2 to 4 m depth. There is no significant effect in rock mass deeper than 8 m. For low stiffness case, similar behavior is seen regarding the effect of time period increase from seconds to minutes. However, the impact is more uniformly distributed though out the joint length and the effect also travels deeper as compared to high stiffness case.

In Fig. 12, it is seen that the zone of influence increases significantly with increasing time period since the transient can travel deep into the rock mass. It is also seen that the peak hydraulic impacts due to time period of 60 s and higher are significantly higher than the peak hydraulic impact due to time period of 3.5 s. The peak hydraulic impact is comparatively smaller in case of low stiffness case, because of higher flow out of the system (and pressure release) due to larger hydraulic aperture and deeper zone of influence.

It should be noted that in these simulations, the pressure amplitudes for all simulations are kept constant by varying the time period only. However, water hammer has a higher potential for pressure increase as compared to the mass oscillations. This can be seen in Fig. 2a where the pressure rise due to mass oscillation is 7% of the static head before shutdown and the pressure rise due to water hammer is 14.5%. Hence, simulation is carried out with a pressure amplitude increase of 7% during transient and time period 200 s to compare the actual difference in hydraulic impact between a water hammer and mass oscillation for this particular waterway system. The results are indicated by dotted lines in Figs. 11 and 12. It is seen that the displacements are still much larger than 3.5 s case, but the hydraulic impact is reduced by almost half in case of both stiffness cases. Hence, the results show that the effect of mass oscillations is much higher than that of water hammer, when compared in terms of a single pressure pulse. However, the cumulative impact is more relevant in terms of block stability over long-term operation, which will be presented in the discussion section. Three major conclusions can be drawn from the above results, which are as follows:

1. The effect in terms of both joint displacements and hydraulic impact increases significantly when the time period increases from seconds to minutes (between a typical water hammer and a mass oscillation event). Joint displacement does not increase significantly when the time period is increased further. The hydraulic impact, however, keeps on increasing when the time period is increased. This is because, the pressure is ¨trapped¨ in the joint for a longer period, even though the aperture remains almost same.
2. In high stiffness case, the joint displacement is smaller but concentrated within shallow depths from the tunnel walls. For low stiffness, the displacements are much larger, and the effect is dispersed more uniformly over a deeper area in the rock mass.
3. The hydraulic impact is smaller in low stiffness case, but the zone of influence is larger.

6.6 Effect of Joint Friction Angle

Results of simulations with joint friction angles of 40° and 25° are presented in Figs. 13 and 14. It is seen that joint friction angle has little or no impact in both joint displacement and hydraulic impact when the normal stress across joint is high. With reduced normal stress, the joint displacements during transients increase significantly. Similar results as in previous simulations are seen along the length of joint because of change in joint stiffness. However, at 300 m static head, the joint deformations are significantly high when the friction angle is reduced, and the effect is seen deep into the rock mass for both stiffness cases.

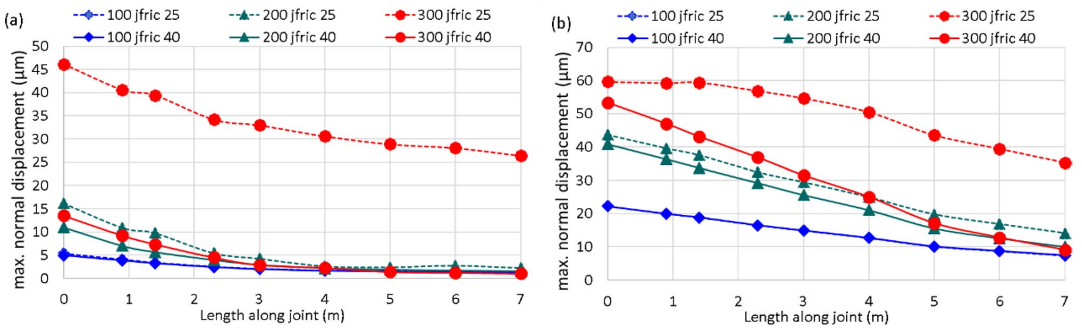

Fig. 13 Normal displacement along the joint due to pressure transients with different joint friction angle for **a** high stiffness (ST1) and **b** low stiffness (ST2)

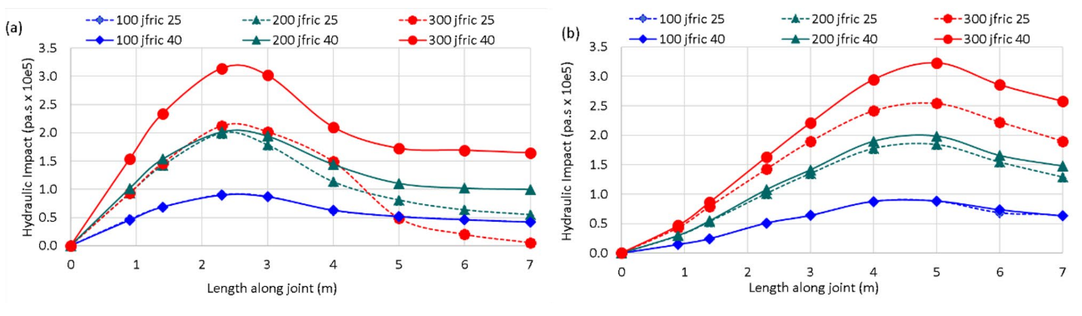

Fig. 14 Hydraulic impact along the joint due to pressure transients with different joint friction angle for **a** high stiffness (ST1) and **b** low stiffness (ST2)

Figure 14 shows that hydraulic impact decreases with decrease in friction angle for higher static heads. This is the result of high apertures formed during the end of tunnel filling. During tunnel filling, smaller friction angle causes larger shear displacement of the joint. Joint dilation during shear displacement causes higher joint aperture at the end of tunnel filling. This phenomenon is more pronounced when the static heads are higher (lower effective normal stresses across joints).

When normal stresses are high, for example at 100 m static heads, the hydraulic impacts are same for both friction angles since hydraulic apertures are also the same.

6.7 Effect of Joint Dilation

Simulations for 500 m static head for both stiffness cases were also run without allowing dilation. This static head is chosen to see maximum effect since it is the maximum allowable static head as per the conventional FoS principle. In Fig. 15a, it is seen that when dilation is restricted, the peak hydraulic impact increases significantly and is shifted closer to the tunnel wall, with a narrower zone of influence. This is logical because the hydraulic aperture after tunnel filling (steady state in Fig. 15b), without dilation will be smaller and thus creates larger flow resistance, which prevents the pressure pulse to travel deep into the rock. This causes high pressure buildup and thus higher hydraulic impact closer to the tunnel wall as compared to when dilation is allowed. In Fig. 15a, it is also seen that the effect of dilation is larger in high stiffness case. This is because the onset of dilation is more sudden and pronounced as compared to a low stiffness joint which allows gradual displacement. Higher dilation effect causes large change in hydraulic aperture and hence, larger difference in hydraulic impact as compared to low stiffness joint.

7 Discussion

In contrast to established understanding that the additional seepage forces during pressure transients (or hydraulic impact) are higher at locations closer to the tunnel, it is seen that the highest impact occurs where there is sufficient pore pressure buildup and also enough flow resistance to cause a significant timelag. Such locations are few meters into the tunnel wall, depending upon joint properties. Further deep into the rock mass, the hydraulic impact starts to decrease as the pore pressure buildup is reduced, even though the timelag is higher. Therefore, in addition to rock mass pore pressure, a more important parameter that needs to be added to this phenomenon is the timelag or transmission delay of the pressure pulse into the rock mass, which depends upon the joint aperture created at the end of tunnel filling.

In relation to tunnel static heads, it is obvious that during water filling, the magnitude of joint deformation will increase with increasing static head. However, it is seen that the percentage increase in joint deformation during pressure transient is found to be the highest when the FoS is between 1.5 and 2.5. This means that tunnels designed within this FoS range will be the one that are most impacted by pressure transients. Palmstrøm (1987) shows that critical locations along the unlined tunnels usually have FoS lying within this range. FoS larger than 2.5 are usually undesirable with respect to economic reasons, whereas FoS smaller than 1.5 are uncommon, since they would require specific stress measurements during construction.

Block failures occur at a certain amount of absolute joint displacement. Such absolute displacement is the cumulative effect of a number of pressure transients over the tunnel operation period, referred to as hydraulic fatigue. For such fatigue to occur due to transients, the joint has to deform further from its initial deformation state at static tunnel pressure or steady state. At the static condition, when the FoS

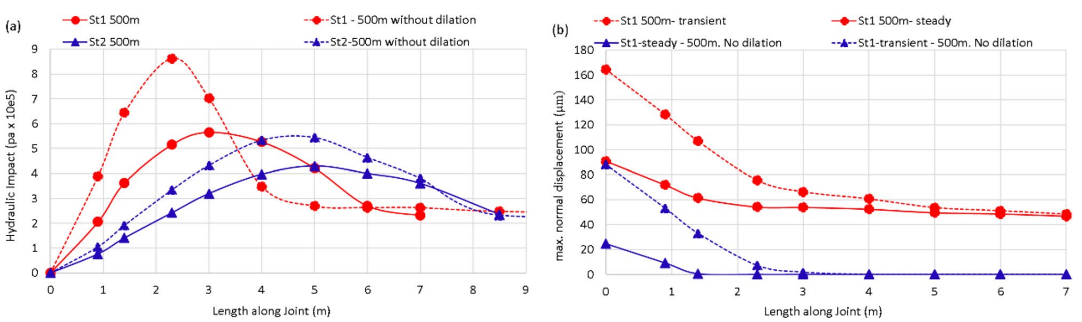

Fig. 15 Effect of joint dilation at 500 m static head on **a** hydraulic impact for both stiffness cases and **b** max. normal displacement along the joint for high stiffness case during steady state and pressure transient

is greater than 1, the block will remain stable. This applies for all static head cases in Fig. 7, even if absolute deformations are higher for higher static heads. During transients, the more a joint deforms from its initial displacement state, the more it contributes to fatigue. As seen in Fig. 7, the cases with FoS between 1.5 and 2.5 show larger relative displacement during transients and hence will have more "loosening effect" on the joint. Hence, percentage increase of deformation during pressure transients is used for comparison because it shows which case of FoS shows more relative joint displacement from their respective steady states after tunnel filling.

This shows that most of the unlined tunnels which are under operation may be impacted significantly in terms of fatigue due to cyclic loading of joints over long-term operation of tunnels, eventually leading to block falls.

This conclusion holds true for both hard crystalline rocks and schistose rocks as represented by two sets of joint stiffness values used in the simulations. Figure 16 shows a typical example of block falls in rock mass with varying joint conditions registered during headrace tunnel inspections in Norwegian power plant under operation over 30 years.

A commonly established concept about impact of time period of pressure transient is that only slower pressure oscillations, i.e., mass oscillations are of significance, when it comes to instabilities occurring during plant operation. Prevalent argument for this is that the time period of oscillation for water hammer is too short to travel deep into the rock mass. However, from the field data and simulations, it is seen that such transients can also travel up to 4 m deep into the tunnel wall, even in stiff joint conditions and sufficiently high normal stresses. Hence, the next question is whether the displacements and hydraulic impact are significant to contribute to block falls. It is seen that the maximum hydraulic impact along the joint due to each cycle of mass oscillation is about 25 times higher than the impact due to one water hammer pulse for high stiffness case and 15 times higher for low stiffness case. The displacements for both cases due to mass oscillation are only about 2 times higher than water hammer. At higher static heads, when the FoS is lesser but still within allowable range, it is likely that hydraulic impact ratio will be almost equal or slightly less since mass oscillation may cause increased leakage and pressure release because of longer time period. However, this still supports the previously established knowledge that mass oscillations cause significantly larger hydraulic impact as compared to water hammers.

The hydraulic impact ratio can vary depending upon different parameters, such as waterway lengths, area, valve closure time, etc. Longer tunnel length between turbine and surge tank (pressure tunnels/shafts) contributes to higher water hammer magnitude and longer tunnel length between surge tank and reservoir (headrace/ tailrace tunnels) causes larger mass oscillations. For Roskrepp waterway, these two lengths are 560 m and 3.5 km, respectively.

It is desirable to place the surge shaft as close as possible to the turbine to reduce the detrimental effect of large water hammer. This means that most waterways have relatively longer headrace/tailrace tunnels and shorter pressure tunnels/shafts. Further, the pressure increase due to water hammer is only limited within the length of pressure tunnels/shafts, but the pressure rise due to mass oscillations applies to the full length of waterway. This means the hydraulic impact of mass oscillations are much large in magnitude and felt deeper into the rock mass. Also, they apply to longer stretch of tunnels as compared to water hammer. Faster valve closure time may contribute to large water hammer magnitude but still the hydraulic impact is smaller because of their small time period. However, the effect of water hammer may not be fully ignored. This is because, for every shutdown event, the number of pressure pulses due to water hammer with significant timelag is higher than mass oscillations. In the case of Roskrepp, for every shutdown event, there are 3–4 mass oscillation cycles with timelag whereas the

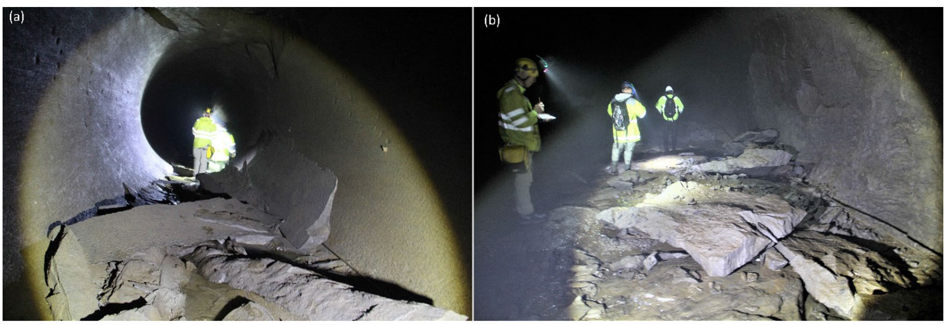

Fig. 16 Block falls registered in **a** hard crystalline rock (Trondhjemite) and **b** schistose phyllite during inspections of unlined pressure tunnel of a power plant in Norway

number of water hammer pulses is much higher (Fig. 17a). Further, during a pressure transient, it is very difficult to make a distinction to when water hammer ceases and mass oscillation start. Hence, the cumulative hydraulic impact of all such pulses is significant regarding long-term stability of waterways systems.

The effect of small but frequent pressure pulses, such as water hammer on the opening of joints, can be compared to the concept of fatigue hydraulic fracturing (FHF) introduced by Zang et al. (2013), to enhance the permeability of fractured rocks in petroleum reservoirs. Hofmann et al. (2018) describes the cyclic soft stimulation (CSS) technique which consists of injection protocol with three types of cycles with different time periods, i.e. Long-term cycles (LTCs, hours or more), medium-term cycles, (MTCs, minutes to hours) and short-term cycles (STCs, minutes and less).

In Fig. 17a, the duration between two valve closure events can be compared to the LTCs, mass oscillations are equivalent to the MTCs and water hammers can be compared to the STCs. In CSS, the short-term cycles are applied to amplify the fatigue and weakening of the rock by inducing additional small fissures before and besides the macroscopic fracture development. The time scale and magnitude of pressures in these different two processes can vary but the basic principle in both cases is hydraulic fatigue.

Hence, in the authors' opinion, water hammers, wherever applicable along the waterway, must be taken in consideration to assess long-term stability of water tunnels subjected to pressure transient. Therefore, it is important to consider the cumulative impact of both water hammer and mass oscillation because the impacts due to water hammers at one cycle, even though small, are measurable and more frequent, as shown by these experiment and simulations. A deeper understanding of hydraulic fatigue is necessary to quantify the effect of water hammers. Till recent time, very limited research has been conducted in this field. A much larger number of theories and experiments exist regarding ¨mechanical fatigue¨ of rock without fluid injection. But whether and how classical fatigue concepts can be applied to hydraulic fatigue remains a subject of further research.

It can still be stated that fatigue causes gradual deterioration of joint surfaces over long-term resulting in reduction of joint stiffness and friction angle. Simulation results with decreasing stiffness and friction angle can provide some insight on how the behaviour changes with change in joint properties. Table 7 presents some results for 300 m static head (FoS 2.1) for different stiffness and friction angles.

The values in Table 7 indicate that when the joint stiffness gradually deteriorates from ST1 to ST2, transients cause much larger joint deformation (7.5 times) than tunnel filling (3.5 times) for 40° friction angle case. Similar results are seen for 25° friction angle as well. But for the same stiffness, when friction angle decreases from 40° to 25°, larger joint deformation occurs during tunnel filling itself and thus the effect during transient is relatively small. This shows that transients tend to cause larger deformation as the joint

Table 7 Simulation results for 300 m static head at 2.3 m joint length for different stiffness and friction angles

Joint friction angle	ST1		ST2	
	40°	25°	40°	25°
Normal displacement, tunnel filling (μm)	2	29	7	26
Normal displacement, transient (μm)	5	34	37	57
Hydraulic impact (pa s $\times 10^5$)	3.14	2.12	1.63	1.43

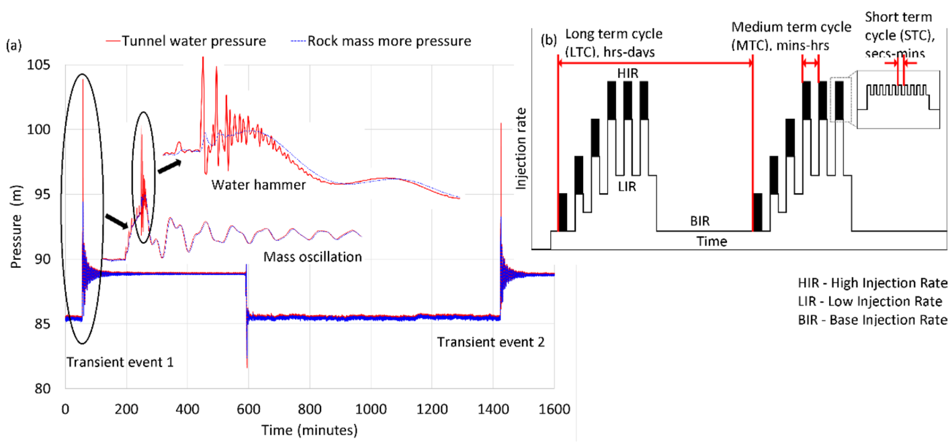

Fig. 17 Comparison between **a** recorded pressure fluctuation in an unlined hydropower tunnel and **b** cyclic fluid injection protocol used in petroleum and geothermal reservoirs (Hofmann et al. (2018)

undergoes gradual reduction in stiffness. But for gradual reduction of friction angle, joints are likely to deform more during filling itself. In real scenario, a simultaneous reduction of these parameters is a more likely case. Hence, a possible combined effect would be such that transients with small time periods, cause ¨gradual loosening¨ of joints in addition to the effect of mass oscillation, thereby reducing the joint strength. Macroscopic movement of blocks occurs during filling/dewatering and mass oscillations.

The orientation of the joint with respect to major principal stresses also plays an important role in the behaviour during tunnel filling and pressure transient because it affects the normal stresses across the joint. If the minimum principal stress is parallel to the joint, the normal stress across joint will be higher because the major or intermediate principal stress will be acting perpendicular to the joint. In such condition, the FoS will be higher and hence the joint deformation during tunnel filling and transient will be reduced. This may result in a non-conducting, tightly closed joint which shows very little or no response during pressure transient. On the other hand, if the minimum principal stress is perpendicular to the joint, this will create a larger joint displacement because of reduced normal stress. If such normal stress is critically low (lower than tunnel water pressure), then the joint will be hydraulically jacked during tunnel filling itself.

8 Conclusion

The results of numerical simulation show that relative joint deformations due to short pressure transients are highest when normal stresses acting across the joints are 1.5–2.5 times higher than the static tunnel water pressure. Since most critical locations in the tunnels are designed to be in this range of factor of safety, it is concluded that such tunnels are significantly affected by pressure transients, eventually leading to hydraulic fatigue. This conclusion holds true for varying joint stiffness values.

The results also confirm that mass oscillations can cause significantly large hydraulic impact on block stability. It has been demonstrated that for a typical hydropower waterway system, the hydraulic impact ratio between mass oscillation and water hammer per cycle can vary between 15 and 25. This ratio can vary, mainly depending on the length of headrace/tailrace tunnel and high-pressure tunnel/shaft. Simulation results show that joint deformation does not increase significantly, when the time period of oscillation is increased from one minute to several minutes. The higher impact with increasing time period is mainly because the pressure is ¨trapped¨ for a longer time.

More importantly, the analysis results show that water hammers, wherever applicable along the waterway can contribute to gradual ¨loosening¨ of the joints since they can travel up to 4 m into the rock mass even in stiff joint conditions and sufficiently high normal stresses. As indicated by results with various stiffness and friction angles, such ¨loosening¨ becomes more prominent when the stiffness of joint gradually reduces. This phenomenon, combined with reduced friction angle over long term, causes larger displacements or block movements during tunnel dewatering/filling and pressure oscillations with large time period, i.e., mass oscillations. Thus, it is concluded that water hammers and mass oscillations have a cumulative effect on the long-term stability of blocks. However, the effect of water hammer is only limited in the waterway length between the surge shaft and turbine but effect of mass oscillations applies to the full length of waterway.

The analyses and results presented above are based on numerical models assuming the rock joint as the interface between two parallel plates. Limitation of these models is that it does not consider the effect of flow tortuosity in rock joints caused by joint roughness, which is still an outstanding issue in numerical simulation of flow processes in rock. However, the achieved results are consistent with basic phenomenon of joint fluid flow and some well-established trends from previous researches. Hence, these findings can be of significance to further understand the issue of block failures caused by frequent pressure transients. Another issue of interest is the ¨hydraulic fatigue¨ due to which the joint properties change gradually over time, which is a large field of research.

Long-term stability of unlined hydropower water tunnels subjected to medium to high static water heads in changing energy market conditions is an immerging challenge, since frequent start stop sequences will cause intensified occurrence of pressure transients. In authors' knowledge, this is the first time such study has been conducted involving hydro-mechanical interactions of frequent pore pressure changes in the rock mass around hydropower tunnels. The effect of frequent pressure transients of both short- and long-time periods (water hammer and mass oscillation) are the main cause for frequent block falls in unlined pressure tunnels.

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Declarations

Conflict of interest The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no financial support for this work that could have influenced its outcome.

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Paper 4

Cyclic fatigue in unlined hydro tunnels caused by pressure transients.

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Cyclic fatigue in unlined hydro tunnels caused by pressure transients

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Regulated hydropower is currently the most capable load balancing technology for the future renewable energy system which will be characterized by a large share of unregulated energy from solar and wind power. Increased application of hydropower for balancing means that more frequent pressure transients will occur in the waterway system. In this article, the authors explain how such pressure transients cause pore pressure fluctuations in the rock mass surrounding the tunnel periphery and may lead to cyclic fatigue. An explanation of the mechanisms that contribute to fatigue is also presented along with the most significant factors that can accelerate the fatigue development. The possible applications and limitations of these findings are discussed and recommendations are given.

For underground construction in hard and brittle rocks, it is usually assumed that the long-term behaviour of the rock mass does not have significant influence on the stability during the serviceable life. However, this notion is changing, as underground structures are expected to serve for longer periods. For example, long duration compression loading is of particular concern for the underground disposal of nuclear waste [Damjanac and Fairhurst, 2010[1]]. In general, underground structures are expected to be fully functional during long-term operation, with as little maintenance as possible. This has created the necessity to improve the understanding of long-term behaviour of fatigue of hard, brittle rocks under different long-term loading conditions. One of the first studies conducted regarding the cumulative damage to rock mass under cyclic stresses was conducted by Burdine [1963[2]] in relation to drilling for mining purposes. The concept of fatigue damage has also been studied in relation to its use for other purposes such as stability of deep-seated landslides caused by cyclic pore pressure variation [Preisig *et al.*, 2016[3]], and by seismic loads [Gischig *et al.*, 2016[4]], hydraulic fracturing [Zang *et al.*, 2021[5]), and crude oil storage [Wang *et al.*, 2015[6]] among others. For hydropower plants, fatigue has been studied in the case of hydraulic turbines [Liu *et al.*, 2016[7]; Trivedi *et al.*, 2013[8]], which are known to occur as a result of pressure transients caused by load changes and start-stops. This paper presents work on cyclic fatigue of hydropower tunnels, caused by the same load changes and start-stops.

Failure in any material under stress occurs as a result of initiation and growth of cracks which can deform in three different modes: tensile; in-plane shear; and, anti-plane shear. Such failures can be the result of a monotonic load that exceeds the strength of the material, or a cyclic load acting for a longer time with cyclic stresses smaller than the monotonic strength. Furthermore, fatigue can also occur because of a sustained load or residual stress acting for a long time, referred to as stress corrosion as described by Schijve [2009[9]]. Fatigue behaviour in natural rock material has been studied by various researchers in the past under static, monotonic, and cyclic loading conditions and is reviewed comprehensively by Cerfontaine and Collin [2018[10]]. The main conclusion is that stress corrosion and fatigue mechanisms are responsible for subcritical cracks on rock specimens, which probably occur simultaneously such that stress corrosion dominates at high mean stress while fatigue mechanism dominates at high-cycle amplitude.

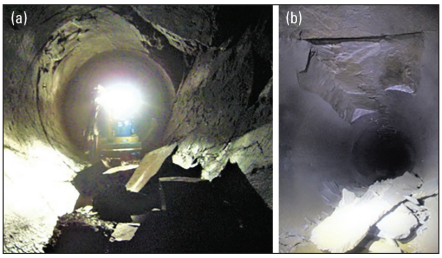
The effect of cyclic loads on rock joints is also important when assessing fatigue failure of a rock mass. Surface degradation of rock joint walls occurs because of shear stress, which gradually reduces their strength for which some constitutive models have also been proposed by Belem *et al*. [2007[11]] and Nemcik *et al*. [2014[12]]. Experimental results [Liu *et al.*, 2018[13]; Tsubota *et al*.; 2013[14], Ferraro *et al.*, 2010[15]; Jaferi *et al.*, 2004[16]] show that the number, frequency, and amplitude of the stress cycles reduce the resulting peak and residual shear strength of the joints subjected to cyclic loading.

Cyclic loads occur in hydropower tunnels during pressure transients, which occur after load changes on the turbine units. It is seen from experimental [Neupane *et al*., 2020[17] and Neupane, 2021[18]] and numerical simulations [Neupane and Panthi, 2021[19]] that such pressure transients can cause increased stresses in the rock mass surrounding the tunnel periphery which can contribute to fatigue. In the previous publications, operational data from several hydropower plants in Norway from the past 20 years shows a high frequency of start/stop sequences and load changes. This is expected to increase further in the future, since hydropower will be used to balance the power production from more non-regulated sources such as wind and solar energy [Catrinu *et al.*, 2011[20]]. This translates to a significant increase in pressure transients of various magnitudes and a corresponding increase in the number of cycles of hydraulic stresses acting on the rock mass, which can contribute to cyclic fatigue, leading to block falls and tunnel collapses in the long run. Brekke and Ripley [1987[21]] mention that if operational requirements lead to frequent pressure pulsations, special efforts may be needed to ensure that blocky, unlined sections remain stable over the operational life of the project. However, this issue has not been addressed from a perspective of rock mass fatigue around tunnels.

In this article, the authors present the observed mechanism which may lead to fatigue in rock masses around unlined tunnels because of frequent pressure transients. The factors contributing to this mechanism are presented. The conclusions are based on observations from dewatered unlined hydropower tunnels and the experimental and numerical studies mentioned above. Possible applications and limitations of these findings are discussed and recommendations are given.

1. Cycle fatigue mechanism

A detailed inspection of 35 km of unlined hydropower pressure tunnels at four hydropower plants, which



Block falls observed in the headrace tunnel of Brattset (a) and Ulset (b) headrace tunnels after more than 30 years of operation.

have been in operation for more than 30 years, has been conducted by the authors. From the number of block falls observed in these tunnels, and the results from an experimental study and numerical simulations, some conclusions are drawn regarding how the failure is initiated and aggravated as a result of frequent pressure transients over long-term operation. The failure mechanism which has caused a relatively stable rock mass to fail during long-term operation as a result of cyclic fatigue is explained. From the tunnel inspections, it is seen that relatively stable and unsupported blocks during construction can also be destabilized over years of powerplant operation.

Fig. 1 presents two typical block fall cases from the unlined headrace tunnels of the Brattset and Ulset hydropower plants located in central Norway where such events have occurred. These powerplants have been in operation since 1982 and 1985 respectively, and the block falls presented in Photos (a) and (b) were observed during inspections carried out in 2015 and 2017 respectively. These are new block falls that had not occurred during the previous inspections in 1992 (Ulset) and 2008 (Brattset). Comparatively fresh fracture surfaces could be seen in these blocks, although the time of these events cannot be narrowed down because of a lack of additional inspections. However, this shows that such failures are time dependent and occur over the duration of powerplant operation and might be accelerated by cyclic loads, in addition to dewatering and filling events. For the block fall at Brattset, a distinct sliding plane is dipping steeply into the tunnel which intersects with another joint in the crown. Failure has been caused by the breakage of the rock bridge and has an estimated volume of 15 m^3, with large rock pieces of 3-6 m^3. For the block falls at Ulset, near horizontal foliation joints intersect with new vertical fracture planes seen on the walls, causing the block fall.

Cyclic loading in hydropower tunnels over several years of operation can be categorized as high cycle loading [Lee and Barr, 2004[22]], which is characterized by a large number of cycles at low stress levels, where the number of cycles could be in thousands. The number of total load changes for Brattset and Ulset are 2000 and 1500 per year respectively [Neupane *et al.,* 2021[18]], which means that thousands of pressure transients occur during long-term operation over many years. Such transients are of different magnitudes, which cause a high variability in the cyclic stress magnitude.

Fig. 1 presents a representative tunnel cross section in rock mass with conducting joints (1a), with a time-series of pressure transients in water and rock (1b), and the resulting idealized pressure loading on an idealized intact rock bridge (1c). In the figure, the existing joints are oriented so that pressure transients can travel into the rock mass, but the condition for blocks to become detached has not been met. An idealized pore pressure situation at four arbitrary locations, along with a flow path following a system of conductive joints, is presented together with the pressure diagram for an intact rock bridge for eight different time points of the time-series.

The pressure transients in hydropower plants can be divided into two phenomena: mass oscillations with a long oscillation period in the order of minutes; and, water hammer with a short oscillation period in the order of seconds [Jaeger, 1977[23]]. The focus in this paper is only on the mass oscillations, as they have a larger impact on the rock mass because the time of application of the hydraulic stress is higher as compared with water hammer. Mass oscillations also influence larger stretches of tunnels in typical hydropower systems, since water hammer is limited to the stretch between the turbines and the nearest upstream and downstream surge tank, whereas the mass oscillations influence the entire tunnel length. Calculations in Neupane *et al.,* [2021[18]] demonstrate that for a specific case study, mass oscillations had a significantly more adverse effect on the pore pressure compared with the water hammer. This is expected to be the typical situation. It is noted, however, that there may be exceptions where water hammer is the main contributor to cyclic fatigue. Such exceptions include unlined hydropower tunnels without surge tanks that do not have mass oscillations.

Experimental and numerical simulation results indicate that the pore pressure magnitude is higher at locations closer to the tunnel and gradually decreases as the length of the flow path increases. At point A, the magnitude is the highest and almost equal to the tunnel pressure, whereas at point D, it is the lowest and this location is far enough into the rock such that the transients do not affect the pore pressure. On the other hand, the time lag is lowest at a point closer to the tunnel and increases as the length of the flow path increases. Thus, the additional hydraulic stress is a combination of these two parameters when the rock mass pore pressure is increased. The pore pressure magnitude and time-lag at any point along the joint depends on the joint void geometry.

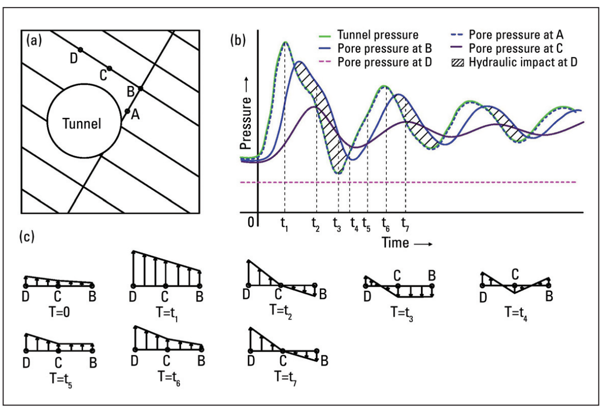


Fig. 1. (a) A representative tunnel section; (b) conductive joints to illustrate pore pressure changes resulting from pressure transients; and, (c) idealized pressure diagrams along an intact rock bridge.

Neupane *et al.,* [2021[18]] define the accumulated hydraulic stress acting on the joints from a pressure transient as the 'hydraulic impact'. More specifically, the hydraulic impact is a destabilizing load (MPa.sec) and is the difference between joint pore pressure and tunnel water pressure integrated over time, when the joint pore pressure is higher at certain periods during the pressure transient. For example, the hydraulic impact at location B during a pressure transient is shown in Fig. 1(b).

Fig. 1(c) shows the difference between tunnel pressure and rock joint pressure at the four locations along the joint at different time points during the pressure transient, and their resultant direction is shown with arrows. Higher tunnel pressure means that the resultant is away from the tunnel cavity and higher joint pressure means that it is towards the tunnel cavity, pushing the rock block away or towards the tunnel cavity respectively. It is seen in Fig 1(b) that the joint pore pressure could be higher or lower than the tunnel pressure at different locations along the joint because of the delayed rock mass response. This causes the resultant pressure direction to vary towards or away from the tunnel cavity within minutes.

During the rise of the first pressure oscillation (time 0 to t_1), the tunnel pressure is higher than the joint pore pressure along the whole joint and the magnitude of resultant pressure increases from 0 to t_1 at all locations because of the quick rise in tunnel pressure. The resultant direction is out of the tunnel cavity and thus induces compressional forces on the rock bridge which pushes the block away from the tunnel cavity. Between t_1 and t_2 the resultant direction at B changes towards the tunnel cavity, but it is still towards the tunnel cavity at C and D, which induces a shear loading on the intact rock block between B and C. Between t_2 and t_3, the resultant direction at C also changes towards the tunnel cavity. This causes point B and C to move towards the tunnel and induces the shear loading between C and D. Between t_3 and t_4, resultant direction at B again points away from the tunnel, inducing further shear loading between B and C in reverse direction as compared to the one between t_1 and t_2. After t_4, the resultant at all locations point away from the tunnel and the resultant magnitude increases, starting a similar cycle of loading as explained between t_0 and t_4, but with reduced magnitude. This pattern is repeated and continues for some cycles until the pressure transient is dampened as a result of friction. The amplitude of the first pressure pulses are the highest and hence the resultant pressure is also highest, which reduces with time as the mass oscillation dampens.

The joint section AB and another joint set located in the third dimension in Fig 1(a) control the stability of the illustrated block. In addition to the compressional and shear loading acting on the intact rock, these joints also undergo cyclic shearing, when the intact rock block tends to displace from its steady state during the transient. Previous research has shown that both rock joints and intact rock mass undergo strength reduction due to cyclic fatigue. For rock joints, the strength reduction occurs because of shearing of asperities. Patton [1966[24]] classified the asperity of rough joints into first and second order, which represent the waviness and unevenness of the surfaces, respectively. According to Fathi *et al*. [2016[25]], during cyclic loading the contact area between joint surfaces increases for the first few cycles, which is named as the contraction effect. On further cycles, this effect decreases and damage of the second order asperities begins, which is degradation. Fatigue cracks initiate in the first-order asperities and then coalesce with each other and the rock joint after a large number of cycles [Liu *et al.,* 2018[13]], which eventually leads to failure.

The damage and degradation of a joint surface caused by pressure transients could be characterized by the reduction of stiffness and frictional resistance and increase in hydraulic aperture. A numerical simulation carried out by Neupane and Panthi [2021[19]] shows that a reduced friction angle causes larger joint deformations already during tunnel water filling or steady state itself. On the other hand, larger deformations were seen during pressure transients when joint stiffness is reduced. Hence, a possible scenario is a reduction of these two parameters resulting from fatigue causing the joints to deteriorate and eventually leading to macroscopic movement during filling/dewatering and/or mass oscillations.

Deterioration of intact rock leads to block falls in areas where they could not have been envisaged because of an intact rock bridge preventing the formation of a wedge. Pressure transients cause a complex mechanical loading on the rock block within a short time, comprising both compressional and shear loading as explained previously. There may also be tensile stresses acting on the intact rock where it is hinged to the rock mass. Further, tensile forces can be generated in the intact rock during compressional loading as well. The magnitude of additional forces during pressure transients are small compared with the strength of intact rock, and thus the failure must be attributed to cyclic loading caused by frequently occurring pressure transients.

Cyclic loading can result in accumulated fatigue damage and failure of the intact rock at a stress level lower than its monotonic strength, which is a result of progressive decohesion and loosening of material caused by microcracks initiating and propagating to form a macroscale crack [Cerfontaine and Collin, 2018[10]]. It can be seen that the results of cyclic loading are different in terms of the crack growth process compared with monotonic loading. Monotonic loading results in a definite crack, while cyclic loading results in a wider fracture zone which creates significant crack and dust [Erarslan 2016[26], and Erarslan *et al.,* 2014[27]]. This is because for monotonic loading, the failure mode is brittle and the rock grains along the failure surface are highly cracked. For cyclic loading, failure occurs along grain boundaries and with intergranular cracks as the primary failure mechanism. Also, the wear and shearing between rock grains starting at the boundaries further leads to intragranular cracks. The failure finally results from the coalescence of many microcracks rather than the growth of a single macrocrack as discussed by Cerfontaine and Collin [2018[10]] and Erarslan [2016[26]]. This may explain the observation reported by Bråtveit *et al*. [2016[28]] where it is mentioned that the average volume of rock blocks registered were 25 per cent smaller when the tunnel systems were subjected to 'hydropeaking'. However, the average total volume and frequency of the rock falls were found to increase. Hydropeaking is defined as an operational mode in which the load changes in powerplants are done multiple times per day to benefit from variable power prices, causing frequent pressure transients in the waterway.

Hence, for a rock block to be unstable, rock joints must significantly deteriorate such that they no longer have sufficient shear strength, combined with the weakening and rupture of intact rock bridge so that a new joint is created, thus fulfilling the block removability condition as described in the next section. This explains how the frequent rock mass pore pressure variation in unlined tunnels caused by pressure transients can affect the stability condition of a relatively stable block over the duration of the plant's operational lifetime. Further, the effect of induced tangential stresses around the excavation is also significant if such stresses are of similar magnitude as the rock mass strength. However, since the failure did not occur during the construction period but after several years of operation, it can be deduced that this is not the primary cause of failures. Since the rock mass is weakened as a result of cyclic fatigue, stress influence may also contribute to failure, even several years after the operation has begun.

2. Contributing factors

This section highlights the significant contributing factors that lead to fatigue of rock mass caused by pressure transients. It also provides some basic guidelines on how this information can be used to make a preliminary assessment of the risk potential for block falls in unlined hydropower tunnels.

2.1 Block removability

Goodman and Shi [1985[29]] point out that a block must be finite, removable, and potentially unstable to be critical to the stability of an underground excavation. Five types of blocks are thus referred to and are presented in Fig. 2.

Out of these five types, types I to IV are finite blocks but only type I and II have the potential for movement. Type I, known as the key block, is removable and its orientation is unfavourable in relation to its stability so it needs support to be stable. Type II may be stable with sufficient friction, and under favourable loading conditions, and is called a potential key block. Type III is stable because of gravity, even without any frictional resistance. Type IV is non-removable because of its tapered shape, even if it can move to some extent. Type V is an infinite block and does not cause any threat to the excavation.

During the construction of unlined tunnels, optimal use of rock support usually dictates that only unstable blocks are supported. Type I is most critical and is the most likely to be detected and stabilized during construction even before the excavation has completely isolated the block. Type II block may or may not be supported depending on joint conditions and the intended use of the tunnel. For unlined hydropower tunnels, such blocks may, if unsupported, become unstable already during the first tunnel water filling or during the first years of operation, due to reduced frictional resistance. This is because of reduced effective normal stress across joints or reduction of joint stiffness due to washing out of joint infilling materials. This condition also applies for block falls associated with weakness zones as mentioned by Palmstrøm [2003[30]], which can be categorized as failure from insufficient support measures in weak rock mass.

Block types IV and V are most likely to remain unsupported because they do not cause any instability problems. However, this is under the basic assumption

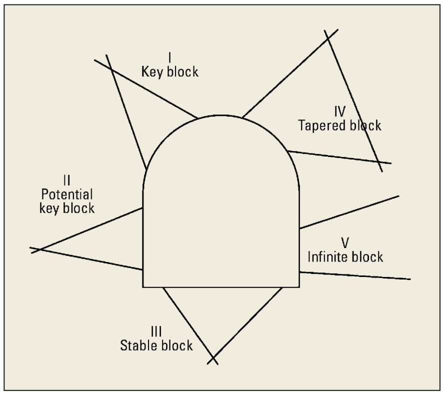


Fig. 2. Definition of blocks according to Goodman and Shi [1985[29]].

that the blocks are incapable of internal cracking. It has been demonstrated in the previous section that this assumption can be violated as a result of cyclic fatigue during long-term operation of unlined pressure tunnels.

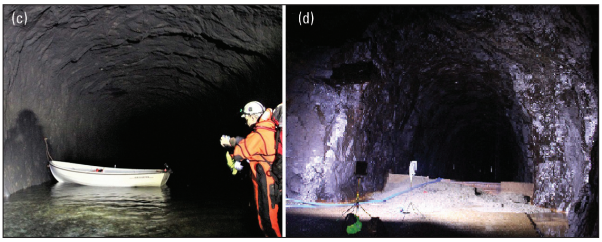
2.2 Joint geometry

Fluid flow through joints is an important parameter for fatigue of rock mass from pressure transients since it is the cause of additional stress acting on the joint surfaces during such transients. The coupling between fracture flow and deformation under normal stress is described using the 'modified cubic law' [Witherspoon *et al*. 1980[31]]. This law implies that hydraulic aperture of a joint is the most critical factor governing the fluid flow which is the idealized opening referring to the parallel plate model, where the fracture is represented by two smooth parallel plates. From the instrumentation at Roskrepp [Neupane *et al*. 2020[17]], it is demonstrated that pressure transients do not have any influence on the rock mass pore pressure if the joints are tightly closed, and do not allow the pressure transient to travel into the rock mass. The pore pressure will not change during a transient in such case and the response will be similar as seen in location D in Fig. 1(b).

Hydraulic aperture is not a physical feature that can be measured directly, and is a function of factors such as mechanical aperture, joint roughness, contact area between fractures and stiffness of fracture surfaces [Hakami, 1995[32]]. For the purpose of assessing long-term block stability in unlined tunnels, exact quantification of hydraulic aperture is not possible in practice because its calculation requires complex investigations requiring significant time and resources. This parame-

Table 1: Description of joint physical aperture according to ISRM (1978)

Aperture	Description	
<0.1 mm 0.1-0.25 mm 0.25-0.5 mm	Very tight Tight Partly open	"Closed" Features
0.5-2.5 mm 2.5-10 mm >10 mm	Open Moderately wide Wide	"Gapped" Features
1-10 cm 10-100 cm >1 m	Very wide Extremely wide Cavernous	"Open" Features



Photos from dewatered headrace tunnels at: (c) the Suldal I powerplant with tight joints and a dry tunnel wall; and, (d) the Roskrepp powerplant with open joints and dripping flow, showing different joint flow conditions.

ter also shows great variability as a result of the varying joint properties mentioned above. For practical purposes, it is necessary that these parameters can be assessed by a field engineer using simple tools available at his/her disposal during construction or inspection of such tunnels. The ISRM [1978[33]] recommendations for measuring physical joint aperture using a feeler gauge can be used for a simplified assessment, which is presented in Table 1. The aperture range which is of interest regarding long-term stability is most likely less than 1 mm because larger apertures may indicate destressed rock mass and must be addressed by the design considerations to limit hydraulic jacking against design static pressure [Palmstrøm and Broch, 2017[34]].

Another important parameter which describes the joint void geometry is the joint roughness, which can be assessed using a Barton's comb. The measured discontinuity roughness can be assigned by joint roughness coefficient (JRC) according to Barton and Bandis [1990[35]]. In general, rough and undulating joints tend to produce larger void geometry compared with planar, smooth joints and will produce larger hydraulic stresses during pressure transients. However, rougher joints will also have higher frictional resistance, which acts as a stabilizing force. Measurement of joint aperture and JRC values conducted at Roskrepp headrace tunnel showed that joint aperture values of 0.1-0.25 mm and JRC values of 4-6 (rough planar) resulted in non-conductive joints. Similarly, joint aperture values of 0.25-1 mm and JRC values of 10-14 (smooth undulating) and 14-18 (rough undulating) resulted in conductive joints. Observation of seepage through joints can also give a good indication of their conductive properties if the joints are connected to a ground water source through interconnected flow channels. As an example, Photos (c) and (d) show two tunnel sections indicating varying degrees of joint void geometry and flow conditions.

2.3 Deformation response of the rock mass

Results of the numerical simulation by Neupane and Panthi [2021[19]] indicate that pressure transients have a much larger zone of influence into a rock mass having relatively smaller deformation modulus and joint stiffness values. When pressure transients travel deeper into the rock mass, it increases the probability for larger instabilities caused by fatigue damage. Further, hard rocks with stiff joints will have higher fatigue limits and hence may be able to endure comparatively larger number of pressure transients. Schijve [2009[9]] defines fatigue limit or fatigue strength as the maximum stress amplitude for which there is no failure of the specimen. According to Cerfontaine and Collin [2018[10]], fatigue limit of 70 per cent of the monotonic strength may be used for intact rocks; however, failure may still occur for a lower stress level but for a very large number of cycles. Similarly for rock joints, Jafari *et al*. [2004[16]] concluded that if the applied cyclic stress amplitude is lower than 50 per cent of the peak monotonic shear strength, the remaining peak shear strength is nearly constant even after experiencing a high number of cycles. This implies that hard rocks with stiff joints are better suited to cope with fatigue, since their fatigue limits are higher as compared with weaker rocks.

For the purpose of assessing fatigue probability caused by pressure transients, it is not currently regarded as practically possible to quantify the deformation modulus and joint stiffness values for rock mass in tunnels which have already been in operation for many years. Also, it is difficult to make use of limited available data given a large variability of mechanical properties and response to loading conditions. Hence, to use the results presented here, the two broad categories of rock type, schistose and hard rocks, can be used as references. Smaller deformation modulus and joint stiffness values may represent schistose rocks whereas higher values may represent hard rocks. For example, the tunnels of Ulset and Brattset are both located in central Norway in the central-southern part of the Scandinavian Caledonides. The Cambro-Silurian rocks in this area such as phyllite, quartz-mica schist, graphitic schist can be categorized as schistose rocks. These tunnels also pass through rocks such as Trondhjemite and Granodiorite formed as result of intrusions due of volcanic activity in the Caledonian mountain belt, categorized as hard rocks. On the other hand, the rock in Roskrepp and Suldal tunnels are located in southern Norway with bedrocks of among the oldest basement rocks in Norway consisting of hard rocks such as Granite and Granitic gneisses. Hence, a subjective judgement of the rock type is necessary to assess this parameter for a particular tunnel in question.

2.4 Operational restrictions

A study of powerplant operational data shows that powerplants that have operation restrictions experience significantly smaller number of start/stops and large load changes as compared with plants without restrictions [Neupane *et al.*, 2021[18]]. Such restriction may be enforced for several reasons such as limitations in the electromechanical equipment. For some hydro plants in Norway, hydropeaking operation is restricted so that frequent and rapid water level and flow in the river downstream of the powerplant can be avoided, to protect the aquatic environment.

Brattset and Driva have operational restrictions of hydropeaking and they experience about 60 and 175 start/stops every year and large/medium scale load changes comprise of 13 per cent and 30 per cent of the total load changes. These values are up to 400 start/stops per year where 50 per cent are large or medium load changes for powerplants of similar sizes without restrictions [Neupane *et al.*, 2021[18]]. Hence, powerplants without operational requirements will be exposed to larger fatigue loads over their lifetime.

2.5 Duration of shutdown/opening

Shutdown duration has been found to be the most significant parameter regarding additional stresses on rock masses during shutdowns or load reductions. It is defined as the time between start of shutdown event and the peak mass oscillation amplitude and is a relative measure of how fast the turbine valves are operated. It is dependent on how fast the turbine valves are operated when changing the load in the powerplant. Measurement from Roskrepp [Neupane *et al.,* 2021[18]] shows that the duration of shutdowns is dependent on the individual powerplant operator, even for similar magnitude of the load changes because of the lack of a standard procedure. It can be seen that shorter shutdown durations result in a steeper rise in mass oscillation pressure and thus give a shorter time for the rock mass to respond to the pressure change in tunnel. As a result, a larger time-lag occurs between the pressure peaks of the tunnel and rock mass, which causes larger hydraulic impact, as shown in Fig. 1, to the rock mass. The effect of the shutdown duration is illustrated in Fig. 3, which shows a comparison of two shutdown events. Transient 1 has a larger load change magnitude than transient 2 but still shows a smaller time-lag with the rock joint pressure as a result of a much larger shutdown duration.

Every single pressure transient in hydropower tunnels is a unique phenomenon, because it is a combination of variables that are continually changing, such as the power output, turbine opening and reservoir levels determined by river hydrology and powerplant operation. Further, the flow into the tunnel from eventual brook (secondary) intakes also vary significantly over time. These parameters affect the static head and headloss before each transient event. The change in flow, headloss and resulting turbine head also causes the turbine efficiency to vary. Hence, the response of rock mass to each transient is unique, contributing to different pore pressure responses during each transient.

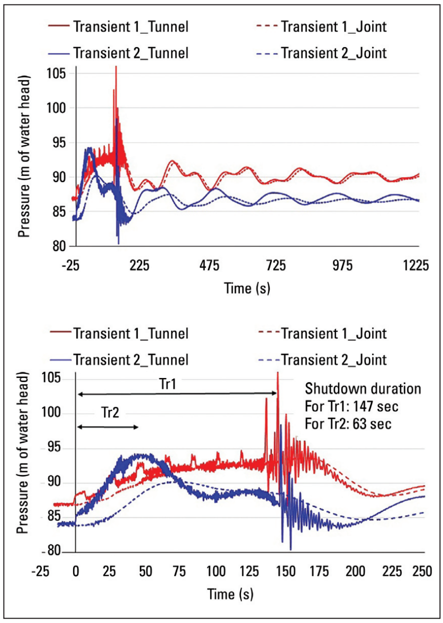


Fig. 3. Comparison of (a) pressure signals of two normal shutdowns; and (b) enlarged view of the first mass oscillation cycle. The lines denoted as tunnel represent the measured water pressure in the main tunnel, while the lines denoted as joint is the measured pore pressure in the joint. The graphs and the calculation of the shutdown duration can be found in Neupane et al. [2021[18]].

2.6 Dewatering and filling

Dewatering and filling of hydropower tunnels, not defined in this work as a pressure transient, but which can lead to fatigue are mentioned specifically as they are the incidents that inflict the largest single event stresses on hydropower tunnels. Since such events are infrequent, ranging from once per year to only once during the operational lifetime (during commissioning), they can by definition not cause cyclic fatigue. However, such events add to the accumulated stresses that the rock mass endures over long-term operation and may accelerate ongoing fatigue or trigger the initiation of a new fatigue process. In other words, the monotonic load from such events may exceed the strength of one or more rock features.

A recommendation for hydropower plant operators in Norway is to fill the tunnel at a rate slower than 200 m head per day during first filling, and slower than 300 m per day for subsequent fillings. Dewatering for both cases should be done at a rate slower than 250 m per day [Palmstrom and Broch, 2017[34]]. After dewatering, a 48-hour waiting time is recommended before allowing personnel to enter the tunnels, to reduce the risk of injuries from block falls. These recommendations are based on previous experience that dewatering causes block fall.

The mechanism that causes stress on the rock mass from dewatering is the same as for pressure transients caused by load changes and start-stop. However, since the water pressure inside the tunnel is completely removed, the destabilizing forces are allowed to act for a long duration until the rock mass is drained. The resulting hydraulic impact thus becomes severe. Furthermore, the complete draining of joints may result in washing out of infill material in joints, exposing these joint to higher hydraulic impact after filling and start of normal operation.

Filling of the tunnel is not regarded as being as severe as the dewatering since the pressure force resultant is then directed into the rock mass. However, the pressure gradient will be significant and may destabilize loose rock blocks by lifting or moving.

Based on experience of long-term fatigue it is a natural conclusion that dewatering and filling of hydropower tunnels should also be reduced to a minimum. These events are the single largest stresses that can be inflicted and should be avoided. At the same time, it is recommended to do periodic inspections to monitor if block falls start occurring more frequently, indicating an elevated risk of tunnel instability and collapse. Such inspections can, however, be made with remotely operated vehicles (ROV) in water filled tunnels to avoid the damaging dewatering. An ROV can be used to inspect the tunnel conditions with both filming and 3D scanning, which improves the documentation and eases the comparison of different inspections. Tunnel inspections and 3D scanning with ROV in water filled hydropower tunnels up to 12 km have been conducted by the authors. This was found to be both time and cost efficient compared with dewatering and manual inspection, and the potentially harmful dewatering process is avoided.

3. Application and limitations

Two possible applications of the findings presented here are envisaged in terms of: (1) the design of unlined pressure tunnels subjected to frequent pressure transients; and, (2) the operation of hydro powerplants. The findings point out the need for an increased conservatism in terms of rock support design with the increase in frequent load change events. Cost-effectiveness and shorter construction time are the main reasons that justify the design of unlined tunnels, such that rock support application is kept to a minimum possible amount, utilizing the in-situ rock stresses. Another design aspect is that minor rockfalls can be tolerated as long as they do not develop significantly and increase the frictional loss or cause blockage in the tunnel. While the existing design principle is relevant to avoid hydraulic jacking, the possibility of increased block falls as a result of operation may need to be considered when deciding the rock support requirement. This is even more important for example, in tunnels which are excavated in schistose rock mass such as phyllite and schist, and specifically in areas where the possibility of block removability could be encountered in the future as a result of damage to the intact rock bridge, with partly open and conductive joints. This scenario, combined with an unfavourable operational regime, could result in an accelerated fatigue rate.

Thus, potential 'future' blocks need to be identified and supported to fulfill the long-term stability condition both for new construction and the maintenance of existing tunnels. It is usual practice to map and record joints during construction and thus potential blocks could be identified by using simple visualization and kinematic analysis tools in 3D. Detailed assessment should be carried out for areas where higher joint conductivity is encountered. The sealing of such joints may be carried out in exposed sections to reduce the permeability so that pressure transients cannot travel into the rock mass. The most crucial parameter for the hydraulic impact on the rock mass is shutdown duration. A helpful strategy in reducing stress on the rock mass is to slow down the load change operations and thus avoiding strong transients occurring over short durations. It may be possible to apply the recommendation from Neupane *et al*. [2021[18]] to slow down the shutdown/opening readily without doing major changes in the powerplant. It is regarded as problematic that the shutdown duration is usually dependent on the individual operator and lack of standard guidelines for speed of load changes. Hence, more emphasis should be given towards keeping the speed of load changes consistently slow. Such measures are expected to reduce the number of block falls and prolong the serviceable lifetime of unlined pressure tunnels and shafts. However, it is also acknowledged that slowing down the load changes may affect the potential power plant income. Specifically for power plants that deliver system services such as frequency restoration reserves (FRR) there may be requirements to how fast the power shall be changed. Tertiary reserves may also demand start-up or shut down within a limited time. Hence, the specific powerplant owner needs to consider what is most reasonable for each power plant. For most power plants, the authors expect that it is possible to reduce the shutdown duration without affecting the income. This should especially be considered in power plants with significant block falls and concerns about the tunnel stability. In hydropower plants with tunnels constructed in relatively stable rock mass, measures to reduce the hydraulic impact should not be necessary. But, if inspections show that rockfall events are becoming increasingly frequent and more severe in specific tunnels, a reduction of load changes and slower load changes is highly recommended.

Every single power plant is unique in its design in terms of design discharge, operating head, and operation. These parameters, when combined with a wide variability of rock mass conditions makes both pressure transients and the subsequent rock mass response very unique over a wide range of load change events. Hence, it is difficult to theoretically quantify the safest range of shutdown duration for a particular power plant. For example, based on the instrumentation results at Roskrepp, shutdown durations larger than 200s seem to give the lowest possible impact with respect to shut down from full load and is the recommended shutdown duration from full load for this power plant to reduce the hydraulic stresses. Similar investigations are needed to quantify the recommended shutdown duration for individual power plants.

Tunnel dewatering and filling should be avoided as much as possible. Dewatering and filling events are the single most severe stresses that the hydropower tunnels are subjected to and may accelerate ongoing fatigue or initiate new fatigue processes. At the same time, it is recommended to do periodic inspections to monitor the tunnel conditions and the extent of block falls. Such inspections are recommended to be carried out using underwater ROV equipped with filming and 3D scanning possibilities. The experience gained so far indicates that use of ROV is both time and cost effective compared with traditional dewatering and manual inspection.

A general recommendation for the inspection interval for unlined hydropower tunnels is difficult to provide since this is case dependent. However, the authors strongly recommend a detailed mapping and 3D scan of the tunnel during construction. The mapping should provide engineering geological information such as fracture intensity, orientation and condition of fractures, overall rock quality description, description of support applied. The tunnel should be periodically inspected during operation. It is recommended and common practice to conduct a physical inspection after the liability period is over. The inspections thereafter are recommended to be carried out alternately with ROV in submerged and physical inspection in dewatered conditions. Since ROV inspection with 3D scans do not provide the possibility to map any changes in the conductivity of the joints and formation of new potential blocks, it is recommended that the tunnel is dewatered, physically inspected and that the necessary support measures are taken every 10 years. In addition, periodic inspections with ROV including 3D scans will be able to document, with accuracy, the historical development of the tunnel conditions and the amount of block falls. Then necessary measures may be considered and planned before a dewatering is necessary.

This paper presents a qualitative assessment of the mechanisms involved, and the contributing factors which can be used as a preliminary guide to assess the possibility of rock mass fatigue in critical locations of unlined pressure tunnels. However, application in quantitative terms demands more data from similar instrumentation at different powerplants, so that a larger database can be created to correlate the observed block fall events and the aforementioned parameters.

4. Conclusion

The basic mechanism of pore pressure changes in the rock mass around unlined tunnels during pressure transients and resulting rock mass fatigue is explained based on field observations, a field experiment and numerical simulations. The main factors contributing to rock mass fatigue have been described. It is concluded that an increasing conservatism may be needed in rock support decisions in potential failure zones relating to rock mass fatigue. It may also be considered to treat permeable zones in critical locations such that pressure transients cannot travel deep into the rock mass and cause additional stresses during load changes. The contributing factors presented here can be used as a preliminary guide to identify critical locations where additional support measures are needed. Several recommendations are provided to the operators of hydropower plants with unlined tunnels. These include reducing the frequency of load changes in hydropower plants with a vulnerable rock mass. Many hydropower plants should consider reducing the speed of load changes so that the hydraulic impact of the rock mass pore pressure during pressure transients can be reduced. Dewatering and filling of hydropower tunnels should be kept to a minimum. This will reduce the long-term aggregated hydraulic stresses acting on the rock mass, and thus slow down rock mass fatigue. Finally, a recommendation for the inspection interval and inspection method for unlined hydropower tunnels has been provided. ◊

Conflict of interest statement

The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no financial support for this work that could have influenced its outcome.

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Appendix B: Co-author statements

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DECLARATION OF CO-AUTHORSHIP

Bibek Neupane applies for the evaluation of the following thesis:

Long-term impact on unlined tunnels of hydropower plants due to frequent start/stop sequences

*)The declaration describes the work process and division of labor, specifically identifying the candidate's contribution, as well as give consent to the article being included in the thesis.

*)
Declaration of co-authorship on the following article:

Effect of power plant operation on pore pressure in jointed rock mass of an unlined hydropower tunnel: An experimental study

Authors: Bibek Neupane, Krishna Kanta Panthi and Kaspar Vereide

Author contributions:

The instrumentation concept was developed by the main supervisor who is the second author of this manuscript. After this the overall instrumentation program was further developed jointly by the authors. The first author developed detailed plan in collaboration with the co-authors and conducted the installation of the equipment. The second author conceptualized the rock mass pore pressure measurement including location, orientation/length of boreholes and packer arrangement for the boreholes. The third author conceptualized choice of sensors and piping arrangement and data logging frequency for all sensors. Third author was also the contact person for gaining access to the power plant and managing installation help needed from the power company. The manuscript was written by the first author except for section 2.1 which was written by the third author. Both co-authors also reviewed and edited the manuscript.

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*)
Declaration of co-authorship on the following article:

Operation of Norwegian hydropower plants and its effect on block fall events in unlined pressure tunnels and shafts

Authors: Bibek Neupane, Kaspar Vereide and Krishna Kanta Panthi

Author contributions:

The first author conceptualized the paper, performed the analysis, and wrote the manuscript. Both co-authors reviewed and edited the manuscript.

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*)
Declaration of co-authorship on the following article:

Evaluation on the effect of pressure transients on rock joints in unlined hydropower water tunnel using numerical simulation

Authors: Bibek Neupane and Krishna Kanta Panthi

Author contributions:

The first author conceptualized the paper, performed the numerical modelling and analysis, and wrote the manuscript. The co-author reviewed and edited the manuscript.

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*)
Declaration of co-authorship on the following article:

Cyclic fatigue in unlined hydropower tunnels due to pressure transients: Observed behaviour, contributing factors and recommendations

Authors: Bibek Neupane, Krishna Kanta Panthi and Kaspar Vereide

Author contributions:

The first author conceptualized the paper and wrote the manuscript. The co-authors reviewed and edited the manuscript.

Statement from the Co-authors:

We hereby declare that we are aware that the papers, of which we are co-authors, will form a part of the PhD Thesis by the PhD candidate who made a significant contribution to the work in the planning phase, research phase and writing phase.

Valgrinda, 07-09-2021
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Place, date

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Signature co-author (Krishna Kanta Panthi)

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