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# 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.

## 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 to 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

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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 tunneling (Section 2), and summarizes relevant international TBM experience (Section 3). In Section 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.

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 (Karlson, 2008, Nilsen, 2014). 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

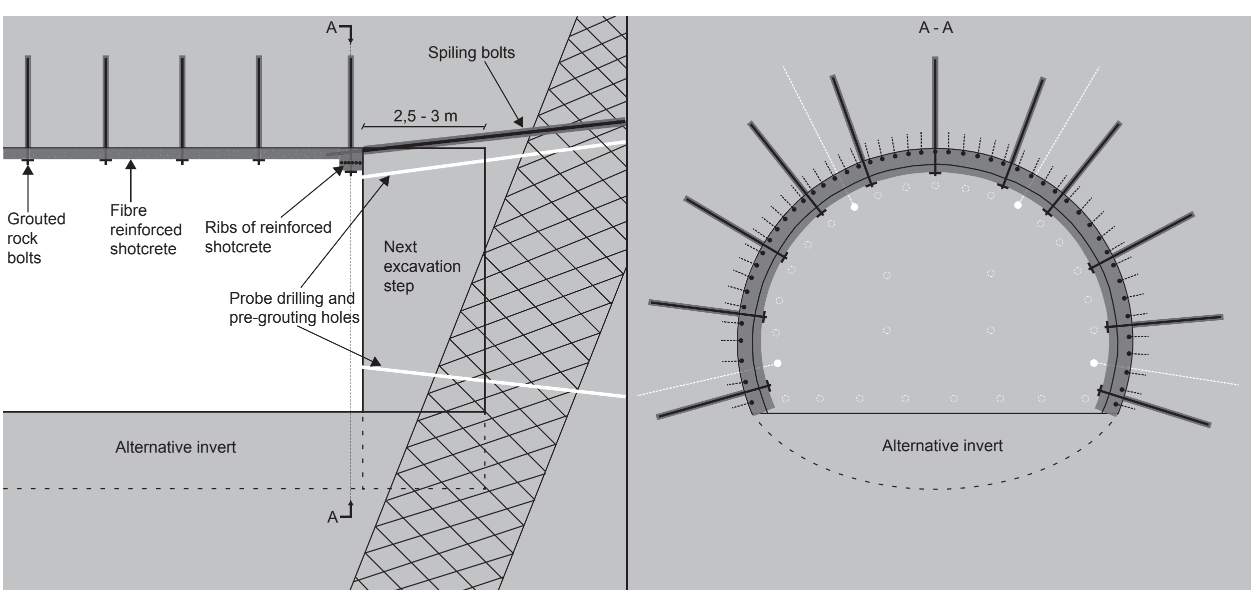


**Fig. 1.** Typical working steps to excavate through weak water bearing ground. White dashed circles make up drill holes for a grout-fan. Decision to grout is normally based on measured water leakage from the probe drilling holes (white filled circles).