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Engineering Geological Considerations for the Planning of TBM Tunnels in Norway

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Doctoral Thesis

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Thesis for the degree of Philosophiae Doctor

Trondheim, June 2017

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Abstract

The use of tunnel boring machines (TBMs) was extensive in Norway during the larger scale hydropower development between the 1970s and the 1990s, and clients, contractors and research institutions contributed to the technological development that allowed stronger and more massive rocks to be bored. Most of the tunnels had small diameters (< 5 m), and the largest was 8.5 m. Currently, there are large-scale infrastructure developments (both public transport and technical) occurring in Norway, which involve the construction of many new tunnels. Some of these tunnels are being bored with TBMs, and it is believed that the use of this excavation method has great potential at future infrastructure projects. The necessary TBM diameters will often be > 9 m. Due to the high traffic (and importance) of new tunnels and increased construction in rural areas, larger emphasis is being placed on environmental considerations, and reliability, availability, maintenance and safety (RAMS). From a life cycle cost (LCC) perspective there is a desire to find the optimal tunnel concepts to ensure the fulfillment of functional requirements.

In Norway igneous and metamorphic rocks such as granites and gneisses dominate, and anisotropy is common with regards to rock properties and in situ rock stresses. Some of the most important engineering geological challenges experienced during earlier tunnel boring in Norway have been related to spalling and rock burst in high in situ stress environments, instability due to highly fractured/faulted rock, and large water inflow. These aspects are also typical concerns in Norwegian drill and blast (D&B) tunnels, but it has been realized that TBMs offer less flexibility, and that more careful planning and investigations ahead of a project are important for bored tunnels. Tough rocks can be considered a geological hazard (threat) with respect to the boring process, but the hazard of encountering very unfavorable rock mass behavior is considered to be equally and sometimes more important, since it may directly impact work safety, construction time and tunnel costs, and in the worst case threaten the feasibility of tunnel boring as an excavation method.

The aims of this thesis are:

- Identify and highlight important engineering geological aspects that should be carefully considered in the planning phase of Norwegian TBM infrastructure tunnels.
- Contribute to research within the respective fields, and provide specific recommendations that project owners and consultants can apply when planning TBM infrastructure tunnels in Norway.

The chosen focus of the thesis is not on the boring process, but on aspects considered to be equally and sometimes more important for the planning of hard rock TBM tunnels in Norway.

The work contributes within the following engineering geological fields of research, with a focus on TBM infrastructure tunnels:

- Waterproofing concepts
- Ground water drainage and settlement control
- Prediction of brittle failure (spalling and rock burst)
- Feasibility of tunnel boring through weakness zones (incl. high pressure, large water inflow)

The main conclusions following the research conducted in this thesis are:

- Engineering geological challenges normally increase with larger excavation diameter, and carrying out investigations and evaluations of rock mass behavior early enough in the planning process are important to assess the technical feasibility of tunnel boring.

- The TBM waterproofing concepts found to comply with the functional requirements of the Norwegian National Railway Administration, and hence believed to ensure a reliable railway operation (also important for other types of tunnels such as for roads and subways), are the concepts most often used in European infrastructure tunnels. That is the single shell concept (gasketed undrained segmental lining) and the double shell concepts (outer shell of shotcrete or drained segmental lining, and inner shell of drainage layer/membrane and cast concrete).

- In order to avoid pore pressure drop in settlement sensitive soils above urban tunnels, and hence to avoid damage to buildings and infrastructure in the future, functional requirements should take into consideration the total future drainage potential from all tunnels and building pits. It is believed that the use of traditional hard rock TBM-S (TBM-singe shield) machines built with the possibility to convert into static-closed-mode (from where pre-excavation grouting can be performed when the face is pressurized) can be an alternative to D&B in order to better control drainage during tunnel construction.

- Existing research on brittle failure and prediction methods have mostly focused on rock types with isotropic behavior, and the effect of anisotropy has not been fully understood. The results of this thesis suggest that existing prediction methods can be used for foliated/anisotropic rocks as long as: rock strength is considered to be directional-dependent; one considers that failure may occur in, and rotate to other parts of the tunnel-periphery than the location of maximum tangential stress; the range of recommended input parameters and confidence intervals of the prediction methods are considered.

- Tunnel boring through weakness zones in deep Norwegian subsea tunnels is considered one of the most challenging scenarios for a TBM. Since a TBM is often less flexible than D&B,

feasibility will directly relate to the identified geological hazards in each case and their potential implication to tunneling. For TBM it is important to identify and plan for the use of available and effective mitigation methods before the onset of construction. Large diameter tunnel boring (> 12 m for road tunnels) through weakness zones in deep Norwegian subsea tunnels has been found to involve a high risk, and hence is not recommended when the water pressure is above ca. 100 m. For lower water pressures TBMs with the option to operate in closed-mode may counteract some of the identified hazards, and can possibly be advantageous over D&B in certain cases.

The main suggestions for future work include:

- Research on new and alternative waterproofing concepts.
- Work to define best practice for construction processes and waterproofing concepts to ensure that ground settlements are minimized, and to limit future legal disputes.
- Continued research on the brittle failure behavior of anisotropic rocks, in order to predict failure with increasing accuracy.
- Continued development of TBM technology and pre-investigation techniques, in order for TBMs to be used in increasingly challenging geology in the future.
- Development of faster and more user-friendly computer programs for 3D numerical analysis, in order to analyze rock mechanical problems in a more realistic way.

Acknowledgements

This is for you Olaug Braamann
(my beloved grandmother)

List of papers and note on contributions

Paper 1: Evaluation of the potential for TBM use in future Norwegian tunneling projects

Authors: Øyvind Dammyr and Bjørn Nilsen

The candidate wrote the paper and carried out the literature review and evaluations. Bjørn Nilsen suggested some of the literature, contributed in discussions, reviewed the manuscript, and contributed some of the photos. The candidate gave an oral presentation of the paper at the Eurock 2012 ISRM International Symposium.

Paper 2: Possible Concepts for Waterproofing of Norwegian TBM Railway Tunnels

Authors: Øyvind Dammyr, Bjørn Nilsen, Kurosch Thuro and Jørn Grøndal

The candidate wrote the paper, carried out the literature review, gathered the data from international projects, and was responsible for the analyses and interpretation of the results. Bjørn Nilsen contributed to discussions and reviewed the manuscript. Kurosch Thuro contributed to discussions and reviewed the manuscript. Jørn Grøndal and the Norwegian National Railway Administration contributed the data from Norwegian railway projects and contributed to discussions.

Paper 3: Pressurized TBM-shield tunneling under the subsidence sensitive grounds of Oslo: Possibilities and limitations

Author: Øyvind Dammyr

The candidate was the sole author of this paper. Bjørn Nilsen and Johannes Gollegger contributed to discussions.

Paper 4: Prediction of Brittle Failure for TBM Tunnels in Anisotropic Rock: A Case Study from Northern Norway

Author: Øyvind Dammyr

The candidate was the sole author of this paper. Bjørn Nilsen contributed to discussions during fieldwork.

Paper 5: Feasibility of tunnel boring through weakness zones in deep Norwegian subsea tunnels

Authors: Øyvind Dammyr, Bjørn Nilsen and Johannes Gollegger

The candidate wrote the paper, carried out the literature review, gathered the necessary data and carried out the numerical modeling, and was responsible for the analyses and interpretation of results. Bjørn Nilsen suggested some of the literature, suggested text to be included in some of the sections, contributed some of the photos, contributed to discussions and with the interpretation of results, and reviewed the manuscript. Johannes Gollegger contributed to discussions and with the interpretation of results, and reviewed the manuscript.

Table of contents

1 Introduction

1.1 Background

Norwegian mainland rocks were formed during the Archean, Proterozoic and Paleozoic eras, and are of both igneous (plutonic and volcanic) and sedimentary origin. The geological history of Norway comprises the formation of the Swecokarelian (Proterozoic) and the Caledonian (Ordovician-Devonian) mountain chains, sediment deposition, igneous activity and intermittent erosion. Metamorphosis, tectonic activity and folding processes have altered the rock mass, and numerous glaciations have contributed to form the mountainous topography. Igneous and metamorphic rocks such as granites and gneisses dominate, but sedimentary rocks such as shales, limestones and sandstones are also common in various parts of the country (Figure 1). Anisotropy is common with regards to rock properties and in situ rock stresses. Spalling and rock burst activity is well known to Norwegian tunnelers, and is typically experienced in tunnels constructed along valley sides at the base of mountains (high sub-vertical stress and low stress perpendicular to the valley side), close to the surface (high horizontal tectonic and low vertical stress) or under large overburdens (high vertical stress). Due to erosion during recent glaciations faults can often be mapped as depressions on the surface. Deep weathering in relation to faults are normal, and weak zones of decomposed rock including clay minerals, which may require extra considerations during tunneling, are often encountered. Excavation procedures and rock support design are continuously evaluated and adapted to the given situation at the tunnel face.

Tunnel Boring Machines (TBMs) were extensively used in Norway between 1972 and 1992. The excavation method was predominantly utilized on hydropower projects, but sewage and water tunnels, and a few road tunnels were also bored. The generally high strength rocks (UCS > 60 MPa, ISRM 1980) made manufacturers, clients, contractors, consultants and research institutions take part in the technological development that ultimately enabled stronger and more massive rocks to be bored. Tunnel diameters (Ø) ranged from 2.4 m to 8.5 m, but most of the tunnels were smaller than Ø5.0 m (Hansen 1998). All TBMs were of the open type (TBM-O), normally with a short shield for cutterhead and work protection (Figure 2). Besides the tough boring process, engineering geological challenges were related to spalling and rock burst in high in situ stress environments and instability and large water inflow in weakness zones. These aspects are also typical concerns in Norwegian drill and blast (D&B) tunnels, but it was realized that TBMs offered less flexibility, and that more careful planning and investigations ahead of a project was important for bored tunnels. The use of TBMs came to an end with the culmination of the large hydropower development in Norway, and 22 years passed before the method was brought back into use at the end of 2013.

Population growth in and around the larger cities in Norway leads to increasing demand for tunnels to host new transport infrastructure and installations for other essential functions such as water, wastewater and power supply. Norway has many rural tunnels where the traffic is low, and where requirements regarding water leaks, groundwater lowering and maintenance intervals have not been given great attention. Environmental concerns and reliability, availability, maintenance and safety (RAMS), are increasingly important for new tunnels. Today the Norwegian tunneling industry places a greater emphasis on these aspects, including the life cycle cost (LCC) of tunnels.

The Norwegian National Railway Administration (NNRA) is currently planning and constructing the inter-city (IC) railway network in southeastern Norway, where the capital Oslo will be the central hub. The NNRA has also evaluated the construction of a high-speed rail (HSR) network between the larger cities in southern Norway and towards Sweden, which may become an extension of the IC network. Such a system would involve the construction of many long tunnels due to the mountainous topography and strict curvature and vertical gradients of high-speed railways.

The Ø7.2 m TBM-O for the Røssåga hydropower project marked the return of TBMs in Norway, and the 7.4 km long headrace tunnel of Nedre Røssåga was bored between late 2013 and late 2015 (Figure 3). The new Ulriken tunnel between Arna and the central city of Bergen in Western Norway, where a Ø9.3 m TBM-O started excavating 7 km of tunnel in early 2016, is the first Norwegian railway tunnel ever to be bored. Late in 2016 four Ø10 m double-shield TBMs (TBM-DS) started boring 19 km of the twin-tube tunnel on the Follo line (Follobanen) railway project, which is part of the IC network between Oslo and the city Ski. Other upcoming tunnels such as new central subway, railway, and other essential infrastructure tunnels in Oslo, and tunnels for future high speed railways between the larger cities in Norway, may also be considered suitable for tunnel boring. The Norwegian Public Roads Administration (NPRA) has shown recent interest in the method, and did a preliminary evaluation of TBM excavation in 2012 for the planned 27 km long and 400 m deep Rogfast subsea road tunnel in Western Norway. TBMs may also be considered advantageous for the upgrade of older road tunnels from one to two tubes, partly because vibrations from blasting can largely be avoided, and hence the existing traffic may be less interrupted. In addition to the development of traditional hard rock TBM technology, there has the last decades been an evolution in TBM pressurization technology, and the earth pressure balance (EPB) and slurry machines are now frequently used in soft ground mixed with sections of harder rock. These machines may offer superior control of water leaks and ground settlements, and in some cases enable tunneling where earlier the only approach thought feasible was through access from the surface.

Geological timescale

Cenozoic
- Quaternary
- Neogene
- Paleogene

Mesozoic
- Cretaceous
- Jurassic
- Triassic

Paleozoic
- Permian
- Carboniferous
- Devonian
- Silurian
- Ordovician
- Cambrian

Proterozoic

Archean

(Ma)
Present
3
23
72
145
201
252
299
359
419
444
485
541
2500
4000

Map labels

Mehamn, Honningsvåg, Berlevåg, Hammerfest, Vardø, Vadsø, Alta, Lakselv, Kirkenes, Karasjok, Kautokeino, Tromsø, Bardufoss, Andenes, Harstad, Sortland, Stokmarknes, Narvik, Leknes, Svolvær, Bodø, Fauske, Arctic circle, Svartisen, Mo i Rana, Sandnessjøen, Mosjøen, Brønnøysund, Rørvik, Namsos, Steinkjer, Trondheim, Stjørdal, Kristiansund, Molde, Ålesund, Andalsnes, Oppdal, Røros, Stryn, Tynset, Lom, Dombås, Førde, Sogndal, Fagernes, Trysil, Lillehammer, Voss, Gjøvik, Hamar, Bergen, Geilo, Hønefoss, Kongsvinger, Odda, Rjukan, Oslo, Haugesund, Drammen, Ski, Stavanger, Larvik, Fredrikstad, Egersund, Evje, Arendal, Mandal, Kristiansand

N

100 km

Legend

Oslo Graben
- Volcanic and magmatic rocks (Carbon. and Perm.)
- Sedimentary rocks (Cambrian to Silurian)
- Sedimentary rocks (Devonian)

Caledonian
- Thrust nappes of Precambrian rocks
- Metamorphic and magmatic rocks
- Thrust nappes of sandstone and shale (late Prot.)
- Basement rocks locally aff. by the Cal. orogenesis
- Basement rocks (Archean, Proterozoic)

Figure 1: Simplified overview of Norwegian geology. Modified after NGU (2016) and translated into English. Geological timescale drawn after information in Cohen et al. (2013; updated)

Recent work on prediction models for hard rock TBM advancement rate and tool wear includes research on the *Alpine model* (research group ABROCK: e.g. Schneider et al. 2012, Thuro et al. 2015), the *NTNU model* (e.g. Macias 2016), the *CSM model* (e.g. Yagiz et al. 2010), and research by many others (e.g. Gong and Zhao 2009, Hassanpour et al. 2011). Recent work to develop even tougher cutter tools has also been done (for example research group FAST-Tunn: Future Advanced Steel Technology for Tunneling – e.g. Vassenden and Grøv 2013). In TBM tunnels tough rocks can be considered a geological hazard (threat) with respect to the boring process. This hazard normally relates to construction time aspects and financial considerations, but can in extreme cases threaten TBM feasibility. What is considered more important is the hazard of encountering adverse (very unfavorable) rock mass behavior in the form of spalling/rock burst, and collapse/convergence/water leaks. Such events may directly impact work safety. They may also impact construction time and tunnel costs, because of the reduction in machine utilization rate. In the worst case these hazards may ultimately threaten the feasibility of tunnel boring as excavation method.

This PhD project primarily set out to focus on TBM tunnels for railways and roads, which typically require larger diameter machines than what was used in the 1970s, 80s and 90s. Most of the results will however be applicable to all types of TBM tunnels, and many could also be relevant for D&B tunnels.
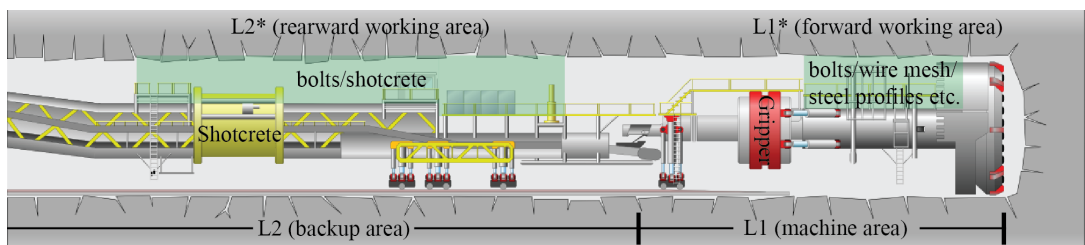
Figure 2: *Illustration of a TBM-O with the L1 (machine) and L2 (backup) areas, and the L1* and L2* working areas (the * is used to denote a working area). Rock support installation in the form of rock bolting, erection of wire mesh and installation of steel profiles is normally done in the L1* area, whilst shotcreting and supplementary rock bolting are normally done in the L2* area. Application of shotcrete with hand-held equipment can be done in the L1* area if needed. Modified after Rebel (2017)*

1.2 Aims and objectives

The aims ("*what is hoped to be achieved*") and the objectives ("*the specific steps taken to achieve the aim*") of this thesis are stated below. As described in the last section there is already significant on-going research with regards to prediction models for hard rock TBM advancement rate and tool wear. The chosen focus of this thesis is not on the boring process, but on aspects considered to be equally and sometimes more important for the planning of hard rock TBM tunnels in Norway.

Aims:

- Identify and highlight important engineering geological aspects that should be carefully considered in the planning phase of Norwegian TBM infrastructure tunnels.
- Contribute to research within the respective fields, and provide specific recommendations that project owners and consultants can apply when planning TBM infrastructure tunnels in Norway.

Objectives:

- Review earlier Norwegian TBM experience, functional requirements for Norwegian tunnels, and recent international TBM experience to assess: Geological hazards that can impact and threaten the technical feasibility of tunnel boring, and; aspects that should be considered in order to satisfy the given functional requirements.
- Use available experience, recent research within the fields, and new investigations to verify and/or to update existing knowledge, with a focus on relevance for Norwegian geological conditions and requirements. Summarize acquired knowledge in a form that is easily accessible to tunnel planning engineers.

This PhD thesis contributes within the following engineering geological fields of research, with a focus on TBM infrastructure tunnels:

- Waterproofing concepts
- Ground water drainage and settlement control
- Prediction of brittle failure (spalling and rock burst)
- Feasibility of tunnel boring through weakness zones (incl. high pressure, large water inflow)

1.3 Organization of thesis

The scientific work and hence foundation of this thesis are five papers. The work leading up to, and preparation of Paper 1 helped identify various engineering geological aspects considered important for Norwegian tunnel boring. The next papers were prepared as stand-alone contributions within the four research fields listed in Section 1.2. A brief summary of the background, aims, research methods, results and conclusions of each paper are presented in Section 2. Important clarifications and supplementary information are also listed there. The reader is recommended to read the papers in full to see all results/conclusions and to be able to take full advantage of the new acquired knowledge within each field of research. In Section 3 the relevance and implications of the new knowledge is discussed, primarily with focus on the planning of Norwegian TBM tunnels. The five papers are included in full length after the references.

Figure 3: Boring of a headrace tunnel at the Røssåga hydropower project in 2013 represented the return of TBMs in Norway. The picture is taken towards the front of the Ø7.2 m TBM-O. Photo: Øyvind Dammyr

2 Main content and findings of papers

2.1 Paper 1

Background, aims and methods:

At the time when this PhD project started in 2011 a larger scale infrastructure development, which involve long (> 5 km) traffic tunnels as well as urban utility tunneling (water, sewer, cables etc.), was on the brinks in Norway. An evaluation of the potential for renewed use of TBM in Norwegian projects was therefore considered timely and important. Paper 1 gives a brief review of the Norwegian TBM history with focus on advantages and challenges that were experienced with the excavation method. It presents recent developments within TBM technology internationally and a few case projects. The aim of the paper was to evaluate to what extent new technology has addressed former challenges, to identify important aspects that need to be given careful consideration when planning TBM tunnels in hard rock, and to evaluate the potential for future use of tunnel boring as excavation method in Norway. The primary method used in the paper was literature review, but knowledge transfer through discussions with key persons in the Norwegian and international tunneling industry, as well as TBM site-visits were also carried out.

Results and conclusions (see original paper for more information and detailed results):

Typical challenges at Norwegian projects were related to the *cutters* (spalling, fracturing, cutter-change, wear) and the *cutterhead* (stiffness/cutter group failure); *high in situ and anisotropic rock stresses in massive rock masses* (spalling/rock burst, work safety, difficulty with early rock support, gripper problems); tunneling through *weakness/fault zones* (tunnel collapse, tedious works); *water* (sometimes in connection with weak zones and other times not, high pressures, large inflows, standstills); *logistics* (transport of muck sometimes too slow); and the *TBM profile* (circular profile not considered optimal for road tunnels). In Paper 1 it is stated that most of these challenges have been properly addressed or are significantly reduced with the introduction of new technology. In retrospect, the author has realized that the above conclusion and the right column of Table 1 in Paper 1 should have been somewhat less "optimistic", especially regarding high in situ rock stresses and water. It is correct that TBM technology has developed and that even more challenging conditions can be handled, but tunnel boring is in many cases still sensitive to challenging geological conditions. It is important to carefully evaluate such aspects at the planning phase, because they will influence the choice of tunneling method and equipment, and may ultimately threaten TBM feasibility. Based on Paper 1 and these points, some important considerations for future TBM projects in Norway have been recognized, and are presented in Table 1.

Table 1: *Important considerations recognized for future TBM projects in Norway*

Planning:	More thorough and comprehensive pre-construction investigations may be needed for TBM tunnels compared to D&B, and in a case where tunnel boring may be an option it needs to be included from the conceptual stage of a project to allow for sufficient time to do the necessary investigations and evaluations.
TBM cutters/ advancement rate:	Further development of tougher cutter tools and methods to predict wear and performance in hard rock will be important in order for tunnel boring to be the preferred excavation method at future projects. Note: This is not the focus of this PhD thesis.
RAMS and LCC:	The Reliability, Availability, Maintenance and Safety and Life Cycle Costs of tunnels are currently central topics in Norway. Especially for high traffic tunnels, where interruptions to the regular operation have large impacts, it will be important to find the optimal rock- and waterproofing concepts regarding maintenance need and service life. In order to protect the environment, as well as buildings and infrastructure above and around the tunnel, it will be important to ensure that groundwater-levels and pore-pressures in the overlying soils are not influenced by tunnel construction in a way that is negative, and which may lead to settlement damages.
Spalling/rock burst:	There is need for careful planning and pre-construction investigations to identify potential hazards and to choose the appropriate excavation method, and in the case of TBM, to choose the right machine-design and equipment.
Weakness zones/water:	Same as for spalling/rock burst.

The large need for new infrastructure in Norway will raise demand for high quality, environmentally friendly and efficient solutions, and hence the potential use for TBM technology is looking more promising than in a long time. With all the new TBM projects that will be constructed or may be planned in the near future (see Sect. 1) it will be important that clients, contractors, consultants and research institutions alike continue to invest in knowledge and innovation within TBM technology.

2.2 Paper 2

<u>Background, aims and methods:</u>

The level of rock reinforcement (rock support) in Norwegian tunnels is normally adapted to and decided based on observations during tunneling. The permanent support normally consists of a single layer system of rock bolts and shotcrete, supplemented with ribs of reinforced shotcrete or cast-in-place concrete where necessary. Most Norwegian tunnels are drained structures where running water and drippings normally are led down to the invert by using a freestanding umbrella-structure of polyethylene (PE) panels covered with shotcrete, or lightweight concrete segments combined with an overlying membrane. The systems have been optimized for D&B tunnels. The need to perform maintenance on these systems (especially the PE-system) has in many cases proven to be significant, and the lifetime has proven to be limited. In addition the freestanding systems have normally required challenging inspections of the gap between the structure and the rock mass. The NNRA and NPRA functional requirements for tunnels are continuously evolving and have become stricter the last decade as a result of increased traffic, realization that maintenance is expensive, and greater demand from the public for reliable transport infrastructure (keeping schedules and avoiding delays and tunnel closures). The aim of paper 2 was to evaluate and compare the durability, life expectancy and maintenance needs of traditional Norwegian waterproofing concepts to the generally more rigid concepts seen in other European countries, and to identify the systems that are best suited to satisfy Norwegian functional requirements. The focus was on waterproofing of rail tunnels, but the results are also applicable and important for other types of tunnels. A qualitative method was used to gather maintenance data from several different tunnels, in different countries, and with different waterproofing concepts. The data was gathered by means of communication/discussions with persons/organizations with detailed knowledge about the concepts and maintenance need of each tunnel. This information was supplemented with a review of existing literature on the topic.

<u>Results and conclusions (see original paper for more information and detailed results):</u>

Ingress of water, even in small amounts, can cause large negative effects for the operation of railway tunnels. The most common consequences of seepage have been found to be ice building and frost related problems, track degradation, track settlements, electrical problems, and calcareous deposit formation. These issues can lead to the need for considerable maintenance. Table 2 summarizes how the investigated waterproofing concepts meet the NNRA functional requirements for railway tunnels and how they relate to cost effectiveness.

Table 2: *Summary of how the investigated waterproofing concepts meet the NNRA functional requirements and how they relate to cost effectiveness*

PE:	Not recommended for Norwegian TBM railway tunnels. Only excels in the low investment cost compared to some of the other concepts. Inspections needed behind the lining. Service life does not live up the requirements for [1]Technical Lifetime. Complete renovation or reconstruction after as little as 15–20 years have been reported.
LCC:	Not recommended for Norwegian TBM railway tunnels. Relatively low-cost in terms of investment and have better capabilities with regards to pressure/suction forces and fire safety than PE. Inspections needed behind the lining. Increased maintenance need due to seepage have been reported. Service life not believed to live up to [1]Technical Lifetime = 80.
European TBM waterproofing concepts (DSCC, SSSL, DSSL):	Generally the most favorable concepts to comply with the NNRA functional requirements, and hence to ensure a reliable railway operation. Higher initial investment cost, but rewarded with a long service life. The SSSL (undrained solution) has shown that the achieved level of waterproofing is closely linked to the quality of the manufacturing and construction processes, and that repairs to achieve the operating requirements seem to be necessary. This is also the case for double layer systems (quality control and methods to repairs/refurbish are paramount). Few experiences with the long-term durability of the SSSL under relatively high water pressures exist. SSSL may be best suited in urban areas where the overburden is relatively low and the sensitivity for groundwater lowering is high. The DSCC and DSSL are likely to be the only feasible options for tunnels with substantial overburdens due to the potential high water pressures.
New innovations (SSSM and IWS):	The single shell lining of fiber-reinforced shotcrete does not necessarily need to be abandoned as permanent rock support. However, durable single-shell waterproofing solutions in combination with shotcrete need to be developed and implemented. New innovations, such as the SSSM or the IWS, represent new and potentially cost effective solutions. More research is required in order to determine if these concepts can live up to the functional requirements for Norwegian railway tunnels.

Notes: PE = polyethylene-panels with shotcrete, LCS = lightweight concrete segments, DSCC = double shell cast concrete, SSSL = single shell segmental lining, DSSL = double shell - outer segmental lining and inner cast concrete, SSSM = single shell sprayed membrane, IWS = insulating waterproof shotcrete and drainage channels

[1]*Technical Lifetime (TL), according to the NNRA, is the time it takes before the waterproofing does not longer fulfill its given function. TL is expressed as the number of years, which are expected with at least 90 % probability. The mean value of TL is assumed to be at least 25 % higher than the requirement (which means 100 years for the waterproofing solution in high traffic tunnels).*

2.3 Paper 3

Background, aims and methods:

The Norwegian capital Oslo is the fastest growing capital in Europe, and there is great demand for underground excavations for trains, roads, subways, power cables, water, and wastewater. The marine clay deposits covering the city are very sensitive to pore pressure reduction, and it will be important to limit drainage to future tunnels to avoid environmental impact and settlement damage to buildings and infrastructure. Many of Oslo´s tunnels are built as drained structures, where leakage of water is mitigated using pre-excavation grouting (PEG) executed ahead of the face of advancing tunnels. The philosophy is to create a waterproof umbrella in which the tunnels can be excavated through, and which functions as a permanent measure against drainage. It has however been realized that drainage, even in small amounts can lead to significant reductions in pore water pressures during the construction phase, and in the permanent phase (if an undrained lining is not used). The aim of paper 3 was (in an Oslo specific context) to discuss the possibilities and limitations of pressurized (closed-mode) TBM-shield tunneling, traditionally used for tunneling in soil and soft rock, and evaluate if this can be an alternative to traditional drill and blast (or open-mode TBM) excavation with PEG; in order to better control drainage in the construction phase. A qualitative research method was primarily used, where much of the information was based on knowledge acquired by the author through site-visits and discussions with experienced persons within the TBM industry. The published scientific literature on the use of TBM pressurization technology in mixed and hard rock geology is limited, but published data from a few recent relevant case projects was reviewed and discussed in the paper.

Results and conclusions (see original paper for more information and detailed results):

Detailed information about the ground conditions, hydrogeology, building foundations, and state of existing infrastructure along the tunnel route should be gathered to define the necessary functional requirements needed to protect the environment above the tunnel and to achieve the desired RAMS for the finished project. First then, the process of project optimization, including the choice of tunneling method should begin. In order to avoid increased settlements in the future it will be important to consider the total future drainage potential from all tunnels and building pits (both during construction and in the permanent state). The different representatives of future infrastructure development should work closely together, and with the municipality of Oslo and the Norwegian Water resources and Energy directorate (NVE), to define and agree on the necessary basic functional requirements for future tunnels in Oslo. Table 3 summarizes some of the possibilities and limitations of pressurized shield tunneling in hard and mixed rock mass conditions.

Table 3: *Possibilities and limitations with pressurized shield tunneling in hard and mixed rock*

Possibilities of pressurized TBMs:	• Face pressure will counteract drainage towards the tunnel and mitigate pore pressure drop in the overlying soils. • Face pressure and TBM shield will better ensure stability (mitigate collapse) in areas of low rock overburden (risks: see limitations), may enable excavation through sections of soil, and in this way, enable alignments that may not have been thought possible with D&B or open-mode TBMs. Feasibility of soil excavation depends on many factors, and should be evaluated for each specific case.
Limitations of pressurized TBMs:	• The cutter-head/tools and screw conveyor and slurry circuit, of EPB and slurry machines respectively, are especially prone to excessive wear when used in hard rock. Frequent interventions and potentially very slow advance rates in hard rock conditions should be expected. • Due to the higher risk involved with hyperbaric interventions maintenance stops should primarily be planned under atmospheric pressure conditions. • Due to the above aspects closed-mode hard rock TBM excavation is normally only planned for a limited length, where pressurization is considered absolutely necessary. Other parts of the drive are normally excavated in open-mode, and hence equal to that of traditional TBM-shield operations. • Design of segment linings partially depends on the thrust force needed for the TBM to operate at the closed-mode design pressure, and this may involve a stronger/heavier lining than would be necessary for open-mode excavation (cost aspect). • There is a risk of blowout/loss of face pressure, over-excavation or loss of rock stability when passing close to foundations or operating infrastructure. There is also less flexibility with respect to PEG, forepoling/spiling, and handling of unexpected obstacles than D&B.

The most realistic alternative for Oslo at present time is considered to be the use of traditional hard rock single-shield machines with the option to convert into static-closed-mode, from which PEG can be performed until water leakage is reduced to a level acceptable for continued advance in open mode. High capacity/flexibility to do PEG will be important. These machines can offer some of the benefits of pressurized machines and the same watertight lining just behind the machine.

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2.4 Paper 4

<u>Background, aims and methods:</u>

Spalling and rock bursts are common manifestations of brittle (extension) failure in hard and massive rock masses. The high proportion of metamorphosed igneous basement rocks combined with large and anisotropic rock stresses have made these phenomena well known to Norwegian tunnelers. Operational challenges for open-TBMs in burst-prone rock are typically related to: safety; blocky rock conditions at the face; jamming of the shield from dilating/failed rock; tedious scaling and/or large rock support needs; damage to equipment from falling rock; shotcrete in the L1 area (health and practical challenges); invert heave; cleaning and transportation of rock overbreak, etc. In some cases, failure may be even more severe for shielded TBMs with segmental linings, because the rock is allowed to dilate longer due to lack of confinement from rock support. Preparation is therefore key in order to predict brittle failure and its severity, and hence to choose the appropriate excavation method and rock support equipment.

Existing research on this topic is abundant and originates through experience from mines and tunnels in countries around the world. The state of practice on brittle failure prediction have sprung out from Canada, especially through the Atomic Energy of Canada Limited (AECL) mine-by experiment, followed by similar research in Sweden (Äspö) and Finland (Olkiluoto), and in the European base tunnels. Numerous researchers have studied rock properties and rock stresses, and how they relate to brittle failure development in situ. However, most of this research has focused on rock types with isotropic behavior, and the effect of anisotropy has not been fully understood. Understanding this effect is believed to be very important in order to predict failure in Norwegian TBM tunnels. Paper 4 gives a review of existing theory and uses a Norwegian TBM tunnel in foliated granitic gneiss as a case to study brittle failure characteristics of anisotropic rock where the rock mass strength is directional dependent. The aim was to investigate how existing prediction methods perform when used for anisotropic rock, highlight important aspects that influence their predictive capabilities, and give some recommendations for future TBM planning and construction stages. The methods used in the paper range from literature review and information gathering from tunnel inspection reports, photos and earlier research, through to field mapping and rock sampling, laboratory testing of rock mechanical properties, and 2D and 3D numerical FEM modeling.

<u>Results and conclusions (see original paper for more information and detailed results):</u>

Prediction of brittle failure can be challenging and involve several uncertainties, and a well-planned investigation and testing program, and thorough interpretations are crucial for the outcome. Table 4 summarizes some of the considerations/recommendations based on the outcome of Paper 4.

Table 4: *Some of the considerations/recommendations based on the outcome of Paper 4*

Existing prediction methods can be used for foliated/anisotropic rocks as long as these conditions are met: Remark: Other features than foliation can lead to anisotropic behavior	• Rock strength must be considered to be directional dependent. The choice of strength should be based on where in the tunnel failure would be expected, which is a function of tunnel boundary stress (compression) and the angle that is formed with the foliation. • Due to rock anisotropy failure may occur in (and rotate to) other parts of the tunnel–periphery than the location of σ_{max} (*max. tangential stress*). • The range of recommended input parameters and confidence intervals of the prediction methods must be considered. These ranges allow for a relatively large spread in predicted failure depths.
In situ stress:	It is important to consider the uncertainties involved in stress determination and the potential for local stress variations (i.e. due to the presence of discontinuities). Both methods for determining the tensile strength of the rock (field and lab) should be used for a comparison when determining the maximum horizontal stress σ_H (i.e. both methods in Haimson and Cornet 2003). In situ stress should preferably be determined at different locations, and measurements during construction can be a good supplement to tests done in planning stages.
Boundary (tangential) stress:	A 3D numerical program is recommended for the determination of excavation boundary stresses, also when the tunnel is circular, if one of the principal stresses is not oriented with trend/plunge equal to that of the tunnel (plain strain conditions). Mesh around excavation boundaries in numerical models should be relatively dense in order not to underestimate the stress level.
Intact rock strength:	σ_c (*uniaxial compressive strength*) test directions (angle between compression "σ_1" and foliation) should be determined for each individual case. As mentioned above, the relevant strength depends on tunnel trend/plunge, strike/dip of foliation, and tunnel boundary stresses. Both σ_{ci} (*crack initiation stress*) and σ_{cd} (*crack damage stress*) should be reported in addition to σ_c. Preferably more than one method for σ_{ci} determination should be used for verification purposes.
Rock mass strength (σ_{cm}):	The lower bound for σ_{cm} can be taken as σ_{ci} from laboratory testing if back calculations are not possible. Verification from more projects is desired.
Uncertainties:	Due to uncertainties involved in determining σ_c and in situ stress, the use of a probabilistic methodology (i.e. Martin and Christiansson 2009) can be valuable.
Failure mitigation and DOF (depth of failure):	Confinement is essential in order to mitigate brittle failure. It has been shown by others and demonstrated in Paper 4 that even moderate support can be effective to limit DOF and bulking/displacements. Installation as early as possible is essential, and choice of TBM, and type/capacity of the rock support system, is important.

2.5 Paper 5

Background, aims and methods:

Norway has a long tradition for building subsea tunnels, which are primarily intended for road traffic. The prevailing rock mass quality is generally favorable for tunneling, but the encounter of weak and/or water bearing zones is normal, and sometimes leads to extreme challenges. A common characteristic of the zones is that they are normally of limited width and intersect otherwise competent rock mass. At the present time, long and deep subsea tunnels are being constructed (e.g. Ryfylke tunnel – ca. 14 km long, max. ca. 300 m below sea level) and planned (e.g. Boknafjord tunnel – 27 km long, max. ca. 400 m below sea level). All tunnels to date have been constructed with the D&B method, but future projects might benefit from continuous excavation. However, due to the less flexible nature of a TBM, more effort with regards to investigations and evaluations in the pre-construction phase will be required. The aim of paper 5 is to assess the feasibility of large diameter (> ca. 12 m) tunnel boring through weakness zones in deep Norwegian subsea tunnels. This paper summarizes some of the extreme challenges encountered in Norwegian subsea road tunnels, and reviews experience from international TBM projects considered relevant for future Norwegian tunnels. The focus is on geological hazards, their implications, mitigation measures and their effectiveness. The authors´ experiences with planning and construction of subsea tunnels represent an important basis for the paper. Some of the methods used are literature review, drill core mapping and mechanical tests of specimens in the laboratory, and 3D numerical FEM modeling.

Results and conclusions (see original paper for more information and detailed results):

Feasibility of tunnel boring through weakness zones in deep Norwegian subsea tunnels is directly related to the identified geological hazards in each case, and their potential implication to tunneling. The first step is always to thoroughly investigate the presence of weakness zones along the tunnel alignment, and to gather as much information about these zones as possible. Even without the detection of adverse zones, there will always be a risk of encountering individual open and permeable fractures, which can threaten feasibility. Mitigation measures such as a D&B section through zones can be possible, but it requires adverse rock mass behavior to be identified in advance. This has proven difficult on earlier projects. Paper 5 concludes that large diameter (> Ø12 m) tunnel boring through weakness zones in deep Norwegian subsea tunnels involves a high risk, and hence that it is not recommended when the water pressure is above ca. 100 m. For lower water pressures closed-mode tunnel boring may counteract some of the identified hazards, and can possibly be advantageous over D&B in certain cases. Table 5 summarizes some of the mitigation measures and their effectiveness for Norwegian TBM subsea tunnels.

Table 5: *Mitigation measures and their effectiveness for Norwegian TBM subsea tunnels*

Probe drilling and PEG:	Probe drilling is important in order to identify adverse conditions ahead of the tunnel face during construction, especially with regards to the presence of water. Possibility for probe drilling and PEG from a TBM, especially through the face, is normally limited compared to D&B. The drilling rig capacity is also normally smaller with TBM, and drilling through the face can be problematic for further boring in the case of jamming of steel drilling rods (glass fiber or plastic rods can be used). Installation of blow out preventers can be challenging and time consuming, especially when the grout umbrella needs a large number of holes.
Pilot tunnel:	A pilot can be excavated as part of the pre-investigation program, or as part of the main construction works. At some international projects a pilot tunnel has been included into the overall tunnel concept. A pilot will give unique insight on rock mass behavior, and will enable other necessary operations (PEG, ground freezing, complete excavation of the fault zone etc.) to be executed ahead of the main tunnel(s). A pilot will considerably increase the cost compared to the traditional D&B tunnel concepts used for Norwegian subsea tunnels.
Optimization of rock mass-/rock support-interaction (squeezing potential)	TBM/ground/support-interaction should be evaluated carefully for weakness zones of "substantial width", which will depend partly on rock properties and in situ stress and have to be determined using engineering judgement. 3D numerical analyses are encouraged to correctly assess the stabilizing effect of the side rock and the effect of twin tunnels on expected deformations. The above can have direct implications regarding the feasibility for and success of tunnel boring.
Closed-mode tunnel boring:	May be advantageous over D&B in some cases (may potentially replace extensive, time consuming and difficult PEG). It is important to remember the challenges related to wear and slow advance rates explained in Paper 3. Choice of tunneling method, and machine type in case of TBM, should be evaluated for each specific case. It will strongly depend on cost effectiveness since preparation for closed-mode TBM tunneling normally would imply the use of a conservatively designed gasketed concrete segmental lining (undrained solution) for the whole tunnel.
Ground freezing:	Requires space for drilling of freeze-holes, space for the freezing installations, and space for a permanent (normally undrained) waterproof lining. It is important that adverse conditions are identified ahead of the face, and before escalation of the excavation problems. A "safe" distance/barrier between the tunnel and the zone in question would normally be required. Ground freezing should not be relied on as a method that can be used to counteract all extreme challenges, and/or where other mitigation methods are unsuccessful.

3 Discussion

3.1 Functional requirements

The basis for all tunnel planning starts with defining the functional requirements. As mentioned earlier the RAMS and LCC may vary based on the type of tunnel, its purpose, its location etc. In Norway the functional requirements have been constantly evolving the last decades, and especially in urban environments and for high traffic infrastructure projects they have become a topic of debate. This is largely because the functional requirements relate to the design of the final tunnel lining, and hence that they have relatively large implications on construction costs. The debate concerning costs is often related to the use of concrete. Concrete is also one of the major contributors to CO_2 emissions from tunnel construction, and there is an increased focus/demand on new projects to limit emissions. Figure 4 shows the rock-grout-segment interface of the single-shell segmental lining in the Hallandsås tunnel. The outer diameter of the tunnel is 10.6 m and the inner diameter ca. 9 m. With 540 mm thick segments there is an average annular gap of approximately 0.25 m that needs to be filled with grout, and the resulting average concrete thickness is ca. 0.8 m.

The author wishes to highlight that the "right" choice of functional requirements is key to a safe, reliable, and cost effective infrastructure in the future, and that more research and cooperation within and between the different bodies of traffic administration in Norway will be important in order to define a more uniform practice for future projects. It is believed that the relationship between functional requirements and social economical aspects/implications are not fully understood, and it is important to realize that today's infrastructure will become more essential, and hence will be increasingly more sensitive to interruptions in the future.

Once the functional requirements are in place the work on how they can best be implemented can begin. If not already defined, the first step will be the decision of tunneling concept (e.g. the choice of a single or twin-tube tunnel, escape tunnels, emergency caverns etc.), and then the concept for waterproofing of the tunnel. In a case where closed-mode tunneling is considered as the only feasible option the choice of tunneling method may have to come at this early stage in the process. For most Norwegian scenarios, however, the choice of tunneling method should not be rushed before several engineering geological aspects (including the aspects of this thesis) have been considered carefully. For more information on the selection of tunneling method see section 3.3.

Figure 4: *Observation of the rock-grout-segment interface of a single-shell segmental lining waterproofing concept (left). The picture was taken in a cross passage between the western and eastern tube of the Hallandsås tunnel in 2011. The segment thickness is 540 mm. The finished waterproof lining in the western tunnel is also shown (right). Photos: Øyvind Dammyr*

3.2 Relevance of results for the planning of TBM tunnels in Norway

The Norwegian adopted European Norm (EN) documents Eurocode 0 (NS 1990) and Eurocode 7 (NS 1997a-b) are central in the planning of civil engineering (in this case rock engineering) works in Norway. Eurocode 0 defines the principles and requirements for safety and serviceability, describes the basis of design and verification and gives guidelines for related aspects of structural reliability (NS 1997b). Based on the Consequence class/Reliability class (CC/RC), and the difficulty level of the works, Eurocode 0 defines the required level of quality verification of the planned design and construction process (this is typically an *internal company* verification or *third party* verification). Eurocode 7 is intended to be used in conjunction with Eurocode 0 and to be applied to the geotechnical aspects of design. It is concerned with the requirements for strength, stability, serviceability and durability of structures, and governs the types of methods and parameters to be used in the planning of rock engineering works. Based on a *geotechnical category level* (ranging from 1-3), it suggests levels of testing and reporting according to the defined category level. According to NBG (2011), which is a guideline for the Norwegian rock engineering community on how to use/interpret Eurocode 7, the *geotechnical category level* can be found as a function of the CC/RC and the difficulty level of the project. An important aspect of Eurocode 7 is that both the short-term and long-term situations shall be verified. NBG (2011) gives some examples of design situations that need to be verified, where the ones most relevant for this thesis are:

- Gravitational block fall
- Tunnel collapse in low consolidated weakness zones
- Spalling/rock burst
- Large water leaks
- Mobilization of swelling pressures from swelling clay minerals
- Squeezing in overloaded and deformable rock masses

The NPRA handbook N500 (SVV 2016a) with guideline document V520 (SVV 2016b) governs the planning and construction of road tunnels in Norway. The NNRA and other public administrations, which are responsible for traffic infrastructure, also use these documents to varying extents in their planning processes. The N500 states that the geological report following a tender should contain an interpretive part evaluating the geological conditions along the tunnel. Some of these shall be:

- Geological boundaries, tectonic structures and weakness zones with possible intersection points along the tunnel. Zones that can lead to excavation problems shall be discussed

- Soil conditions - consequences relating to surface slides, settlements and the environment. Mitigation measures shall be discussed

- Hydrogeological considerations – wells and water magazines, and requirements on maximum water leakage to avoid pore pressure reduction. How this impacts tunnel excavation shall be discussed

- Likelihood of encountering water that can create excavation problems, and the likely amount of pre-excavation grouting. All potential problem areas shall be discussed together with mitigation measures

- The probability of encountering rock stress problems. The impact on tunnel excavation shall be discussed. Relates to challenges with both high and low stress situations

- Uncertainties or special risk scenarios. All uncertainties shall be described

In order to satisfy the requirements of the above documents, it is important to have sufficient information about the ground conditions in order to do the required evaluations/verifications. Due to the lower flexibility of tunnel boring compared to D&B this will be increasingly important for future projects where TBMs are being considered. In this respect, it must be emphasized that ground-investigations and careful interpretations are the key to minimize risks, and hence also to mitigate disputes between the client and the contractor. The papers of this thesis contribute towards the evaluation of many of the above aspects, and hence they can aid tunnel engineers in the planning process of TBM tunnels.

3.2.1 Waterproofing concepts

The choice of concept is closely related to the tunneling method. A path towards the use of more heavy concrete structures will even out the cost difference between the typical Norwegian D&B water proofing concepts and the concepts normally adopted for TBM tunnels internationally. There is certainly a need for more research on the waterproofing of Norwegian tunnels, and a path towards heavier concrete concepts is not necessarily the optimal solution. However, there are few well-documented alternatives to the European concepts that seem to satisfy the strict functional requirements of today's modern infrastructure.

It is relatively difficult to obtain detailed maintenance and cost data from different tunnel projects. A publicly available database with such information from Norwegian infrastructure tunnels would probably encourage more research on the topic. It would enable independent researchers to come up with new ideas, and enable more detailed and reliable analyses, including cost benefit analysis of existing functional requirements. More research on this topic is considered very important because of the large-scale infrastructure development that is currently ongoing in Norway.

3.2.2 Groundwater drainage and settlement control

The choice of concepts for PEG and permanent waterproofing today varies between projects. For the city of Oslo in particular, there is a need to define a more uniform practice to avoid groundwater drainage and lowering of porewater pressures. Decisions and acceptance towards a common safety level with corresponding concepts should be discussed in the industry and between decision makers. Without agreement on best practice, disputes regarding settlement damages will probably increase in the future, and it will be difficult to hold one specific project responsible for a specific damage.

The experience with pressurized TBMs in hard rock conditions is limited, and development of machines will be necessary in order to excavate longer stretches of hard rock in closed-mode. Shielded hard rock TBMs with the ability to close the cutterhead relatively quickly (and for example pressurize the face with air) in the case of unacceptable water leakages or pore pressure drops seem to be the most realistic option for hard rock today. TBM excavation with this approach will give a tight lining directly behind the machine, but it will probably still require continuous PEG during tunneling. Grouting efficiency from a TBM is normally lower than with D&B. The same is true with regards to the flexibility in terms of drilling positions. Future development of grouting technology, especially towards higher capacity on the TBM, will be important.

3.2.3 Spalling and rock burst

Only a limited database of carefully documented case histories on brittle failure exists, especially for anisotropic rock, and further studies (especially case projects) will be very valuable for predictions to become more accurate. In order for predictions to have real value careful pre-construction investigations in the form of rock stress measurements and testing of rock mechanical properties have to be done. There are many planned (and already constructed) tunnels and caverns in metamorphic igneous rocks in Norway, and hence there are many opportunities to carry out research on brittle failure in anisotropic rock. More accurate predictions will be valuable for future TBM infrastructure tunnels in Norway, and findings will be important contributions to the international scientific research within the field.

Typical rock support types to control spalling and rock burst on a TBM-O consists of a combination of wire mesh and rock bolts directly behind the cutterhead and application of shotcrete, ideally in the L1 area for early confinement, but most practical in the L2 area. Bolts should preferably be yieldable, yet high capacity, and be effective immediately upon installation. The short cutterhead shield inhibits rock support to be installed before ca. 5 m behind the face, and this is a weakness compared to D&B where it is possible to excavate with reduced round lengths and apply shotcrete already at the face. The use of shotcrete in the L1 area of a TBM may also be problematic

with regards to health and safety, wear on the machine, and time consuming cleaning. Thin spray-on lining materials (TSLs) may contribute to addressing these problems. Archibald and Dirige (2006) reported that the majority of TSL products currently available are highly effective for mitigating rockburst damage, with the ability to substantially deform and constrain fragment or loose rock ejection created by energetic spalling. TSLs have rapid set, tenacious adhesion, high deformability and high tensile strength, and Archibald and Dirige reported that they were able to generate support performance equivalent to that of bolts and welded wire mesh, and thin shotcrete linings. More research into the use of TSLs is encouraged.

3.2.4 Weakness zones and water

The geological hazards related to excavating through weakness zones may be independent of excavation method, but their impact and available mitigation measures can be quite different. Due to the lower flexibility of tunnel boring it may involve a higher risk than D&B, and future projects tend to be planned in more extreme environments. It is therefore important to have detailed knowledge about the ground conditions, and it may be necessary to use other methods and more detailed analyses in the planning phase of TBM tunnels than what has normally been done for Norwegian projects. This is especially important for wide and potentially water bearing weakness zones such as often found in subsea tunnels, since deformation and water bearing characteristics of the zones may threaten TBM excavation feasibility. Such zones should be investigated in detail with regards to sqeezing potential and how deformations will impact the TBM operation. This is especially important for the planning of twin-tube tunnels, where one needs to analyze how the distance between and the excavation of the two tunnels impact each other, in order to ensure that the planned rock support measures will have sufficient capacity. It will also be important to evaluate the possibilities/limitations with regard to PEG and face pressurization, since the feasibility of tunnel construction with TBM ultimately relies on the function/effectiveness of these mitigation measures.

In order for new extreme projects, such as the Gibraltar strait railway tunnel, to be realized pressurized TBM technology and lining design will need to develop further. Lombardi et al. (2009), by taking into consideration the long-term loads (determined by water pressure, the residual effective pressure and the swelling of the rock mass), have estimated that the total natural stress state that can act on the lining of the Gibraltar strait tunnel is 7.4 MPa. They reported that this will result in the need for 0,8 m to 1,2 m thick segments with concrete strengths in the order of 100 MPa to be installed in the service, safety, and in the rail tunnels. Though the technical aspects may be resolved in the future, the decisions to go forward with these extreme projects will probably relate strongly to cost factors and political will.

3.3 Selection of tunneling method

Selection of tunneling method should be based on a thorough evaluation of functional requirements, tunnel and waterproofing concepts, engineering geological hazards, and aspects such as construction cost and time. As discussed by Spiegel and Schneider (2011) and others the selection of method firstly depends on the technical feasibility relating to the geotechnical conditions, and secondly on economic criteria (cost and risks) and project specific aspects (e.g., tunnel length, access points, construction time). Lemmerer (2011) underlines that each method of tunneling (conventional and mechanical) has its particular advantages and also disadvantages, and that it is the task of the designer and the tunnel engineer to evaluate and compare these and then select and implement one method. The example in Figure 5 illustrates how the four main aspects covered in this thesis can be used in the process of evaluating technical feasibility of different excavation methods for a railway tunnel. The different parts of the figure is discussed below.

Basic project information: Even with only initial information at hand, it is possible to do preliminary assessments, and even eliminate certain concepts and tunneling methods. In the example of Figure 5 one may argue that TBM-O may be advantageous over D&B and TBM-S with regards to performance, but more information is needed in order to evaluate how suitable the granitic gneiss formation is for boring (not the focus of this thesis, but will basically include specific testing of rock parameters and mapping of fracture sets and their frequency). The potential for high water pressures up to 1000 m rules out the use of an undrained waterproofing concept and the use of a TBM built with a bulkhead for face pressurization (hence the "non feasible" tag for this type of TBM). The potential for water inflow and unstable ground in fault zones may be a disadvantage for TBMs due to the typical lower flexibility of the method, and limitations with regards to pre-excavation grouting compared to D&B. A TBM-S will probably be better suited than a TBM-O with regards to frequent intersection of weak zones (the photos shown in Figure 6 are taken at a TBM-O and TBM-S).

Functional requirements and tunnel concept: Based on the functional requirements, the tunnel concept (including waterproofing concept) can be chosen. It can be seen in the figure that the use of one larger tunnel (with a parallel escape tunnel) is not considered optimal from an operational and maintenance perspective since both tracks would have to be closed in the order to do maintenance work. There is also a "feedback process" here from the evaluation of geological hazards in the next step, where it is was found that smaller cross sections are preferred due to the potential for spalling/rock bursting in massive rock and deformation in fault zones. The optimal tunnel concept is therefore a twin-tube tunnel with sufficient distance between tubes to reduce potential deformations in weak zones. The only waterproofing concepts considered to satisfy the

functional requirements are the double-shell concepts. For D&B and TBM-O this will be an outer shell of shotcrete for rock mass stability, and an inner shell consisting of a drainage layer/membrane and cast-concrete for waterproofing. For TBM-S this will be an outer segmental lining for rock mass stability, and a similar inner shell as the one mentioned above for waterproofing.

Geological hazards: The evaluation of possible geological hazards, their severity, and risks is very important, especially for TBMs, because this relates directly to the technical feasibility of the method. For these evaluations to be sufficiently accurate, detailed information on rock mechanical properties may be required. Hence, the earlier the investigations and testing are done, the earlier one can evaluate and conclude on technical feasibility. A timely conclusion will be important since there may be an important "feedback process" involved in the choice of tunnel concept. The evaluations do not necessarily relate to a specific planning phase, and they should ideally be performed whenever new and more detailed/relevant data becomes available. In the current example the geological hazards are not location specific, but normally evaluations will be done for different sections of the tunnel alignment. The risk of spalling/rock burst behavior is regarded to be high due to the σ_{max}/σ_c ratio (found from investigations/testing/analysis). The lack of early rock mass confinement on a TBM-S (not before behind the shield and after filling of the annular gap) leads to a risk of large bulking displacements, which could involve jamming of the machine. The use of a TBM-S is therefore not considered feasible in this specific case. Analysis of squeezing potential resulted in a recommendation towards the use of two smaller tunnels with sufficient distance between tubes (minimum two diameters distance). Some friction along TBM shields is expected, but is not believed to threaten feasibility. Fault zone material is considered to be weak, but consolidated and with a high proportion of clay minerals, so for the most part the zones are thought to be dry. If collapse development should occur it is believed to be gradual. It is considered more likely with only local fallout, which is believed to be in favor of the TBM-S compared to the TBM-O. The risk of large water inflow at high pressure adjacent to fault zones is considered to be higher for tunnel boring compared to D&B, due to normally having less flexibility and capacity with regards to pre-excavation grouting.

Technical feasibility: Once the investigations and evaluations are considered sufficient, one can draw conclusions about the technical feasibility. Then the secondary aspects (including cost and construction time) can be evaluated before making the final choice of tunneling method. In the given example both D&B and TBM-O are shown as feasible methods, although TBM-O is considered to involve a higher technical risk. In such a case, both methods could be tendered alongside each other. In practice the final choice of method may depend on factors such as the

contractor's experience with the different methods and the availability of tunneling crews and delivery times for machines and equipment.

Feasible | **Intermediate** | **Not feasible**

Evaluation based on initial information			
D&B	**TBM-O**	**TBM-S**	**[1]TBM-S w/bulkh.**
			Not feasible
	?	?	?

Basic project information
- Train tunnel: 15 km (two tracks) (no intermediate access possibilities)
- Overburden (w. press.): 300-1000 m Partly below open water (larger lake)
- Rock type: Granitic gneiss
- Tunnel to cross many fault zones

[1]*With bulkhead to allow for pressurization*

Functional requirements
- No drippings on rails
- Low downtime for maintenance
- 100 years servicelife for WP concept
- WP must handle large press./suction forces
- Lowering of water-level in lake prohibited
- Emergency escape passages every 250 m

Tunnel concept

Single-tube double track tunnel (*110 m² for D&B and 133 m²/Ø=13 m for TBM*) **with parallell escape tunnel:** Not chosen from an operational and maintenance perspective (in order to do maintenance both tracks must be closed). The evaluation of geological hazards also showed that a larger diameter tunnel involved a higher risk.
Twin-tube single track tunnels (*65 m² for D&B and 79 m²/Ø=10 m for TBM*): Considered optimal. The distance beween tubes should be minimum two diameters.
Waterproofing: Double shell cast concrete (D&B and TBM-O) or double shell segmental lining (TBM-S) considered the only concepts to meet the functional requirements.

"Feedback": Due to potential for spalling/rock burst and squeezing two smaller tunnels with sufficient distance (min. 2D) should be used.

Investigations
Geological hazards
Evaluations

Evaluation considering geological hazards			
	D&B	**TBM-O**	**TBM-S**	**Comments:**

Hazard	Description	D&B	TBM-O	TBM-S	Comments:
Spalling/ rock burst	High potential:$\sigma_{max}/\sigma_c>0.6$ DOF>1 m (bulking displ.)			Not feasible	TBM-S high risk of jamming
Squeezing	[2]Strain $\varepsilon < 2\%$ (unsupp.) Wall deformation < 10 cm				Friction along shield expected
Tunnel collapse	If occurring thought to be gradual. Most likely restricted to local fallout				Shotcrete in L1 area not optimal for TBM-O
Sudden large water inflow	Core drillings indicate high risk of encountering large water inflow at high pressure adjacent to faults				TBMs less capable with regards to pre-excavation grouting

[2]*Calculated for the smaller tunnels*

Technical feasibility

	D&B	TBM-O	TBM-S	Comments:
			Not feasible	D&B and TBM-O feasible

Figure 5: *Example showing how the aspects covered in this thesis can be used in the process of evaluating technical feasibility of different tunneling methods, in this case for an imaginary double track train tunnel*
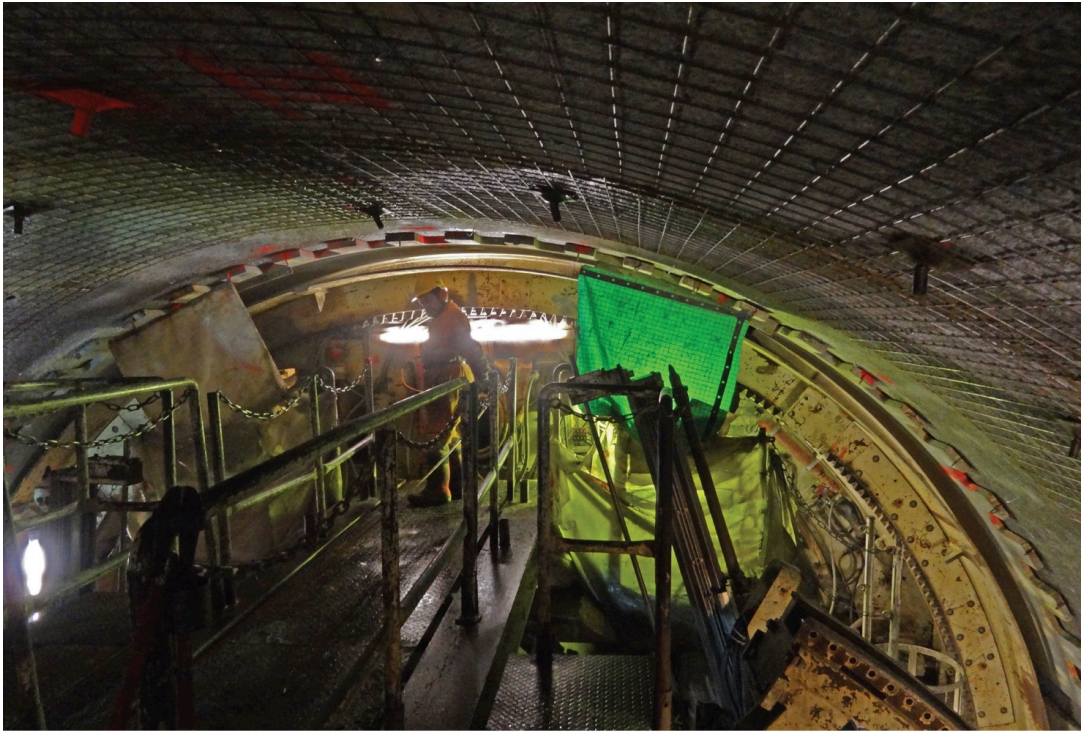
Figure 6: View of the L1* (forward working area) on the TBM-O at the Nant de Drance hydropower project in Switzerland (top) and the ring building area within the shield of the TBM-S at the Hallandsåsen tunnel project (bottom). Photos: Øyvind Dammyr

4 Conclusions

Following the objectives listed in Sect. 1.2 the aims set out for this thesis have been answered through five research papers. Through the work summarized in a compressed form in Section 2, important engineering geological aspects that should be considered in the planning phase of future Norwegian infrastructure TBM tunnels have been identified and highlighted.

- *Investigations:* Earlier experience with tunnel boring in Norway is extensive, but the tunnels were generally of a smaller diameter than modern infrastructure tunnels. The manifestation of engineering geological hazards are normally more severe in larger tunnels, and investigations and evaluations of aspects covered in this thesis will be increasingly important for the planning of future TBM tunnels of larger diameter. It is important that the necessary investigations are done early enough in the planning process in order to be able to do thorough evaluations of technical feasibility. If there is a lack of investigations, or there is not enough time for sufficient evaluations, tunnel boring may be considered too high of a risk. The potential benefits with the use of TBM will then be lost, since it is not likely that the contractor or the client is willing to take this extra risk.

- *Waterproofing concepts:* The TBM waterproofing concepts found to comply with the NNRA functional requirements, and hence to ensure a reliable railway operation (also important for other types of tunnels such as for roads and subways), were the typical European concepts (double shell cast concrete, single shell segmental lining and double shell segmental lining). The achieved level of waterproofing is closely linked to the quality of the manufacturing and construction processes, and repairs seem to be necessary to achieve the operating requirements.

- *Ground water drainage and settlement control:* In order to avoid increased settlements and damage to buildings and infrastructure in the future (the focus has been on the city of Oslo, but this will also be important for other cities and urban areas with settlement sensitive soils), it will be important to consider the total future drainage potential from all tunnels and building pits (both during construction and in the permanent state) when defining the functional requirements. Pressurized TBMs (EPB and Slurry) have limitations with regards to high wear when operated in closed-mode in hard rock. At present time the use of these machines are not considered to be realistic in Oslo. The most realistic alternative to D&B is considered to be the use of traditional hard rock TBM-S machines built with a bulkhead, and hence with the option to convert into static-closed-mode, from which pre-excavation grouting can be performed until water leakage is reduced to a level acceptable for continued advance in open-mode.

- **Prediction of brittle failure:** Existing prediction methods can be used for foliated/anisotropic rocks as long as the following conditions are considered. **1.** Rock strength must be considered to be directional dependent, and should be based on where in the tunnel failure would be expected. This is a function of tunnel boundary stress (compression) and the angle that is formed with the foliation. **2.** Due to rock anisotropy failure may occur in (and rotate to) other parts of the tunnel-periphery than the location of σ_{max} (maximum tangential stress). **3.** The range of recommended input parameters and confidence intervals of the prediction methods must be considered. These ranges allow for a relatively large spread in predicted failure depths. Other aspects/recommendations relating to the prediction of brittle failure in anisotropic rock have been addressed in this thesis (Table 4). For example, a 3D numerical program is recommended for the determination of excavation boundary stresses, also when the tunnel is circular, if one of the principal stresses is not oriented with trend/plunge equal to that of the tunnel (plain strain conditions).

- **Feasibility of tunnel boring through weakness zones:** Feasibility has been investigated for what is considered the most challenging cases, subsea tunnels. Feasibility is directly related to the identified geological hazards in each case, and their potential implications for tunneling. The first step is always to thoroughly investigate the presence of weakness zones along the tunnel alignment, and to gather as much information about these zones as possible. The different mitigation measures and their effectiveness for Norwegian TBM subsea tunnels has been summarized in Table 5, and it has been shown that a TBM is often less flexible than D&B. Large diameter tunnel boring (> Ø12 m for road tunnels) through weakness zones in deep Norwegian subsea tunnels has been found to involve a high risk, and hence is not recommended when the water pressure is above ca. 100 m. It is important to remember that the largest water leaks can be related to individual open fractures and not necessarily fault zones. For lower water pressures, TBMs with the possibility to operate in closed-mode may counteract some of the identified hazards, and can possibly be advantageous over D&B in certain cases.

5 Suggestions for future work

In addition to the suggestions already discussed in chapter 3, the following aspects are considered to need further investigation:

- In addition to research into the cost aspects of operating tunnels (maintenance-/service life-cost), which is important for the choice of functional requirements and tunnel concepts, research into new and alternative waterproofing concepts is considered very important.

- In urban areas such as Oslo, future project owners should work together with administrative bodies to define the best practice for construction processes and waterproofing concepts, in order to ensure that ground settlements are minimized and to limit future legal disputes. For the future use of TBMs, continued development within flexibility and capacity for pre-excavation grouting will be important. A TBM-S with grouting capacity/efficiency similar to that of D&B could be advantageous over D&B due to the possibility to install an undrained lining just behind the machine in an industrialized process.

- In order to predict brittle failure with increased accuracy, more research is needed on anisotropic rocks. Investigations and testing of rock mechanical properties, and measurements of in-situ stresses, should be encouraged at projects that experience stress problems (both D&B and TBM projects). In this way observations of overbreak can be correlated with rock mechanical data, and results compared with existing research.

- In order for TBMs to be used in increasingly challenging geology, such as the crossing of subsea weakness zones, it will be important to have continued development towards higher capacity/efficiency and flexibility for pre-excavation grouting, and increased performance with regards to face pressurization. It is believed that the option to drill longer core-drilling holes will be important in order to make TBM a potential choice for future subsea projects. This would ensure a longer proportion of the alignment to be covered by a direct method, and provide a better basis for comparison with indirect methods such as refraction seismic.

- The use of 3D numerical models should be encouraged in order to analyze rock mechanical problems in a more realistic way, and hence with better accuracy. The development of faster and more user-friendly computer programs will be important.

On my way into the TBM at a site visit to the Røssåga hydropower project (Northern Norway) in September 2014. Photo: Shannon Martin

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Paper 1

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Paper 2

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Paper 3

Pressurized TBM-shield tunneling under the subsidence sensitive grounds of Oslo: Possibilities and limitations

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Abstract: The marine clay deposits covering the bedrock under the Norwegian capital Oslo are very sensitive to pore pressure reduction. Existing tunnels under the city are draining the ground to varying extents, which have led to settlement damages to buildings and infrastructure. Oslo is the fastest growing capital in Europe, and there is great demand for underground excavations for trains, roads, subways, power cables, water, and wastewater. It will be very important to limit drainage into new tunnels during the construction phase and in the permanent state. This paper discusses the possibilities and limitations of pressurized (closed-mode) TBM-shield tunneling as an alternative to traditional drill and blast (or open-mode TBM excavation) with pre-excavation grouting. Earth pressure balance and slurry shield machines with gasketed precast concrete segmental linings, which are normally used in soil and soft rock tunneling, may give superior drainage control compared to non-pressurized (open) methods; but these machines have challenges with high wear when used in hard rock. The use of traditional hard rock single-shield machines built with the option to convert into closed-static-mode is another, and probably a more realistic alternative, which should be considered in order to better control drainage during the construction phase.

Keywords: TBM; pressurization; hard-rock; EPB; slurry; drainage

1 Introduction

It is well established that drainage as a result of underground infrastructure and building pit construction in the city of Oslo, can cause surface subsidence. Porewater pressure reduction in the marine clay deposits covering the city leads to compaction, which for buildings with soil foundations, results in settlements that can go on for decades and give massive damages. This is not a direct problem for buildings with bedrock pile foundations, but settlements of the surrounding ground lead to problems for the local infrastructure (roads, tramlines, walkways, sewage and water pipes etc.) in between the buildings (Fig. 1). Rapid population growth in Oslo raises the demand for new underground transportation and technical infrastructure such as the Fornebu metro line, new central metro tunnels, new train tunnels, tunnels for power cables, tunnels for water transfer and waste water. Future projects may benefit from the possibilities of pressurized TBM-shield excavation in order to reduce and better control drainage in the construction phase. Gasketed single-shell concrete segmental linings, which are typically used in combination with these TBM machines, will also leave behind a permanent waterproof undrained structure. The maximum overburden is seldom more than ca. 100 m, and normally below ca. 50 m for tunnels in Oslo, and this type of segment lining has proven its function under these water pressures. As opposed to the drained structure of many tunnels in Oslo, the undrained structure can better preserve the natural hydrogeological situation without the use of artificial methods (i.e. the use of water infiltration wells to counteract porewater pressure reduction above tunnels). This paper covers some of the possibilities and limitations, in an Oslo-focused context, for the excavation method in question.

Fig. 1: Settlements induced by construction activity causes problems for the tramline in the street of Storgata (October 2015). Photo: Ø. Dammyr

2 Geological setting

The geology of the Oslo region involves a transition from Precambrian basement rocks to Cambro-Silurian sedimentary rocks, which were later intruded by Permian volcanic and plutonic rocks. A thin layer of conglomerates is found on top of the basement rocks, overlain by thicker alternating shale and limestone layers (Fig. 2). These layers were later folded and faulted during the Caledonian orogenesis. The fold axis direction was ca. NE-SV and orthogonal to the thrust faulting (compression) direction, which was ca. NV-SE. In Carboniferous - Permian time extension led to the formation of N-S trending faults, and volcanic and plutonic rocks cut through the rock mass and formed larger massifs on top of or within the Cambro – Silurian rocks. These rocks today form the elevated terrain of "Nordmarka", a much loved recreational area for the people in Oslo. As a result of this activity, many smaller (meter scale) and wider (10+ meter scale) intrusion dykes are found within the Cambro –Silurian rocks. Thinner dykes are typically composed of diabase and meanite, whilst the thicker ones are composed of syenite.
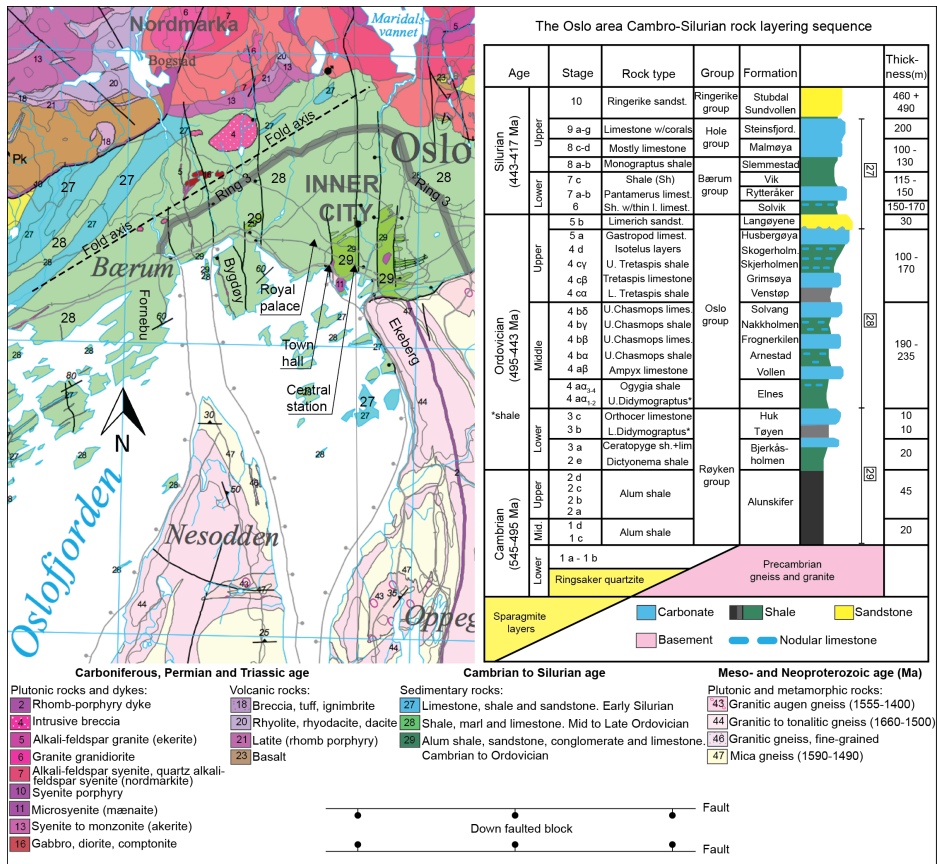
The Oslo area Cambro-Silurian rock layering sequence

Age	Stage	Rock type	Group	Formation		Thickness(m)
Silurian (443-417 Ma) Upper	10	Ringerike sandst.	Ringerike group	Stubdal / Sundvollen		460 + 490
	9 a-g	Limestone w/corals	Hole group	Steinsfjord.		200
	8 c-d	Mostly limestone		Malmøya		100 - 130
	8 a-b	Monograptus shale		Slemmestad		
Lower	7 c	Shale (Sh)	Bærum group	Vik		115 - 150
	7 a-b	Pantamerus limest.		Rytteråker		
	6	Sh. w/thin l. limest.		Solvik		150-170
Ordovician (495-443 Ma) Upper	5 b	Limerich sandst.		Langøyene		30
	5 a	Gastropod limest.		Husbergøya		100 - 170
	4 d	Isotelus layers		Skogerholm.		
	4 cγ	U. Tretaspis shale		Skjerholmen		
	4 cβ	Tretaspis limestone		Grimsøya		
	4 cα	L. Tretaspis shale		Venstøp		
Middle	4 bδ	U.Chasmops limes.	Oslo group	Solvang		190 - 235
	4 bγ	U.Chasmops limes.		Nakkholmen		
	4 bβ	U.Chasmops limes.		Frognerkilen		
	4 bα	U.Chasmops shale		Arnestad		
	4 aβ	Ampyx limestone		Vollen		
	4 aα3-4	Ogygia shale		Elnes		
	4 aα1-2	U.Didymograptus*				
Lower *shale	3 c	Orthocer limestone		Huk		10
	3 b	L.Didymograptus*		Tøyen		10
	3 a	Ceratopyge sh.+lim		Bjerkås-holmen		20
	2 e	Dictyonema shale	Røyken group			
Cambrian (545-495 Ma) Upper	2 d / 2 c / 2 b / 2 a	Alum shale		Alunskifer		45
Mid.	1 d / 1 c	Alum shale				20
Lower	1 a - 1 b	Ringsaker quartzite		Precambrian gneiss and granite		

Legend: Carbonate, Shale, Sandstone, Basement, Nodular limestone; Sparagmite layers

Fig. 2: Geology of the Oslo area. Geological map (left) with explanation (bottom) compiled from Lutro and Nordgulen (2008). Cambro – Silurian rock layering sequence (right) redrawn and translated based on Bjørlykke (2004)

The Cambro-Silurian sediments are today exposed around the city of Oslo. Weaker fault rocks and shales have often been eroded to a lower level than the stronger limestone layers, leaving deep trenches that were later filled with marine sediments during/after the last glaciation in Holocene. The terrain has risen more than 200 m since then and washout of salt in these clays has resulted in many areas where the clay structure is extremely weak (quick clays), and may collapse (becomes liquid) if stirred or overloaded. The rock surface is typically uneven, and depths from the terrain to the rock surface can vary significantly over short distances. Fig. 3 shows a shale-

limestone outcrop, and a flatly lying syenite intrusion below shale at the face of the

Løren road tunnel during construction.

Fig. 3: Alternating shale and limestone layers of stage 3a (left) from Nærsnes southwest of Oslo. Photo: Kjetil Lenes (CC BY-SA 3.0). Syenite intrusion and shale of probably stage 4b exposed in the Løren road tunnel (right) north of the city center. Pregrouting can be seen ongoing. Photo: From Høien and Nilsen (2014)

3 Tunneling experience in Oslo

There is extensive experience from tunneling in the rock formations under the central

city of Oslo and the surroundings. Tunnels for subways, roads, trains, power lines,

water and wastewater have been excavated using conventional drilling and blasting

(D&B). During the 1970s a 40 km network of smaller diameter (3 – 3.5 m) sewer

tunnels were also excavated through the Cambro-Silurian sediments with the use of

open TBMs. The conditions for TBM advancement rate and cutter wear in the

Cambro-Silurian rocks are generally considered to be favorable, whilst the Permian

rocks are somewhat tougher to bore (Table 1 and 2).

Degree of fracturing and fracture-orientation in relation to the tunnel drive is a

major component that can influence TBM advancement rate. Degree of fracturing

should be expected to vary, but is generally thought to be favorable. Fig. 4 shows part

of a geological log from the Løren road tunnel. The RQD value varies between 60 and

80, and the number of fracture sets is typically 3, sometimes with additional sporadic

fractures. Q – values outside fracture and fault zones typically vary between ca. 1 – 10

for tunnels in Oslo (author's experience). The log of Fig. 4 also shows grout

5

consumption for each of the overlapping grout umbrellas, which were performed

systematically during tunneling.

Table 1: Some numbers reported (by SINTEF 2006) for the properties of the different rock types in the Oslo region (BWI also included in relation to D&B). As shown, large variations within the different rock types should be expected. See Table 2 for categorization of the NTNU drillability indices

Rock type	Drilling Rate Index (DRI)	Bit Wear Index (BWI)	Cutter Life Index (CLI)	UCS (MPa)
Shale	42-85	7-32	44-197	25-119
Sandstone	58	34	6	130
Limestone	43 - 58	10 - 21	69 - 136	40 - 91
Diabase	28 - 39	7 - 39	16 - 138	66 - 178
Mænaite	32 - 54	16 - 52	105 - 110	-
Syenite porphyry	36 - 52	15 - 34	10	125 - 183

Table 2: Category intervals for drillability indices, after Bruland (1998)

Category	DRI	BWI	CLI
Extremely low	- 25	- 10	< 5
Very low	26 - 32	11 - 20	5.0 - 5.9
Low	**33 - 42**	**21 - 30**	**6.0 - 7.9**
Medium	**43 - 57**	**31 - 44**	**8.0 - 14.9**
High	**58 - 69**	**45 - 55**	**15.0 - 34**
Very high	70 - 82	56 - 69	35 - 74
Extremely high	82 -	70 -	≥ 75
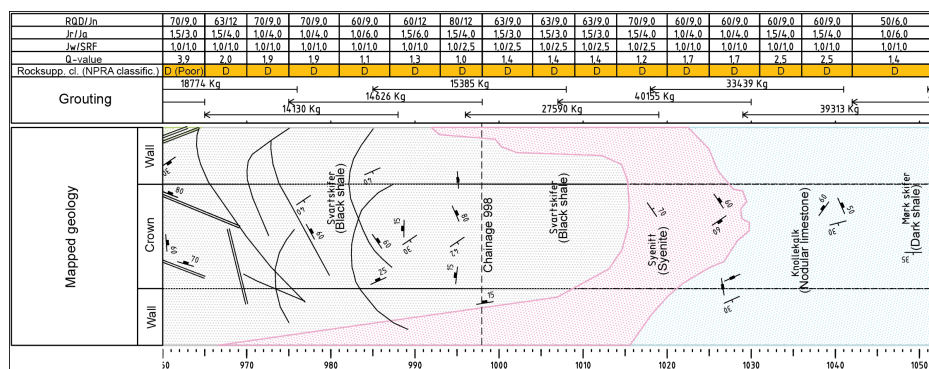
Fig. 4: Part of a geological log from the Løren road tunnel (tunnel A). Translated into English and used with permission from the National Public Roads Administration (NPRA). Chainage from which photo in Fig. 3 (right) was taken is shown

3.1 Rock mass stability

Rock mass surface weathering, low rock overburden with large variations over short

distances, and crushed/fault zones are typical concerns for tunnel planners in Oslo.

Most of Oslo's transport infrastructure lies relatively close to the surface, and there is

often a need to plan tunnel alignments through potentially weathered rock mass.

Norway also has areas (including the Oslo area) that have been influenced by deep weathering, typically relating to fault and fracture zones. During the Mesozoic, the climate was tropical and weathering products can still be found deep (+ 100 m) in the rock mass. Other engineering geological challenges are related to intrusion dykes and their adjacent rock mass, which can be fractured/crushed and highly permeable, and lava tuff layers (found in some formations) where alteration products such as swelling clay materials can give rise to stability issues. It has normally been possible to achieve a stable, self-bearing rock mass only with the help of traditional rock reinforcement such as rock bolts and shotcrete. In fault zones and in areas where the rock overburden is low, or the tunnels are close to the surface or other infrastructure, the use of ribs of reinforced shotcrete has normally been sufficient. In parts of future tunnels and caverns close to the surface and/or close to operating infrastructure, more rigid/stiffer excavation support, such as pipe umbrellas in combination with sequential short blast rounds (1-2 m) and cast concrete at the face might have to be used. In order to optimize tunnel alignments, future projects may also have shorter sections planned through soil.

Due to strict requirements relating to vibrations, typically resulting from blasting and machine scaling of tunnels, techniques such as drill and split or TBM may be considered advantageous. Drill and split is at the time of writing in use at the new Follo-line railway project (2 x 19,5 km train tunnels) for passing of road tunnels and oil storage caverns in the Ekeberg-hill (Ekebergåsen).

Alum (potassium aluminum sulfate) shale makes up the lower sedimentary shale layer in the Oslo area. This highly organic black shale was deposited in oxygen poor waters and has a high content of heavy minerals such as uranium, and other minerals such as sulfur. It is considered as the primary source of the radioactive gas

Radon (intermediate decay product of Uranium) in many houses in Oslo, which above a certain concentration gives a risk towards the development of lung cancer. If alum shale is exposed to oxygen several chemical processes initiate. In the processes sulfuric acid is precipitated, which can attack concrete and steel (important consideration for tunnel linings and reinforcement). Another alteration product is gypsum, which requires more space and can lead to swelling pressures as large as several MPa. Due to the heavy toxic minerals and acidic runoff alum shale must be transported to special deposits, in which there exist very few today, and the cost for transport and deposition is high. In light of the above, it is important with local geological knowledge in order to predict where alum shale can be encountered. Specific investigations (e.g. core sampling and stratigraphic interpretation) should be considered for each project.

3.2 Drainage, settlements and functional requirements

Settlements have been a general concern for tunnel builders since construction of the Holmenkollen subway-line tunnel between Nationalteateret and Majorstuen in the start of the 1900's. Naturally occurring subsidence in the marine sediments is still ongoing, but the rate is believed to be slow compared to settlements induced from earlier deposition of "town-fill" material and drainage from construction activity. Buildings and infrastructure located on top of or in vicinity to the deeper sediment trenches are the ones most prone to settlements, since settlement potential increases with deposit thickness. The larger damages are often seen for buildings where the foundation lies on rock or a thin deposit on one side and a deeper deposit on the other. This leads to uneven settlements, and introduces shear forces in the structure of those buildings.

Table 3 shows the most relevant requirements of the Norwegian National Railway Administration (NNRA) and the NPRA concerning drainage and waterproofing. They are not very specific and are open to individual evaluations for each tunnel project. In many cases this is beneficial, since most tunnels have individual purposes/functions and are different with regards to location, geology and hydrogeology. Most tunnels in Oslo serve very important functions and therefore it is strengthened focus on Reliability, Availability, Maintainability and Safety (RAMS). It is also important to consider Life Cycle Costs (LCC) in the choice of solutions. Waterproofing (including frost protection) must ensure that running and dripping water is handled, and that the required water-tightness is achieved without the need for costly interruptions to do maintenance works. In 2015, 94,4 M subway trips were undertaken (Ruter 2016) in Oslo. The combined population of Oslo and the neighboring county Akershus (from where many people commute to work in Oslo) is expected to increase by more than 20 % within 2030, from todays 1.2 M to 1.45 M. In 2060, this number is expected to have risen to 1.7 M (Ruter 2015). This illustrates the importance of selecting the right solutions for tunnels.

Table 3: Selection of essential text from the functional requirements in NNRA (2016) and NPRA (2016). Translated into English

NNRA	NNRA Technical rules - Tunnels/Planning and construction/Tunneling Sect. 2 General: *Tunnels of the NNRA shall normally be built as drained constructions, where drainage of groundwater is allowed in amounts that are acceptable in order not to inflict significant damage from drought or settlements to the overlying nature and constructions.* Tunnels/Planning and construction/Waterproofing Sect. 2 Technical lifetime: *Technical lifetime[1] of the waterproofing construction shall be 80 years. Life cycle costs shall be calculated over a time period corresponding to the technical lifetime.*
NPRA	NPRA Handbook N500 – Sect. 3.5 Requirements to limit drainage: *Consequences for the environment as a result of drainage of groundwater shall be mapped and evaluated as part of the pre-investigations. Based on these the need for watertightness requirements shall be evaluated.*

[1] Technical lifetime (TL) is the time it takes before the waterproofing does not longer fulfill its given function. TL is expressed as the number of years, which are expected with at least 90 % probability. The mean value of TL is assumed to be at least 25 % higher than the requirement (which means 100 years for the waterproofing solution)

The choice of water-tightness requirements for tunnels in Oslo has been based on empirical relationships between measured water leaks in the tunnels and/or pore pressure reductions in wells, versus recorded surface settlements/damages. Some of the wells are only established down to the rock surface (soil well), while others are drilled further into the rock mass (rock well). The wells are normally established several years before a project starts in order to record the annual groundwater level and pore pressure fluctuations. Due to the clay's low permeability, it is normally not a direct correlation between pore pressure reduction at the bedrock and the groundwater level above during construction. Typically a leakage criterion (given in l/100 m tunnel/min) is set and inflow measured at constructed thresholds in the tunnels at some distance from the tunnel face. This is done during stoppage events in tunneling. Due to vaporization on tunnel surfaces (can be substantial, especially in ventilated tunnels) and other assumptions that are challenging to verify (i.e. accuracy of the measurement, will all the water be collected at the thresholds?), this type of measurement may be biased with large uncertainties. Average leakage measured over a longer tunnel stretch is also not necessarily representative of actual pore pressure reduction, since drainage typically is concentrated and linked to local fractures. It is important to be aware that drainage during tunneling (before the desired water-tightness is achieved) can have large impact on settlements, and the end result might not be seen until long after the tunnel is finished. Measurements of pore pressure-drop in wells should therefore be used extensively (i.e., a sufficient amount of wells should be used) in order to continuously assess the risk of surface impact from tunneling, and to document the achieved water-tightness.

The preferred method to seal leaks, and reduce drainage potential, has traditionally been pregrouting (continuously, or selectively based on leaks measured

from probe drilling holes) with the use of cement-based products executed with relatively high pressures in front of the advancing face of tunnels. The goal is to construct a permanent impermeable ring in which the tunnels are excavated through. Each grout umbrella is typically 18-24 m long, with minimum overlap between umbrellas of about 8 m. The center of the face is also grouted, something that is important during construction in order to prevent lateral drainage. Drilling of grout holes, grouting, curing, and drilling of verification holes is time consuming, and it has been realized that an impermeable seal against water influx cannot be achieved in this way. Especially in, or in vicinity of fault zones and stiff and fractured rock intrusion zones, permeability is often high and large water leaks have been experienced, which have been tough to seal. In addition it has been realized that even smaller leaks, over time, can have a substantial impact on settlements. Once the tunnel has passed, the only way to seal leaks is additional post-grouting behind the face (proven to have reduced effect) or to construct a watertight (undrained) concrete lining.

Only a few of Oslo's tunnels are constructed as watertight undrained structures. In some specific areas of drained tunnels that have had large leakage problems, undrained solutions have been executed selectively. It should be noted that before the lining is in place, substantial soil compaction might already have occurred. Infiltration of water has been used to a large extent during the construction period of several projects in order to counteract pore pressure reduction in the overlying soils. For several tunnels, these countermeasures have also had to be used in the permanent state. The recommendation after a larger study of the effects of water infiltration (NPRA 2003) was that infiltration primarily should be used as a temporary measure, and that permanent infiltration should be avoided and only used as an emergency measure.

4 Pressurized TBM-shield tunneling

TBM pressurization technology was originally developed for tunneling in soils, where a support pressure is often necessary to counteract groundwater-flow and stabilize the tunnel face. Increasing need for underground construction, especially in urban areas, resulted in the need to handle more complex and mixed ground conditions, and deal with stricter environmental requirements. In order to meet these demands TBM technology has developed to be more versatile. New potential projects in Oslo will typically involve a mixture of different rock types, shifting rock mass quality, very limited ground cover, and in special cases, might have passages where the face is partly excavating in rock and partly in marine clays (or even full-face excavation in clay). The main purpose of this article is to investigate pressurization technology with regard to limiting drainage when excavating in rock. The aspect of face stability is therefore given little focus.

There are two main types of pressurized TBM-shield machines available. These are the Earth Pressure Balance (EPB) and slurry machines. Both machine types have been evolving and both types can handle a variety of ground conditions. This means it is not necessarily only one machine type suitable for a specific project. The choice of technology may also depend on cost factors and the contractors experience with the different types of machines. In the next sections the basic EPB and slurry technologies are explained. Note that different TBM manufacturers may operate with different names for more or less the same technology. TBM machines are typically tailor made for a specific project, and a combination of EPB and slurry technology in the same machine is being offered. A very important aspect in the following discussion is that EPB and slurry machines are still primarily being used for tunneling

in soils, and that experience from pressurized tunneling in hard rock conditions is still

relatively limited. Extreme wear to cutterheads, tools and mucking systems on these

machines, have resulted from closed-mode tunneling in hard rock (see Sect. 4.3).

A third machine type that should be included here, and the one best suited

with regards to the hard rock boring process, is the traditional hard rock single-shield

TBM. The outer shield and thrust system is basically the same as for the EPB and

slurry machines. It has a cutterhead optimized for hard rock, and a traditional

conveyor belt for muck transport. A single-shield TBM can be designed with a

retractable conveyor-belt and a bulkhead. In this way the forward part of the machine

can be closed and pressurized relatively fast. During this static-closed-mode,

conventional pre-grouting can be performed until water leaks are reduced to

acceptable levels for further advance.

4.1 EPB technology

The working principle and parts of an EPB-TBM can be seen in Fig. 5 (left side). The

cutterhead is structurally designed and equipped with excavation tools suitable for the

expected ground conditions (normally rolling cutters for hard rock). The working

chamber is filled with excavated muck, and a stable face pressure (sensors in the

working camber) is achieved by advancing the machine while at the same time

controlling the rate of muck extraction through the screw conveyor. An advantage of

the basic EPB-TBM system is that no other equipment is necessary. EPBs have

traditionally been used in cohesive soils with sufficient clay or silt content (i.e. in low

permeability ground). In granular soils and rock, and under high water pressure, it can

be a challenge to form the impermeable "plug" in the screw. Material can potentially

flush through the screw leaving the face unpressurized. In recent years the application

range of EPB technology has widened with the introduction of muck conditioning in

the form of for example bentonite or polymers (foam). It is also possible to connect

the screw conveyor directly to muck pumps, which can control the discharge rate, and

for example, lead the muck into a traditional slurry circuit. In the excavation chamber

of EPB-machines there is normally no room for stone crushers, and boulders must be

transported through the screw and be removed from "boulder traps".
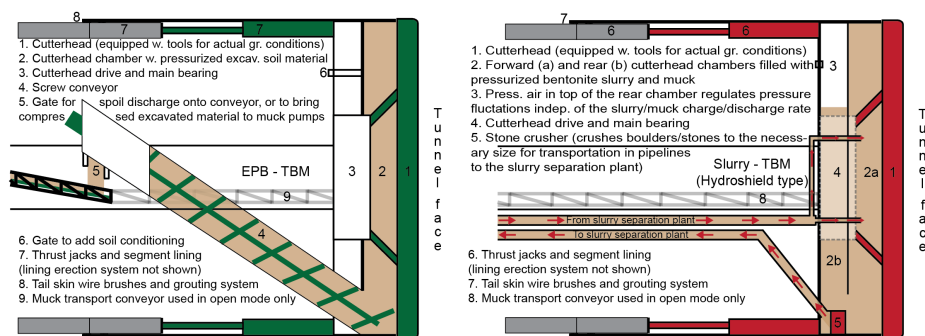

Fig. 5: Illustration of the basic EPB- (left) and slurry- (right) TBM machine technologies. The shown slurry-TBM is of the so-called "Hydroshield" type

4.2 Slurry technology

The working principle and parts of a slurry-TBM can be seen in Fig. 5 (right side).

Similar to the EPB-machine, the cutterhead is constructed for, and equipped with

suitable excavation tools for hard rock. In slurry-TBMs the working chamber is filled

with slurry (bentonite or a water-clay mix), which is typically fed through pipes (Fig.

6) from a slurry separation plant at the surface. Larger stones and boulders are

crushed by a stone crusher before the suspended muck is sucked out at the invert and

transported back to the surface plant for separation. The charge and discharge rate of

the slurry material is what regulates the pressure at the working face of traditional

slurry-TBMs. Another variant (the one shown in Fig. 5) is the "Hydroshield" where

the working chamber is divided into two parts, which are connected through a

submerged wall. In this system a "bubble" of pressurized air in the top of the second

working chamber enables the face pressure to be controlled independently of the

charge/discharge rate. Slurry-TBMs have traditionally been used in granular soil,

heterogeneous permeable ground, and where high water pressures are expected.

Fig. 6: Slurry transportation pipes, waterproof single shell segment lining and normal belt conveyor (for open mode tunneling) in the western-tube of the Hallandsås tunnel (Sweden). The inner diameter of the tunnel is 9 m. Photo: Ø. Dammyr

4.3 Experience with closed-mode excavation in mixed and hard rock conditions

The published scientific literature on the use of pressurized TBMs in mixed and hard

rock conditions is still relatively limited. In order to illustrate some of the challenges

involved in closed-mode tunneling, some experiences are presented below.

Sturk et al. (2011) reported on closed-mode slurry excavation at the

Hallandsås Ø10.6 m TBM tunnel project, where tunneling was performed through

predominantly Precambrian gneisses and amphibolites, locally highly weathered or

totally decomposed. Closed-mode excavation was initially done for 153 rings (ca. 337

m), but severe damages to the slurry circuit (e.g. pierced steel pipes) were

experienced after less than 50 m. Despite time-consuming repairs it was reported that

it was difficult to keep the slurry circuit operational.

Shirlaw (2016) wrote that over the last 20 years, there has been a significant amount of pressurized TBM tunneling in weathered rock, where much of the tunneling has been for infrastructure construction in cities such as Hong Kong, Singapore, Kuala Lumpur, Bangalore, Shenzhen and Guangzhou. Some experiences with pressurized TBMs in mixed face conditions, resulting from tropical weathering of igneous rock in Singapore, were reported by Shirlaw (2016) and are summarized here. In brief, some of the typical challenges associated with closed-mode EPB and slurry machines in mixed and hard rock include:

- Inability to maintain the face pressure.

- Ground loss.

- Sinkholes or local areas with large settlement over the tunnel.

- Slow rates of tunneling.

- Rapid tool wear, damage to tools, mixing arms and other parts of the TBM.

- Very frequent and long interventions.

- Clogging, either by sticky clay or by coarse-grained particles.

The degree to which these problems may affect a particular drive will, according to Shirlaw (2016), depend on many factors, including the extent of the mixed ground and types of rock encountered, the type and design of the TBM and cutting tools, and the operation of the TBM. It was reported to be especially challenging when the rock is strong and abrasive, and that one of the important considerations is that a TBM which is to be driven through mixed face rock conditions resulting from weathering has to be able to excavate rock and soil, while applying pressure to maintain face stability. The TBM is therefore, inevitably, a compromise. Tunneling rates in either soil or rock are typically significantly slower than might be achieved with a machine designed only for soil or only for rock. Using two tunnels as examples, excavated

with a Ø7.46 m slurry and a Ø9.23 m EPB machine, Shirlaw (2016) showed that clogging of the cutterhead and resulting wear to cutterhead and tools can be a problem for both slurry and EPB machines, but that EPB machines are especially prone to excessive wear when the proportion of rock at the face is high, especially when tunneling in closed-mode. For the EPB example, only 1.65 % of the tunnel length had the most onerous conditions, but the time required for TBM advance and interventions was one third of the total time for the whole drive. It was reported that on average, interventions were carried out for just over every 3 m of tunnel when the rock comprised over 85 % of the area of the face, and many of the interventions were of very long durations.

Kenyon (2016) reported on challenges with extreme wear during closed-mode EPB tunneling for the Bangalore Metro north-south line with a Ø6.5 m EPB machine. At this project, 35 months were used to excavate 970 m of tunnel (average tunnel advance of less than 1 m/day). It was reported that much of the delay had been caused by difficult mixed face ground conditions along an alignment that also featured sections of hard rock. Severe jointing resulted in rock chunks continually coming loose from the face. The cutterhead would continue rotating without the material being crushed, which created an imbalance in the machine and ultimately high wear to the cutterhead, tools and mountings. A recovery shaft had to be excavated from the surface in front of the machine in order to replace the cutterhead. After restart of the machine, full-face soil and mixed face conditions, as well as a challenging 46 m stretch of solid rock (UCS up to 120 MPa), were encountered. Kenyon (2016) reported that during this rock section, frequent interventions to perform cutting tool replacements were necessary, one for every 3.3 m of advance on average, and almost all interventions had to be performed in hyperbaric (pressurized) conditions.

Wallis (2014) reported on the construction of the Bangalore Metro east-west line, where slurry TBMs excavated through an undulating interface between fresh granite bedrock and weathered granite layers beneath soft wet residual soils at the surface. The TBMs operated at a relatively shallow depth (one to two machine diameters) and at an operating pressure of about 0.2-0.7 bar, always less than 1 bar. The complex geology at the tunnel alignment was reported to comprise: A mix of hard massive granite at more than 300 MPa UCS and few joints or fissures in the bottom of the face; layers of weathered granite in the center with many fresh granite blocks and boulders in the weathered matrix; and residual soils of wet running sands, clays and gravels at the top. Tough TBM tunneling conditions were reported to slow top advance rates of eight to ten rings per day in full faces of residual soils or soils and weathered rock materials, down to two or three rings a day when excavating hard, dry massive rock. Wallis (2014) reported that the slurry system was used in both situations, providing a cooling effect on the TBM when working at full power, as well as assisting to reduce wear through the lubricating effect of the polymer-based slurry.

Hard rock slurry technology is evolving, illustrated by the completion of the Lake Mead intake No. 3 tunnel (see e.g. Anagnostou 2014) and the Bosporus strait crossing road tunnel (see e.g. Bäppler 2014), which involved long drives in closed slurry mode. However, similar to the experiences discussed above, excessive wear and time-consuming interventions were necessary. More research and literature is welcomed on this topic, and there is no doubt that further evolution and innovations with respect to wear-reduction in hard rock is welcomed for both EPB and slurry machines.

4.4 Possibilities and limitations

Fig. 7 illustrates some key features and challenges of future tunnel projects in Oslo. The TBM is shown operating in open-mode (without pressurization). With the introduction of face pressure, drainage towards the tunnel is counteracted and the resulting negative surface effects removed. In weak rock and soil, and where the overburden is low, it is especially important to maintain the correct face pressure (the system must react quickly to possible pressure fluctuations), and to have strict control of the mucking operation in order to avoid over-excavation and the creation of voids in front of or above the cutterhead.

A D&B or open-TBM tunnel alignment would normally be planned a fair bit deeper than the alignment shown in Fig. 7. Due to rock mass weathering, permeable intrusion zones, fault zones and low/no rock overburden, a D&B or open-TBM operation with pregrouting would be challenging with the shallow alignment, and might result in insufficient watertightness during construction (a watertight lining would probably have been chosen for the permanent state). Other challenges would be potentially very sensitive clays in a section of the crown (possible collapse) and exposure of alum shale to oxygen.

The shallow alignment demonstrates some of the possibilities of pressurized TBMs. In addition to drainage control, the face pressure and TBM shield will better ensure stability in areas of low rock overburden, and *may* enable tunneling through the sensitive clays without collapse. In the recent smaller diameter *Midgardsormen sewage-project* (Føyn and Lauritzen 2014) air pressure was used for about 200 m length to ensure sufficient face stability during pipe jacking of a Ø2.4 m tunnel through sensitive clays. Surface settlements were normally small and acceptable, but steering problems occurred in some areas (Føyn and Lauritzen 2014). Pressurized

TBMs can enable alignments that may not have been thought possible with D&B or open-TBMs, and not considered possible by other than surface excavation methods such as cut and cover. However, feasibility of tunnel boring through stretches of soil depends on many factors, and should be evaluated for each specific case. Thorough ground investigations are in this respect very important in order to choose the appropriate method. D&B excavation under pressure is possible, and has been done for the purpose of soil stability in parts of the subway development around Grønland subway station in Oslo in the 1960s. However, large-scale operations could be logistically challenging (workers, machines and muck must pass through the airlock), pressure fluctuations cannot be handled in the same way as with a TBM, and a waterproof lining will normally not be installed before at some distance behind the face. With respect to closed-mode tunneling through Alum shale, since the whole system is "submerged" no or limited oxygen exposure of alum shales would occur (during construction and in the permanent state), something that can be considered as favorable.
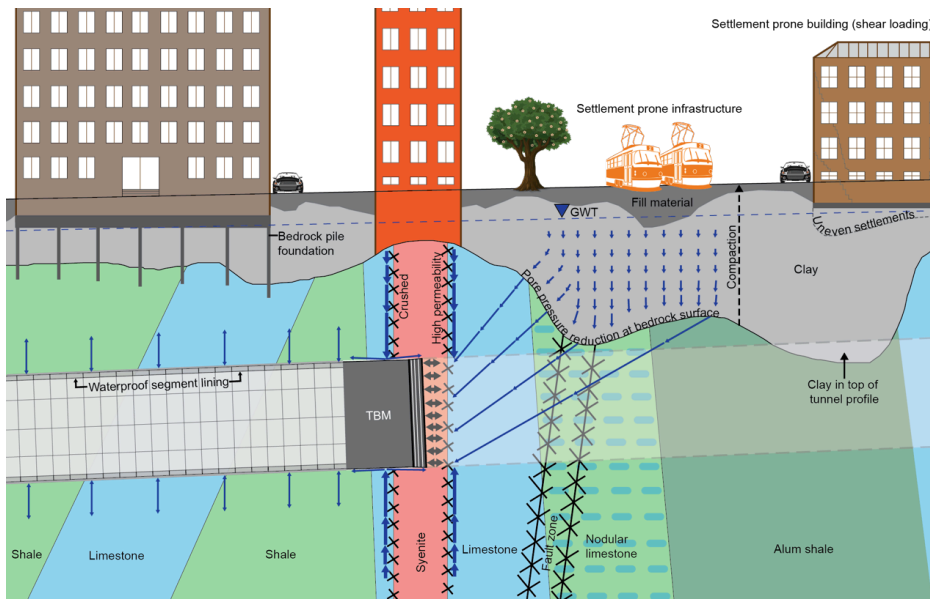
Fig. 7: Illustration of key features and challenges of future tunnel projects in Oslo. The TBM is shown in open-mode, and drainage leads to soil compaction and surface settlements (for local infrastructure and buildings with soil foundations). Closed-mode TBM operation equalize the pressure gradient between the tunnel and the ground, and theoretically no flow of groundwater is then to be expected

Hard rock tunnel boring involves the need for cutter changes and necessary cutterhead repairs. As shown in Sect. 4.3, cutterheads, cutter tools, screw conveyors and slurry circuits of EPB and slurry machines respectively, can be especially prone to excessive wear when used in hard rock. Maintenance stops should primarily be planned under atmospheric pressure conditions. It is possible to undertake repair works under pressure, but there is always a higher risk involved with this type of work and such interventions are normally avoided if possible. In permeable ground, a typical pregrouting scheme can be performed from the TBM ahead of a maintenance stop in order to create a "waterproof" location. Due to the above considerations, closed-mode hard rock TBM excavation is normally only planned for a limited length, where pressurization is considered absolutely necessary. In other parts of the drive the TBMs normally excavate in open-mode, and the muck is transported on a belt conveyor equal to that of traditional TBM-shield operations. Another important aspect

is that the design of the segmental concrete lining depends on the thrust force needed for the TBM to operate at the closed-mode design pressure. This may involve a stronger/heavier lining than is necessary for open-mode excavation (cost aspect).

Open-mode excavation in sensitive areas would normally involve a pregrouting operation in front of the advancing tunnel (regardless of TBM type). However, EPB, slurry, and also single-shield TBMs designed with a bulkhead, have an advantage when operating in open-mode compared to traditional TBMs. If surprised by water inflow and/or falling pore pressures, the working chamber can be closed relatively fast (depends on the design) and the TBM put in static-closed-mode. Pre-grouting can then be executed with the face pressurized, until the water leaks are reduced to within acceptable limits for further advance in open-mode. This method was found to be most practical/economical for water control at the Hallandsås Ø10.6 m TBM tunnel project, which was mentioned in Sect. 4.3.

There are scenarios where pressurized excavation might be considered too high of a risk for reasons other than machine wear. This can for example be the risk of blowout and risk of over-excavation or rock stability when passing close to foundations or operating infrastructure tunnels. A TBM is also typically less flexible than D&B with respect to pre-grouting, the need for forepoling/spiling, and handling of unexpected obstacles (such as non-registered groundwater wells). Other challenges with pressurized tunneling relate to the conditioned muck, which normally needs to be cleaned before deposition. In an urban environment it can be challenging to find the necessary TBM access points and logistical space for all the support functions. The latter is one of many aspects that need to be given attention in the planning process of a potential TBM drive. This article does not include a review of such aspects, which are project specific.

Excavation with the use of EPB TBM machines has earlier been evaluated and compared with D&B for the Lysaker – Sandvika railway (NNRA 2006, SINTEF 2006) project west of Oslo. The positive benefits with respect to rock stability, increased groundwater control (during and after excavation) and reduced environmental impact (regarding settlements, blasting vibrations and gasses, and reduced material transport on local roads) were overshadowed by the higher cost and the fact that TBM excavation was not considered to be faster. In these evaluations it was assumed that functional requirements could be achieved with the use of pre-grouting, D&B excavation, and the drained umbrella waterproofing-structure of freestanding polyethylene- (PE) panels, which are often used in Norway. Recent studies (e.g. Dammyr et al. 2014) indicate that functional requirements might not always be achieved with the use of current methods, and that large variations with respect to maintenance costs and service life of different waterproofing concepts occur. In order to make sound comparisons, detailed studies into technical and social economic costs (life cycle costs, cost relating to environmental impact above the tunnel, public- and goods-transport cost/inconvenience resulting from tunnel maintenance etc.) relating to the different concepts are warranted.

5 Concluding remarks

The intention of this article has not been to recommend a universal tunneling method or waterproofing solution for tunnels in Oslo. The first step should always be to gather detailed information about the ground conditions, hydrogeology, building foundations, and state of existing infrastructure along the tunnel route, and to define the necessary functional requirements needed to protect the environment above the tunnel and to achieve the desired RAMS for the finished project. Following this, the process of project optimization, including the choice of tunneling method can begin.

Oslo's need for new infrastructure will require many new tunnels, and in order to avoid increased settlements in the future it will be important to consider the total future drainage potential from all tunnels and building pits (both during construction and in the permanent state). This is a very demanding task, both technically and legally (e.g., which project is to be blamed for a specific damage?). To address this challenge it is recommended that the different representatives of future infrastructure development work closely together, and with the municipality of Oslo and the Norwegian water resources and energy directorate (NVE), in order to define the necessary basic functional requirements for future tunnels. One approach to solve this problem is to construct all tunnels as watertight undrained structures (i.e. plan the tunnels as tight as technically possible) and to require an absolute minimum of drainage in the construction phase. If such an approach is to be followed, pressurized TBM-shield tunneling may be part of the solution. However, evolution of EPB and slurry machine technology for hard rock will be necessary in order to improve the wear characteristics of these machines in closed-mode operation. The most realistic alternative at present time is considered to be the use of traditional hard rock single-shield machines with a static-closed-mode option, equipped with high capacity/flexibility to do pre-excavation grouting. These machines can offer some of the benefits of pressurized machines and the same watertight lining just behind the machine.

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Paper 4

Is not included due to copyright

Paper 5

Feasibility of tunnel boring through weakness zones in deep Norwegian subsea tunnels

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Abstract: Norwegian subsea tunnels have all been excavated with the drill and blast method. The prevailing rock mass quality is generally favorable for tunneling, but the encounter of weak and/or water bearing zones is normal, and sometimes leads to extreme challenges. Future Norwegian subsea tunnels might benefit from the use of tunnel boring machines (TBMs), but the less flexible nature of a TBM will require more effort with regards to investigations and evaluations in the pre-construction phase. This paper summarizes some of the extreme challenges encountered in Norwegian subsea road tunnels, and reviews experience from international TBM projects considered relevant for Norwegian tunnels. The focus is on geological hazards, their implications, and mitigation measures. The aim is to assess the feasibility of tunnel boring through subsea weakness zones. Due to uncertainties and limitations with pre-construction investigations/interpretations for subsea tunnels, there will always be a remaining risk of encountering difficult ground. It is shown that it can be hard to predict adverse rock mass behavior ahead of the face during tunneling. Based on recent state of the art large diameter (>12 m) TBM technology, it is concluded that closed-mode excavation may be considered feasible for water pressures up to ca. 100 m. Pressurized TBMs can reduce risk and may enable excavation through unfavorable rock mass conditions, but this will require continuous installation of a gasketed segmental concrete lining (undrained solution), which can mean a conservative lining design for the rest of the tunnel. Adverse rock mass behavior and/or sudden large water inflow at high pressure can be challenging to handle with open-face TBMs. Based on the above large diameter tunnel boring is considered to involve a high risk for water pressures above ca. 100 m, and is therefore not recommended. The use of a pilot tunnel to investigate and treat the ground ahead of the main tunnel(s) can be a way to reduce risk. In order to reduce contractual risk, the inclusion of a drill and blast section to be used in the case of extreme challenges, can be wise. The potential for squeezing should be evaluated for weakness zones of substantial width, and 3D numerical analysis are encouraged for zones where squeezing challenges are expected.

Keywords: Norway; hard-rock; weakness-zone; TBM; water-pressure; tunnel-stability

1 Introduction

Close to 50 Norwegian subsea tunnels are currently in operation, with more in construction or planning phases. Their purpose is mainly for road traffic, but some are for the transport of oil and gas. All tunnels have been constructed with the conventional drill and blast (D&B) method, but future projects might benefit from continuous excavation. Excavation with a TBM-O (open gripper/main-beam TBM with a short cutterhead shield) was considered for the North Cape road tunnel (opened 1999), but was not selected due to the risk of large water inflow and an estimated higher cost. There is extensive experience with small to medium diameter (Ø) on-shore TBM tunnels in Norway (mostly smaller diameter, but ranging from Ø2.3 m - Ø8.5 m), and some experience from crossing of problematic weakness zones and dealing with high water pressures similar to that encountered in deep subsea tunnels exists for TBM-O machines. However, all experience from subsea TBM tunnels and TBMs of larger diameter are from outside of Norway. Modern Norwegian subsea road tunnels will typically need a diameter of more than 12 m if TBMs are to be used (based on NPRA 2012).

Compared to conventional on-shore tunnels, subsea tunneling involves a greater risk due to less detailed knowledge about the geology and rock mass properties in the planning phase. This knowledge is to a large degree acquired through indirect methods, such as reflection and refraction seismic, and to some extent core drillings (direct method). Permeable zones with unlimited supply of water from the sea can pose a real threat of flooding such tunnels. In addition, auxiliary access/exit tunnel possibilities are few or non-existent. Due to the less flexible nature of a hard rock TBM compared to D&B (i.e. limitations related to probe drilling and pre excavation grouting (PEG); few possibilities of accessing and directly observing the tunnel face; large machinery and backup that cannot be easily/fast removed, modified or repaired), adverse rock mass conditions and large water inflow may threaten excavation feasibility. On the other hand, the potentially faster excavation rate of a TBM and industrialized rock/water support possibilities (especially with shield TBM) may offer the best solution. Shielded hard rock TBMs can also incorporate closed-mode (slurry or earth pressure balance - EPB) or static closed-mode (built with a bulkhead, but cannot excavate under pressure) technology. This enables the application of a face stabilizing and water counteracting pressure, which in some cases (for example when tunneling through fault zones with highly permeable cohesionless material) can be advantageous over D&B.

There has so far been limited research involving detailed discussion and evaluation on the feasibility of tunnel boring through weakness zones intersecting otherwise hard competent rock mass at great depth. This paper gives an overview of geological hazards specifically related to Norwegian subsea tunnelling (Sect. 2), and summarizes relevant international TBM experience (Sect. 3). In Sect. 4 are implications to tunnel boring, possible mitigation measures, necessary pre-construction investigations and ultimately TBM feasibility evaluated. The main focus is on three types of challenging weak zones, and large water inflow at high-pressure (sometimes connected to weak zones, other times not). A common characteristic of the zones is that they are normally of limited width and intersects otherwise competent rock mass favorable for hard rock TBMs.

2 Challenges in Norwegian D&B subsea tunnels

The locations of fjords and straits in Norway are in most cases defined by major faults or weakness zones in the bedrock, and most Norwegian subsea tunnels encounter sections of very poor rock quality (Nilsen 2014). During tunneling continuous probe drilling and tunnel face mapping, supplemented with core drillings when needed, are used to adapt excavation procedures and rock support design to the prevailing conditions. Normal working steps to excavate through weak water bearing ground in Norway can be seen in Fig. 1. Probe drilling (and sometimes core drilling) is used to map the rock mass quality ahead of the face, and PEG is used to deal with water. Probe drilling is done systematically, and a software system is normally used to continuously process, store and graphically present data from the drillings. Engineering geologists use observations at the tunnel face and drill-data to make decisions regarding rock support and continued excavation. PEG is either done systematically, with holes in a grout fan typically overlapping each other by 5-10 m, or is carried out based on expected weakness zones and water leakage measurements from the probe drilling holes. Most subsea road tunnels are close to, or more than, 10 m wide, and have cross-sections ranging from about 60 m^2 to 80 m^2. With few exceptions (Tromsøysund and Ryfast tunnels) the tunnels constructed to date are single-tube tunnels (escape is normally only possible through the tunnel entrances).

2.1 Zone type 1

This type of weakness zone is characterized by heavily crushed rock with gouge (see Fig. 2 top), often containing active swelling clay, and is the one most frequently encountered in Norwegian subsea tunnels. Water seepage in this type of zone may dramatically reduce the stand-up time and thus increase the excavation problems.
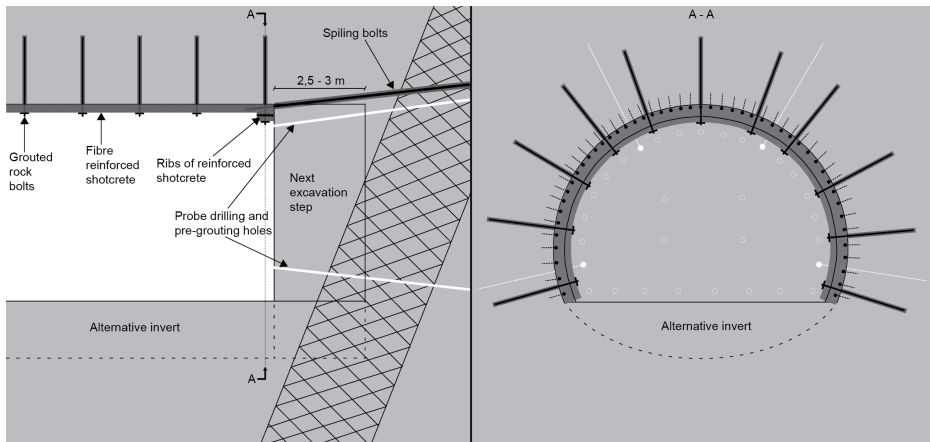
Fig. 1: Typical working steps to excavate through weak water bearing ground. White dashed circles make up drill holes for a grout-fan. Decision to grout is normally based on measured water leakage from the probe drilling holes (white filled circles)

During construction of the Atlantic Ocean tunnel, between Averøya and Kristiansund, a 25 m wide zone of heavily crushed rock was encountered at 225 m depth below sea level (Karlsson 2008, Nilsen 2011). Probe drillings had showed little water, and the zone had a seismic velocity similar to other zones in the tunnel, which had been crossed without severe stability problems. As a precaution PEG had been carried out to seal off potential water influx, and excavation had commenced with reduced blast lengths and procedure similar to that shown in Fig. 1 (although not with ribs of reinforced shotcrete). An unexpected progressive cave in developed in this area (the rock cover was 45 m), which could only be stopped by backfilling of excavated material and shotcrete at the face and pumping of concrete into the resulting cavity, which was estimated to have progressed to 10 m above the crown. Probe and core drillings to map the zone showed a considerable amount of water and full hydrostatic water pressure (500 l/min leakage through one probe drilling hole). An extensive and time consuming grouting campaign and stepwise excavation/support was executed, with a permanent concrete lining installed in short sections as the works progressed (see Fig. 3). The occurrence of swelling clay in the weakness zone significantly contributed to the problem, primarily by reducing the internal friction of the rock mass. It took 10 months to pass 20 of the 25 m from the west side. The last 5 m was excavated from the east side in a similar manner.

Example of zone type 1 (fault zone)
RQD = 10
Q = (10/20)x(1/8)x(1/10) = 0.006
GSI = disintegrated/very poor = 10-15
→Presence of water will reduce strength
Dry: indented by thumb pressure
Wet: easily indented by finger

Example of zone type 3 (fault zone)
More coarse grained and cohesionless
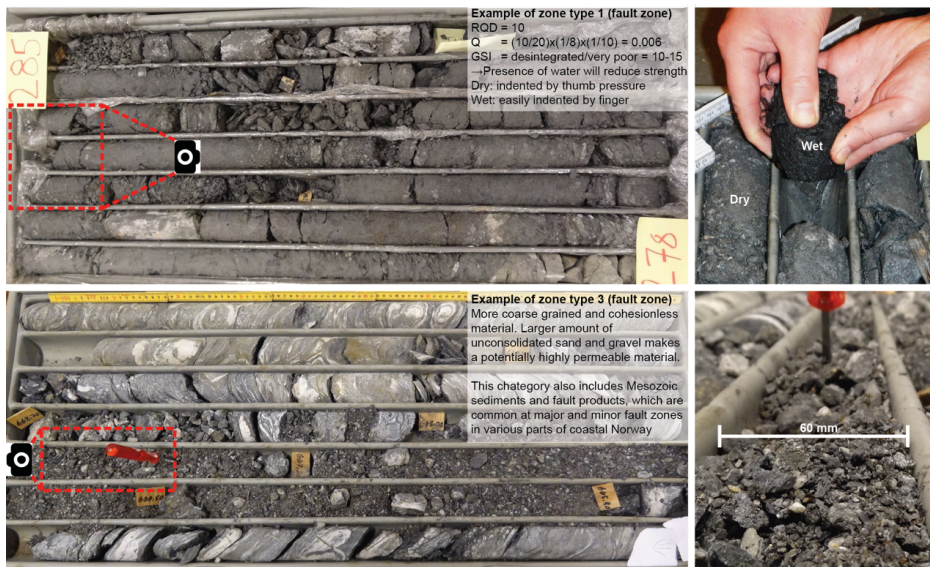material. Larger amount of
unconsolidated sand and gravel makes
a potentially highly permeable material.

This chategory also includes Mesozoic
sediments and fault products, which are
common at major and minor fault zones
in various parts of coastal Norway

Fig. 2: Example of weakness zone type 1 (top) and type 3 (bottom). The cores have been drilled through two fault zones at the planned twin-tube Rogfast subsea road tunnel project in western Norway. The tunnel, which will shorten the travel time between the cities of Stavanger and Bergen substantially, will be the world's longest (ca. 27 km) and deepest (ca. 400 m) subsea road tunnel. The fault zones in this figure are located within black shales, but typically also occur within gneissic and granitic rock formations. Photos: Ø. Dammyr and S. Todnem

Swelling of clay minerals sometimes leads to tunnel convergence and overloading of installed rock support. The buildup of swelling pressure is closely related to the amount and type of clay minerals, the presence of moisture, and whether or not the support system allow for (flexible support) or restrict (stiff support) rock mass volume expansion. The occurrence of swelling in Norwegian tunnels is normally not considered a threat to excavation feasibility, but is rather representing an extra challenge for rock support design and installation timing.

Tunneling through zone type 1 rock with a substantial overburden, or where there are high horizontal stresses, may cause tunnel squeezing. Squeezing was experienced in the Byfjord subsea tunnel, where fractured zones of clay rich phyllite, locally completely disintegrated, were encountered over a stretch of several hundred meters. Some water seepage was present, but no flow. In the area of the lowest rock mass quality (Q = 0.003-0.004) instability in the form of cave in at the face was followed by squeezing behind the face. At one location, 0.3 m of inward convergence was measured in one of the walls of a non-reinforced cast concrete lining (the invert had not been concreted at that time). The cave in stopped developing, but remediation work and the construction of a reinforced concrete invert were necessary. There have later been issues with continued deformation in this area.

The ground pressure (p_0) in the area was estimated to ca. 3.5 MPa (Pedersen 1997). A similar zone (see Fig. 2 top) to that of the Byfjord tunnel has been identified from core drillings at the nearby planned Rogfast subsea tunnel project (weakness zone width of >30 m). The theoretical ground pressure is there estimated to ca. 6.4 MPa. The rock mass strength (σ_{cm}) is believed to be similar to the Byfjord tunnel weakness zone. Considering this, and that two parallel tunnels are planned relatively close to each other at Rogfast (ca. 20 m pillar width, less than 2-diameters), challenges relating to squeezing may be experienced. This will be discussed further in Sect. 4.
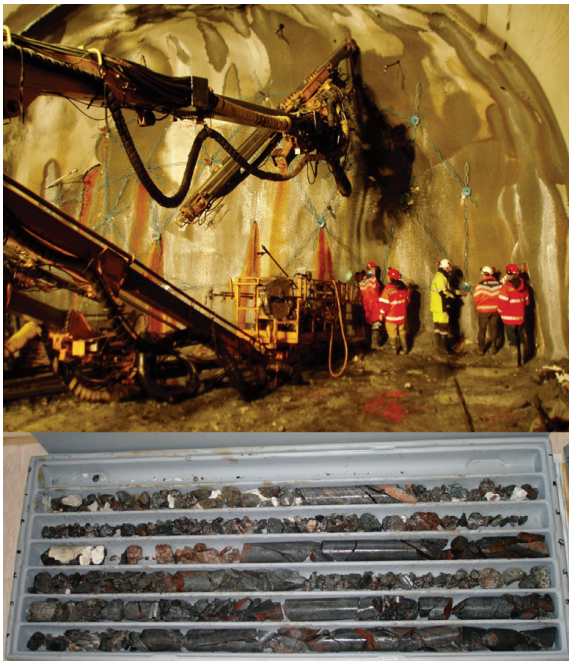
Fig. 3: Situation at the tunnel face when excavating through the zone in the Atlantic Ocean tunnel (top), and drill cores from the central crushed zone (bottom). Photos: B. Nilsen

2.2 Zone type 2

The second zone type does not relate to fault zones, but to glacial material (e.g. moraine) deposited in for example glacial erosion channels (see Fig. 4), which can be deep, and have an extent that can be hard to precisely predict during the pre-investigation phase.

During excavation of the Oslofjord subsea road tunnel, a glacial erosion channel filled with highly permeable glacial deposit material was encountered at 120 m depth below sea level. The material made up the upper part of the face over a length of ca. 15 m. The middle and lower part of the tunnel profile consisted of hard fractured rock. The lower portion of the glacial material consisted of well-rounded non-layered moraine material with fractions from sand and up to boulders of 3 - 4 m^3

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(Andreassen 1999). The top portion consisted of glaciofluvial material without larger stones. The zone

was unexpected, because pre-investigation core drilling had followed the centerline of the tunnel, and

unfortunately too low to detect the glacial material. However, based on probe drilling the zone was

identified well in time before tunneling into it. Grouting was deemed unsuccessful since extensive

efforts did not reduce the water leakage, and ground freezing to create a frozen zone over a length of

about 46 m was therefore established. A cavern had to be constructed to accommodate the freeze

equipment and for drilling-space for the freeze holes, and the tunnel profile had to be enlarged from 80

m^2 to 130 m^2 to accommodate the thick (>1 m) reinforced undrained cast lining (Andreassen 1999).

After an initial 3 months of investigations, planning and grouting, it took 10 months to drill the 115

freeze and temperature monitoring holes (12 holes were lost due to collision with broken steel rods

etc.). The brine-freezing operation (including setup) took five months. Due to the salinity of the

porewater, an unusual low freeze temperature of -28°C had to be reached for the frozen structure

(Berggren 1999). Stepwise excavation (blast lengths of ca. 1.5 – 3.0 m) took about 3.5 months, and the

whole operation about 2 years. Due to the possibility to construct a deep bypass tunnel under the zone,

this project could finish more or less on schedule.
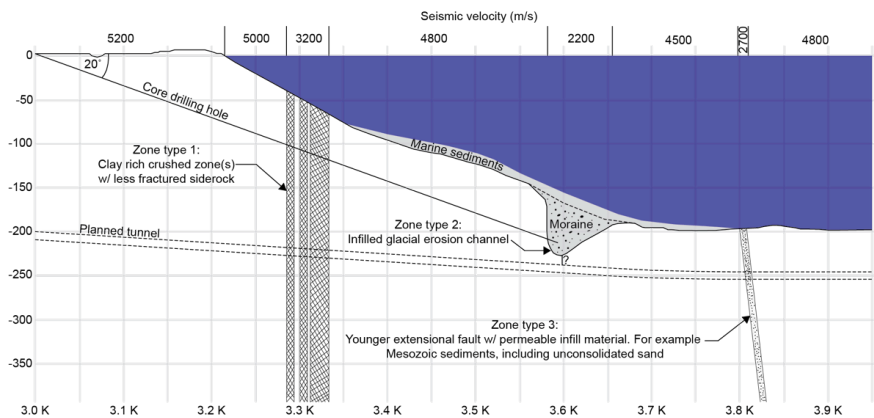
Fig. 4: Longitudinal profile for an imaginary planned subsea tunnel with the three types of discussed
weakness zones illustrated. Also shown are seismic velocities and a core-drilling hole, which are the
most common pre-investigation techniques for Norwegian subsea tunnels

One of the lessons learned from the Oslofjord tunnel incident was that probe/core drilling

should always be done well above the planned location of the tunnel crown. A similar incident was

prevented at the Bømlafjord tunnel (7.8 km long, 240 m deep) in the west coast of Norway, where

directional drilling of about 900 m long core drilling holes made it possible to detect a deep erosion channel in time sufficient to adjust the planned alignment.

2.3 Zone type 3

The third type of zone relates to highly permeable cohesionless fault material. It can simply be crushed rock material with a coarser grain size and little fines, or it can be Mesozoic sediments and fault materials, which are common at major and minor fault zones in various parts of coastal Norway (Bøe et al. 2010).

During tunneling for the Bjorøy subsea road tunnel, located in Precambrian gneissic rock in western Norway, a 4 m wide (25 m crossing length) sub-vertical extensional fault zone of Jurassic age was unexpectedly intersected 70 m below sea level (no seismic refraction or core drilling had been done in that area). It was discovered 10 m ahead of the face by probe drilling, which caused 5 m^3 of sand to flush into the tunnel at full hydrostatic water pressure and at a rate of 200 l/min through a single 51 mm drillhole. Core drilling from the tunnel revealed that the zone consisted of crushed rock mixed with clay, sand and coal fragments (Fig. 5). The sedimentary rocks of what today is called the Bjorøy formation occur in a ca. 10 m thick zone within the Proterozoic basement, in an area without any trace of post-Palaeozoic sedimentary rocks (Bøe et al. 2010). Planning and passing of the Bjorøy fault was time consuming and demanding, and took 9 months. It was done with extensive PEG through blow out preventers, draining of the rock in front of the excavation face, and the use of heavy rock support.

2.4 Water inflow

Norwegian subsea tunnels are with few exceptions drained structures, and hence not designed to be completely watertight. Dripping water is normally handled with a freestanding umbrella structure of polyethylene (PE) panels (shotcrete is used for fire protection), or lightweight concrete segments with an overlying membrane. Only in exceptional cases (such as in very weak and permeable zones) an undrained cast-in-place lining is constructed.

The situation in the Atlantic Ocean tunnel (Sect. 2.1) developed into a serious water inflow problem, and the main challenge was the large, high pressure (up to 23.5 bar) water inflow and the difficulty of achieving efficient sealing in such conditions of the heavily fractured rock mass with unpredictable flow/leakage channels. More than 1,000 tons of grout (mainly micro cement) was required to seal the leakages. In some cases the grout curtain had to be supplemented with several rounds of new holes, and in one extreme case it counted a total number of 153 holes.

High pore/water pressures are not specific to subsea tunnels, they also occur in on-shore tunnels with substantial overburdens. However, in subsea tunnels they may occur in combination with a small rock cover. As the head difference between the water level and the tunnel has to be dissipated within a smaller distance, the pore pressure gradients and consequently the destabilizing seepage forces are higher (Anagnostou 2014). This is most serious where the ground is weak, such as in weakness zones. However, water inflow in general, weather it is related to a weak zone or to independent permeable fractures in an otherwise competent rock mass, represents a major geological hazard that ultimately can threaten the feasibility of tunneling. If the pumps cannot cope with the amount of water the tunnel will eventually be flooded, and in the worst case lost.

Fig. 5: View of part of the Bjorøy tunnel face after PEG with epoxy (top), and cores from core drilling at the face (bottom). Photos: B. Nilsen

3 Challenges in relevant international TBM tunnels

Few international TBM subsea projects have the hard rock geology and the high water pressures found in Norwegian subsea tunnels. However, subsea tunnels excavated through fault zones or weak water bearing rock are faced with many of the same engineering geological hazards. Table 1 summarizes data

from the discussed weakness zones in Sect. 2, and data from challenging international TBM experience

considered relevant for Norwegian subsea tunneling. The TBM tunnels, which will be discussed in the

following, include both on-shore and subsea tunnels. Note the large spread in tunnel diameters, which

can have profound impact on excavation feasibility. Only a few of the TBM tunnels are larger than

Ø12 m (ca. 110 m^2), because of the limited experience with these diameters for tunnels considered

relevant in a Norwegian context.

Table 1: Summary of project data for the discussed subsea weakness zones of Norwegian D&B
projects, and relevant challenges in international TBM tunnels. TBM-O (open), S (shield), DS (double-
shield), EPB (earth pressure balance), and Slurry. Numbers in columns 6-11 are estimations based on
the discussed literature

Tunnel (excavation completed)	Type	Excavation type	Cross-section	Main rock type (in discussed area)	Hydrostatic head (at discussed area)	Rock cover (at discussed area)	Theoretical overburden pressure (at discussed area)	Area-width (crossing length)	Advance rate through area	PEG
			m^2		m	m	MPa	m	Days/m	Tons/m
Byfjord (1992)	Road	D&B	70	Phyllite	170	85	3.6	41	1	-
Bjorøy (1996)	Road	D&B	53	Gneiss	70	35	1.4	4 (25)	11	10
Oslofjord (2000)	Road	D&B	79	Gneiss	120	0	1.4	15	49 / [1]16	47 / [1]15
Atlantic Ocean (2009)	Road	D&B	71	Gneiss	225	45	3.2	25	15	67
Rogfast (planned)	Road	D&B / TBM	2 x 79 / 2 x 110 m^2 (Ø12 m)	Shale & Gneiss	300	200	6.4	> 30 (45)	-	-
Great Belt (1997)	Rail	TBM-EPB	60 (Ø8.8 m)	Till and Marl	20	15	0.35	5	54	-
Hong Kong SSDS Tunnel F (2000)	Sewage	TBM-O	9 (Ø3.35 m)	Granite and tuff with intrusion dykes	125	30	-	268	1	5

Kárahnjúkar TBM No. 2 (2006)	Water	TBM-O	41 (Ø7.2 m)	[2]Altered Hyaloclastite (Móberg)	200	200	5.7	50	4	N/A
Gotthard Faido (2010)	Rail	TBM-O	71 (Ø9.5 m)	Flatly bedded gneiss with high mica content	1500	1500	40	500	0.25	-
Niagara (2011)	Water	TBM-O	163 (Ø14.4 m)	Shale (mudstone and siltstone)	140	140	3.0	3300	0.2	-
Hallandsås (2013)	Rail	TBM-Slurry	88 (Ø10.6 m)	Precambrian gneiss, amphibolite/dolerite	Max. 140	Max. 140	Max. ≈ 3.8	-	-	-
Lake Mead (2014)	Water	TBM-Slurry	41 (Ø7.2 m)	Tertiary sedimentary rocks (conglomerates, breccias, sandstones, siltstones and gypsiferous mudstones), metamorphic rocks (amphibolites, schist and gneiss), and basalts	Max. 140	Min. 20	-	-	-	-
Eurasia tunnel (2015)	Road	TBM-Slurry	147 (Ø13.7 m)	Rock (mudstone, sandstone, magmatic dykes) and marine deposits (clay through to cobbles)	Max. 100	Min. 25	-	-	-	-

[1]Calculated based on the total length of the 46 m freeze ring instead of the zone length
[2]Hyaloclastite: Volcaniclastic deposits formed by explosive magma-water fragmentation and non-explosive granulation of glassy lava rims; used for both unconsolidated and consolidated deposits. Móberg: Consolidated, mafic to intermediate, hyaloclastite (see Jakobsson and Gudmundsson 2008). Kröyer and Arnalds (2004) reported that this younger rock mass was typically more fragmented and heterogeneous than the basaltic lava layers, which occurred in other parts of the project area

Channel *subsea* tunnel (TBM-DS and TBM-EPB): These tunnel experiences are elaborately discussed elsewhere (see e.g. Warren et al. 1996, and Voirin and Warren 1996) and will not be repeated here. Although a milestone in tunneling no critical events or extremely challenging ground conditions, which could have threatened the feasibility of tunneling, where experienced during the subsea part of this project. The use of a smaller diameter pilot tunnel enabled important ground investigations and ground improvement works to be performed ahead of the main running tunnels.

Great Belt *subsea* tunnel (TBM-EPB): Biggart and Sternath (1996) reported that during screw repairs a hydraulic connection formed rapidly and unexpectedly in glacial till deposits between the

working chamber and the seabed (only ca. 15 m vertical distance). It was not enough time to close the screw inspection hatches or the working chamber bulkhead door, and inrush of water at a rate of 180.000 l/min led to the flooding of both eastbound tunnels and the Sprogö worksite. The resulting hole could be plugged from the seabed (only ca. 5 m water depth) using bentonite slurry and concrete, and the water could be pumped out. TBM repairs took 9 months. The bulkhead door to the screw conveyor was after this incident always closed during repairs, and work ahead of the bulkhead was normally done under air compression. Dewatering wells drilled from the seabed ("Project MOSES") lowered the maximum expected water pressure from 80 m to 30 m at this project, and enabled crucial cutterhead repairs to be done under "more reasonable" pressures (20 m - 30 m).

Hong Kong *subsea* SSDS Tunnel F (TBM-O): McLearie et al. (2001) reported on the passing of several fault and dyke zones. The most severe was the 268 m stretch through the Tolo Channel fault zone, which consisted of a repeating succession with rock of poor to very poor quality (decomposed granite, continuous joints, and ground water). Considering the small (Ø3.35 m) excavation diameter, the required and extensive PEG campaign (up to 20 holes of 54 m length in one round with close overlap) and heavy rock support in the form of steel ribs and lagging, excavation of this tunnel would likely have been extremely difficult with a TBM-O of larger diameter (authors remark). McLearie et al. (2001) stated that Tunnel F exemplifies the probable limit of what can be done using an unshielded TBM with extensive PEG, continuous ground support, experienced supervision at the tunnel heading, and the determination to complete the tunnel. It took 10 months to pass the area.

Kárahnjúkar *on-shore* TBM No. 2 tunnel (TBM-O): Kröyer et al. (2007) reported on the encounter of three fault zones over a 50 m stretch in the younger rock types of the Móberg formation (altered Hyaloclastite). The ground was blocky with clay infilling in joints. Void formation above the cutterhead started to form, and a sudden water inflow (9000 l/min) led to more ground loss and the loss of cutterhead rotation in the most severe area of this stretch. Advancement with spiling, shotcrete directly behind the cutterhead, steel ribs with 0.4 m spacing, and steel lagging behind ribs in the crown and walls, was not sufficient. Consolidation of the rock mass in the form of PEG with cement and rapid reacting polyurethane foam was tried. However, with the water and loss of cutterhead rotation it was necessary to remove steel arches and move the TBM backwards in order to fill the void and install a concrete plug (620 m^3 of concrete). Washout of concrete was severe, and a complicated process involving drainage relief holes, injection of polyurethane foam to seal off water influx, and further

concrete backfill/grouting operations was performed. When boring commenced, a similar void-formation challenge was encountered. It required spiling with, and grouting through Ø90 mm 12 m long steel pipes, backing up of the TBM, PEG and installation of self-drilling bolts, before construction of a second concrete plug/backfilling. Normal excavation could commence after 6 months. In this area both refraction seismic and magnetic surveys were undertaken to try and predict potential further challenges. A considerable variation in seismic velocities was reported, but the interpretation was difficult and inaccurate. The magnetic surveys was however found to be quite accurate in this area, and proved to be very useful to predict potential problematic zones.

Gotthard *on-shore* tunnel (TBM-O): *Lot Faido*. Gollegger et al. (2009) reported that large deformations and damage to the temporary rock support (including the invert, where a larger section had to be replaced) was experienced over a stretch of 500 m in the flatly bedded part of the Lucomagno gneiss formation (high mica content). Deformation processes in the form of softening and buckling of individual strata, and shear loading on joints, which can lead to brittle failure along the plane of bedding or schistosity, were described. It was recorded that in some places 50 % of the total convergence was experienced already at the end of the finger shield, about 5 m behind the face (see Fig. 6 top for a similar TBM). Since support could not be installed before this location, the timing and properties of the support proved to be essential to avoid failure of the support and keep the deformations within acceptable limits. The temporary support in L1* (the working area closest to the cutterhead), which was found to work best, was sliding steel TH ribs (including friction couplings and brakes) and 10 to 15 cm shotcrete including styrofoam inserts to create displacement slots. These support measures ensured sufficiently high advance rates to avoid jamming of the cutterhead shield. However, when the TBM of the second parallel tunnel reached this location increased deformations occurred, even though the pillar-width between the tunnels where approximately three tunnel-diameters. This was somewhat unexpected since such unfavorable rock properties were only expected over short stretches, and hence the influence of parallel tunnels had not been considered to pose greater challenges. Convergence of more than 20 cm and failure of the temporary support in both tunnel tubes was experienced. Extensive efforts had to be taken in order to ensure stability and safety, and large-scale remediation works (including the invert) had to be undertaken. Considering the difficulties, acceptable advance rates of between 3 – 5 m per day were achieved in this section. *Lot Amsteg*. Ehrbar (2008) reported on TBM jamming in a slightly water bearing fault zone of loose material in the western

tunnel tube. The eastern tube could be excavated without problems through the area, and the decision was made to excavate a tunnel from the eastern tube in order to grout the fault zone and free the TBM in the western tube from the front. The operation was successful and the TBM was freed after 5 months. Ehrbar (2008) reported that experience from the Gotthard project showed that the response to changing rock conditions (ground improvement, ground reinforcement, and dewatering) is easier and faster with conventional D&B compared to TBM excavation. However, the TBM drives were about twice as fast compared to what had been considered possible with D&B. The cost was considered to have been more or less the same.

Niagara falls *on-shore (short stretch under the river)* tunnel (TBM-O): Gschnitzer and Goliasch (2009) reported on excavation of the first 3.3 km of a Ø14.4 m water pressure tunnel through Queenston shale (mudstone/siltstone sequence of Silurian age). Large difficulties with gravity induced key block failures, as well as overbreak in the crown area (> 3 m) and in the invert (up to 1 m) induced by high horizontal stress, was described. The TBM had to be modified in order to allow for a new spiling system to pre-support the rock above the cutter head and roof shield where large block failures were experienced, keeping large blocks in place and preventing further relaxation of the rock mass and overbreak. Each umbrella consisted of 20 to 40, 9 m long grout pipes (3.2 m to 4.5 m overlap between umbrellas), subsequently completed with shotcrete, steel ribs and rock bolts. In areas with stress induced failure loose rock were removed, flexible steel channels, rock bolts and wire mesh were installed, and shotcrete applied in L1* by handheld nozzle. Advance rates of around 5 m per day under severe overbreak exceeding 3 m in the crown were reported. At the intake an 8 m wide, 7.5 m high, and 400 m long D&B tunnel was excavated from the other side under the Niagara River in order to perform PEG and seal the ground from substantial water inflows ahead of the TBM, which would later drive along the alignment with the cutterhead within the crown of the grouted tunnel.

Hallandsås *on-shore* tunnel (TBM-Slurry): Sturk et al. (2011) and Burger and Duduit (2009) reported on the sometimes hard, sometimes strongly faulted, weathered and permeable rock mass, and the selection and design of the TBM-Slurry (see Fig. 6 bottom) constructed for 140 m water pressure in static closed-mode (bulkhead closed, but no excavation). Main challenges were wear on cutters and cutterhead from blocky ground, the need for extensive PEG in order to limit ground water ingress, to control outwash of segment backfill material, and to stabilize the very poor rock in the Mölleback Fault Zone (MBZ) with ground freezing. Due to high wear on the slurry circuit it was found most reasonable

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to excavate the tunnel in open-mode, and to use static closed-mode with PEG whenever water leakage was close to the allowed limits, or when the flow of water became a problem for continued excavation. Flowing water washed out mortar in the longitudinal direction outside the segments after backfilling. This was tackled with pea gravel as the primary backfilling material and mortar only in the top part (where water was not circulating). Watertight barriers (formed with grouting or inflatable seals) were later constructed at certain intervals and post grouting was then executed. Due to possible flowing ground behavior, ground freezing over a length of 140 m and 233 m for the east and west drives respectively, was established for crossing of the MBZ (Schubert 2013). Freezing could be done well in advance of excavation due to access from the northern portals through a new 600 m long D&B tunnel. No problems were encountered during excavation of the completely decomposed material in the MBZ, but Sturk et al. (2011) reported that it would most likely have exhibited flowing ground behavior if it had not been frozen.

Fig. 6: The view at the rear of the shield at a TBM-O (top) and TBM-Slurry (bottom). Nant de Drance (Switzerland) and Hallandsås (Sweden) tunnel projects. The tunnels are ca. 9 m and 10 m in outer diameter respectively. Photos: Ø. Dammyr

Lake Mead Intake No. 3 *subsea* tunnel (TBM-Slurry): Anagnostou (2014) reported on considerable difficulties in the first part of the alignment through metamorphic rocks (not yet under the lake), caused by an unfavorable combination of high water pressure, extremely high rock permeability and the presence of an unexpected fault zone. The fault, which was oriented sub-parallel to the tunnel, consisted of almost cohesionless material, and it was necessary to operate the TBM in closed-mode at 14 bars (130 m) of pressure for several hundred meters (first time ever done). It should be mentioned that the TBM starter tunnel, which was excavated with D&B, was inundated twice from large water inflows in raveling ground conditions. The construction of a new starter tunnel and realignment (consequently with large delays) was necessary. Tunneling in the sedimentary rocks under the lake could partly be done in open-mode with the use of advance drainage (see Anagnostou et al. 2010 for details) to increase the standup time. However, very challenging permeable and unstable ground conditions led to a substantial increase of closed-mode tunneling compared to what was planned. The largest problem was to find favorable ground conditions (or to improve the ground sufficiently) in order to allow maintenance works on the cutterhead and in the cutterhead chamber to be done under atmospheric conditions.

Eurasia *subsea* tunnel (TBM-Slurry): At this Ø13.7 m double-deck road tunnel project excavation was done entirely in closed-mode (3.3 km), through a combination of rock (including intrusion dykes with UCS up to 125 MPa) and marine sediments, and with up to 100 m of water pressure (Wallis 2015). The whole operation took 420 days (ca. 8 m per day on average) to complete. Some of the most challenging tasks (especially under these pressures) are typically to change cutter tools, and to execute other necessary repair works in the pressurized part of the cutterhead. A disc cutter rotation monitoring system provided data to the control cabin about cutter rotation and temperature, and the cutterhead was equipped with hydraulic lines in order to detect wear on the cutterhead structure (Bäppler 2015). The TBM was designed with the concept of accessible cutting wheel arms, which allowed the rolling cutters and a large number of the cutting knives to be changed in atmospheric conditions. About halfway through the advance, material-wear on the intake screen and the jaw crusher was discovered, which had to be repaired (Herrenknecht 2015). Professional saturation divers had to enter the cutterhead and worked under 9 bars of pressure in order to repair the intake screen and the jaw crusher. This work was completed in 3 weeks.

4 Evaluation and discussion

4.1 Geological hazards

Crucial for the planning of subsea tunnels are identification of geological hazards (threats/dangers), evaluation of risks (probability of occurrence) and potential implications to tunneling associated with these hazards (can be different for D&B and TBM), identification and practical implementation of mitigation measures to reduce or at best remove the risks (includes the choice of tunneling method and type of machinery), and how to deal with remaining uncertainties. Although project specific, the potential risk of encountering very challenging conditions in Norwegian subsea tunnels has been established in Sect. 2. It is important to be aware that risks in tunneling are not solely related to the geology/rock mass and the excavation method. This also depends on the quality of the contingency plans and the experience of the client, the planning consultant, and the contractor. In case of an unforeseen event it can be the knowledge and experience of the on-site personnel that has the primary effect on the outcome. The contract and the economic situation of the client and/or contractor can also be influential. Even though these aspects are important, this is topic for a separate discussion and will not be addressed further here.

Fig. 7 summarizes the most important geological hazards of Norwegian subsea tunnels, and the potential implication for tunnel boring. Hazards and their potential implication in the different zone types will somewhat vary, but available mitigation measures/actions are largely the same. Fig. 7 shows that all zone types involve hazards that can give challenges for tunnel boring if not mitigated properly. In the following the effect and limitations of different mitigation measures will be discussed together with the necessary investigations that should be performed in the planning phase of subsea TBM tunnels. The above forms the basis to evaluate feasibility of tunnel boring through the different types of zones.
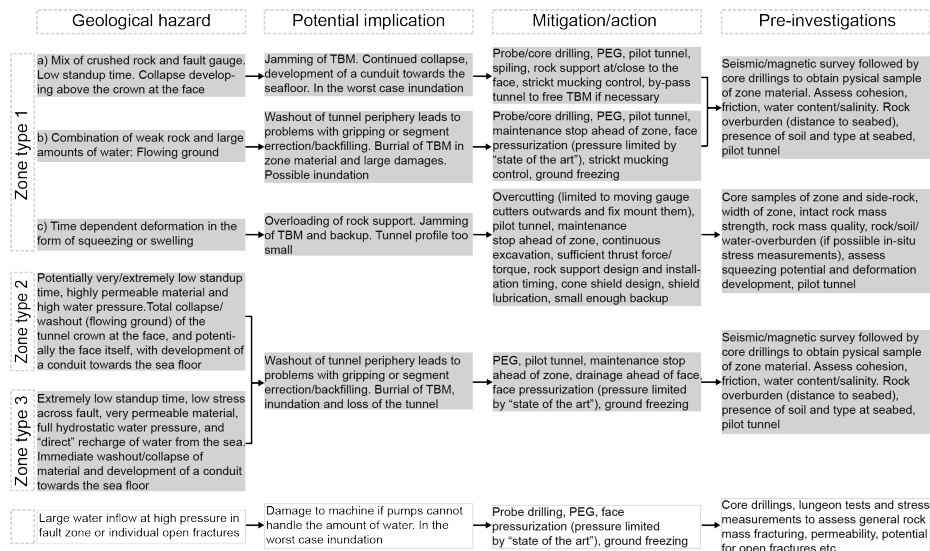
Geological hazard	Potential implication	Mitigation/action	Pre-investigations
Zone type 1 a) Mix of crushed rock and fault gauge. Low standup time. Collapse developing above the crown at the face	Jamming of TBM. Continued collapse, development of a conduit towards the seafloor. In the worst case inundation	Probe/core drilling, PEG, pilot tunnel, spiling, rock support at/close to the face, strickt mucking control, by-pass tunnel to free TBM if necessary	Seismic/magnetic survey followed by seismic drillings to obtain pysical sample of zone material. Assess cohesion, friction, water content/salinity. Rock overburden (distance to seabed), presence of soil and type at seabed, pilot tunnel
b) Combination of weak rock and large amounts of water: Flowing ground	Washout of tunnel periphery leads to problems with gripping or segment errection/backfilling. Burrial of TBM in zone material and large damages. Possible inundation	Probe/core drilling, PEG, pilot tunnel, maintenance stop ahead of zone, face pressurization (pressure limited by "state of the art"), strickt mucking control, ground freezing	
c) Time dependent deformation in the form of squeezing or swelling	Overloading of rock support. Jamming of TBM and backup. Tunnel profile too small	Overcutting (limited to moving gauge cutters outwards and fix mount them), pilot tunnel, maintenance stop ahead of zone, continuous excavation, sufficient thrust force/torque, rock support design and installation timing, cone shield design, shield lubrication, small enough backup	Core samples of zone and side-rock, width of zone, intact rock mass strength, rock mass quality, rock/soil/water-overburden (if possible in-situ stress measurements), assess squeezing potential and deformation development, pilot tunnel
Zone type 2 Potentially very/extremely low standup time, highly permeable material and high water pressure.Total collapse/washout (flowing ground) of the tunnel crown at the face, and potentially the face itself, with development of a conduit towards the sea floor	Washout of tunnel periphery leads to problems with gripping or segment errection/backfilling. Burrial of TBM, inundation and loss of the tunnel	PEG, pilot tunnel, maintenance stop ahead of zone, drainage ahead of face, face pressurization (pressure limited by "state of the art"), ground freezing	Seismic/magnetic survey followed by core drillings to obtain pysical sample of zone material. Assess cohesion, friction, water content/salinity. Rock overburden (distance to seabed), presence of soil and type at seabed, pilot tunnel
Zone type 3 Extremely low standup time, low stress across fault, very permeable material, full hydrostatic water pressure, and "direct" recharge of water from the sea. Immediate washout/collapse of material and development of a conduit towards the sea floor			
Large water inflow at high pressure in fault zone or individual open fractures	Damage to machine if pumps cannot handle the amount of water. In the worst case inundation	Probe drilling, PEG, face pressurization (pressure limited by "state of the art"), ground freezing	Core drillings, lungeon tests and stress measurements to assess general rock mass fracturing, permeability, potential for open fractures etc.

Fig. 7: Geological hazards, potential implication for tunnel boring, possible mitigation measures/actions to deal with the hazards, and necessary pre-construction investigations needed in order to assess feasibility of tunnel boring through the zones. The bottom row in the diagram represent the hazard of large water inflow at high pressure, which can occur in any of the three zone types, or be related to individual open fractures in an otherwise competent rock mass

4.2 Mitigation measures and pre-construction investigations

Geotechnical interpretive/baseline report (GIR/GBR): Identification of potential challenging zones and assessment/prediction of rock mass behavior needs to be given great attention in the planning of TBM tunnels. Results and interpretations should be continuously updated throughout the pre-construction phase and summarized in the *GIR*. Experience has shown that full disclosure of geotechnical information will reduce the risk for both the owner and the contractor, and thus the project cost (Munfah et al. 2004). The intent of such disclosure and the use of a geotechnical baseline report (GBR) are to allocate and share underground construction risks between the owner and the contractor.

Seismic/magnetic surveys, probe/core drilling and down-hole tests: Probe drilling is normally done systematically during subsea tunneling (except when tunneling in closed-mode), and successfully helped to identify adverse conditions ahead of the face in the Oslofjord and Bjorøy tunnels. Probe drilling is especially important with regards to detect potential water leakage problems, because the general experience is that none of the main pre-construction investigation methods for the undersea section of the planned tunnel (core drilling and refraction seismic investigation) give reliable prediction of the risk of large water ingress (Nilsen 2014). Nilsen illustrated the same for the much used Lugeon test. With regards to tunnel stability, seismic surveys and probe drilling can never precisely predict the

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rock mass behavior, illustrated by the unexpected collapse in the Atlantic Ocean tunnel. Core drilling in the pre-construction phase, and/or during construction, is important and can give more reliable information. Magnetic surveys should also be considered in order to correlate and verify interpretation of seismic surveys. For several completed Norwegian tunnels there have been indications, however not conclusive, that the magnitudes and orientations of rock stresses have distinct effect on water ingress (i.e. steep joints oriented perpendicularly to the minor principal stress give largest ingress) (Nilsen 2014). Hydraulic fracturing or overcoring tests, typically done in core drilling holes in the planning phase, or from within the tunnel during construction, are encouraged in order to get a basic understanding of the stress regime. This information can also be valuable for evaluation of other aspects, such as the potential for squeezing.

PEG: Possibility for probe drilling and PEG from a TBM, especially through the face, is normally limited compared to D&B, and the larger TBM cross section (for road tunnels) involves a larger rock volume that needs to be grouted. A TBM has fixed locations and a finite number of drilling-positions/angles available, compared to the highly flexible and "unlimited" options with D&B. The drilling rig capacity is normally also smaller with TBM, and drilling through the face can be problematic for further boring in the case of jamming of steel drilling rods (glass fiber or plastic rods can be used). Installation of blow out preventers can also be challenging and time consuming, especially when the grout umbrella needs a large number of holes like at the Bjorøy and Atlantic Ocean tunnels.

Pilot tunnel/fixed D&B section: If pre-investigations in the form of seismic surveys, core drillings etc. cannot give sufficient information about the expected conditions, a pilot tunnel can be used. A pilot can be excavated as part of the pre-investigation program, or as part of the main construction works. It can be a thoroughgoing tunnel or a shorter exploration tunnel ahead of the main tunnel(s). A pilot will give unique insight on rock mass behavior, and will enable other necessary operations (PEG, ground freezing, complete excavation of the fault zone etc.) to be executed ahead of the main tunnel(s). At some international projects a pilot tunnel has been included into the overall tunnel concept. At the Channel tunnel a centered pilot was used for exploration and ground conditioning ahead of the main running tunnels in the construction phase, and today functions as an emergency and maintenance tunnel in the operating phase. At the Brenner basis tunnel project, between Austria and Italy, an exploratory pilot tunnel is constructed below the two main running tunnels. It will

be used as an emergency, maintenance and drainage tunnel in the operating phase. Construction of a by-pass or pilot tunnel from the main TBM tunnel can be time consuming, because the TBM and the backup are blocking access for effective machinery (e.g. automated multi-boom drilling rigs etc.). A by-pass/pilot excavation might have to start behind the TBM backup (L3) and hence it could take long for the pilot to reach the critical area once a challenging event has initiated. The most logical option would be to map potential zones well ahead of the TBM with probe and core drillings, and to have provisioning's in the contract that allows for the construction of a D&B section. The challenge will however be to identify the most hazardous zones or leakage channels ahead of the face with sufficient certainty. In this respect core drilling during tunneling might have to be relied upon to a greater extent.

    Rock support, squeezing: The possibility to install radial rock support on a TBM-O is limited to about 5 m behind the face (could be somewhat reduced to ca. 3.5 m). The same is true for spiling bolts. The cutterhead and shield of both TBM-O and TBM-S will in the event of a collapse to some extent limit the free space for the rock mass to move (stabilizing), but the increased friction resulting from the collapsed material must be considered. In the event of void formation the possibility for rapid backfilling and shotcrete/concreting at the face and above the shield is limited, and the delayed rock support installation can give problems in squeezing rock. Ramoni and Anagnostou (2011) reported that trouble-free application of over-boring technology seems to be possible only in very soft rocks, and Gollegger et al. (2009) reported problems with hydraulically extendable cutters for the TBMs in the Gotthard base tunnel. Overcutting possibilities is considered to be limited to the fixed design of the TBM cutterhead, and the possibility to move the gauge cutters outwards and fix-mount them (possible for approximately 50 mm extension). The non-variable dimensions of TBMs and the delayed possibility to install radial rock support makes tunnel boring sensitive to large displacements. Relatively small convergences of 10–20 cm may lead to considerable difficulties in the machine or in the back-up area (Ramoni and Anagnostou 2011). In order to assess TBM feasibility where squeezing is expected, and for input to machine selection and design, it is crucial to evaluate ground/shield/support-interaction and installation timing in the planning phase. A first approximation of squeezing potential and degree of tunneling difficulty can for example be found using the Hoek and Marinos (2000) strain (ε) vs. rock mass strength/in situ stress (σ_{cm}/p_0) plot for unsupported tunnels (see Sect. 4.3.2). The plot is based on data from documented case histories and a simple closed-form analytical solution for a cylindrical tunnel in a hydrostatic stress field. The overburden represents the

magnitude of in situ stress, and the internal support pressure is assumed to act uniformly on the entire perimeter of the tunnel. Hoek and Marinos (2000) recommended that the tunnel should be subjected to detailed numerical analyses where significant potential for squeezing problems is identified. In order to evaluate these problems, and to investigate the interaction between the ground and the rock support, the convergence confinement method (CCM) (see e.g. Carranza-Torres and Fairhurst 2000) is often used. In the planning phase a ground reaction curve (GRC) is typically obtained from staged (core relaxation – simulating gradual tunnel advance) 2D plain strain numerical analyses. In order to correlate a given point on the GRC to a specific distance behind the face, a longitudinal displacement profile (LDP) is needed, which can be obtained from proposed analytical methods, and axi-symmetric or full staged 3D numerical analyses. A support characteristic curve (SCC) is used to find the appropriate support and installation timing in order to keep convergence within the required limits. As discussed, the available support measures and installation timing differs between D&B and TBM, and between different types of TBMs. The GRC and LDP are calibrated with actual measured deformations during construction, but in order to choose the appropriate tunneling method (and TBM specifications), it is important that estimations in the planning phase are as accurate as possible. Vlachopoulos and Diederichs (2009) showed that the ultimate size and shape of the plastic zone around an advancing tunnel can significantly influence the rate of displacements, and that previous analytical functions for LDPs are inadequate for tunnel analysis in very week ground at great depth (could lead to significant errors in the specified installation distance from the face, and hence the potential for failure of the temporary support). They proposed equations and a graphical template, which can be incorporated directly in analytical solutions, or used to calibrate staged 2D numerical models. The traditional use of the CCM assumes that installation of rock support does not change the rock mass response. Amberg (2011) described that re-compression of the ground occur when rock support is placed during ongoing deformations, and that this can lead to a support pressure equilibrium lying above the GRC. It was pointed out that this effect can lead to a significant increase of the "true" rock pressure in squeezing conditions, and that simulations of the tunnel advance in three dimensions are required. Ramoni and Anagnostou (2011) presented basic considerations and decision aids (helpful nomograms) for TBM tunneling in squeezing ground, including assessment of the required thrust force, lining design, and the decision of TBM type. Various publications by these and other authors can be helpful in the planning phase of a TBM tunnel through squeezing ground.

Maintenance stop: Should be planned ahead of where adverse conditions are expected, in order to prepare the TBM for continuous excavation through the area without any longer stops. This is especially important in the case of closed-mode excavation (to avoid the need for maintenance under pressurized conditions) and in the event of short stand up time and squeezing (to be able to install rock support as fast as possible and avoid development of ground instability/convergence).

TBM pressurization: Pressure is limited to "state of the art", which for larger diameter TBMs (Ø >12 m) is considered to be ca. 100 m (with reference to the Eurasia tunnel project). For higher pressures the limits for segment lining and TBM design with regards to e.g.; watertightness, handling-weight, component sealing, and thrust-force will be significantly challenged. It should be emphasized that, dependent on design, it can take some time (sometimes several weeks) to convert a machine from open- to pressurized- mode. Fast conversion was possible for the Lake Mead TBM with the use of a screw conveyor for muck discharge in open-mode. In order for a shielded machine, which is generally intended to excavate in open-mode, to have the possibility to excavate in closed-mode or convert into static closed-mode (in order to perform PEG before further advance in open-mode), the segments must be dimensioned for the expected water and ground pressures. This can mean a conservative lining design for the rest of the tunnel.

Ground freezing: Requires space for drilling of freeze-holes, space for the freezing installations, and space for a permanent (normally) undrained waterproof lining. It is important that adverse conditions are identified ahead of the face, and before escalation of the excavation problems. In a case where water is already flowing into the tunnel, to create a frozen ring can be very difficult or at best take a very long time. A "safe" distance/barrier between the tunnel and the zone in question would normally be required. Freezing should in light of the above not be considered as a method that can be used in all cases where extreme challenges are encountered, and/or where other mitigation methods are unsuccessful.

4.3 Feasibility of tunnel boring in deep Norwegian subsea tunnels

4.3.1 Zone type 1 - subtype a) and b)

Events in zone type 1a (increasing difficulty with presence of water) and type 1b (large amounts of water) can for open-mode TBM excavation be very time consuming and difficult to handle, shown by the type 1a zones of the Gotthard Amsteg tunnel (relatively dry), the Hong Kong SSDS Tunnel F (wet), and the Kárahnjúkar tunnel (very wet). It is important to emphasize that none of these tunnels are close

to the excavation diameter needed for future Norwegian subsea road tunnels. Experience from the large diameter Niagara project shows that large overbreak in a dry tunnel with unfavorable rock mass quality (however not a fault zone) *can* be handled with substantial efforts. However, large diameter TBM-O excavation through adverse conditions such as those experienced in the Atlantic Ocean tunnel will probably be extremely challenging. Quick action to provide face stability and to backfill voids above the tunnel crown will be hindered, and stepwise excavation and construction of a concrete lining at the face will be very difficult. The lower flexibility and capacity of PEG, and the larger required circular grouting area would add to the challenges. In order for ground freezing to mitigate collapse the zone must be detected at a distance ahead of the face. This will also be important in order to mitigate water leakages. Still, in order to drill the freezing holes and allocate space for the freezing operation, such works will probably involve the use of D&B excavation to construct a cavern, pilot tunnel etc. (see Fig. 8 for an illustration of the space required).
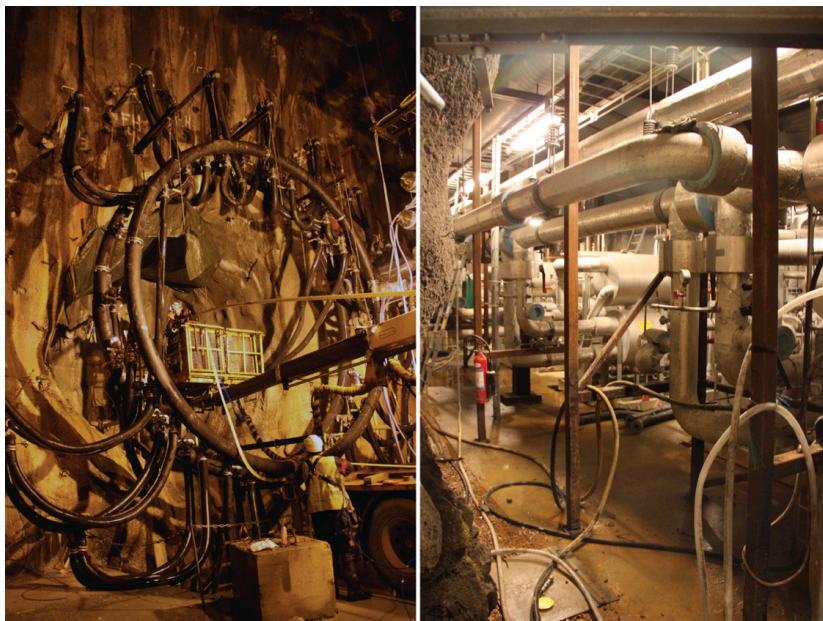

Fig. 8: Freezing operation (left) and freezing plant in a separate cavern (right) used to pass the MBZ in the Hallandsås tunnel. Photo: Ø. Dammyr

Closed-mode excavation through zone type 1 a) or b) is theoretically possible with the use of a TBM-Slurry or TBM-EPB (slurry the more likely option because of the potential high permeability and water pressures) optimized for hard rock conditions, and a gasketed segmental lining. However, closed-mode excavation through the long MBZ zone of the Hallandsås tunnel (zone type 1b) was considered

too high of a risk, and ground freezing was established. High wear to pressurized machines and slower advance rates in closed-mode would in practice mean the use of open-mode excavation for other parts of the tunnel. During a critical event it could be a challenge to achieve the conversion from open- to closed-mode fast enough (although some solutions exist). Even so, the maximum water pressure in many future subsea road tunnels in Norway will exceed 100 m (it will often be above 200 m), which would rule out the use of closed-mode excavation. Static closed-mode and maintaining a lower pressure would help to counteract water inflow rates while performing PEG, but the use of gasketed segmental linings above 200 m of water pressure would still be to push the limit of current technology, and there is no experience with long-term durability of the gaskets at these pressures.

Based on the discussed aspects in this paper, large diameter (>Ø12 m) tunnel boring through Norwegian subsea weakness zones of type 1 a) and b) is generally considered too high of a risk for water pressures above ca. 100 m. The use of pressurized TBMs could help to make shallower tunnels feasible, but possibilities and limitations of the excavation- and required waterproofing-method must be carefully evaluated. The construction of a smaller diameter pilot tunnel (with TBM or D&B) for investigation purposes before the selection of main tunneling method could help reduce risk and make large diameter TBM excavation feasible at higher water pressures. In areas of adverse ground conditions, PEG and other preparation measures can be executed from the pilot tunnel. The pilot could also serve a permanent purpose (escape, maintenance tunnel etc.), and there could be provisioning in the contract in order to allow for a D&B section in the main tunnel(s).

4.3.2 Zone type 1 - subtype c)

Methods to predict squeezing (Sect. 4.2) typically assumes that the rock mass can be regarded as having isotropic behavior. However, for the planning of future Norwegian subsea tunnels through potentially squeezing ground it is important to consider the potential stabilizing effect of the side-rock. Depending on the width of the weakness-zone, the more competent side-rock will to a smaller or larger degree contribute to stability. The largest deformations will theoretically occur in the center of the zone and decrease towards the side-rock. Another important aspect is the influence of parallel tunnels (twin-tube tunnels) on deformation development. Parallel tunnels could lead to increased convergence and decreased stability, but this will depend on the rock mass, the distance between the tunnels, the stress conditions, and the excavation and rock support sequence.

Fig. 9 shows 3D numerical modeling (Rocscience RS[3]) results from an unsupported Ø10 m circular tunnel excavated through a weakness zone of limited, but variable width, which intersects the tunnel perpendicular to the alignment. The characteristics of the Rogfast weakness zone (Fig. 2 top) and side-rock have been used as basis for the material parameters (see Fig. 10). The chosen values for UCS, E-modulus and Poisson´s ratio of the side-rock are from laboratory results obtained by Todnem (2014) on specimens from the "Krågøy BH-03-11C" core drilling hole (see p. 34 in that thesis). The properties of the weakness zone are obtained from the program Roclab (Rocscience), which uses mapped GSI values for the weakness zone to degrade the intact properties of the side-rock. Post-peak parameters were selected based on recommendations in Crowder and Bowden (2004). A number of runs with varying weakness zone width (10 - 50 m), and different isotropic in-situ stress conditions (2.5, 5.0, 7.5 and 10 MPa), were done in order to assess deformation potential. For the 50 m wide weakness zone the effect of twin-tube excavation, with different distance between the tunnels (1, 2 and 3-diameter pillars), were also investigated. The model is restrained on all ends (no deformation possible there) and comprise of 50 m side rock on each side of the weakness zone (making the model range from 110 m to 150 m in length). The height and width of the model is 200 m x 200 m, independent of the pillar thickness. The tunnel(s) were only excavated in one step (one step for each of the tunnels in the case of twin-tunnels). Due to very long computing times no GRC or LDP (which requires stepwise excavation) were obtained, and hence detailed rock support analyses are not part of this exercise. It is assumed that the obtained displacement values ("total displacement") relate to radial inward deformation (convergence) only, and that no deformation occurs in other directions. Only immediate squeezing effects are analyzed, and hence no time-dependent or possible swelling behavior was investigated.
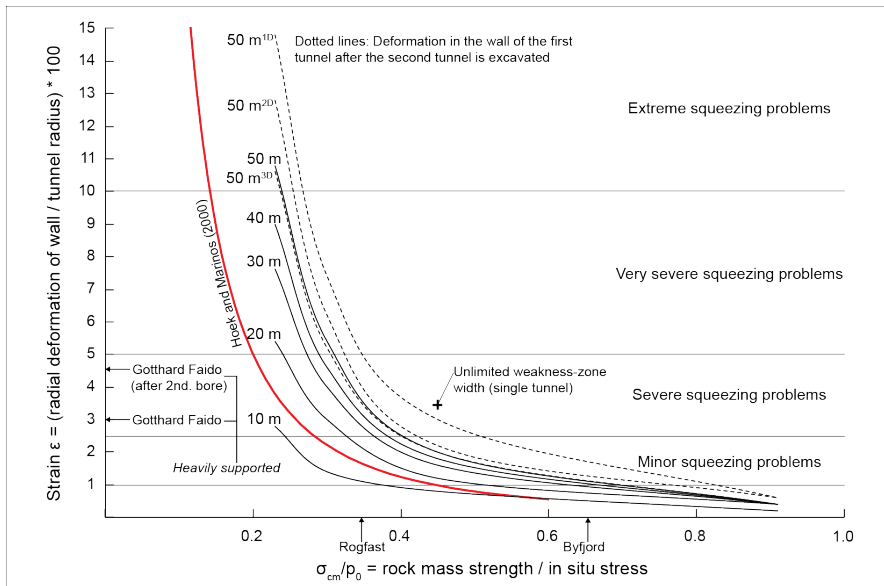
Fig. 9: Maximum tunnel convergence (unsupported tunnel) from 3D numerical modeling (Rocscience RS[3]). The red relationship line is from Hoek and Marinos (2000) and the background plot is slightly modified after the same authors. The solid black lines represent different weakness zone widths of 10 m to 50 m for a single tube tunnel. The dotted lines represent a weakness zone width of 50 m for a twin-tube tunnel with varying distance in between the tubes (1, 2 and 3-diameter pillar thickness). The black cross represents maximum deformation for a single-tube tunnel through a weakness zone of unlimited width (150 m) at 5 MPa of in situ stress

Fig. 9 clearly illustrates the effect of increasing weakness zone width and in situ stress. The theoretical σ_{cm}/p_0 ratio (0.35) for the Rogfast weakness zone is indicated. It can be seen that potentially severe squeezing problems (>2.5 % strain = >15 cm wall convergence for a Ø12 m tunnel) may be encountered if the weakness zone has a width of more than ca. 25 m. If the weakness zone is 50 m wide and parallel tunnels are excavated with a distance of only 1-diameter, very severe squeezing problems (>5 % strain) may be encountered. It is important to note that this maximum strain applies to one of the tunnel walls in the center of the zone. As can be seen in Fig. 10 the largest convergence will occur in the inward facing wall of the first tunnel after the second tunnel is excavated (keep in mind this is for unsupported tunnels). However, both tubes experience increased convergence compared to a single-tube tunnel. If the pillar thickness is increased to 3-diameters the effect of the second tube is in this case eliminated.

It is noticeable that the calculated deformations of Fig. 9 exceed those predicted by the red line of Hoek and Marinos (2000). This is especially clear by comparing the weakness zone of unlimited width with the corresponding point on the red line below. The validity of the chosen input parameters

cannot be verified from back calculations before tunnel construction, but the squeezing problems encountered in the nearby weakness zone of the Byfjord tunnel (assumed to have similar rock mass strength) indicate that the chosen parameters may be representative. Due to the squeezing potential this particular weakness zone of the Rogfast project should be subjected to more detailed analyses in the case of TBM excavation.

The above shows that assessment of TBM/ground/support-interaction is important in order to evaluate feasibility of tunnel boring through squeezing ground. It is further essential in order to choose the right TBM machine and equipment, and to optimize the pillar thickness. When the predicted wall convergence exceed 10 - 20 cm the choice of TBM excavation should be evaluated carefully. The GRC/LDPs are here vital in order to assess the convergence and ground pressure to be expected in the shield area. It will also be important to include provisioning's in the contract to be used in special situations; such as necessary remediation works of failed rock support and other concrete structures.
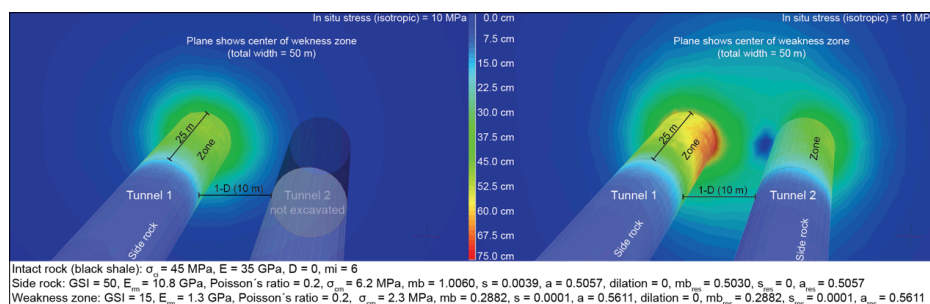

Fig. 10: 3D numerical model (Rocscience RS^3) and input parameters as basis for results in Fig. 9. This particular model shows excavation through a 50 m wide weakness zone (left), and the same situation for twin tunnels with 1-diameter pillar thickness (right). In-situ stress (isotropic) is 10 MPa. Intact rock properties (UCS, E-modulus and Poisson´s ratio) are from Todnem (2014)

4.3.3 Zone-type 2 and 3

The crossing of zone types 2 and 3 can be extremely challenging, regardless of excavation method. The potentially very low strength (sometimes cohesionless) and high permeability material, in combination with high water pressures, can result in flowing ground behavior. The consequences can be total collapse of the working face, followed by burial of the TBM and inundation of the tunnel. The substantial efforts needed to excavate safely through zone types 2 and 3 was illustrated by the Oslofjord and Bjorøy tunnels, where ground freezing and PEG had to be used at 120 m and 70 m depths respectively. Many future Norwegian subsea tunnels will be deeper, and hence potential water pressures will be higher. This can lead to high seepage forces, which can further destabilize the already unfavorable-quality rock mass of such zones.

As was pointed out for zone type 1, the low flexibility of a TBM with regards to space for extraordinary interventions (for example room to establish a large number of blow out preventers, or a cavern to drill freeze holes) makes tunnel boring more vulnerable than D&B in these situations. However, for weak zones at moderate depths (in this paper considered to be <100 m for large diameter TBMs) closed-mode tunneling may actually be advantageous over D&B. If the Bjorøy weakness zone had been encountered with a closed-mode capable TBM, it is possible that the zone could have been passed in a few days/weeks rather than in 9 months. Considering the inundation of the D&B starter tunnel and the long closed-mode TBM drive through a fault zone with cohesionless material at Lake Mead, adverse conditions may be tackled better with closed-mode TBM tunneling than with D&B. At water pressures well above 100 m the only feasible option will be open-mode tunnel boring, which will involve a very high risk in such zones. Based on the above, large diameter (> Ø12 m) tunnel boring through zone type 2 and 3 is generally considered to high of a risk for water pressures above ca. 100 m. The use of a pilot tunnel or a fixed planned D&B section can to some extent reduce this risk.

4.3.4 Water in fault zones or in individual fractures

The hazard of water leakages alone (i.e. no destabilization of the rock mass), can be reduced substantially by a systematic probe drilling campaign and having good PEG capacity on the TBM, as well as sufficient pumping capacity to handle water inflow. However, very high water pressures and open fractures may require the use of a large number of blowout preventers. Work to install such preventers, which in some cases also needs to be installed in the face, is a complicated process due to the restricted space around the machine and cutterhead. In order to grout a specific fracture the fixed and non-flexible drilling positions on a TBM could also be a disadvantage compared to D&B.

During the freezing operation in the Oslofjord tunnel a waterproof "door" was constructed at some distance behind the advancing face. In the event of a possible face collapse and inrush of water the door could be closed, and functioned in this way as an extra barrier to avoid flooding of the tunnel. Such preventive measures are difficult with tunnel boring, because the backup hinders such constructions to be built close to the face. The continuous need for support functions at a TBM, in the form of cables etc., could also make the door closing procedure take too long.

The identification of all permeable fractures cannot be counted on in the planning phase, and before the selection of tunneling method. Therefore, the conclusion regarding feasibility of tunnel

boring is the same as for the other zone types: The use of a TBM is generally considered too high of a risk for water pressures above ca. 100 m.

5 Conclusions

There has so far been limited research involving detailed discussion and evaluation on the feasibility of tunnel boring through weakness zones intersecting otherwise hard competent rock mass at great depth. This paper includes a comprehensive summary of TBM experiences relevant for hard rock TBM subsea tunneling, and discusses an evaluates the feasible limits for tunnel boring when weak and/or water bearing zones have to be crossed. The main conclusion is that the feasibility of tunnel boring through weakness zones in deep Norwegian subsea tunnels is directly related to the identified geological hazards in each case, and their potential implication to tunneling. The first step is always to thoroughly investigate the presence of weakness zones along the tunnel alignment, and to gather as much information about these zones as possible. If seismic/magnetic investigations are done, and directional core drilling follows the whole tunnel alignment (not always feasible due to the necessary long length, and collapse of the holes in weaker zones) without the identification of zones that are thought to give unfavorable rock mass behavior, tunnel boring may be considered feasible. The risk of encountering individual open and permeable fractures is however always real, and can potentially threaten feasibility.

Where weakness zones have been detected in the pre-construction phase the risk of very challenging events have been identified. A planned D&B section through such zones can be possible, but it requires the zones to be identified as having adverse rock mass behavior in advance, something that has proven difficult. In light of the above it has been concluded that large diameter (>Ø12 m) tunnel boring through weakness zones in deep Norwegian subsea tunnels involves a high risk, and hence that it is not recommended when the water pressure is above ca. 100 m. For lower water pressures closed-mode tunnel boring may counteract some of the risk aspects identified here, and can possibly be advantageous over D&B in certain cases (for example; closed-mode tunneling could potentially replace extensive and time consuming PEG). The single-shield TBM with option to convert into static-closed-mode may also be beneficial in the case of instability and large water inflow. However, these aspects must be evaluated for each specific case, and will strongly depend on cost effectiveness since it normally would imply the use of a conservatively designed gasketed concrete segmental lining (undrained solution) for the whole tunnel. Excavation of a pilot tunnel may be a way

to mitigate the increased risks associated with deep subsea tunnel boring, but will considerably increase the cost. The potential for squeezing should be evaluated for weakness zones of substantial width, and 3D numerical analysis are encouraged for zones where squeezing challenges are expected. What should be considered as "substantial width" will depend on e.g. rock properties and in situ stress, and will have to be determined using engineering judgement. Due to uncertainty normally involved in the pre-construction phase an exercise like the one shown in Fig. 9, where weakness zone width and in situ stress vs. ground pressure ratios can be varied, can form a good basis for further decisions.

If tunnel boring and D&B are both found to be feasible the choice of excavation method will often ultimately depend on cost factors. In this lie also the risk aspects, which the contractor has to evaluate for his bid. In this respect, thorough investigations that focus on obtaining important information for both D&B and TBM from an early stage and throughout the project, will be important to evaluate feasibility and to get the "correct" bidding price for the project. Further development with respect to probe drilling, PEG and pressurization may make the use of TBMs more suitable for the excavation of Norwegian subsea tunnels.

Acknowledgements

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