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Underwater tunnel piercing

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PART 1

The master report

ABSTRACT

This thesis is divided into two parts. Part 1 – *Master report* is the introduction of the information and evaluation from the result of the research. Part 2 – *Underwater tunnel piercing - Project report* is the product of the study and is a separate report of underwater tunnel piercing. This report explains both the procedure and the calculation basis.

Underwater tunnel piercing is a process that is often found in construction of hydroelectric plant and shore approach for oil-, gas- and water supply projects. The tunnel piercing is a method that is used to blast the last part of the water tunnel out in the reservoir. Hence, it is a very critical and important operation which has many uncertain elements.

The purpose of this thesis is to make an updated document that collects relevant data and information. The study for the thesis have been the to find relevant data, see if there has been any change since the last report and do an evaluation of the calculation basis for the pressure condition in the blast. To achieve this study, it was used methods like literature research, discussions with specialists and on site-investigation.

The newest improvement that was found was principally the technological equipment, like; multi eco sonar, the use of electronic detonators for tunnel piercing and a discovery of the calculation basis.

The calculations done in Lysebotn 2 at both Strandvatn and Lyngsvatn the calculations where not similar to the measurements. From the study of the calculation formulas and the calculation used in real projects showed that there are still many unsure elements. The study showed that elements like cross section of the piercing opening was not included in the calculation. A possible cause of the error of the tunnel piercing at Strandvatn is possibly the weak condition of the rock mass that made the blast bigger than expected. This shows the impact these conditions have for the result.

Further work for this thesis would be to do a comprehensive study of only the calculations and collect several measurements from blasts of future tunnel piercings. A study of the gas development and a simulation could improve a better tunnel piercing calculation basis.

PART 1

Front page picture: Lyse AS

SAMMENDRAG

Denne oppgaven er inndelt i to deler. Del 1 – Masterrapporten er en introduksjon av informasjonen og vurdering av funn fra studiet. Del 2 – Utslag under vann – Prosjekt rapport er et produkt av oppgavens studie, og er en separat rapport om utslag under vann. Denne rapporten forklarer både fremgangsmåte og beregningsgrunnlaget.

Utslag under vann brukes ved utbygging av vannkraftverk, ilandføring av olje og gass prosjekter og bygging av vannforsyningsanlegg. Utslaget er en metode hvor den siste delen av vanntunnelen sprenges ut i magasinet/sjøen. Det er derfor en veldig kritisk og viktig operasjon som fortsatt i dag har mange usikre momenter.

Hensikten med oppgaven er å lage et oppdatert dokument som samler all relevant data og informasjon om emnet. Studiet for denne oppgaven har derfor vært å finne relevant data, se om det har vært noen endringer siden forrige utgivelse av denne type rapport og gjøre en vurdering av beregningsgrunnlaget av trykkforholdet under utslaget. For oppgaven ble det brukt metoder som litteratursøk, diskusjoner og samtaler med spesialister og befaring.

De nyeste forbedringene som ble funnet var hovedsakelig av teknologisk utstyr som; multistråle ekkolodd, bruk av elektroniske tennere til tunnel utslag og nye funn i beregningsgrunnlaget.

I Lysebotn 2 ble det utført en beregning for to utslag ved Strandvatn og Lyngsvatn hvor ingen av beregningen som ble gjort stemte overens med målingene som ble gjort. Fra en studie gjort av Solviks formler ble det oppdaget at diameteren på utslagsproppen ikke var inkludert i formlene. Begge disse funnene beviste at det fortsatt er mange usikre elementer ved beregningen. En mulig årsak for feil ved utslaget på Strandvatn er trolig fjellets dårlige kvalitet som gjorde at sprengningen ble større en forventet. Dette viser også at de geologiske forholdene også har en del å si for resultatet.

Videre arbeid for dette studiet vil være å gjennomføre en omfattende studie av beregningsgrunnlaget. En studie av gassutviklingen og simulering kunne også vært med på å utarbeide et bedre grunnlag for utslagsberegningen.

PART 1

PART 1

ACKNOWLEDGMENTS

This master thesis is my final work of the master studies at the faculty of Engineering at the Norwegian University of Science and Technology (NTNU), program for 2 years at Bygg og miljøteknikk. The thesis has a workload of 30 credits and written in spring 2018.

I will like to thank my supervisor Amund Bruland for his guidance and feedback during the writing period. I would also like to thank my external supervisor Olaf Rømcke for his great cooperation and interesting input. Iver Hauknes and Espen Hugaas has also been essential for helping me understand the calculations.

Thanks to Rune Sigmar Lien for having me at Smisto powerplant and for teaching me about the process of tunnel piercing. This have been a great indicator for this thesis.

Trondheim, 15 June 2018

Marta Horpestad

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PART 1

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PART 1

1.0 INTRODUCTION

1.1. Background

Underwater tunnel piercing is a Norwegian speciality that have been practiced for over 120 years. The tunnel piercing is the breakthrough round out towards the sea or reservoir. The operation is very critical and demands precision and control (NTNU-Anleggsdrift, 1999). The process contains a lot of uncertain factors which are important for a successful tunnel piercing. Like the investigation procedure, geology conditions and calculation of the pressure situation of the blast. From previous studies there have been found a lot of knowledge for both procedure and calculation, but the information is very scattered and not systematized. The last report that was made for this topic was made in 1999. KILDE?

Through the years there have been discussions between companies associated with the calculation of tunnel piercing, due to spread knowledge between companies. In the resent years it has hence been a wish from the industry to collect all information and clarify the method of underwater tunnel piercing.

1.2. Goals

The purpose of this thesis is to choose experience and knowledge of good quality through a literature research and introduce the knowledge in a summarized document for future use. The main goal is to cover the important aspects and find updates for both calculation and operation of the tunnel piercing. The updates are planned collected from companies and other informative sources. The goal of the thesis is to collect, pursue, and update this information, for the industry to use when planning a tunnel piercing. The main workload of the thesis it to:

- Find relevant and the latest information of the topic.
- Review what has changed through the years for this method.
- Verify the hydrodynamic calculation method.

1.3. Delimitation

The delimitation has been to concentrate on finding new literature and not carry out any model test or simulations of the calculation. The author's basis knowledge for the calculation have also limited the depth of the research and the time it took to understand.

1.4. Previous studies

A previous study of the author was done in autumn 2017, which made it clear that the information of this topic is not easy to find in public source engines. To find more data it has therefore been necessary to find other sources.

A lot of the older calculations are based on the work of Øivind Solvik. His work concentrated on describing the calculation of the piercing blast and the piercing methods that are possible to use. These reports have given a good access of the calculation basis and the description of the tunnel piercing process. Halvor Kjørholt and Jann A. Sandvik are also, among other people, someone who showed a lot of pronounced work for underwater tunnel piercing.

Other earlier studies that have been made was made in 1987 of a working group formed at the Division of Construction Engineering at The Norwegian Institute of Technology at the University of Trondheim. This was made by Amund Bruland, Odd Johannessen, Karsten Myrvold, Aud Røsbjørgen and Jarle Øverland.

The following edition of this work was made for a diploma thesis by Aud Røsbjørgen and Jarle Øverland in 1988.

In 1991 the project report was completed in both a Norwegian and an English version. The latest report was then made in 199 by Steinar Roald and Gry Helle Nakstad.

Part 2 – *Underwater tunnel piercing – Project report* is hence supposed to be the updated next version of this report.

1.5. Reading guidance of the thesis

The thesis is divided into two parts, Part 1 - *Master report* and Part 2 - *Project report*. The Part 2 - *Project report* holds a lot of theory that is essential for the understanding of the *Result*, *Discussion* and *Conclusion*. It is hence recommended to read Part 2 – *Project report* after *2.0. Methodology* from Part 1. The reason for this division is to enable Part 2 to be used separately from the master thesis for future use. Part 1 and Part 2 is therefore numbered with individual chapters and appendixes.

The preliminary chapters for Part 1 – *Master report* are the *Abstract* and *Introduction*. The following chapter *Method*, describes the work process of this thesis and an analysis of it. The next chapter *Result* will present the literature and data of interest that have been found. For the next chapter it is written an evaluation of the results in the *Discussion* and thereby summarized in a *Conclusion*. A suggestion for further work of the topic is written in the end of the conclusion.

Part 2 - *Project report* includes topics from planning process to calculation. In the end some project examples and calculations are described in the appendixes. This report does not contain figure lists and table lists, since it is a separate report.

PART 1

2.0. THEORY

This master thesis is as mentioned divided into two parts. The Part 2 – *Project report* is the product of the literature research and contains a lot of theory and knowledge of underwater tunnel piercing. It is hence recommended that the reader studies Part 2 – *Project report* after this chapter, before reading the *Results*, *Discussion* and *Conclusion*. This chapter is hence only describing the general and central concepts of the underwater tunnel piercing and the situation today.

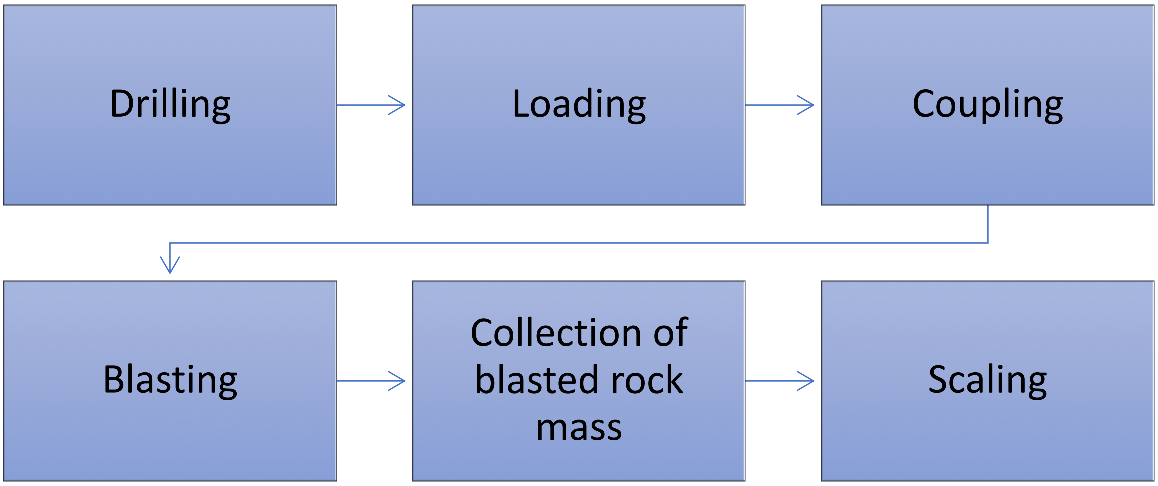
2.1. Tunnelling

The landscape in Norway is the main reason for the rich tunnel development in this country. The long fjords and high mountains make Norway a challenging country to develop infrastructure. As the tunnels started to replace the road and ferries, it became one of the most important infrastructures in the country. The landscape does also hold great opportunities, like hydroelectric projects, which had a great increase after 1950. This was a vital development for the industry in Norway.

The tunnelling in Norway started already back in the 17^{th} century, where mining was an important reason for the Norwegian economy. Skills and experience of the cavernous spaces and complex geometry from these times where important qualifications for the Hydro Power development in the 20^{th} century (NFF, 2017).

The most common excavation method in the tunnelling industry is the conventional drill- and blast method. This gives a great flexibility in handling the water leakage with conditional probe drilling and pre-grouting. The second method is the Tunnel boring Machine (TBM), also known as a full-face tunnelling method. The benefits of this method are the smooth tunnel contour which is favourable for reducing head loss (NFF, 2014).

The conventional method cycle of a round in a road tunnel:



2.2. Hydroelectric projects

99% of a total annual production of 140 TWh of electric energy in Norway is generated from hydropower. The principle of hydropower project is to capitalize on the energy of flowing water. The inflowing water will make sure that the turbine produces energy, where mechanical energy from the turbine is transmitted to a generator and transformed it into electric energy. Hydropower is a flexible energy source, due to the water storage in water reservoir. This means that the water can be used to produce power when needed. Another way to get water to the power station is by transferring water from rivers into the power station, also known as run-of-river power station (Statkraft, 2009).

The design of the hydropower plant does in many cases have several reservoirs and water tunnels. The reason for the lower location of the power station is to optimize the height of the reservoir. The height increases the speed of the water into the pressure shaft as described the figure below. One of the challenges is to have the right head loss for the system. The head loss is the energy in the water that is lost to the friction in the tunnel walls and turbulence. For larger cross sections the head loss is less (Statkraft, 2009).

(NFF, 2017)

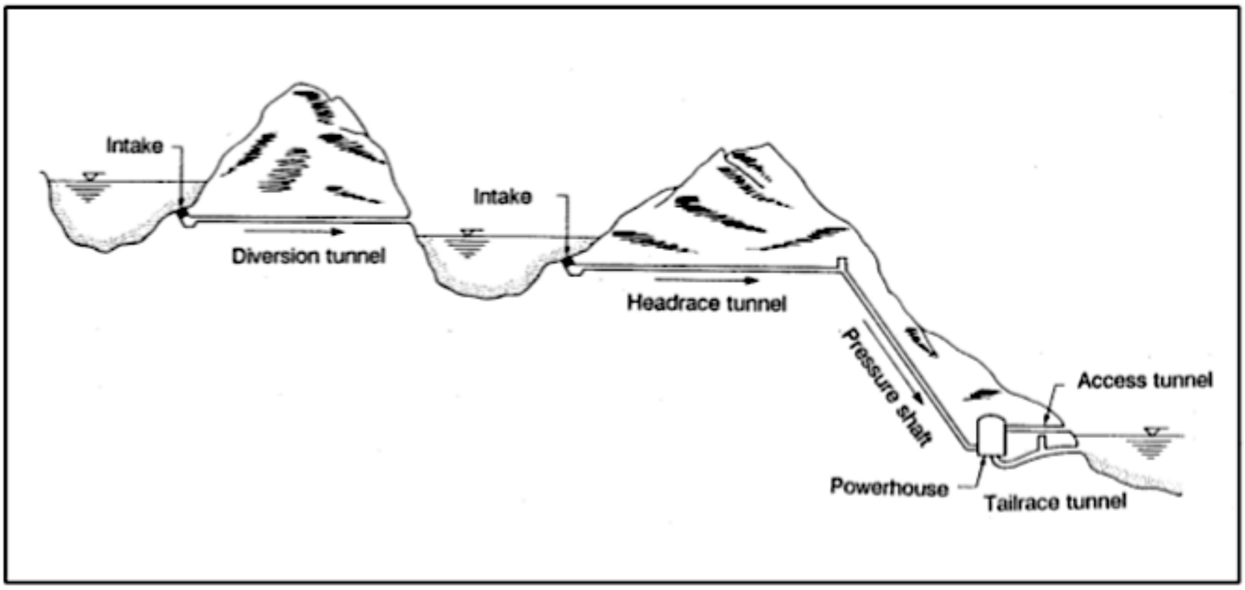


Figure 1: An example of a typical design of a hydropower plant (NTNU-Anleggsdrift, 1999).

2.3. Underwater tunnel piercing

The tunnel piercing or also known as the intake, will transfer the water from the reservoir into the tunnel. The closer the excavation gets the tunnel piercing area the more carefully the excavation needs to be carried out. Modifications like shorter round to minimize the blasting vibration and more frequently grouting is done to secure the rock mass and increase the safety of the area. The last blast is risky and demands high securement and control. In the blast of the piercing round, increased pressure and rock mass pushes into the tunnel. The challenge for the operation is to minimize the pressure and collect the rock mass (Solvik, 1995).

Underwater tunnel piercing is a technical solution which at first was only used and designed for hydropower projects. The method is a result of years of testing and modelling. The method is not only used for hydropower plants but also for water supply-, oil-, and gas industry.

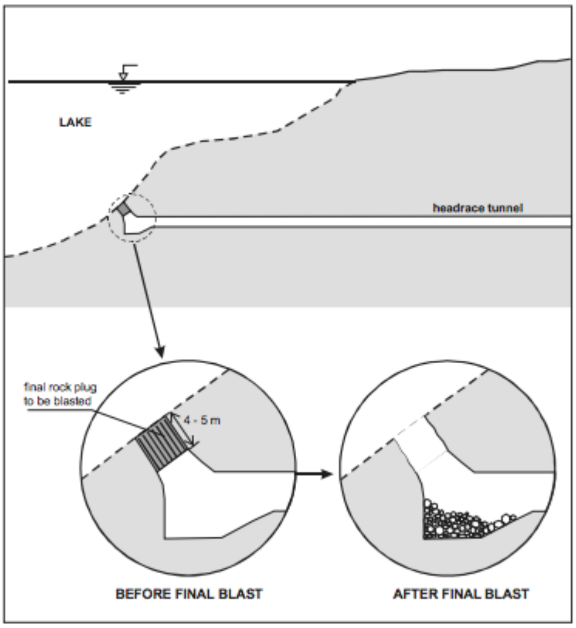


Figure 2: Illustration of the blasting situation for underwater tunnel piercing (Palmstrøm, 2002)

For the hydrodynamic considerations it is important to maintain control of the calculation to prevent damage on the gate, gatehouse, tunnel, and piercing area. All of these are damages that could cause a huge increase on the projects budget and reduce the safety of the workers (Solvik, 1986). However, the hydrodynamic calculation methods have a lot of uncertain factors and are therefore not always correct.

2.4. The history of underwater tunnel piercing

For several times, Norway have been under a glacier, which have eroded deep valleys and fjords. The varied landscape is one of the reasons why Norway is such a dominating country for hydroelectric power and tunnelling (Solvik, 1986). Over 600 piercings have been performed since the first one, which was carried out in Norway in Hardangerjøkulen in Lake Demmevatn, around 1890th. Ten years later, the first hydro-electric power station tunnel piercing was performed (Hugaas & Rømcke, 2017).

Very little experience was collected before the 1950th, which was caused due to the uncertainty of the piercing method at this time. An unsuccessful tunnel piercing was not something any contractors wanted published in the news, and hence a lot of important information was lost.

After the last World War, a rebuilding of the country started up. In this period, it was a specific concentration on hydroelectric power development. Due to lack of documentation the operators the builders could only use knowledge from their own experience (Solvik, 2007). The gained experience was useful knowledge for a while, but since all tunnel piercings were different, it was difficult to compare one tunnel piercing to another. Hence, it was not easy to create a template for this operation. The uncertainty caused a lot of danger to projects and people working there. This proved that more knowledge was badly needed (Solvik, 1986).

From 1963 to around 1994 it was done several studies as modelling experiments, counselling, and field measurements (Solvik, 1995). Laboratory test where carried out at SINTEF-NHL in Trondheim, where they did a scale model test. This resulted in the creation of the two main methods for hydroelectric projects, open and closed tunnel piercings (Solvik, 2007).

In the last 30 years, numbers of tunnel piercings have also been used at the North Sea in Norway, for gas and oil shore approaches. These piercings do usually have a much deeper depth than the hydropower reservoirs, where the lowest is measured at 200 meters at the North Sea (Hugaas & Rømcke, 2017).

2.5. Today

Today the calculation basis of underwater tunnel piercing is still not as exact as wanted. The study done by Øivind Solvik is furthermore the most used calculation basis for this operation, and little new information of the study has been done since. This calculation method does only give an estimation of what the results will be and cannot be used for exact answers. For this kind of calculation there are so many uncertain elements that can make an impact on the results. Through the years many underwater tunnel piercings has been successful, but also unsuccessful. Even though a lot of element for the tunnel piercing have been improved through the years, there are still many uncertain aspects that are jet to be solved.

3.0. METHODOLOGY

3.1. Literature study

A preparatory study was done before the master thesis. This study confirmed how scattered the information of this theme is and that little information was possible to find through public search engines. Most of the data that finally was found was project reports from NFF of projects that had carried out an underwater tunnel piercing. This study was informative in the way that it showed that it was necessary to make a further and more comprehensive study to find more suitable data.

It was hence necessary to contact specialists and companies that had experience and could have an idea of how to find more data. Through my co-supervisor Olaf Rømcke a lot of useful data was found. He has a lot of contacts that he has worked with on projects that has carried out underwater tunnel piercings. He also found informative reports made by Øivind Solvik, and most of the calculation for the master study is based on his studies carried out at the Norwegian Hydrodynamic Laboratory (NHL). The reports are therefore considered as credible and objective. Other sources of information material were project reports from resent projects from Implenia AS, Veidekke, Norconsult, which contained measurements, graphs, illustrations and evaluation of the underwater tunnel piercing process. A lot of these reports where very informative and where god examples of the different implementations of underwater tunnel piercing. This literature study is hence reckoned as the most enlightened.

3.2. Dialogs

An important source of knowledge has been through dialogs with specialists like my co-supervisor Olaf Rømcke and Iver Hauknes, Espen Hugaas. My supervisor Amund Bruland, have also a lot of knowledge, and have been of good help for discussions. Jon Martin Tangvik from Implenia AS and André Reynaud from Norconsult has also been contacted for further explanation and information of the Lysebotn 2 tunnel piercings. In March it was also done an onsite-inspection at Hæhre, where a lot of information of the tunnel piercing procedure was gained from discussions with the staff working at the project.

The material that have been collected from these conversation and discussions have been important for the authors understanding. Olaf Rømcke, Iver Hauknes and Espen Hugaas are all working for Orica AS and have been working with the calculations and blasting for the underwater tunnel piercing for many projects. They have a lot of experience and knowledge of the topic, and the information gained from them are considered as credible. The knowledge gained from these conversations where collected through phone calls, mail and meetings, and was a good help for the understanding of the calculation and the implementation. Some of the information gained could in some situations have been misunderstood and thereby not appear as precise as published reports. Some information in this thesis might therefore be less credible.

3.3. On site-inspection

In March 12th -15th an on-site-inspection at a hydropower project Smisto in Rødøy was completed. The principle of this site-inspection was to get a better understanding of the construction of the tunnel piercing method, which was of good help. Because the project was not finished, no data from this project was used.

4.0. RESULTS

To be able to understand this chapter the reader must first have read Part 2 – *Underwater tunnel piercing – Project report.*

A lot of the literature that was found from the research study is around 30 years old. One of the reports that was most used was "Metodevalg og Beregning" by Øivind Solvik. This report contains calculation basis for the pressure ratio of the tunnel piercing blast. This calculation is also known as the "Solvik model" and is still one of the main calculation basis of the piercing situation. Even though it is an old report is still the last comprehensive study that has been done of this topic.

From other material that has been found, it is discovered that the technology has the main improvement for the tunnel piercing process. For the investigation methods of the sea bottom the major change is the use of multi eco sonar. This makes the work easier and gives a very detailed investigation, due to several beams that can measure 15 different depths (Association of french sedimentologist, 1998). The core drilling has also developed new equipment to make it possible to log while drilling, which gives a good indicator of how the rock conditions is while drilling (Ludvigsen, 2017).

Use of electronic detonators is also an improvement for the tunnel piercing. The electronic detonator has existed since 1980s, but it is not until recently that it has been used for underwater tunnel piercing. The detonator makes it much easier to excavate a controlled blast. It also gives other advantages as smoother contour, less scaling, less overbreak, flatter stone pile, less rock support-, loading- and transport and better control of the vibration (Hauknes, 2018). For the tunnel piercing at Lysebotn 2 they used this type of detonator, which gave the opportunity to measure and have full control of all the detonators and thereby check if all the detonator was in place. Due to millisecond intervals the contour blast was very successful and gave the opening an optimal shape.

For the tunnel piercing at Lyngsvatn (Part 2 - Appendix F) the calculation based on Solviks method and the measurements from the blast where not similar. This shows that Solviks calculation method has some errors. The maximum pressure from the blast was calculated lower than what the measurements indicated. Talking to experts from Norconsult and Orica AS it was assumed that this error was caused by the unsureness of the calculation of the maximum pressure. An overview of Solviks method shows that the cross section of the tunnel opening is not included in the formulas, which could be another reason for the unsuitable results. It was also found that the water level in the air pocket was lower than assumed and therefore gave the air pocket bigger volume than what was used for the pre-calculation. However, the gas development from the blast with 50% development fitted the results from the calculation nicely, so the assumption of the 50/50 gas development was right.

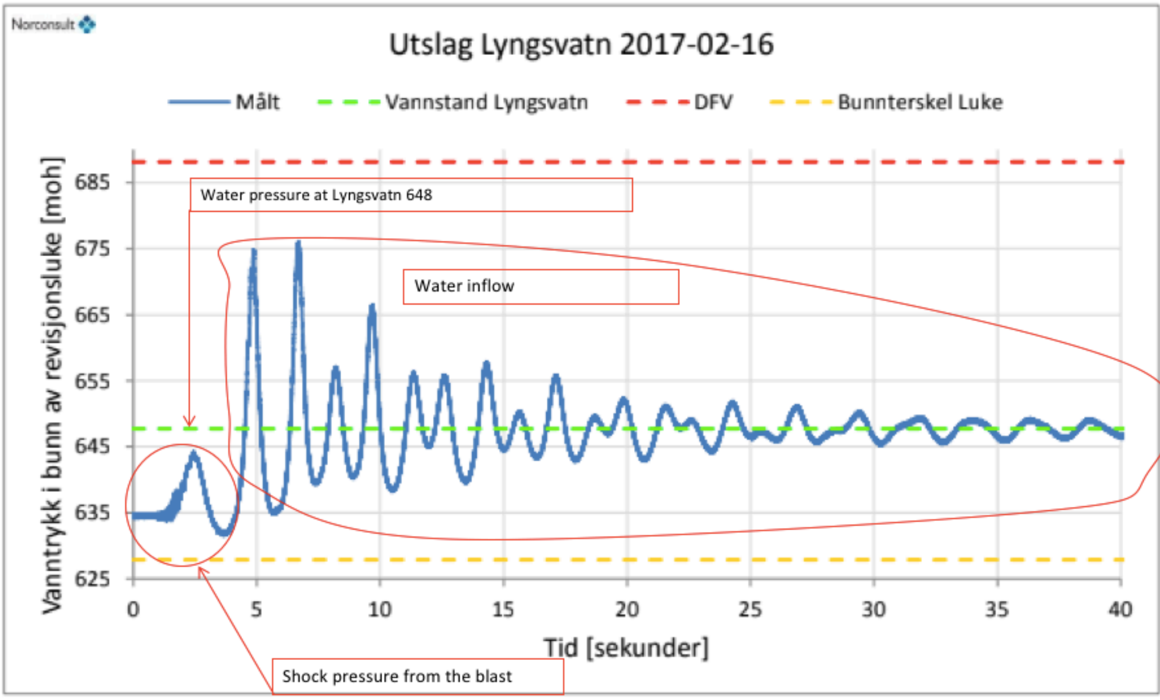


Figure 3: Pressure measurement from the tunnel piercing as Lyngsvatn, (Norconsult).

Table 1: Results from the calculation and measurement of the maximum pressure and the amount of gas development for the tunnel piercing at Lyngsvatn.

	Measured	Calculated
Maximum pressure	5.70 bara	3.08 bara
Gas development	3.18 bara	3.11 bara

In fig.3 the first wave is the gas development only, and the second wave includes the force from the inflowing water. This shows how big of an impact the inflowing water has for the blast. This is also shown in table 1, where you can see the difference between the calculated answers and the measured.

Another piercing blast from the same project in Lysebotn 2 at Strandvatn did also have errors with the calculations compared to the measurements. This tunnel piercing was very similar to the piercing done in Lyngsvatn but got a totally different result. This result went 11.7 meter over the dimensioning pressure. This shows that the maximum pressure is much higher and peaked over the dimensioning pressure, shown in fig. 4. The results from the blast the showed that the opening of the intake was larger than expected. The reason was later believed caused by a geological weakness zone, and thus a larger area had been blasted. With a larger opening than calculated, more water would stream into the air pocket and thereby cause a large pressure increase. This reveals the importance of the investigation of the geology for the results to be correct.

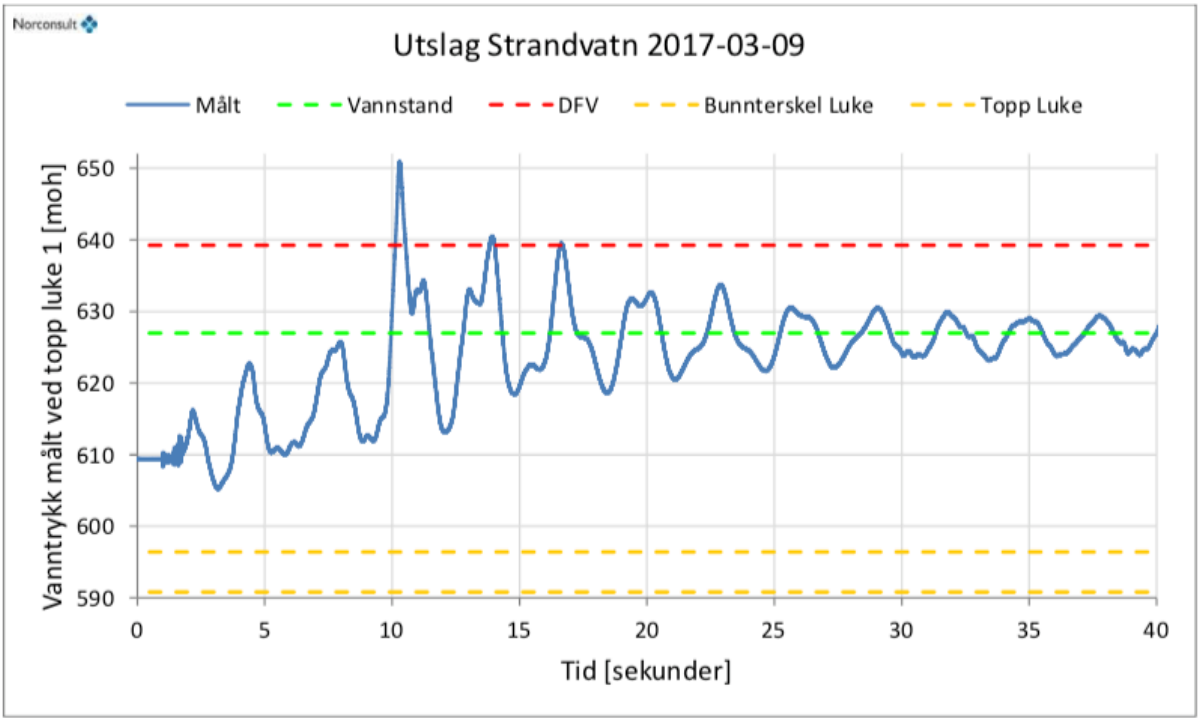


Figure 4: Result from the pressure measurement of the blast at Strandvatn.

5.0. DISCUSSION

For the calculation literature the prescribed source was evaluated to be scattered and a bit comprehensive. The challenge was then to simplify and structure it in a well-arranged document. Several reports of the calculation basis described the same method but with different parameters (P_2 og P_{max}). This was more confusing than informative and showed the importance of tidying the material.

The improved technology has in many ways made the investigation procedure both simpler and faster. Improved equipment is essential for a better tunnel piercing. With the multi eco sonar it is possible to measure the sea bottom more detailed and in a more efficient way. Another improved equipment is the core drilling, where the new improvement is the "measuring while drilling". These are all important improvements for a better and safer tunnel piercing.

Electronic detonators were invented in the 1980s, and the blasting results from these detonators are highly successful. The Lysebotn 2 project was the first project that used these detonators for an underwater tunnel piercing and had great success. A video of the opening after the blast did also show a satisfactory contour. The electronic detonators do also, among other advantages, have control over the leakage to earth and high precision.

Contacting companies who had access to data have been of great value. The data have mainly been project reports that are shown in appendixes in Part 2 and gives an estimation of how the tunnel piercing is done today. Data from Lysebotn 2 is especially very important material and illustrates the problems concerning the calculation method. The most interesting discovery of the calculation was the pressure increase caused by the water inflow. For the appendix *Pressure calculation of tunnel piercing at Lyngsvatn* it was found that the calculated maximum pressure was not the same as the measured pressure, which indicated incorrect calculations. As mentioned the size of the intake was not included in the formulas, which is assumed to have an impact on the inflowing water and pressure increase. Especially the relation between the cross section of the opening, external water pressure and volume of air pocket. With a small air pocket and a large opening, the efficiency of the air pocket will be decreased. But how much of an impact this ratio has on the pressure, must be further investigated.

PART 1

Before the tunnel piercing at Strandvatn, there had been other problems with the geological conditions, that had caused change of the placing of the tunnel. Even though the piercing area was moved to an area with better conditions, it was probably still not ideal when the round was blasted. The peaked maximum pressure from the results was thus probably caused by the conditions of the rock mass. If this is the only reason for the error from the blast is hard to say. But it does show that the geological conditions are often an unsure element and it also shows how important the investigation of the piercing area is.

The examples from Lysebotn 2 are just two of many tunnel piercings and can thus not make the basis of the problems for all tunnel piercing. But the discoveries that was found are however of great value and proves that there is still a lot of study to be done.

Some other unsure factors for the tunnel piercing are also found but are only speculations and are not based on any proofs. One of these is the amount of water filling in the tunnel. The more water there is in the tunnel the smaller the air pocket volume will be. The uncertainty increases with less volume of the air pocket. To know the size of the air pocket is very important, because smaller volume compared to the size of the opening and external water pressure could make the air in the air pocket flow out faster. Then the efficiency of the air pocket will not last as long as wanted.

Another uncertain element is the depth of the external water. It is important that this is measured from the bottom of the plug and not the top of it. But it is also and speculation of if this length also should include the extra meters from the top of the water-filling in the tunnel. The external water has a great impact on the pressure increase in the tunnel and could have a higher impact than is today known.

The gas development has also its uncertainties. The calculation uses 0-100% gas development, where the normal assumption is that the 50% goes into the tunnel and 50% goes out in the reservoir. But this information is only gained on experience and should hence been proved in a comprehensive study.

6.0. CONCLUSION

A lot of the literature that was found for the thesis was old but considered as the best information that exits. The main changes that have developed since the last report of this topic is the technological equipment. The multi Eco Sounder has improved better investigation and the use of electronic detonators for underwater tunnel piercing have also improved the blasting results and coupling procedure, due to the precision and accuracy.

The tunnel piercings at Lyngsvatn and Strandvatn, shows that this underwater tunnel piercing needs further study. The lack of certainty of calculation are in these examples proved, due to different answers of calculation and measurement. The reason for the geological errors at Strandvatn also shows the importance of good investigation of the tunnel area.

For further work it is after writing this thesis suggested to make a study on the uncertain elements that have been mentioned in this thesis. The water filling in the tunnel, gas development in the air pocket, pressure increase in the blast and external water pressure. These cases are only speculations but do show that a new study is imperative for a better knowledge. More data is also needed to be analysed and more updated information needs to be gathered.

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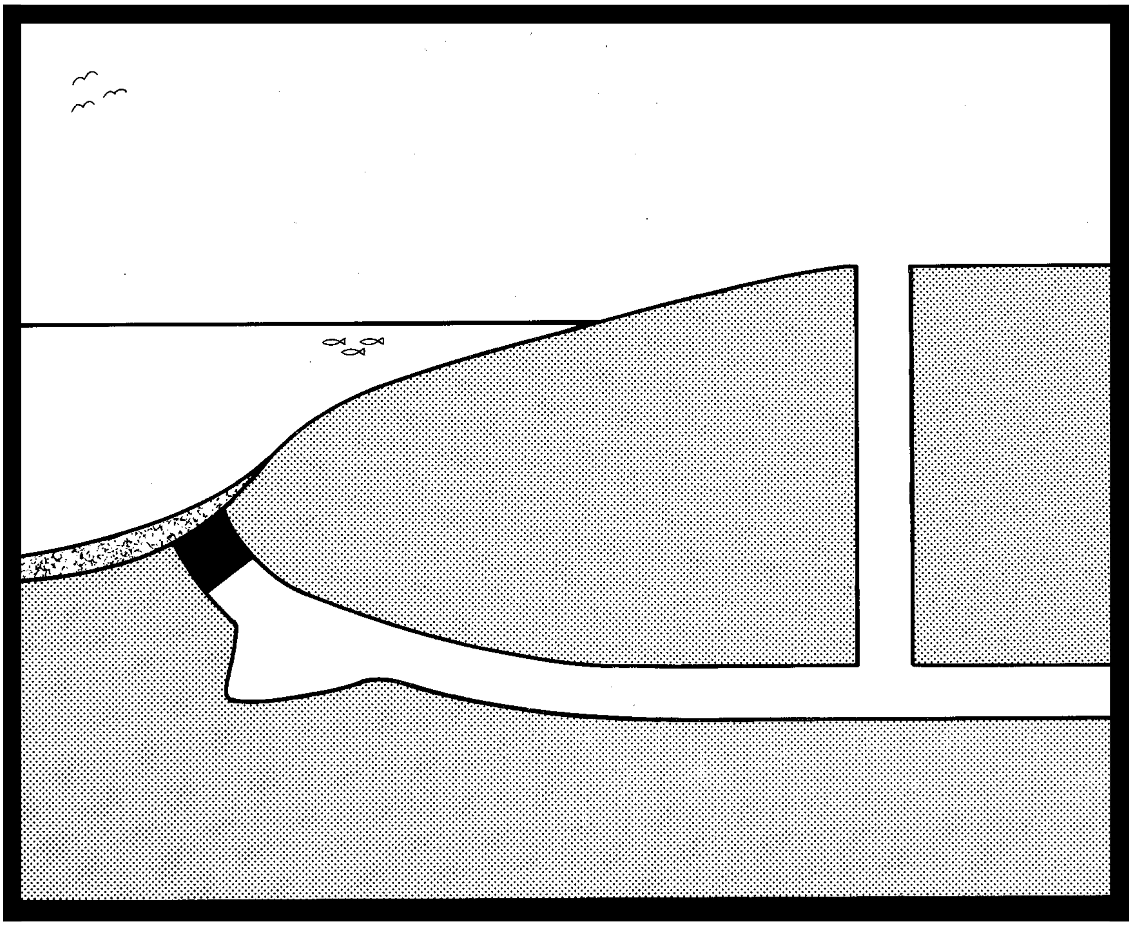
PART 2

Underwater tunnel piercing - Project report

PROJECT REPORT 20-18



UNDERWATER TUNNEL PIERCING

NTNU

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1.0. INTRODUCTION

Over 600 underwater tunnel piercings have been carried out in Norway since the begging in 1890s. Most of the tunnel piercings that have been carried out is for hydroelectric projects. For these projects the water is transferred through a tunnel towards the turbine. The method is also used for other projects like water supply project and shore approach for oil and gas pipe lines project in the North Sea. For future use it is estimated to be used for underwater tube bridges (NTNU-Anleggsdrift, 1999).

For the tunnel piercing there are several ways of carrying out the procedure. For hydroelectric plant there are two main methods. The difference is which gate are being used for stopping the inflowing water. Closed method uses the “revision gate” for closing, while the open method uses the “main gate” (see figure X).

Every tunnel piercing is unique and it is therefore difficult to create a template of how to carry out the excavation. The reason is due to the uncertainty geological conditions, blasting technique and hydraulic approximation of the blast (NTNU-Anleggsdrift, 1999).

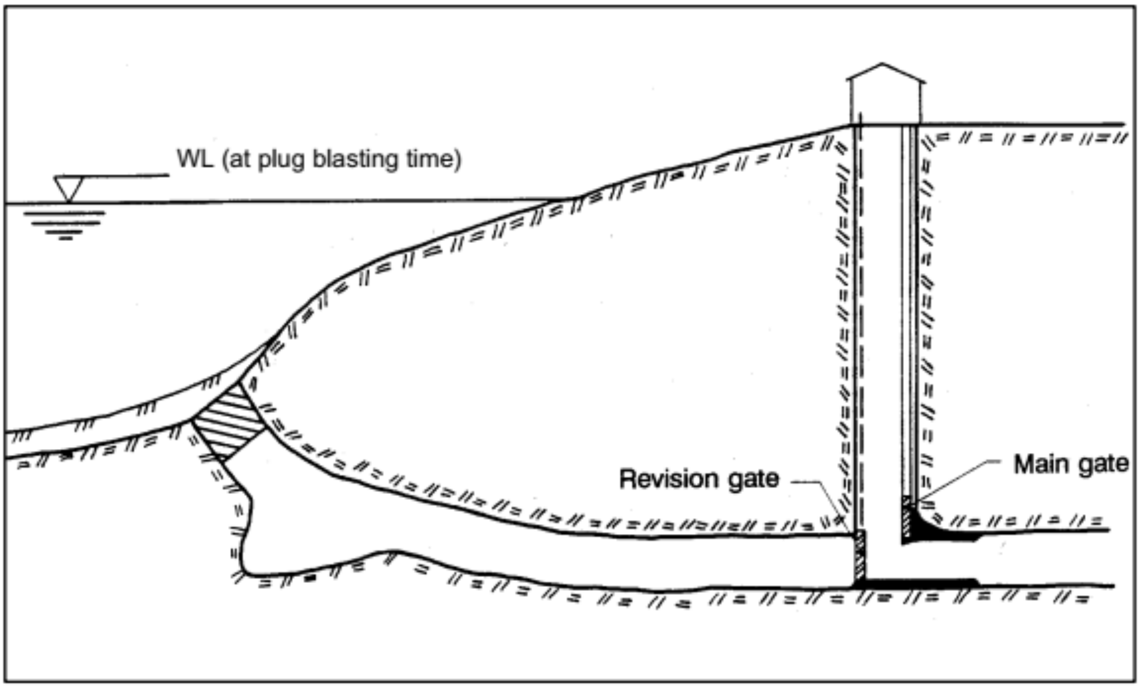


Figure 1: Revision gate and Main gate (NTNU-Anleggsdrift, 1999).

1.2. Open method

This system uses the main gate and allows the water to upsurge in the shaft, as shown in figure 2. The tunnel has an open connection to the atmosphere through the shaft up to the gatehouse. The shaft is also filled up with water and the water-filling will create an air pocket in front of the plug, which together with the water will work as a cushion on the explosion. The water-filling is done to protect not only the gate, but also the gatehouse. With no water-filling the damages could be much larger (NTNU-Anleggsdrift, 1999).

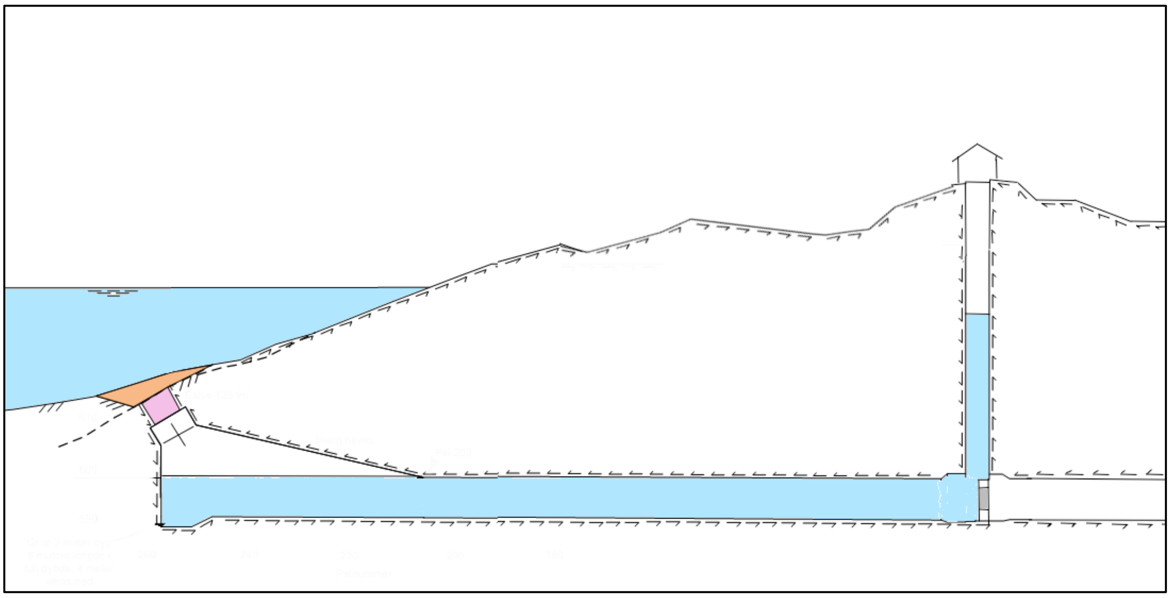


Figure 2: Example of a open method.

1.2.1. Tunnel piercing with concrete plug

Another method casts a concrete plug in the tunnel downstream of the gate construction. The advantages with this method are that the gates are spared from the pressure from the blasting and hence considered as the safest method to use. First the concrete plug is casted, then the tunnel is filled with water. Afterward the gate is pulled down and the water between the gate and the plug will then be pumped out and leave this part of the tunnel dry. The concrete plug will then be removed, (fig. 3). This method is often used in cases where the tunnel length from gate to plug is short. However, the concrete plug demands long setting time and so does the removing of it. In most of these cases the inflowing water will reach all the way up to the gatehouse, due to little retardation of the water. The tunnel is partly filled with water to reduce the inflow of the water (Norconsult).

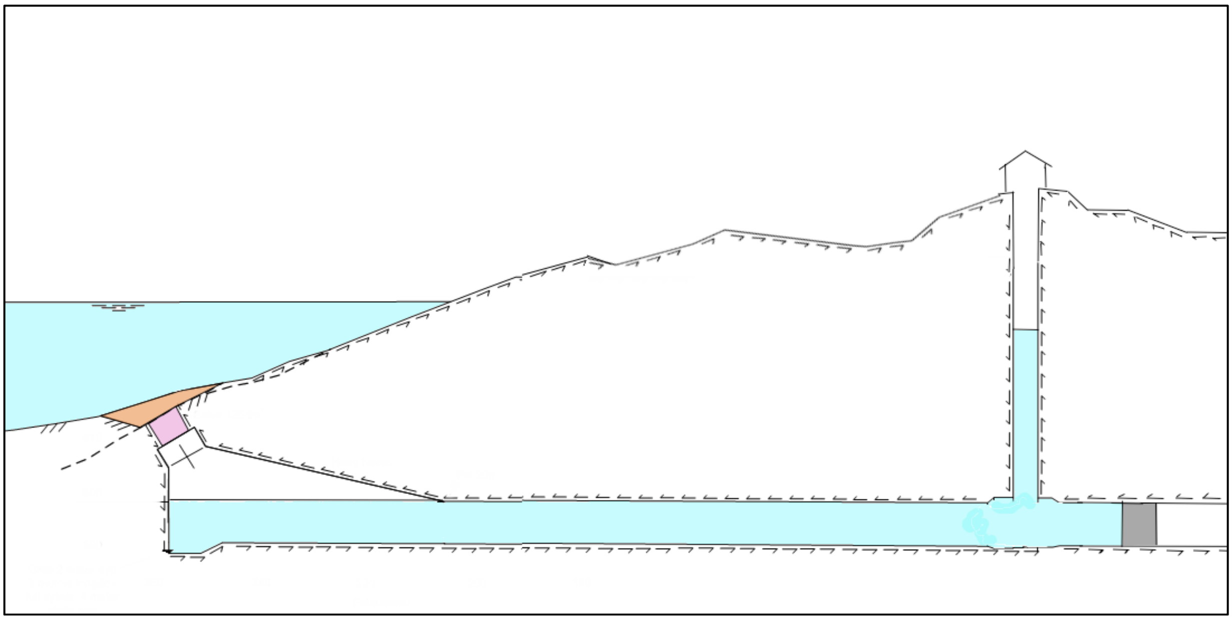


Figure 3: Example of a open method with concrete a plug.

1.3. Closed system

This method is divided into two categories, which is high and low water-filling. Low water-filling is very time-saving, due to less time used for filling (NTNU-Anleggsdrift, 1999). Overall, this method is less complicated and cheaper to preform than the open method. Due to the concern of blasted rock mass reaching the gate, the rocks are usually not of a large size due to the overcharging of the plug. Therefore, the problem is in fact estimated to be quite small (Solvik, 1995).

1.3.1. Dry tunnel piercing with low or no water-filling

It is possible to perform this method with or without pre-compression of the air. This method with can be done with none or little water-filling. A tunnel without pre-compression can only be accomplished if the tunnel is 20-30 times longer than the depth of the external water pressure (Solvik, 1995). For the longer tunnels, the collection of the blasted rock mass is hard to calculate and difficult to carry out, due to high velocity of the inflowing water.

Many projects that have used this method have had great success. The reason is that the water level in the reservoir often have been low and that the friction caused a lot of reduction on the pressure because of the tunnel length (Solvik, 1995). Side 28-29.

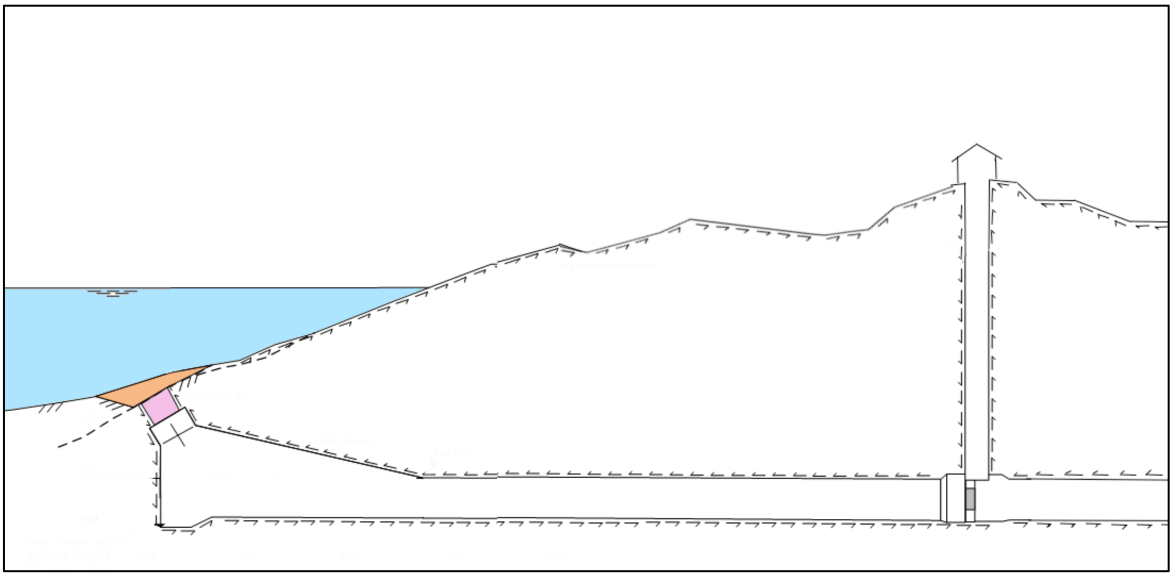


Figure 4: Example of a closed dry method.

1.3.2. Partly water-filled tunnel with pre-compression of the enclosed air

This method is more flexible due to the adjustment of water-filling which is made to get requested volume of the air pocket. Calculation of the pre-compression, gas pressure and the waters after-compression must be as accurate as possible. But due to many uncertain elements there are many elements to consider.

The velocity of the inflowing water will be incredibly lowered if the pressure after the explosion gets close to the external water pressure. This might result in unfortunate collection of the blasted rock mass, depending on the slope of the tunnel piercing. Therefore, if the piercing area is placed in a steep placing, the blasted rock mass collection is usually better (Solvik, 1995).

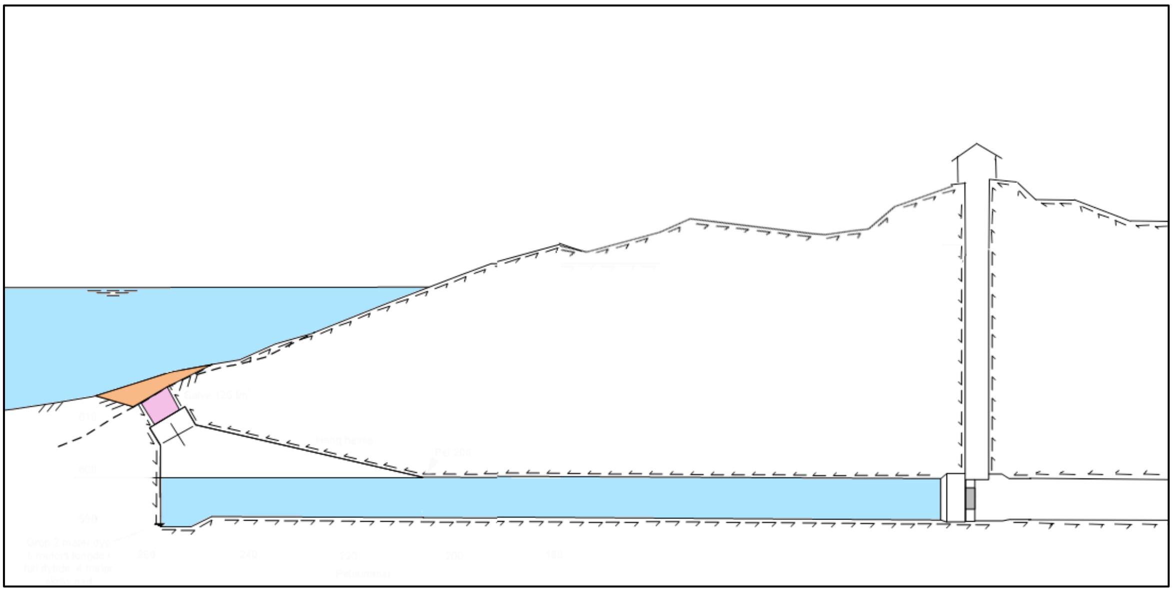


Figure 5: Example of a closed water filled tunnel piercing.

Table 1: A comparison of open and closed method.

Method	Open method	Closed method
Advantages	• Simple hydrodynamic conditions. • Largest pressure against the gate may be estimated with low uncertainty. • Good deposition of blasted rock mass.	• Time efficient. • Cheaper. • Little waiting time before blasting. • More flexible.
Disadvantages	• Demands more time. • More complicated to perform. • High costs. • Long latency before blasting.	• More complicated hydrodynamic conditions. • Requires longer tunnel between plug and gate. • Bad deposition of blasted rock mass. • Experienced personnel. • More demanding pre-calculation. • Requires explosives that withstands high pressure.

1.4. Tunnel piercing in closed chamber

For other projects like oil and gas, it is used another type of method. The method can be described as a short, closed intake. In the opening area where the installation of the pipe is constructed, it is sealed with a concrete wall to prevent water from leaking in and damaging the construction due to the blast. A possibility of the after compression of the air after the blast must also be accepted. In this type of projects high overcharging of the piercing round is normal, and the pressure is hence calculated the same power as a misfired round. No chances can be taken in this kind of a tunnel piercing and hence it is common to calculate higher pressure then assumed (Solvik, 1986).

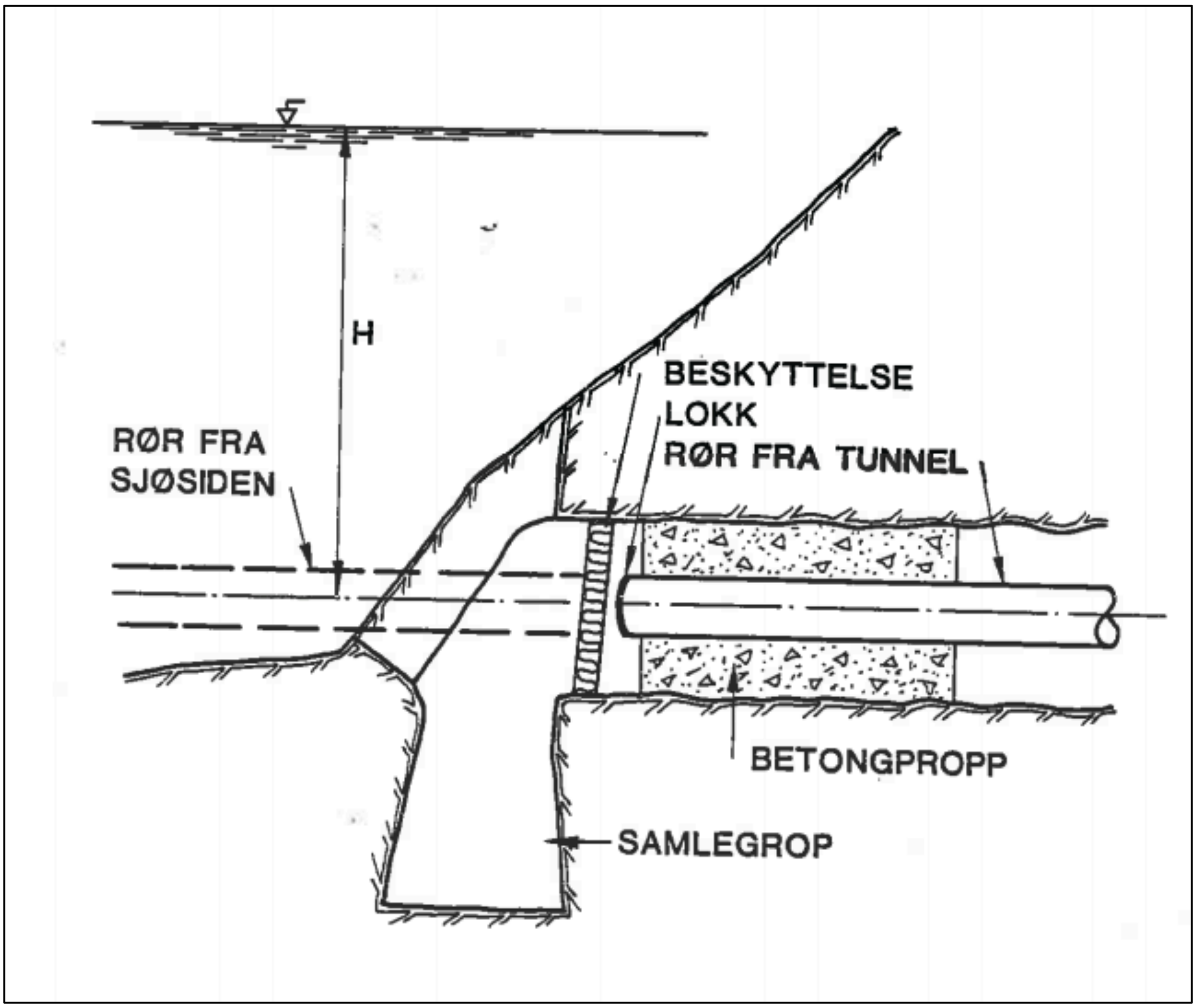


Figure 6: A illustration of a tunnel piercing in a closed chamber (Solvik, 1986).

Future use of tunnel piercing

A new use of underwater tunnel piercing is for tube bridges. The principle is that the bridge is placed into the water with solid rock mass on each side. To enter the tube bridge there must be excavated a tunnel from each side out in the fjord, and it is for this area that the tunnel piercing is used (NTNU-Anleggsdrift, 1999).

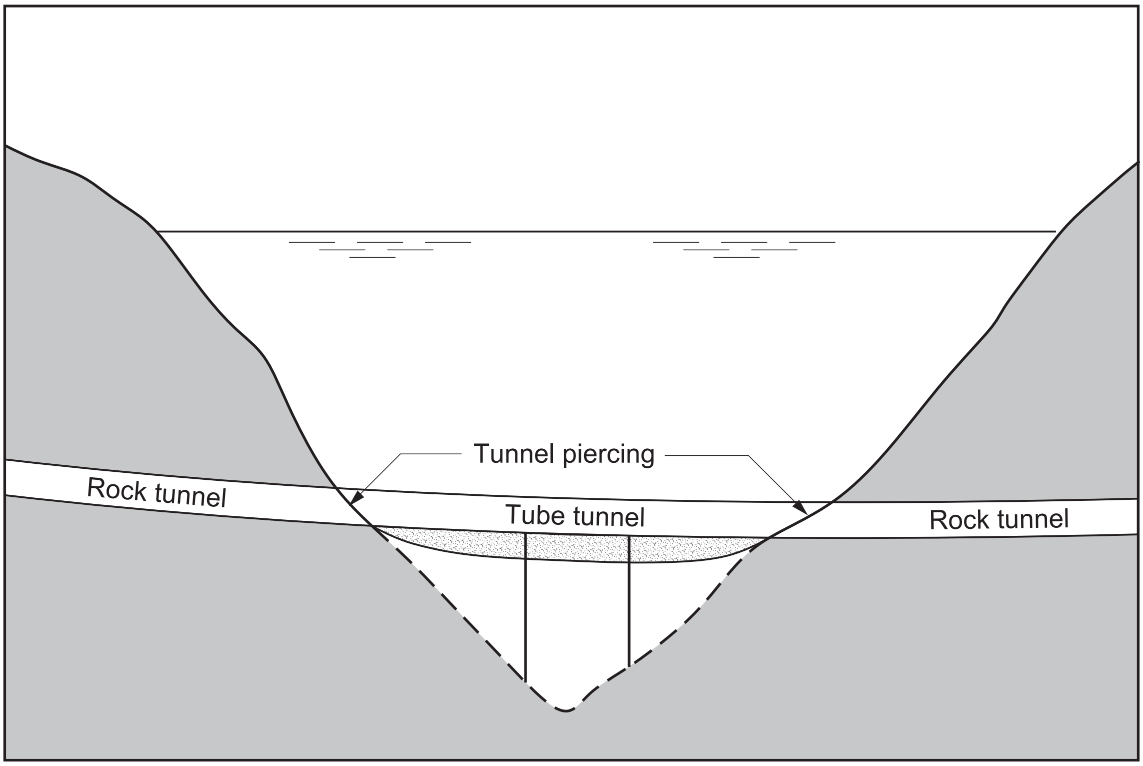


Figure 7: An illustraion of a tube bridge in a fjord (NTNU-Anleggsdrift, 1999).

2.0. PREPARATORY WORK

2.1. Planning

A tunnel piercing is never similar for any projects, due to of the various locations. The water depth is always different, as well as geotechnical conditions, area of the closing gate, rock mass quality and different methods of the construction design.

The most important choice when planning under water tunnel piercing is to find out what kind of method to use. It is also important to consider the reservoir requirement based on hydraulic conditions and financial profitability. Therefore, it is important to look at the topography of the sea bottom to find out where it is most suitable to place the piercing. Using maps and flight photos of the reservoir area gives an estimation of what the sea bottom looks like. Another type of planning is to consider the consequences if lowering a reservoir for constructional reasons, for example for construction of a dam. When lowering the reservoir, this generates a higher possibility of landslides or wave erosion. Rock landslide could in worst case appear due to vibrations of the blasting and could clog the tunnel piercing opening. For a worst-case scenario for both full or lowered reservoir with bad rock quality and risk of landslides, a solution could be to move the piercing to another area or stabilize the rock masses. Other sea bottom issue is the amount of sediments and the contents of fines, which can in large amounts harm the gate when entering the tunnel (Solvik, 1995).

2.2. Investigation methods

To be able to discover the issues mentioned above, it is crucial to implement some investigation other than maps and flight photos of the construction area. The investigation of the rock mass is one of the most important tasks of the tunnel piercing. It makes it possible to expose what to presume ahead of the tunnel face, which will reduce the cost of the project and increase the safety. Ahead of the tunnel face many unknown conditions as faulty zones, bad geology, water leakage etc, are possible to find with use of the right instruments and technics. Because the geology varies from area to area, there is no standard investigation procedure for the piercing area. The last decision of what method to use is usually based on the cost and the constructional situation.

From 1980-2000 there was a systematic improvement of the geophysical site investigation technique due to the increase in sub-sea tunnelling activity. One of the most vital improvements were acoustic profiling which gave the thickness measurement of the soil. The other improvement was refraction seismic that is a tool to find the rock mass quality and more exact location of the rock mass surface (Palmstrøm A. , 2002).

Investigation at deeper levels are more difficult, for example for oil and gas land-based tunnel piercings. For a tunnel piercing at a depth of several hundred meters the challenges increases. One of them is the problem facing the evaluation of geological conditions, like weathering, weakness zones, groundwater conditions, stress measurements, etc. (Palmstrøm A. , 2014).

METHOD	MAIN INFORMATION	MAIN LIMITATIONS	APPLICATION
Seismic refraction	- Thickness of soil layers - Location of ground water table - Location of rock surface - Approx. quality of rockmass	- "Blind zones" (if velocity does not increase with depth) - Side reflection	Extensively used on land and sea
Seismic reflection	- Locations of different layers (soil, rock, sea bottom, etc.) - Soil/rock structure	- "Blind zones" - Side reflection - Interpretation for great depths	Limited use (mainly used for sub-sea tunnels)
Crosshole tomography	- Rock mass quality - Karst caverns etc.	- Interpretation uncertainty	Increasingly used
Electric resistivity	- Location of ground water table/rock surface - Approx. character of weakness zones	- Interpretation - Stray current/buried metal	General use
Electromagnetic (radar)	- Location of ground water table/soil structure - Openings	- Restricted mainly to soft ground	Limited use
Magnetic	- Structural geology	- Interpretation	Minimal use
Gravitational	- Structural geology	- Interpretation	Minimal use

Figure 8: Table collected from Rockmass as. Shows an overview of the relevant geophysical methods (Palmstrøm & Stille, 2010)

A lot of unwanted situations can appear which are impossible to foresee. For many situations it is often best to wait and see how the conditions are before planning. This can save the project a lot of money and time.

2.2.1. Probe drilling

The probe-drilling is carried out to collect information of the conditions ahead of the tunnel face, like water inflow, faults- and weakness zones. The probe holes are drilled before the round drilling, to check if there are any necessary adjustments to carry out before continuing blasting. These holes are drilled into the tunnel face much longer than the normal drilling holes, normally they are 25 m and longer. How many probe holes that are needed, depends on the need of investigation (Grøv, 2017).

2.2.2. Seismic

To find the depth of the reservoir, weakness zones, amount of sediments and the depth to solid rock it is common to use seismic investigation. There are two types of seismic investigation methods; reflection seismic and refraction seismic (NTNU-Anleggsdrift, 1999). The reflection seismic uses an equipment that sends out sound waves into the water, which reflects when hitting jointed areas. The source of pulse is the transmitter, and the hydrophone is the receiver. By installing a radio transmitter at shore, it is possible to find the position of the boat. When the wave hits the rock or seabed surface it will reflect in different velocities. If the rock mass is of good quality it reflects higher velocity, typically higher than 5000 m/s. Lower rock mass qualities will then give 4000 m/s and lower (Grøv, 2017). This method can reach the depth of 1000m (NTNU-Anleggsdrift, 1999).

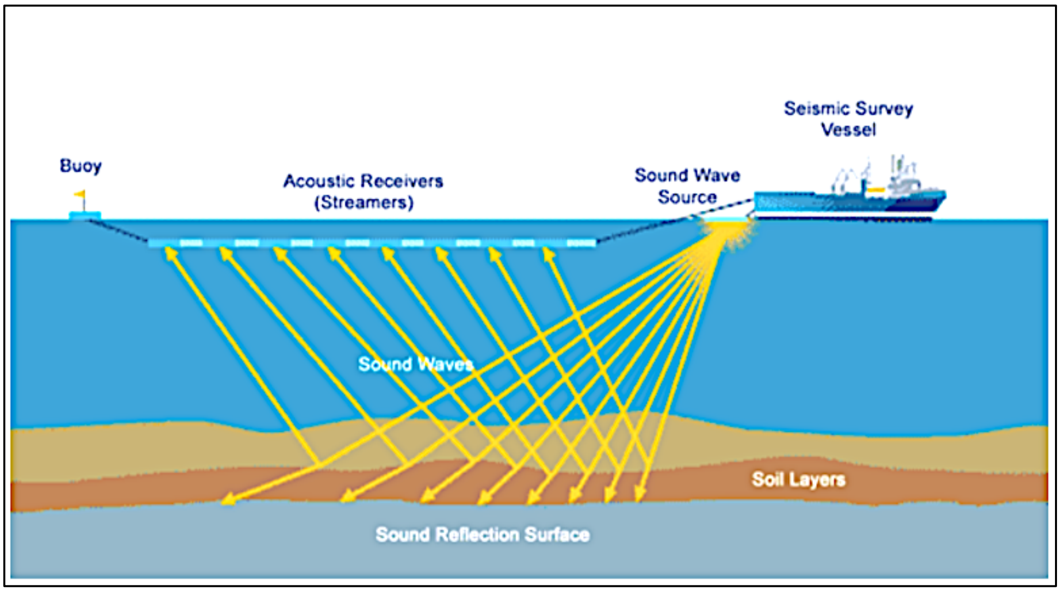


Figure 9: illustration of a reflection seismic (FishSafe, 2017)

The refraction seismic on the other hand is not as good as the reflection seismic but less expensive (Environscan, 2018). It has the same principle as reflection seismic, but the difference lays in the equipment. The geophones are placed in several detectors floating behind the boat, which receives the signals. Explosives generates the shock waves that transmits the pulses into the water. For a best result, the hydrophones measures on the sea bottom. This method can only reach depths less than 400 m (NTNU-Anleggsdrift, 1999).

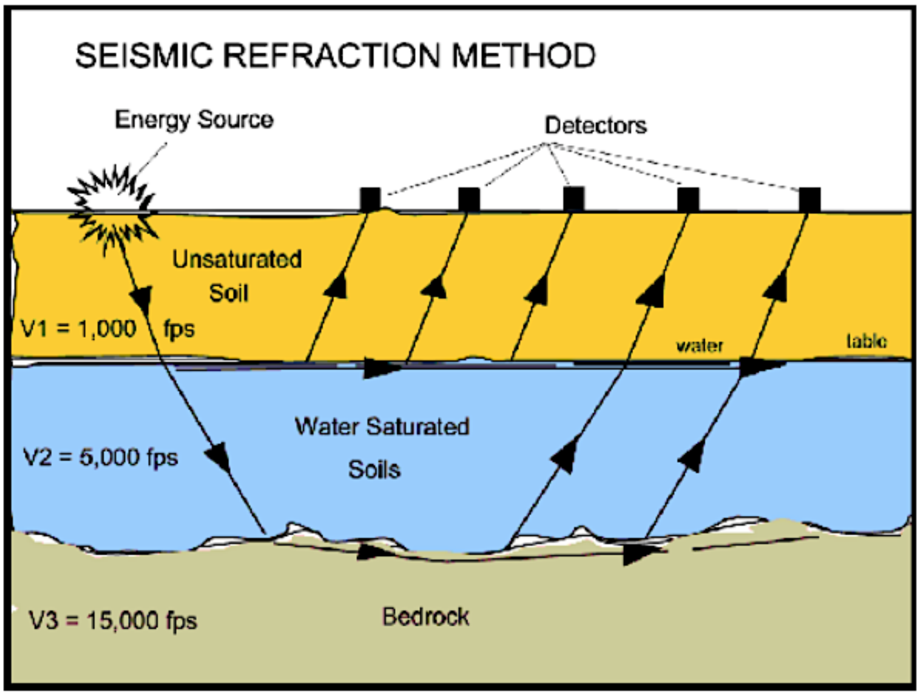


Figure 10: A illustration of the refraction seismic (GeologicResources, 2017)

2.2.3. Sonar Echo sounder

Todays most used equipment for sea bottom scanning is a tool called Echo sounder.

To be able make a surface cover of the sea bottom it is recommended to use a multiray echo sounder or a single echo sounder, depending on the size of the area.

- Multiray echo sounder
- Singelray echo sounder

Multiray is used for larger scans like finding the volume of the reservoir. A mulitray has several echo-sounders with 16 beams that has an aperture of 2°, and can measure up to 15 different depths. The equipment can measure the travelling time of the waves over oblique distances, as long as the propagation velocity of the water is known. This method and system is simple and has an automatic tracing of the seabed level, topographic profiles, block diagrams in perspective and isoslope lines (Association of french sedimentologists, 1998).

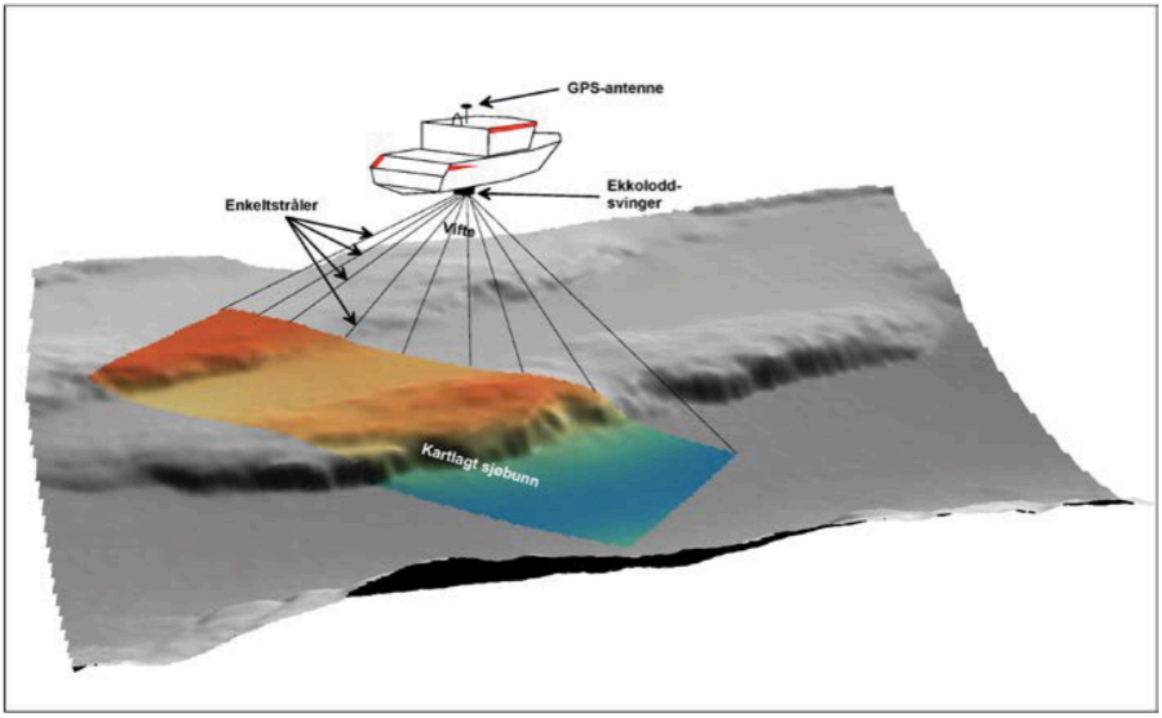


Figure 11: Illustration of the multi echo radar operation (Geosubsea).

2.2.4. Core drilling

This method is ideal for a more detailed inspection of the site and preferred when looking for fracture zones, leakage, and swelling clay. With the use of this method it is possible to reach the length of several hundred meters with core drilling. (NTNU-Anleggsdrift, 1999) However, the method is expensive due to use of special equipment and has a lower drilling speed than other drilling equipment (Ludvigsen, 2017). It should therefore only be an option when there is need for more detailed information. Core drilling is usually not used for deeper piercings; because these areas usually holds good quality rock mass (Solvik, 1986).

A new equipment made in the oil industry combines the use of boring tube and cable. The use of Logging While Drilling (LWD) makes it possible to do an examination continuously while drilling (Ludvigsen, 2017).

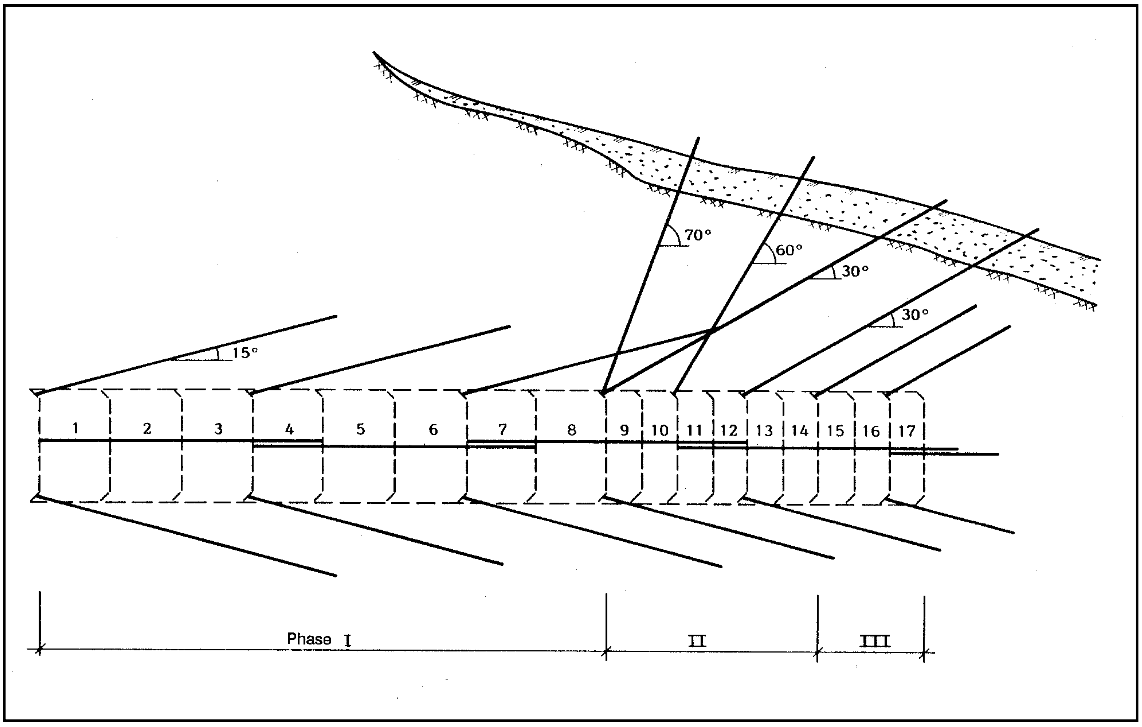
2.2.5. Visual inspection

Other ways to check the tunnelling area are by mapping, registration, and observation. From a map, it is possible to find information like strike, dip, swelling clay areas, fractures, and rock stresses. These parameters can give an estimation of what the rock conditions are at the face, but also further into the rock. The Q-system can also be used in this case to find out what kind of rock support that is needed for the specific area of the tunnel (Nilsen, 2016).

It is also possible to do a visual inspection of the sea bottom. A diver can be used, if the water depth is less than 50 meters. For deeper levels than 50 meters this method is inefficient because of time needed for decompression. Therefore, for deeper investigation a submarine vehicle (ROV) is recommended. ROV is an underwater robot, which is controlled by an operator. A ROV is running a custom C++ mapping software. The software is then recording sonar, depth, and compass and uses this to build cistern maps (The.Eurographics.Association, 2009). This machine can get a closer view of the area which is very detailed, but an overview of the area can be more difficult. After the blasting of the tunnel piercing it can be necessary to perform another investigation to check if the blasting was successful (NTNU-Anleggsdrift, 1999)

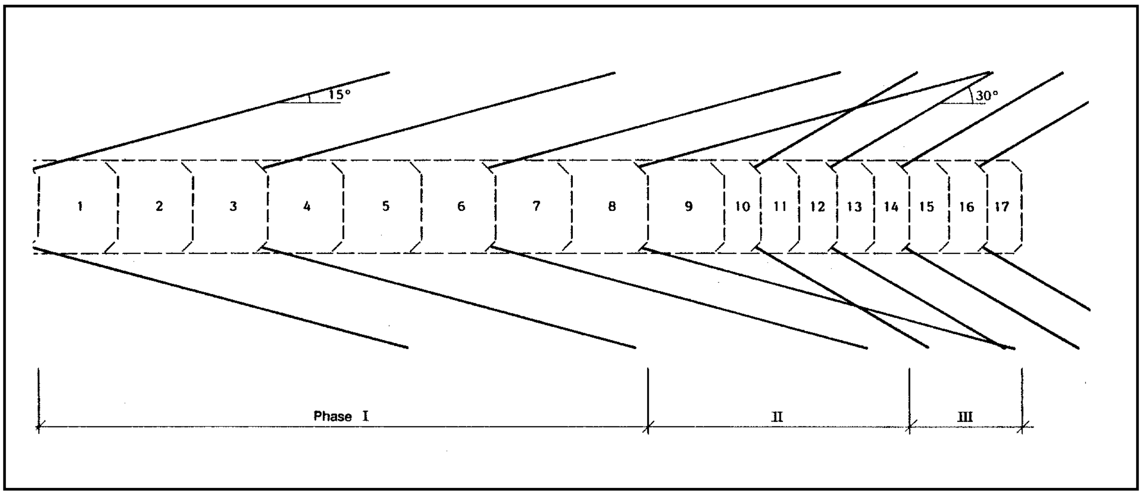
3.0. TUNNELLING TOWARDS THE PLUG

With only 50-100 meter left to excavate in the tunnel before the plug, there are some necessary adjustments of the excavation method to carry out. The reason for these kinds of modifications is to prevent unwanted cases like unsteady rock mass due to vibration from the blasting or due to unexpected poor geology. The vibrations could weaken the rock mass in the plug area and in worst-case lead to uncontrolled water leakage. The tunnelling procedure has therefore been divided into three adjusted phases; in the first phase, pre-grouting is done for every third round. This is to stabilize the rock mass and to minimize the ground water pressure. In the next phase, shorter blasting rounds are used to minimize the vibration and grouting are still also done. As for the last phase, the probe drilling is shortened to avoid penetration into the reservoir. In that way there will not be any uncontrolled water leakage. For the blasting of the rounds cautious blasting is done with shorter rounds than in phase 2. Pre-grouting is also still done in this phase (NTNU-Anleggsdrift, 1999). In fig X more upright probe holes are drilled to check the remaining length to the reservoir.



Side view

Figure 12: Side view of a suggested probedrilling for the last 100 m (NTNU-Anleggsdrift, 1999).



Top view

Figure 13: Top view of the probe drilling for the last 100 m (NTNU-Anleggsdrift, 1999).

3.1. Pre-grouting

Pre-grouting is an equipment to make the excavation process safer by lowering the water pressure and stabilize the rock mass. It is also a method to create a psychical "safe" atmosphere for the staff working in the tunnel. The most common grouting mass are cement based suspension and chemical media, where the cement base is the main grouting mass (NFF, 2011). The mass will fill all the densely cracked rock, weakness zones and chutes with uncompacted material.

The injection holes are drilled systematically in the contour and arranged in an angle to create an outward fan. If clay is found in the water from the boreholes, grouting should be a solution. This is due to the possibility of an increase of leakage in weakness zones with clay.

If there is a lot of water leakage, the grouting pressure must be higher than the water pressure, to prevent the grouting mass from being flushed out. This is not a case if the rock mass is poor, then the pressure should not be too high in case of damaging the rock mass even more. The solution for this case could be to drill more holes, rather than increasing the grouting pressure (NTNU-Anleggsdrift, 1999).

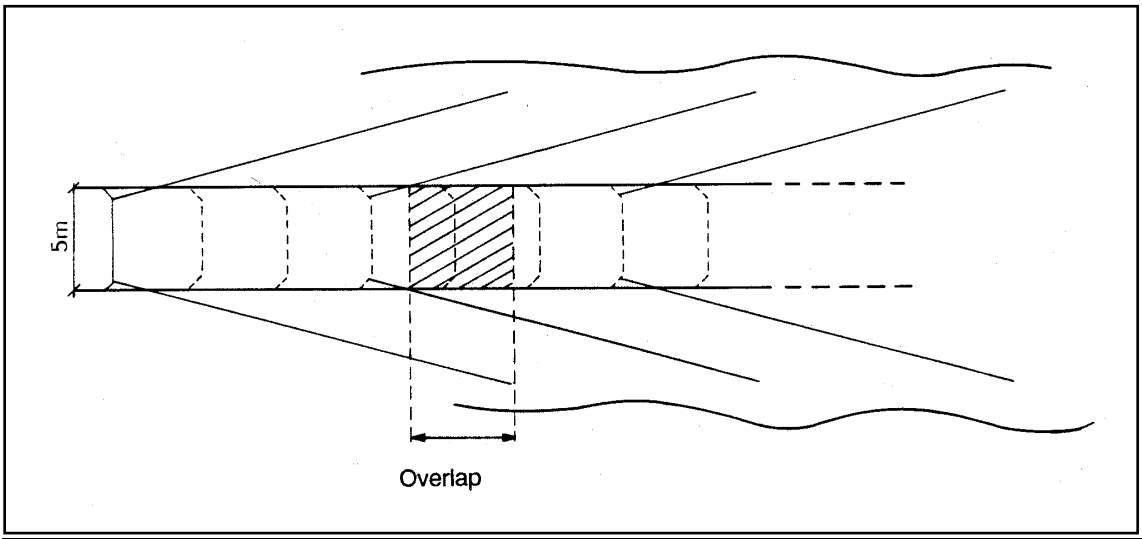


Figure 14: Illustration of the overlap of the probedrilling (NTNU-Anleggsdrift, 1999).

Water leakages

For the work conditions to be satisfactory it is necessary for the tunnel piercing area to be dry (water temp. 4°). There is no limit for water leakage in this area, but the water leakage is usually higher due to penetration of probe holes and it is more complicated to keep the leakage under control. An ongoing leakage is hard to stop and the best way to prevent the leakage is with pre-grouting. But if the leakage is already ongoing the leakage is harder to seal, and the following methods could be used to stop the water (NTNU-Anleggsdrift, 1999):

- **Wedging** – Placing a packer if leakage occurs in a drill hole or specific joints.
- **Cover the face with shotcrete** – Not recommended, due to little adhesion between wet surface and shotcrete.
- **Drilling of relief holes** – Where the water pressure is very high, relief holes are drilled in roof and walls behind the face, to lower the pressure at the face.
- **Grouting with low setting time** – Used in crushed and jointed areas.

3.2. Smooth blasting

At the end of the excavation of the tunnel, it is important to not damage the rock mass more than necessary. If the rock mass cracks up it increases the chances leakage, which might be caused by the vibrancy from the blasting. Some important adjustments are to excavate the contour-holes 50 cm shorter than the other holes and drilling the cut boreholes longer than the rest of the holes, to create a bow shaped tunnel face. The inner contour holes are placed in parallel with the contour holes. Other adjustment is to reduce the confinement of the holes and the amount of explosives per detonator number. If the rock quality is poor, the blasting rounds should be adjusted by drilling the round shorter, and for some cases, it could be reduced to 1 m length per round. In worst case a solution could be to split up the rounds in two, to reduce the vibrancy even more or use all the available numbers of the detonators (NTNU-Anleggsdrift, 1999).

3.3. The piercing area

For the piercing area it is very important that the rock mass is good. Unfavourable rock mass in this area could result in moving the piercing area to another area in the reservoir, which will increase the cost and time.

The geometry of the piercing must form a favourable air pocket underneath the piercing plug to make the blasted rock mass fall into the collection pit. The rock pit should be blasted before drilling of the charging holes for the plug is done. To reach the piercing plug, the collection pit is usually filled with blasted rock mass for the bore rig to stand on (NTNU-Anleggsdrift, 1999).

A detailed probe drilling programme must be prepared to make the tunnel plug and the placing of it as suitable as conceivable. The shaft should be directed oblique upwards from the bottom. The numbers of drill holes should in every case be decided before drilling. The size and placing of the rock pit depends on the type of piercing that is chosen, the thickness of the sediments, size of the plug and tunnel (NTNU-Anleggsdrift, 1999).

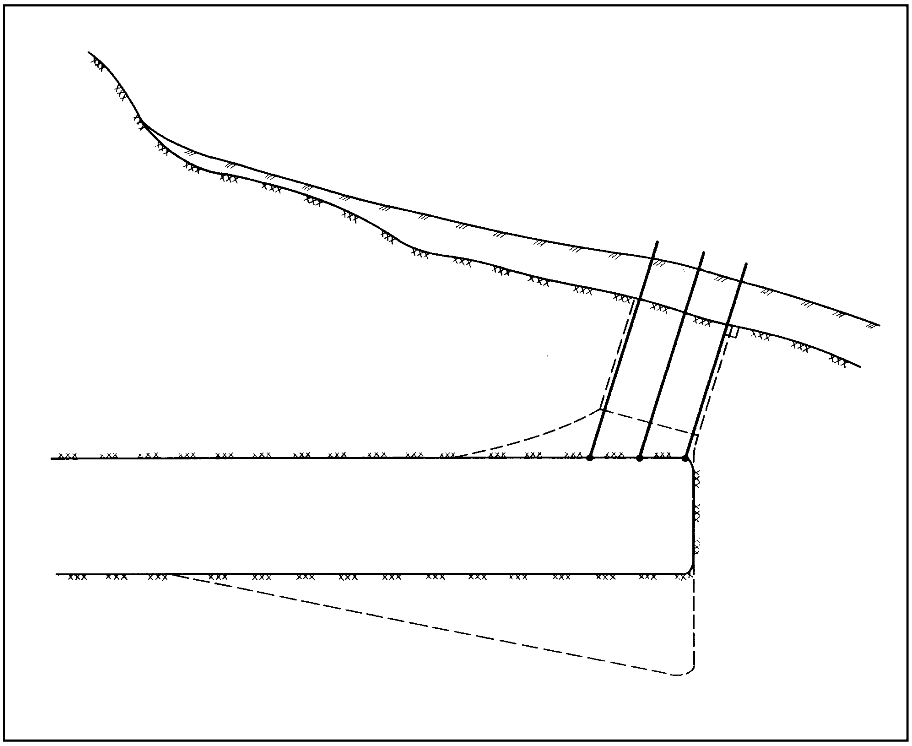


Figure 15: Side view of the mapping of the tunnel plug and the last probe holes (NTNU-Anleggsdrift, 1999).

4.0. THE BREAKTHROUGH ROUND

The breakthrough round, also known as the tunnel piercing, is the final blast before the water flows into the tunnel. If the piercing of the tunnel is unsuccessful, the cost of the project could increase dramatically and result moving the tunnel piercing to a new area in the reservoir. This operation is considered a serious task and an operation that needs to be done carefully and with accuracy. For most tunnel piercing the plug is angled in a shaft up towards the reservoir. The reason for this angle is to increase the efficiency of rock mass falling into the collection pit and to create an air pocket in front of the plug. The air pocket is made for hydrodynamic reasons, which can be read more about in 5.4. Air pocket.

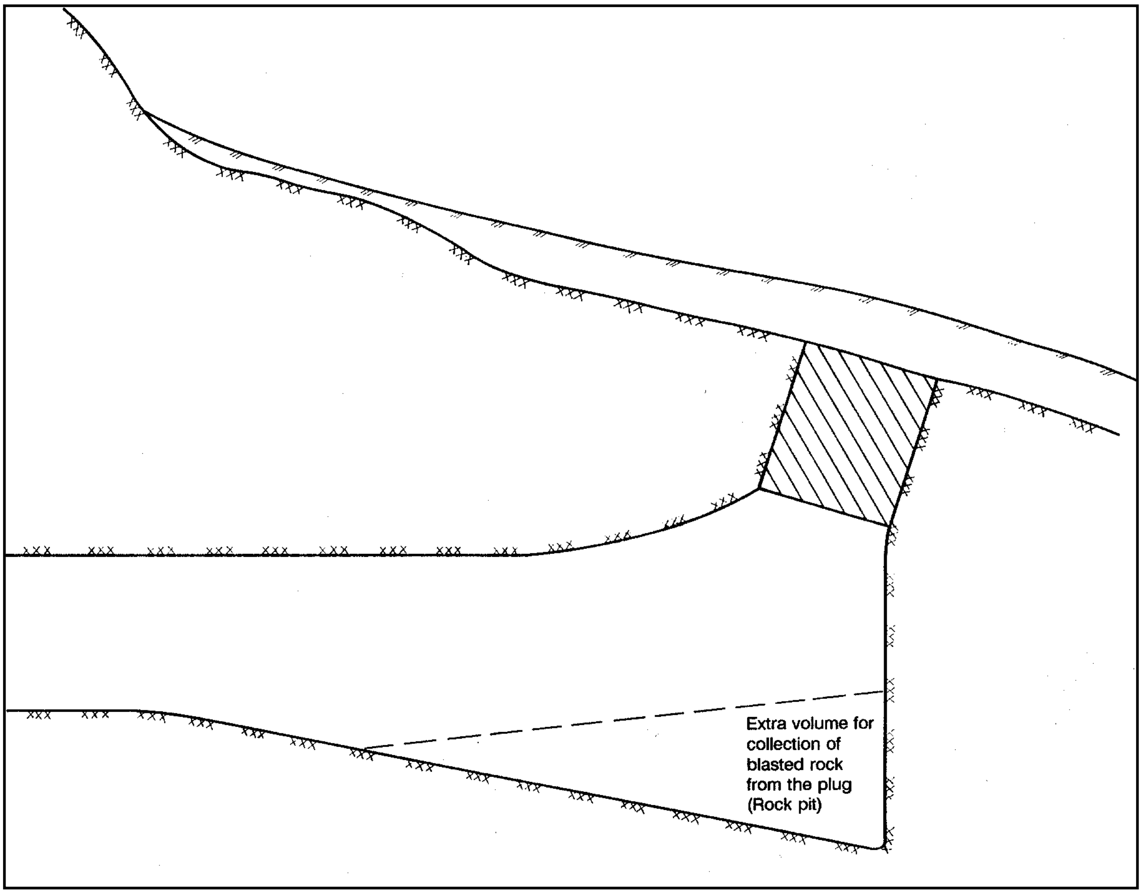


Figure 16: Side view, the tunnel plug and the collection pit with placing of probe holes (NTNU-Anleggsdrift, 1999).

The size of the tunnel is chosen from the type of tunnel piercing method, volume of the plug, and the thickness of the sediment layer (NTNU-Anleggsdrift, 1999). When selecting the thickness of the plug a simple calculation is used:

$$\boldsymbol{Thickness} = 1{,}2 * wp\ (m)$$

$$\boldsymbol{wp} = plug\ width, i.e\ the\ shortest\ side\ of\ the\ plug\ cross\ section.$$

The plug cross section should be smaller on the waterside, than the inner side. This is to make sure that the blocks will loosen and fall the right way, which is into the trap (NTNU-Anleggsdrift, 1999). It is important to mention that the shape and size depends on the rock mass conditions. This calculation is just an estimation.

4.1. Drilling

Probe drilling

The probe holes that are drilled to check and map the sea bottom will in all cases penetrate trough into the reservoir to check the maintaining length. With this information it is possible to decide the length of the loading holes. The minimum is five probe holes, one in the middle and one in each corner (RØMCKE). If there is a need for further investigation more probe holes can be drilled. When drilling the holes water will flush into the tunnel. However, the water does not always come from the reservoir. Joints in these areas does in some situations hold a lot of water. Thus, if the probe drilling reaches a joint filled with water this can be mistaken for being the reservoir. It is therefore recommended to drill another rod just to be sure. Afterwards all holes must be sealed with packers, with or without water leakage. This is done in case of water leakage from the boreholes in the time used for water-filling (could be up to one week)

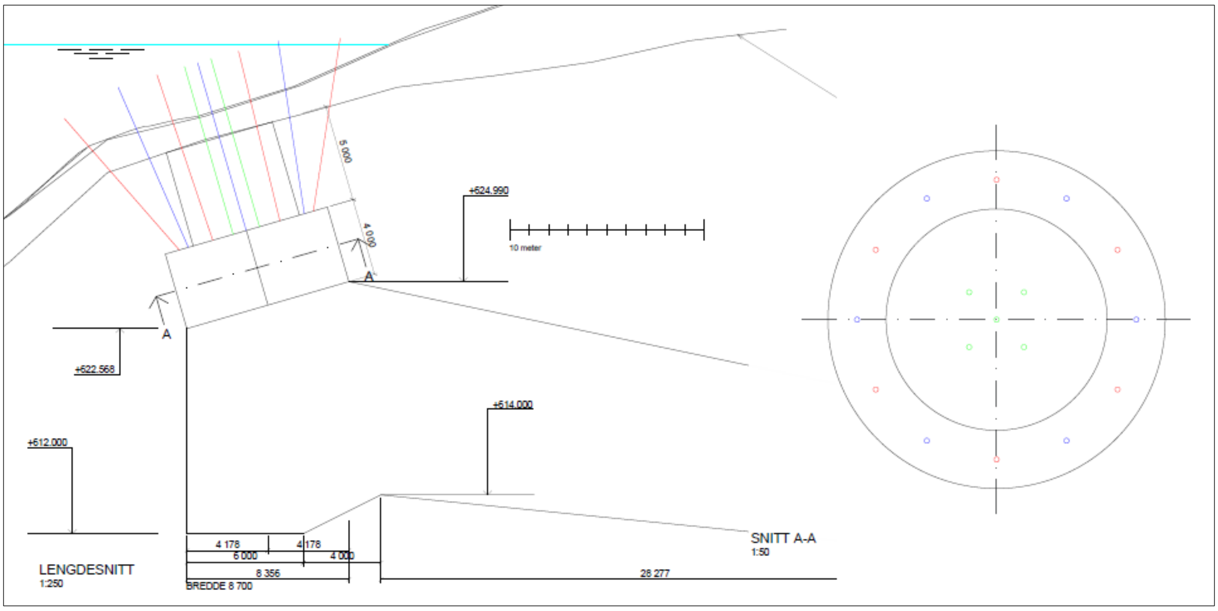


Figure 17: Example of probedrilling plan at Lyngsvatn, uses 17 probe holes for the last round (Implenia AS).

Round drilling

Before the drilling can start there must be carried out a detailed probe-drilling program to map the area of the tunnel plug. This will help to place the plug at the most favourably area. When drilling for the final piercing, the process must be precise. Each hole needs to be checked after it has been drilled to make sure that they are long enough, strait and in the right place. The work must be done properly, or the blasting could be unsuccessful.

The holes should be drilled with a guiding tube on the drill rod to achieve the best result. It is mostly important that the cut drilling have the required accuracy, to avoid the holes from intersecting each other. Another importance is to maintain the crew the same from start to end, because of their knowledge of the rock mass conditions through the project (NTNU-Anleggsdrift, 1999).

The design of the drilling pattern for the plug does often have a circular shape or square with soft corners that is designed to prevent intersection. To make it easier to control the holes the pattern is often designed as simply as possible and with the same length between the drill holes. It is also important to place the cut in an area where the rock mass is of good quality, not only to give the best drilling results, but also to give a precise drilling and avoid intersection (NTNU-Anleggsdrift, 1999). The size of the empty cutter holes is usually 102 mm and the loaded holes are 48 mm for the other bore holes (Rømcke, 2018).

4.2. Charging

For the final blast it is essential that the charges are checked and installed with accuracy. Explosives and detonators should have good water resistance and maintain their sensitiveness when subjected to the static pressure of the air pocket in water filled tunnel piercing. When the hole is charged, a wooden dowel is inserted (NTNU-Anleggsdrift, 1999).

4.2.1. Detonators

The detonators are a device that are used to trigger the explosives. There are three types of detonating systems:

- Electric systems
- Non-electric systems
- Electronic systems

Electric systems

The electric detonator was the first modern detonator to be used. It was first on the market in 1895, and back then it was considered the safest initiation system (Orica Norway AS, 2014). Today the electric detonators are not considered as the safest system anymore, due to its sensitiveness of electric tension. The system should therefore not be used in an area with high voltage wires or under a thunderstorm.

Another disadvantage is the systems limitation, due to few interval numbers. The delay accuracy is 1-2% of the nominal delay time (Olsen, Rock Quarrying, 2009). Electric systems are usually used for smaller blasting operations, such as building sites and ditches. The system uses pyrotechnic delay compositions to achieve a nominal delay time.

The electric detonators are equipped with a resistance, which is possible to measure this to check error with a special ohmmeter. But finding a cut on the wire could be difficult, and with 2000-3000 volt through the wire, this could cause a hazardous leakage. This leakage can be so high that there might not be enough firing current left to initiate the rest of the detonators in the circuit, and it could also be very dangerous in areas with water and bulk explosives (which are good leaders for electricity) (Hauknes, 2018).

Non – electric system

The non-electric system (NONEL) was first used in 1973 and are today used all over the world. It is considered as one of the most important invention in the blasting technology from the last century. The system has been used for demanding blasting for constructions with untraditional detonation systems (Orica Norway AS, 2017).

The main advantage with this system is the flexibility due to the combination of individual delays and surface delays making no restriction on the number of delays in a blast. The only limitation is the accuracy and precision of the delay element (similar to all pyrotechnic detonators) increasing the delay between numbers as the delay time increases. It can also be used in areas with strict rules of vibration (Olsen, Rock Quarrying, 2009).

Electronic system

The electronic system is an all-round system that can be used in many cases. Due to its price it has normally only been used for demanding situations. Advantages that have been experienced with this system is smoother contour, less scaling, less overbreak, flatter stone pile, less rock support-, loading- and transport costs and also better vibration control. The main disadvantage of electronic detonators is the price (Orica Norway AS, 2017).

Electronic detonators use electronic components (microprocessor and capacitor) in the detonator for two-way communication, unique IDs, and freely programmable delay times (Orica Norway AS, 2017). The system has a span from 0-30 seconds with millisecond accuracy and due to this it is possible to pick whatever delay preferred (Olsen, Rock Quarrying, 2009). The detonators, wires and connectors can be tested for errors and leakage with the two-way communication system. This can be done continually during water-filling (NFF, 2010). After firing a blast, a report is created that shows id number, delay in m/s, test and initiation status on every detonator. If a shot has been fired with a faulty detonator this information can be used to localize the position of non-initiated explosive. Electronic detonator gives up to 1000 times more accuracy and larger interval flexibility than other pyrotechnics (Orica Norway AS, 2014).

Table 2: An overview of the advantages and disadvantages of the different detonators (Hauknes, 2018).

Detonators	Electric	Non-electric	Electronic
Advantage	- Number of detonators can be calculated by measurement of the resistance. - Cheap. - Insulation leakage to earth can be measured.	- No limitation of the size of the blast. - Easy to use. - Safe against stray current power lines, RF and static electricity. - Cheap.	- High precision. - 0-30.000 millisecond in 1 millisecond interval. - Safe against stray current power lines, RF and static electricity. - Safe both-way communication. - Testing of detonators and leakage to earth. - High pression delays improves vibration, fragmentation, contour and muck pile shape control. - Static pressure 10 bars. - Easier storage and logistics. - Less misfires and unfired explosives. - Detonator can only be programmed and initiated by advances electronic equipment. - Less plastic waste in the muck pile. - Detonator blast report after every blast.
Disadvantages	- Can be initiated by stray current, RF, Power lines, static electricity. - Considered less safe. - Limited delays restricts the size of the blast. - Less accuracy. - Static pressure 3 bars. - Detonator can be initiated by anyone with a battery.	- Not possible to test. - Less accuracy. - Standard static pressure 3 bars. - Plastic waste in the muck pile.	- Expensive. - Demands longer coupling time then the other detonators (depends on system and producer). - Extensive training of operators needed. - Advanced electronic equipment necessary to program and fire blast.

4.2.2. Explosives

For the break-through it is important that there are enough explosives with enough power, to be certain that the blasting will be successful. It is also important that the explosives tolerate high pressure, water and can release enough power.

For many years Extra Dynamite was used due to its waterproofness and because it was very powerful. The waterproofness was good because of the high NG content of 56%. Normal dynamite does only have 28% (Rømcke, 2017). However, the production of Extra Dynamit was stopped in EU because of changes in ADR (transport of dangerous goods) regulations making Extra Dynamit to sensitive too transport. A detonating cord is used together with the explosive after the Extra Dynamite was stopped.

Today it is used dynamite for under water tunnel piercing. This is due to its high energy, its water resistance and toleration of high air pressure, approximately up to 2 bars. The dynamite is placed in plastic tubes, which also makes them easier to load. The most common civil explosives for underwater tunnel piercing are:

- Eurodyn™ 2000
- Eurodyn™ 3000
- NSP 711

The difference in Eurodyn ™ 2000 and Eurodyn ™ 3000 is that the 3000 have a little higher NG-content and the main difference is therefore the energy it releases and the price.

Another solution is the NSP 711. This explosive is made, among other factors, for underwater blasting. It is very water resistant and has a long durability (Orica Mining Services, 2008). For the deeper piercing the air pressure in the air pocket is usually higher than 3 bars. In this case military explosives have been essential to use because it tolerates higher pressure (Rømcke, 2018).

A detonating cord is always used with two detonators in the holes to secure successful blast for the last round. The detonating cord is placed along the explosives in case there is water/air between the detonators that prevents the other detonator to initiate. The detonating cord will then make sure at least one of the series are initiated.

By excessing the charges, the probability to secure a successful tunnel piercing is increased. This is also done with consideration due to extra loads from sediments on the sea bottom and a few centimetres of rock mass in the end of the round that have not been charged. However, excessing the charges to much does also have its disadvantages, like increased pressure in the air pocket and possibly bigger opening for the intake (Solvik, 1986).

4.3. Coupling

The plug is often placed in the sealing and the work conditions are therefore poor, with inflowing water from drill holes, joints, and the struggle to reach the area. To make the coupling safe and simple, a manual coupling with tape or special coupling clips are recommended (NTNU-Anleggsdrift, 1999). It is also important to check each of the intervals with a certified ohmmeter. The drill holes are installed with tapered formed wooden dowels to make sure the explosives do not fall out (figure 6). The wooden dowel has the effect of sucking up the water, expand, and clog the drill hole. The wooden dowels are hammered in so they should not fall out. The detonator wire is tread through a hole in the wooden dowel and out to the coupling.

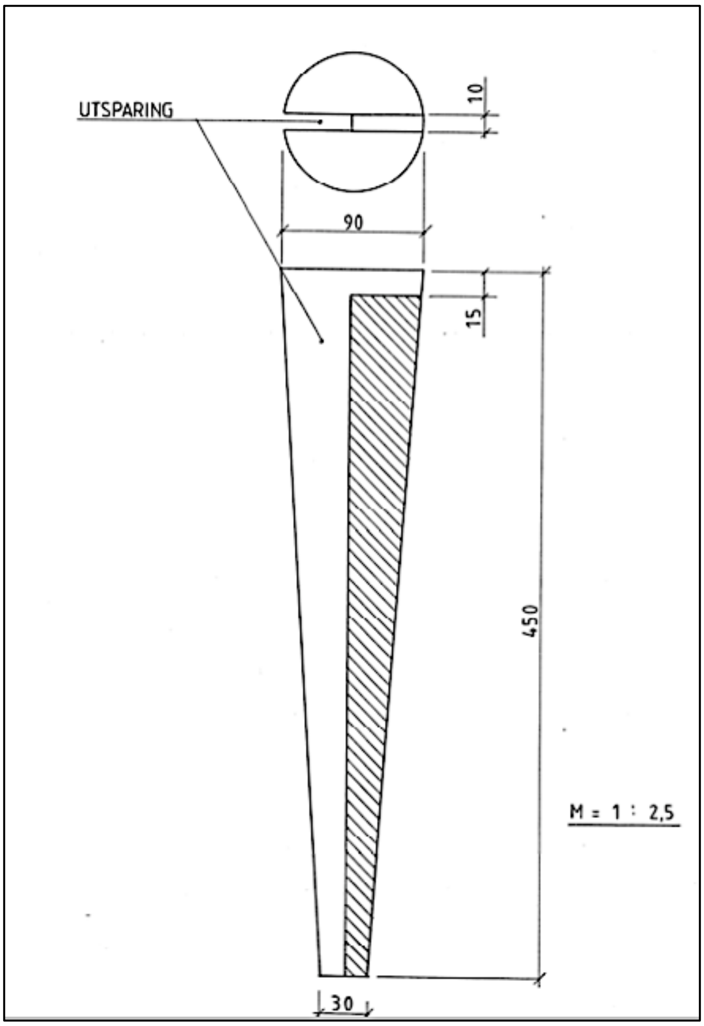


Figure 18: An illustration of a wooden dowel (Dyno/Orica AS).

After coupling all the wires together, the work continues by securing the cables all the way up the shaft to the gatehouse where the blasting wire is initiated. The cables are put in tubes and fastened to the hanging wall in the tunnel to prevent it from damage during the remaining work.

Electric detonator

To make sure that the blasting is successful, one regulation is necessary to achieve. By installing two detonators for each drill hole with the same number, it is a higher possibility of a successful break-through. The two detonators in each hole are coupled with two separate series which again is coupled into a parallel couple. Thereafter they will be coupled to a new double connecting cable. For this coupling method, it is important to mark the different cables, to avoid mixing them up. The resistance between the two cables must not get higher than 5%. If it exceeds this limit, there is a chance that only one of the series is initiated, if the series is the lowest resistance (NTNU-Anleggsdrift, 1999).

Nonel detonator

The Nonel detonators are also coupled with two detonators for each hole, which is done to increase the possibility of full initiation and to secure successful blast (Hauknes, 2018).

Electronic detonator

For the electronic coupling it is also used two detonators in each hole. It is also possible to use to separate detonation system just in case of error on one of them.

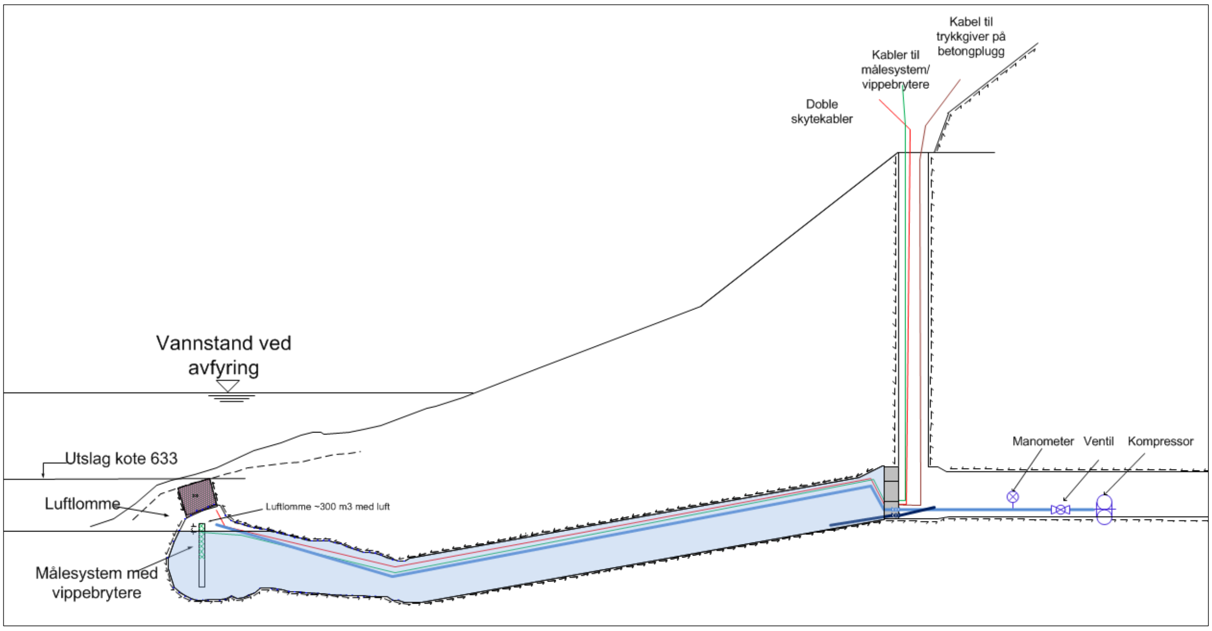


Figure 19: An example of a coupling done at Strandvatn in the Lysebotn 2 project (Implenia AS).

4.4. Blasting

The initiation of the blasting wire is done from the gatehouse or at the other side of the gate, where it is estimated to be safe.

Electric blasting

It uses careful measurement for each loaded hole with an ohmmeter to check the coupling and wiring. What kind of blasting machine that is chosen depends on the type of detonators, what kind of group used, how many numbers of detonators, the resistance of each detonator, the resistance in the connecting cable and what type of coupling used (NTNU-Anleggsdrift, 1999).

Nonel blasting

For the Nonel system the initiation must be done from either electronic or electric detonators.

Electronic blasting

Uses its own initiation system.

5.0. HYDRODYNAMIC FACTORS

5.1. Introduction

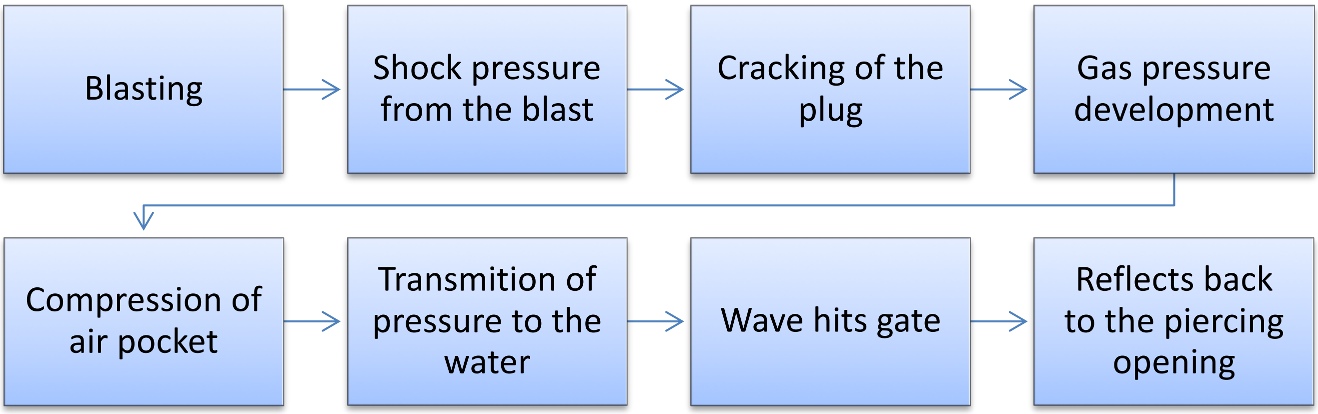
There are many factors to adjust and consider when planning a tunnel piercing. This chapter will present these different elements and explain the impact it has for the tunnel piercing. The hydrodynamics in this operation is often very complex due to the three-phase flow of water, gas, and blasted rock.

A closed tunnel piercing is more complex than an open piercing, considering calculations, model testing and laboratory tests. Many of these elements are added into the decision making and are essential to avoid damage on the constructions. This chapter contains calculation and model developed from The River and Harbour Labouratory at Sintef and NTNU, Trondheim by Øivind Solvik.

Describing the hydrodynamic process after the blast is not easy. First, a shock pressure from the blasting moves towards the gate. When the gas from the explosives starts to develop, a pressure increase in the air pocket is evolved. The gas has the effect of cracking the rock mass. The angle of the plug does also make the blasted rock mass fall into the pit. At this point the pressure in the air pocket will have reached so high that it starts to push the water in the tunnel, at the same time as the water starts flushing in. The pushing of the water is also described as the after-compression, which means the pressure the inflowing water causes the still water in the tunnel and creates a surge towards the gate. When the surge hits the gate, the maximum pressure is reached. Afterwards the water is pushed back and forth and out in the reservoir again, in a disorderly flow-situation (NTNU-Anleggsdrift, 1999).

For a dry tunnel piercing the process is a bit different. With no compression of the air it can only be done with limited pressure increase if the tunnel is 20-30 times longer than the distance between the water level and the tunnel. The pressure increase from the gas explosion can be neglected due to the volume of the "air pocket." The collection of blasted rock mass is also unefficient in this case, due to high velocity of the water inflow and the time it uses reaching the gate (Solvik, 1995).

The process of the blasting of the tunnel piercing is as follows:



5.2. Water-filling

Water-filling has the purpose to ensure favourable collection of the blasted rock masses, reduce the pressure from the blast and in that way prevent the gate from damage (NTNU-Anleggsdrift, 1999). With a long tunnel, it is in some cases only necessary with low water-filling to reduce the transportation of the blasted rock mass. Filling a longer tunnel with water will be time and cost demanding and therefore only recommended for short tunnels.

Open system

This method requires water-filling of the tunnel to reduce the transport of the blasted rock mass towards the gate, which also limits the upsurge in the shaft. The whole tunnel is filled except for an air pocket in front of the piercing area. For this method it is favourable that the tunnel is long (NTNU-Anleggsdrift, 1999).

Closed system

20% of the tunnel volume is standard for low filling (NTNU-Anleggsdrift, 1999). For a closed method, the simplest solution is to use low or no water-filling. The design of the tunnel will determine how the water will stand in the tunnel. If the tunnel has an increasing slope from the gate to the plug, the water will stand against the gate. By doing this the water will give a shelter of the gate, which is favourable (Solvik, 1995)

5.3. Collection of blasted rock mass

If the blasted rock mass is not collected in an effective way, the blasted rock mass might hit the gate and damage it or in smaller tunnels block the tunnel and reduce the water stream into the power station. Different techniques are used to get the best collecting results for the specific tunnel piercing situation.

- The angle in the intake shaft.
- The size and geometry of the collection trap.
- Geometry and slope of the tunnel.

By making a model, it is possible to get an estimated solution of the most efficient size. The best angle for the plug is in an upward angle, to make sure the blasted rock mass falls down by its own weight into the pit. However, if the piercing area lies horizontal with the tunnel, the situation will be more challenging. The importance of the collection is high because removing the blasted rock mass afterwards is expensive and difficult (NTNU-Anleggsdrift, 1999). Due to overcharging of the piercing round, the size of the blasted rock mass is usually so small that the risk of blocking the tunnel is low (Solvik, 1995).

Open piercing

With an open piercing the water velocity is lower than for a closed method, the reason for this is that the water can move freely up the shaft. Collection of blasted rock mass is hence rarely a problem and most of it will usually fall into the trap by its own weight (Solvik, 1985).

Closed piercing

For a closed piercing the transport of blasted rock mass depends on the pre-compression in the tunnel. With a dry closed piercing the water inflows velocity is very high immediately after the piercing, which makes the collection much more difficult (Solvik, 1986). For this method the blasted rock mass must be allowed to spread into the tunnel, without reaching the gate. On the other hand, high water-filling in the tunnel makes the catching of mass easier. Generally, it is the water that hits the gate first, and then large boulders might hit the gate during the water jets and turbulence in the inflowing water. Another thing that can benefit is to make sure that the geometry is of a favorable size by making a model to test which design is best. This is only done in demanding situations (Solvik, 1995).

5.4. Air pocket

The importance of the air pocket is to prevent water hammer. The air pressure in the air pocket is especially essential for the closed method. The air pocket is designed to reduce damage and pressure increase from the explosion and inflowing water. Destructive shockwaves can occur when explosives initiate directly in water. The energy from the blast will then be transferred directly into the water and cause an even bigger pressure surge. A gap of air between the water and the charges is therefore necessary and is another reason for the placing of the air pocket. In some cases, the gap/air pocket must be filled with more air due to the required pre-compression. The amount of required pre-compression can be calculated, see *6.0 Calculations* (Kjørholt, 1986).

If the pressure is higher than the external water pressure the air will in some cases penetrate through fractures in the rock mass and thereby decrease the pressure (NTNU-Anleggsdrift, 1999). This situation can also appear in an open method, due to too high water level in the shaft which increases the pressure in the air pocket (Kjørholt, 1986). Due to these situations it is therefore important to measure the pressure in the air pocket.

The air pressure in the pocket can be measured by using several electrical circuits on a vertical column. When the water reaches the first electrical circuits, it will short-circuit and give information of the height of the water level (NTNU-Anleggsdrift, 1999). A demonstration of the equipment and installation is shown below.

5.5. Explosive gas pressure

The explosive gas that is released during the explosion will cause a sudden increase of the pressure in the air pocket. This pressure varies with the volume of the air pocket and the amount of explosives in the round. When detonating the explosives, it is estimated that 1 kg of explosives forms approximately 0,8 Nm^3/kg of gas in atmospheric pressure. The increase of the pressure from the blast will transmit the energy into the water in the tunnel and develop a surge towards the gate, which reflects when reaching the gate, which is only a case for water filled tunnels. This reaction happens so fast that the pressure in the air pocket is said to be of constant volume (NTNU-Anleggsdrift, 1999).

$$1 \text{ kg} = 0{,}8 \text{ Nm}^3$$

It is essential to mention that there is an uncertainty with calculation of the development of gas. The uncertainty is low with smaller volume and increases with larger volume. The uncertainty concerns where the gas will go. Because some of the gas penetrated out in the reservoir, some into the rock mass and the rest in the air pocket. How much gas that ends up in the air pocket is therefore hard to estimate. But from experiences the recommended amount is 25-50% (Solvik, 1995).

The largest increase of the pressure will happen if the blast end in a misfired round. In this case it is assumed no energy loss and thus all energy will be transferred into the tunnel. All of the energy from the blast will be used to push the water in the tunnel. This situation can therefore cause great damage on the gate and the gatehouse. A combination of large external water pressure and short tunnel is normally one of the worst situations for this case (Solvik, 1995).

5.6. Retardation

One of the main retardations of the pressure increase is the water-filling in the tunnel. The calculation of the retardation of the pressure is easy to calculate for a pipe, but for a tunnel the calculation is difficult and over-complex. The reason for this difference is because of the inflowing in the shaft, the air pocket, the disorderly flow situation caused by the gas and inflowing water, the tunnel cross section, over coverage of the tunnel, the ground water level, wall roughness and the variability of the stiffness of the geology (Vassdragsregulantenes-forening, 1986). If the rock mass is very solid the reduction of the pressure is less than if the rock mass is soft. Overburden, tunnel cross section and groundwater level are other factors that have an impact on the pressure reduction. For the calculation, it is safest to expect the rock mass to be harder than predicted (Solvik, 1995).

For a non-existent head loss/friction calculation, it is possible to calculate the maximum pressure. The calculation will however give a higher pressure than in real life, but this has the advantage of being a safe estimation. However, predicting a too high pressure will usually cause higher costs due to time used for air compressing in the air pocket.

With friction, the pressure will decrease because the kinetic energy will turn into singular loss in the inflow opening and friction loss from the periphery of the tunnel and blasted rock mass accumulation. The loss from the periphery in the tunnel is selected with the Mannings roughness coefficient to make the calculation easier. For dry tunnel M=30 and for waterfilled tunnel M=34.This is a number that tells how rough the surface of the periphery in the channel is and is often used in calculations of the inflow of the water (Solvik, 1995).

5.7. Model testing

Model testing have been an essential part in the development of the tunnel piercing calculation and design. The increasing knowledge have in many ways made tunnel piercing a safer and more achievable profession to carry out. Underwater tunnel piercing is hydraulically very complicated, especially for closed method. The two main challenges that have been tested is to; minimize the transport length of the blasted rocks to prevent it from reaching the gate and minimize the pressure from the blast to spare the gate construction (Sandvik, 1985).

One of the main problems with modelling is to get the atmospheric pressure in the right scale. To enable this a vacuum tank was demanded. Problems like boiling and steam pressure might be caused by to too high temperature. Several methods have been done to conclude the best suited method (Sandvik, 1985).

Laboratory modelling and testing was a process that went on for many years. Lots of tests was done to support the hypothesis and calculation that was made. A result of this process was a calculation model. This model has in later years turned out to be very useful for a number of different piercing situations, like open and closed methods (Solvik, 1995).

To accomplish a test the model needs to be the right size. It has therefore been used Froude model to make it equal in model and prototype (NTNU, 2017). Froude's model law describes the ratio between inertia and gravitation.

A model test for efficient collection of the blasted rock mass was also made. The experiment tested the efficiency by filling the pit with water. However, this turned out to be of the opposite effect, since the water reflected the rock mass and thereby reduced the collecting effect (Solvik, 1995).

5.8. Shaft upsurge

The shaft upsurge is only a case for the open method. The water in the tunnel and shaft will work as a U-pipe, where the water from the reservoir is stationary and the water in the tunnel is in motion and creates a surge. It is important to prevent this surge from flowing over the top of the shaft to spare the gatehouse and the equipment in it. The reason that the gatehouse already is installed when blasting the last round is because the tunnel piercing is usually the last operation that is done on a project like this. This is more time and cost consuming than doing it afterwards. The flow loss in the turn up to the shaft will be of grate scale. This part of the tunnel will be quite narrow and efficiently slow down the velocity. This area can be narrowed even more by installing special made wallboard. Without this energy-loss of the inflowing water, the upsurge could reach the same length as the water depth of the intake (Kjørholt, 1986).

It is important to calculate the upsurge of the shaft to avoid overtopping. The basis of the calculation is the difference in water level between the gate shaft and the reservoir (NTNU-Anleggsdrift, 1999). More information of the calculation can be found in the chapter 6.2. Open method, water-filled tunnel.

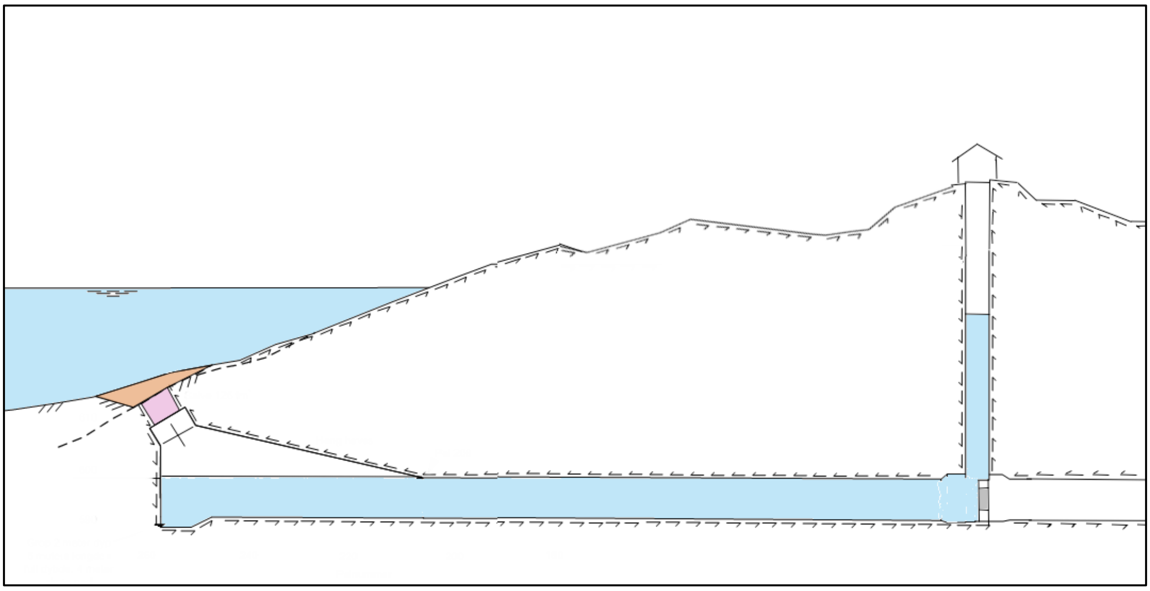


Figure 20: Illustration of an open method with water-filling.

6.0. CALCULATIONS

6.1. Introduction

In the previous chapter a description of all the essential factors to considerate for a successful tunnel piercing where described. This shows that tunnel piercing is not an easy task and that there are lots of elements to consider before the final blast.

The main reason for calculating the pressure for tunnel piercing is to get an estimation of the results of the piercing blast. The last indicator that is possible to adjust is the air pressure in the air pocket in front of the plug. This can be done through a pipe from the gate that will fill in air into the air pocket, which is mentioned in *5.2. Water-filling.*

The reason for the calculation is to prevent unsuccessful blast and damage on the gate. For the different piercing methods, the damages are distinctive.

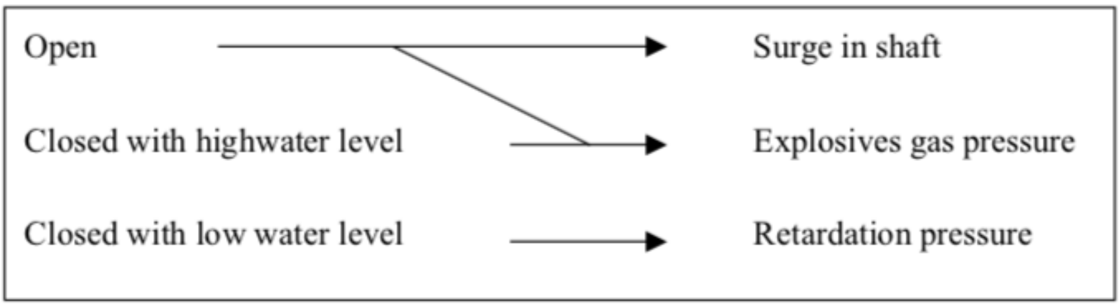
For a closed method, avoid:

- Damage on the gate construction.
- Mass transport into the tunnel towards the gate, and in worst case damage the gate.

For an open method, avoid:

- Damage on the gatehouse and shaft.
- Mass transport into the tunnel that can reduce the inflow and damage the gate.

Design pressure (pressure calculation) for the different methods are shown below:



6.2. Open method, water-filled tunnel

For the open method, the only resistance the inflowing water will meet on its way out is the wall friction, singular loss in change of direction, head loss from inflow up the shaft and narrow areas at the gate construction. In some cases, the pressure from the gas might damage the gate. But the volume of the air pocket is usually large, so that the gas development will not make any harm. If the tunnel piercing had no water-filling, the pressure and shock wave from the blast would normally not do any harm, due to the long distance between gate and plug (Solvik, 1995). Water-filling is for this method essential. If there is no water in the tunnel to capture the incoming water and rock mass, the rock might be transported to the gate and the inflowing water might reach the gatehouse. Early planning of the procedure and calculation of the upsurge can thereby prevent over topping from happening (Solvik, 1986).

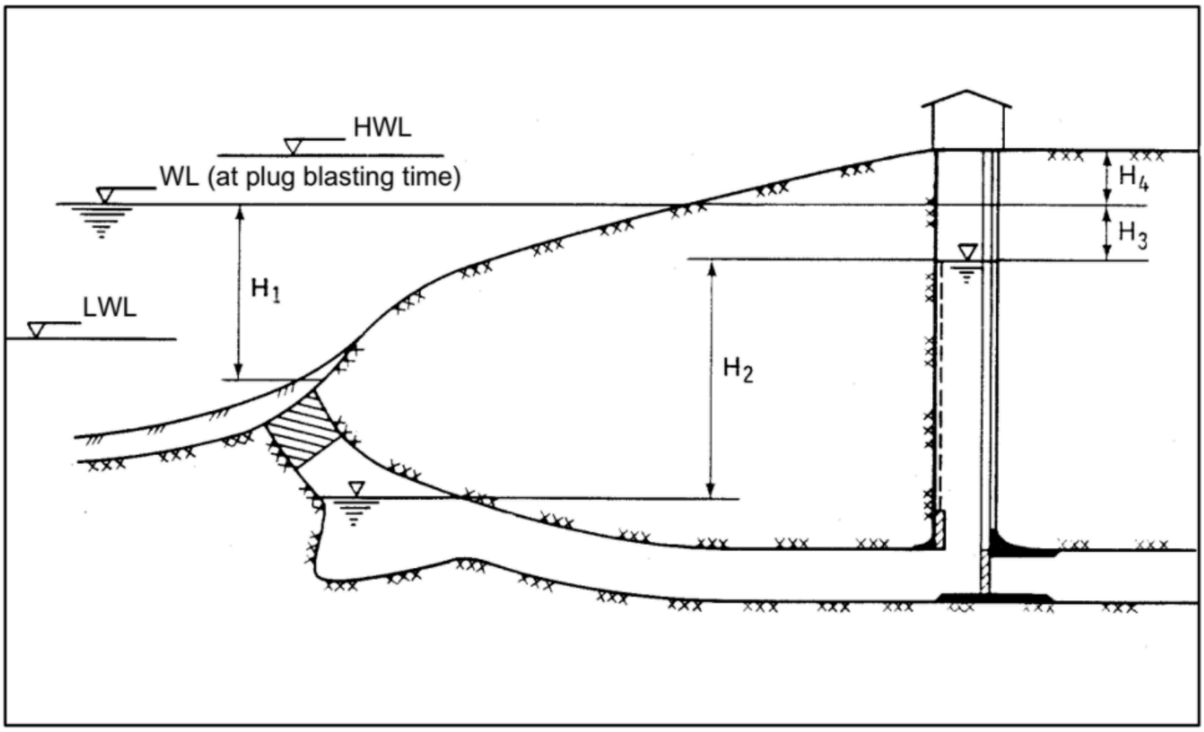


Figure 21: Illustration of an open method with depth measurements (NTNU-Anleggsdrift, 1999).

Criteria's

$$\text{Criteria:} \qquad H2 < H1$$

H1: Water depth at the tunnel plug

H2: Air pressure air pocket. Difference in water level between the gate shaft and the plug.

The water pressure in the air pocket must not be higher than the external water pressure. This is due to air leakage in the rock mass at the plug area, as mentioned earlier. Another problem for the water-filling is the level between the shaft and the reservoir. If the water is higher in the reservoir than in the shaft, a larger surge will occur. This can be estimated by using this criterion:

$$\text{Criteria:} \qquad H4 > c \cdot H3$$

H3: Difference in water level between the water in the reservoir and in the gate shaft.

H4: Difference in water level between the gate house and the water in the reservoir.

C: Constant (0.7 – 0.9)

Small volume of air pocket in an open tunnel piercing is not common, but in some situations it might occur. In these kind of situation it is important to find the pressure development in the air pocket. A small air pocket will reach higher pressure against the gate. Open method will have the same calculations as for a closed method.

In a situation where the air pocket is bigger, which is more common, the highest pressure rise against the gate will appear when the surge reaches its highest level in the shaft. An action is to get the water-filling in the shaft as high as possible to reduce the pressure increase as much as possible within the criteria (Solvik, 1995).

Calculation of the upsurge in the shaft

The parameters that are used in this calculation are, (NTNU-Anleggsdrift, 1999):

δH	=	the difference in water level between the gate shaft and the reservoir
δT	=	the length of one time step (iteratin step)
ΣL/F	=	the sum of length divided by the cross section of each part of the string of water involved.
Q_n	=	the flow of water in the tunnel at the end of time step n
Q_{n-1}	=	the flow of water in the tunnel at the end of time step n-1
H_n	=	the water level in the gate shaft at the end of time step n
Hn-1	=	the water level in the gate shaft at the end of time step n-1
δQ	=	change in the flow of water during time step n
Q	=	average flow of water in the tunnel during time step n
Fi	=	cross section of the narrow pass at the bottom of the shaft
Fs	=	cross section of the gate shaft
g	=	acceleration of gravity = 9.81 m/s^2
C	=	Singular loss coefficient for the narrow pass expressed as velocity head. Primarily, the coefficient is dependent on the shape of the narrow pass, but the value of the coefficient will always be near 1.0. In most cases, the value 1.0 is used.

The equations of the upsurge are

Change in the flow of water during time step n:

$$\delta Q = \frac{g}{\Sigma \mathrm{L} / \mathrm{F}} (\delta H - C \frac{Q/F_i \, |Q \,/\, F_i|}{2g}) \delta T$$

Finding the flow of the water in the tunnel at the end of time.

$$Q_n = Q_{n-1} + \delta Q$$

Finding the average flow of water in the tunnel during time step:

$$Q = \frac{Q_n + Q_{n-1}}{2}$$

In the end find out the difference in water level between the gate shaft and the reservoir. This makes it possible to find out how high the water will reach in the shaft.

$$\delta H_n = \delta H_g - \frac{Q}{F_s}\delta T$$

6.3. Closed method

Compared to the open method, this method reaches higher pressure increase. A simplified description of the pressure distribution is to describe the air pressure in the air pocket as a cushion. As mentioned in chapter *5.0. Hydrodynamic factors* there are a lot of elements to consider for tunnel piercing. These elements are therefore also important to conclude in the calculations.

If the calculation is friction free, it is possible to find the maximum pressure. This because the energy that the water liberates when it moves down into the tunnel, will be found in the energy used for compression in the air pocket (Solvik, 1986).

Another adjustment to the calculation is to use stationary current instead of non-stationary current which is a current that changes through time. For a non-stationary current, it is more complicated to do a calculation and difficult to find the measurements needed (Solvik, 1995).

The lowest gas development of the explosion gives the highest after-compression, while the highest gas development results in no after-compression at all. As mentioned in 6.2. Open method, the pressure in the air pocket must not be higher than the external water pressure due to air leakage in the air pocket.

A brief summary of how to find the different factors for the calculation:

Explosives	• What is needed to blast the tunnel plug
Gas development in air pocket	• Through calculation and measurement of volume
Air pressure in pocket	• Measuring the air pressure
External water pressure	• Measured close to the tunnel piercing date.
Dimensioning pressure for the gate	• Often given by gate producer, and are set to HRV, DFV or can also be found with calculation.
Maximum pressure from blast	• Calculation

6.3.1. Calculation of maximum pressure from the explosives

The main purpose of these calculations is to find the pressure increase in the air pocket and the maximum pressure at the gate, to check if it is below the dimensioning pressure for the gate. For the calculation we assume that 1 kg explosives will develop to 0.8 m^3 gas in atmospheric pressure, and that the energy loss is neglected. The symbols for the calculation is:

1 kg explosives = 0,8 m^3 gas

Pressure:
P_a = atmospheric pressure
P_S = external water pressure
P_0 = pressure in air pocket before blasting
P_{max} = maximum pressure of the after compression (at the gate)
P_2 = Pressure after the blasting, but before the after compression (air pocket)

Volume:
V_1 = volume of air pocket before and after blasting, including explosive gas.
V_g = volume of the developed gas at atmospheric pressure
V_x = volume that the air captures right after blasting
V_1 - V_x = volume that the gas captures right after blasting
V_{min} = the volume when the pressure reaches P_{max}
G = kg explosives for the round

Description of the process

1. V_1 is the volume and P_1 is the pressure in the air pocket before the blasting. P_s is the external water pressure.
2. As soon as the explosion has started the gas develops with the volume V_g into volume V_1.
3. This will make the volume of the air pocket change to V_x, which is part of the incarcerated air V_1.
4. The gas will then have to replace the volume V_1-V_x and the original pressure P_1 will then increase to P_2.
5. V_{min} is the volume in the air pocket when the pressure P_{max} have reached its maximum.

6. The energy that is supplied from the external water pressure P_s multiplied with the volume that is filled up when the pressure has reached its maximum, V_1-V_{min}.

For the following equations 1.), 2.) and 3.) the contributions of the gas are explained, before the water flows into the tunnel.

For the gas the equation is:

1.) $$P_a\, V_g^{\kappa} = P_2\, (V_1 - V_x)^{\kappa}$$

And the air:

2.) $$P_1 \cdot V_1^{\kappa} = P_2 \cdot V_x^{\kappa}$$

If we put β = V_g/V_1, where V_g = 0,8 we get 3) by combining 1) and 2):

3.) $$P_2 = P_a \cdot (\beta + \left(\frac{P_1}{P_a}\right)^{\frac{1}{\kappa}})^{\kappa}$$

Assuming Atmospheric pressure, Pa = 1:

3.) $$P_2 = \left(\beta + P_1^{\frac{1}{\kappa}}\right)^{\kappa}$$

Illustration of the process:

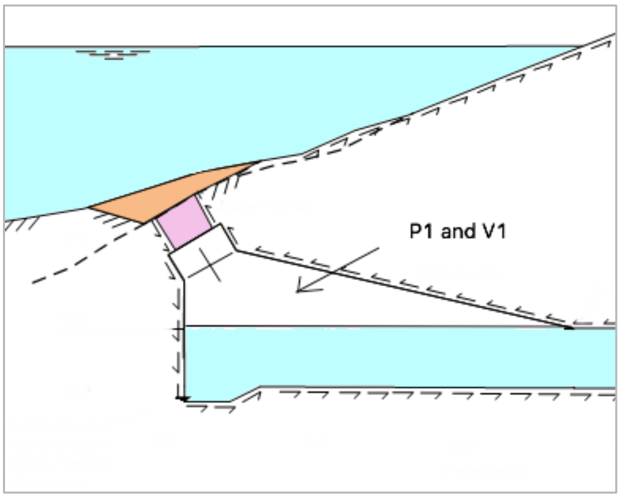


Figure 22: P1 and V1 before the blast.

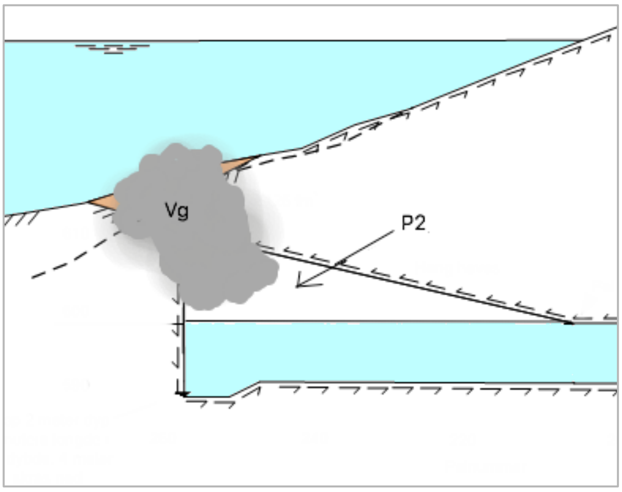


Figure 23: Gas development for P2.

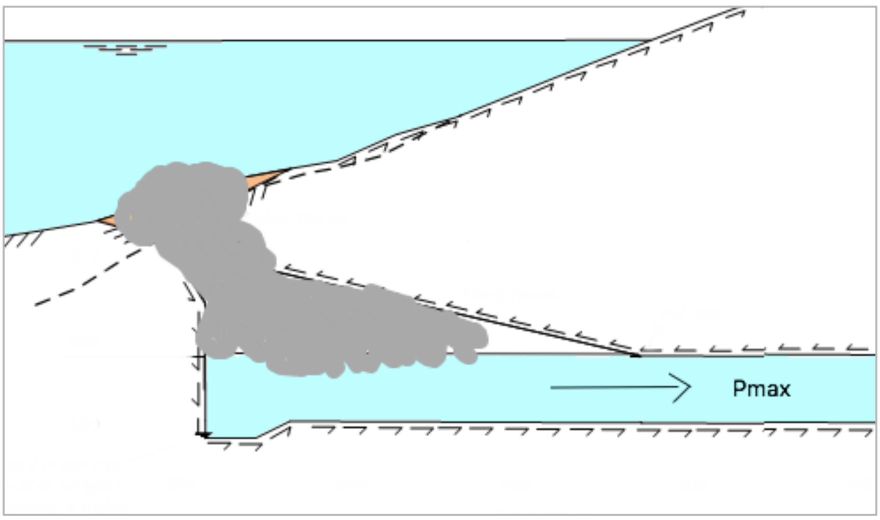


Figure 24: The maximum pressure Pmax.

6.3.2. Design pressure against the gate

If it necessary to calculate the design pressure at the gate, this is another method to estimate it. For this calculation it is only the pressure increase caused by the gas development that is included and not the inflowing water.

The calculation presupposes that all the explosive gas is transferred into the air pocket. The volume of the air pocket for this reaction is assumed to be constant because it happens so fast. It also assumes that there is no head loss for the surge towards the gate (NTNU-Anleggsdrift, 1999).

The pressure increase in the air pocket from the gas development:

$$P_e = P_2 - P_1 \qquad (bar)$$

This will be the theoretical size of the pressure wave that will start to move towards the gate. The static pressure at the gate before the blast is characterized to P_{gate}. This is possible to measure once the water is filled in the tunnel. By the time the wave hits the gate, the pressure of the wave will have increased to double size:

$$\mathrm{P_w} = 2\,(\mathrm{P_2} - \mathrm{P_1}) \qquad (bar)$$

P_2 = Maximum pressure increase from gas after detonation

P_{gate} = Static pressure against the gate before blasting

P_1 = Pressure in air pocket before blasting

P_d = Design pressure against the gate

P_e = Pressure increase in the air pocket

P_w = Pressure increase at the gate

The design pressure at the gate will then be:

$$P_d = P_{gate} + 2\,(P_2 - P_1)$$

To get the best calculation possible it is extremely important that the input data is correct. It is therefore necessary to check the amount of explosives while charging, and check the volume of air in the air pocket while-filling the tunnel with water (NTNU-Anleggsdrift, 1999).

6.3.3. The waters after-compression effect

First the gas will develop and then the inflow of the external water will cause the waters after-compression. The after-compression is the process that leads to the maximum pressure at the gate. All the elements from the surge towards the gate, to water flowing back and forth in the tunnel is part of the after-compression.

In the following calculation we assume adiabatic change of state. For the calculation of the gas development in the air pocket we then get the following equations of state:

$\varkappa = 1.4$ → adiabatic exponent for dry air.

The liberated energy is:

4.)	$P_S\,(V_1 - V_{\min})$

Due to no friction or turbulence loss in this calculation, this energy will be equal to the energy that will be used to compress the air in the air pocket. This process is calculated adiabatic (without transfer of heat) and can be expressed as:

5.)	$\frac{1}{\kappa - 1}\,(V_{min}P_{max} - V_1 P_2)$

The energy equation will then be:

6.)	$\frac{1}{\kappa - 1}(V_{min}P_{max} - V_1 P_2) = P_S\,(V_1 - V_{\min})$

When using adiabatic calculation, the adiabatic equation of state must be added (κ=1,4):

7.)	$P_2\,V_1^{\kappa} = (P_{max}V_{\min})^{\kappa}$

By combining the volume from 7) and put it into 6), gives the next equation, 8), without the volume ratio:

8.)	$\frac{P_S}{P_2}\left(\left(\frac{P_{max}}{P_2}\right)^{\frac{1}{\kappa}} - 1\right) = \frac{1}{\kappa - 1}\left(\frac{P_{max}}{P_2} - \left(\frac{P_{max}}{P_2}\right)^{\frac{1}{\kappa}}\right)$

Maximum pressure of the after compression (P_{max}), the total pressure increase caused by the gas pressure:

9.)
$$\frac{P_{max}}{P_2} = \left(\frac{P_s}{P_2}\right)^{f(x)} \rightarrow P_{max} = \left(\frac{P_s}{P_2}\right)^{f(x)} \cdot P_2$$

Where:

$$f(x) = 2\left(\frac{P_s}{P_2}\right)^{0,081}$$

6.3.4. Calculation method for closed system with neglected energy loss

The total procedure for a closed system with neglected energy loss are described:

1. Finding out how much water is necessary to fill the tunnel with to prevent blasted masses reaching the gate.
2. Calculate the incarcerated volume of air in the air pocket.
3. When the explosives are definite, it is possible to calculate the pressure increase that the gas from the explosives can cause (1 kg = $0{,}8m^3$/). With the gas pressure it is possible to find the after-compression of the inflowing water and give the maximum pressure on the gate. The pre-compression and the gas pressure amount will need to be adjusted at the same time.

The only part that is possible to adjust, once the water is filled in the tunnel, is the pre-compression of the air pocket. The amount of pre-compression is important to choose with consideration of the amount of gas development. But since the gas development in the air pocket is very variable and hard to find exact it is essential that the all possible pressure amount of gas development is used in the calculation (Solvik, 1995).

The graph below shows an example of the gas development and maximum pressure for the given gas development. The equation 9.) is not only made to find <u>one</u> maximum pressure, but for every amount of explosive gas from 0-100% which is used in the graph below. This is an effective way of finding which pre-compression that suits the specific tunnel piercing best.

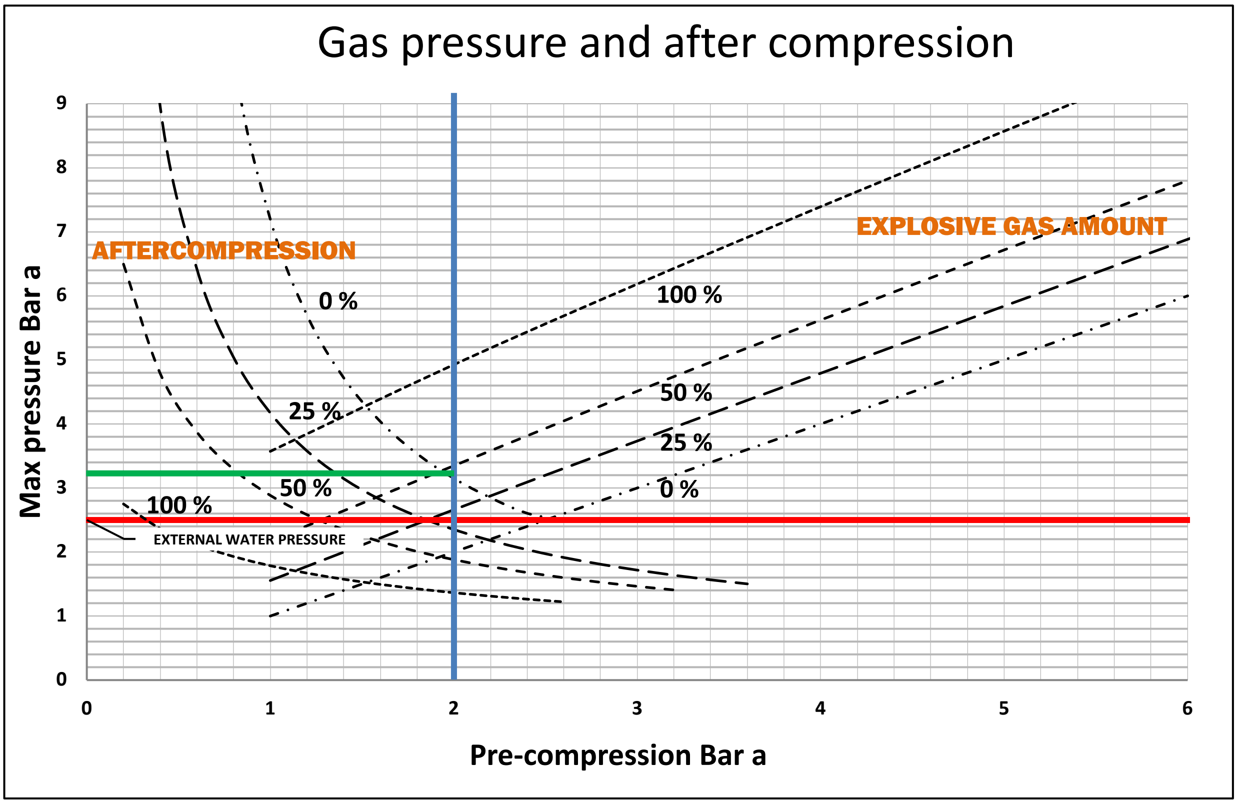


Figure 25: An example of an calculation, with use of the graph.

Example

If the pre-compression is 2 bara in the example above (blue line), the pressure at 50% explosives gas amount will be 3.4 bara, and for 100% 5 bara. The maximum pressure is reached when the after-compression is 0%, the maximum pressure is therefore, in this example, 3.2 bara (green line).

Maximum pressure for given pre-compression

To find the maximum pressure (P_{max}) for the specific pre-compression can be found when choosing the after-compression to 0%. The maximum pressure (P_{max}) will, in worst case, occur if there is no gas development from the explosion. This is a situation that does rarely appear but is necessary to add for the calculations.

From the equation 2.) (P_2), we can see that if we put the gas development (β) to 0%, P_2 will be the same as P_1. This will then give the maximum pressure (P_{max}) for that given pre-compression. The pressure inside the air pocket cannot be smaller than P_1 and the maximum pressure is therefore given when P_2 is the same as P_1 (pre-compression). A higher pressure (P2) in the air pocket will only give lower values for P_{max}.

0% after-compression = P_{max} (for given pre-compression)

By reducing the volume of the air pocket, the impact the volume has on the after-compression and the gas development are shown below. With reduced volume of the air pocket the after-compression and gas pressure will increase higher and opposite with increased pressure.

Smaller volume:

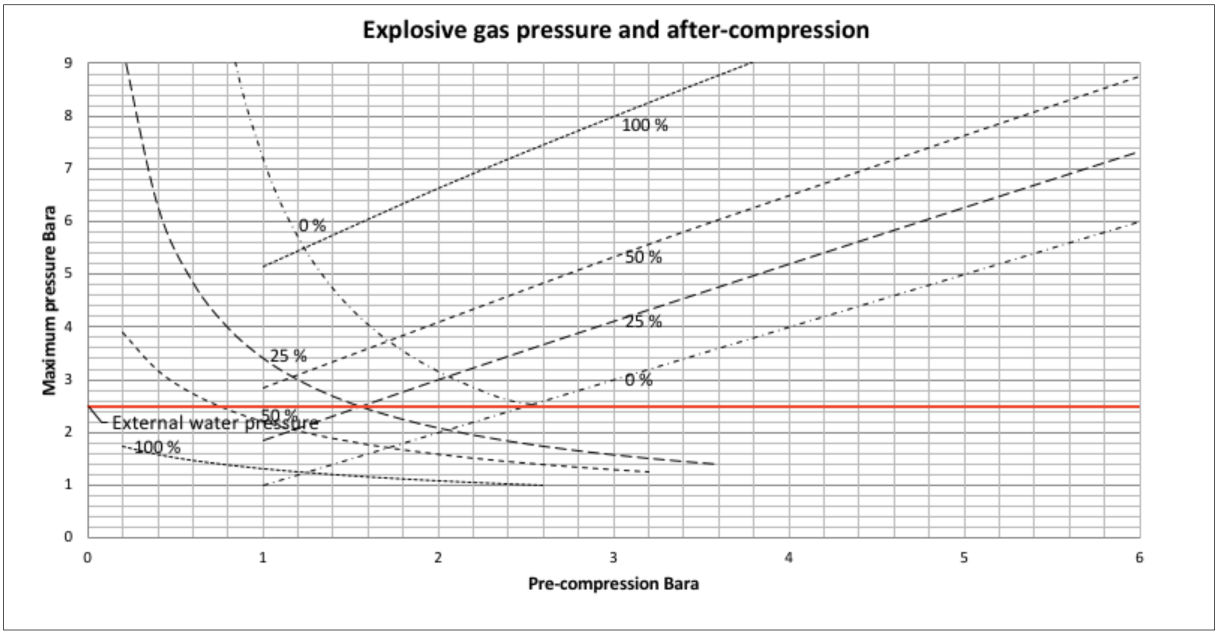


Figure 26: The graph shows the relation with small air pocket volume.

Larger volume:

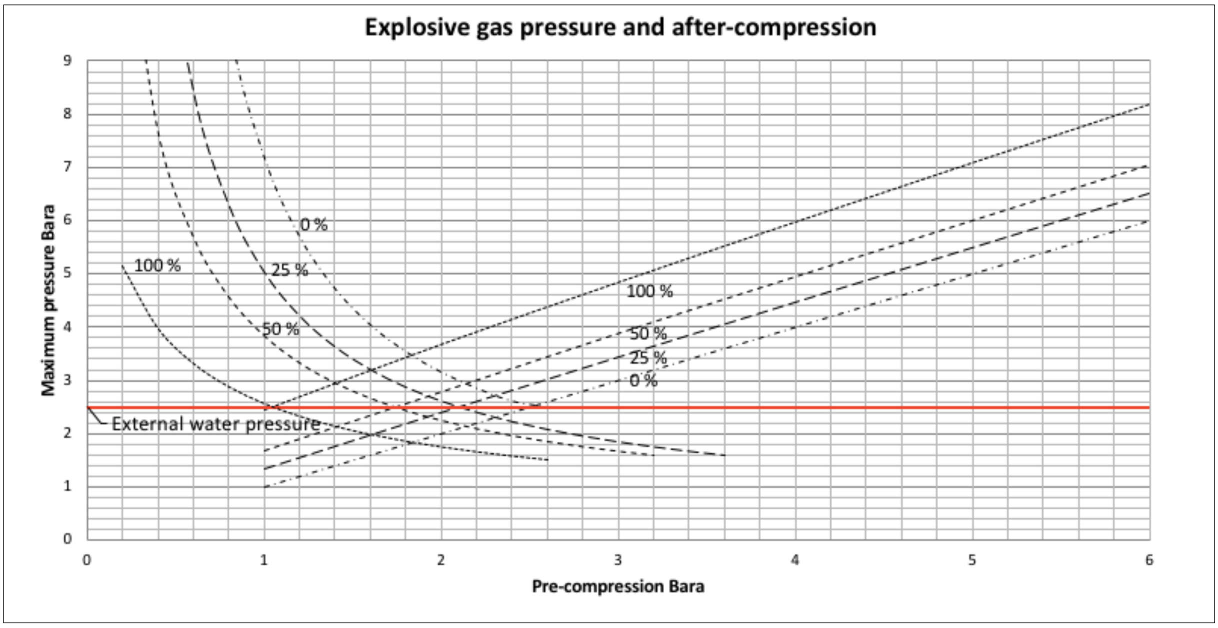


Figure 27: The graph shows the relation with bigger air pocket volume.

It is important to mention that the calculation for this method have many unsure factors, and it is therefore important to keep in mind that the calculation is only a estimation of the result.

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APPENDIX

A. **Research partners**
B. **Some recent tunnel piercings**
C. **Hjartøy – oil project**
D. **Midgardsomen water supply – Open method**
E. **Tunnelling towards the plug - Lyngsvatn**
F. **Pressure calculation of tunnel piercing at Lyngsvatn**
G. **Example of application for explosive gas pressure**
H. **Blasting with short rounds – Cost estimation**

A. RESERACH PARTNERS

- **Orica AS**
- **Hæhre AS**
- **Veidekke**
- **Norconsult**
- **Multiconsult**
- **Implenia**

B. SOME RECENT TUNNEL PIERCINGS

1. Kaarstø – open tunnel piercing (1999)
2. Kvitebjørn Rich gas pipeline project Kollsnes landfall – Tunnel shore approach (2002) depth 68m.
3. Melkøya – cooling water tunnels – piercing blast (2003) depth 80m. 1075m long tunnel.
4. Ormen lange – Intake and outlet tunnel for cooling water. Intake, depth 82m, length 1.345m and outlet depth 40m, length 980m (2004)
5. Sauda hydroelectric extension project (2009)
6. Fossan Kraftverk AS, hydroelectric project (2012)
7. Midgardsormen, water supply, open tunnel piercing (2015)
8. Lysebotn 2, hydropower plant (2017)
9. Smisto, hydropower plant (2018)
10. Tjeldbergodden gas power plant

C. HJARTØY – OIL PROJECT

The Hjartøy tunnel piercing was completed in 1987 and was the first tunnel piercing into the ocean. The oil was lead from Osbergfield to Sture in Øygarden. The tunnel was 2335 m long and ended in the ocean at a depth of 80 m. The tunnel piercing for this project was very successful.

The tunnel piercing for this project was special due to two reasons:

1. The short distance from plug to concrete construction.
2. The shape and drilling pattern of the last round.

Data	
Depth	80 m
Dimensioning pressure for the gate	25 bar
Pre-compression	4,5 bar
Volume of air pocket	3500 m^3
Total number for holes in “plug”	222
Explosives	2600 kg

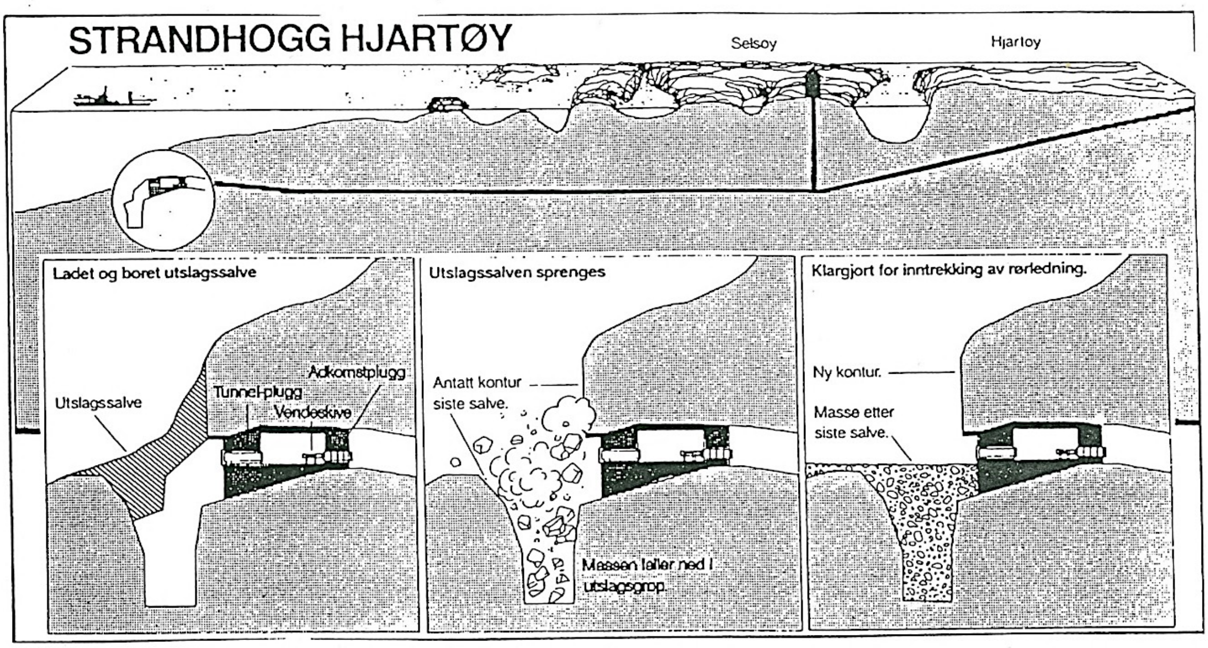

Fig. C.1: An illustration of the blasting process at Hjartøy.

Pit solution

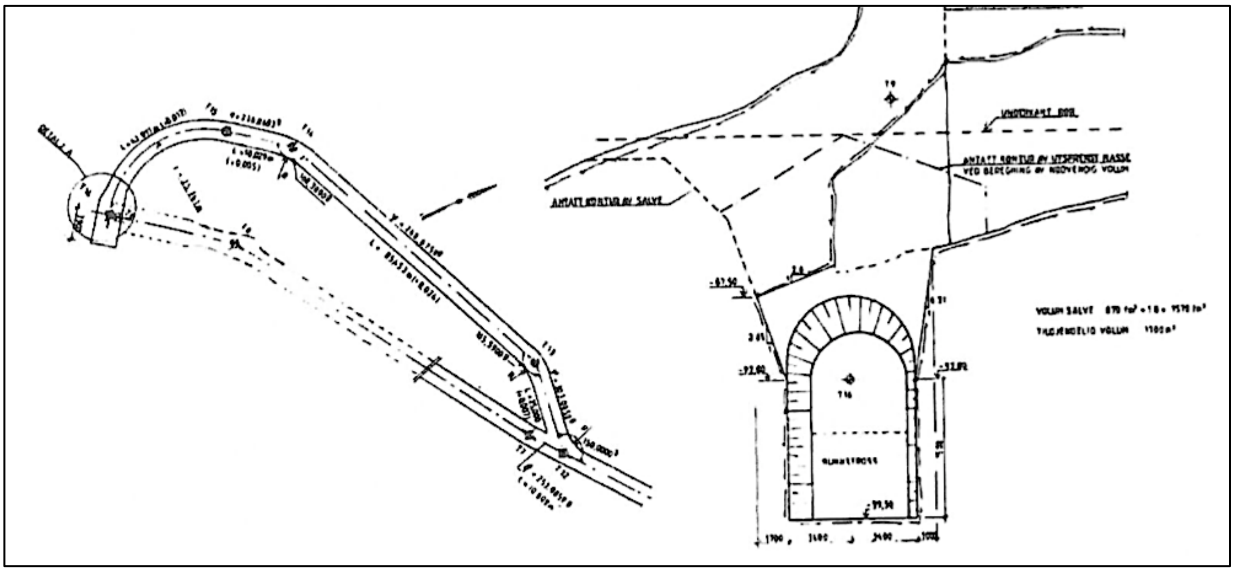

Fig. C.2: An illustration of tunnel and tunnel piercing design at Hjartøy.

The design of the tunnel was adjusted from the original plan to make it easier to excavate the collection pit. This pit had to be large enough to collect all of the blasted rock mass of 600 fm^3. The easiest way to make the pit was to excavate another short tunnel along the main tunnel, which was excavated deeper to reach the pits level. This solution was considered as the most efficient and cheapest way (see figure).

The tunneling towards the plug for both the main and the extra tunnel had to reduce the round length and even split some rounds in two. This was done to minimize the vibration and making sure the process was as safe as possible, and was at all time followed up with vibration monitoring. Amount of explosives for each round was 50 - 80 kg.

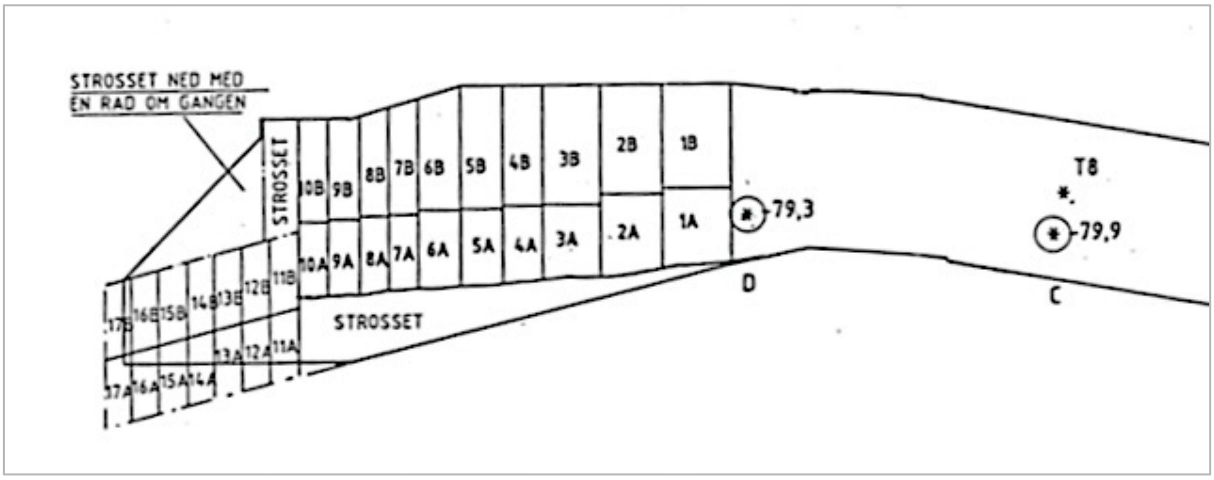

Fig. C.3: An illustration of the rounds in front of the plug, side view.

The solution for the tunnel piercing method

The solution of the method was decided in an early stage. The oil pipe was going to be drawn into a sluice between to concrete plugs. The outer plug was sealed between the oil- and bushing pipe. The sluice was equipped with valves and pipe so it would be possible to fill or empty with water if needed.

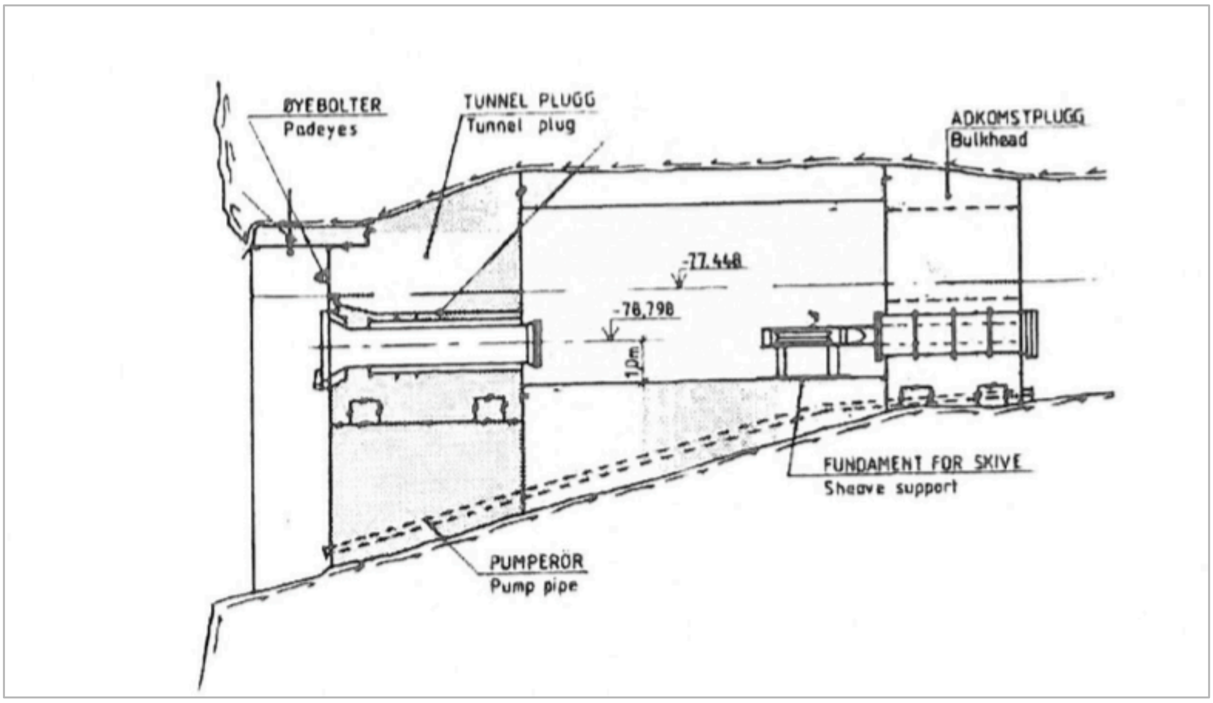

Fig. C.4: An illustration of the pipe constrction.

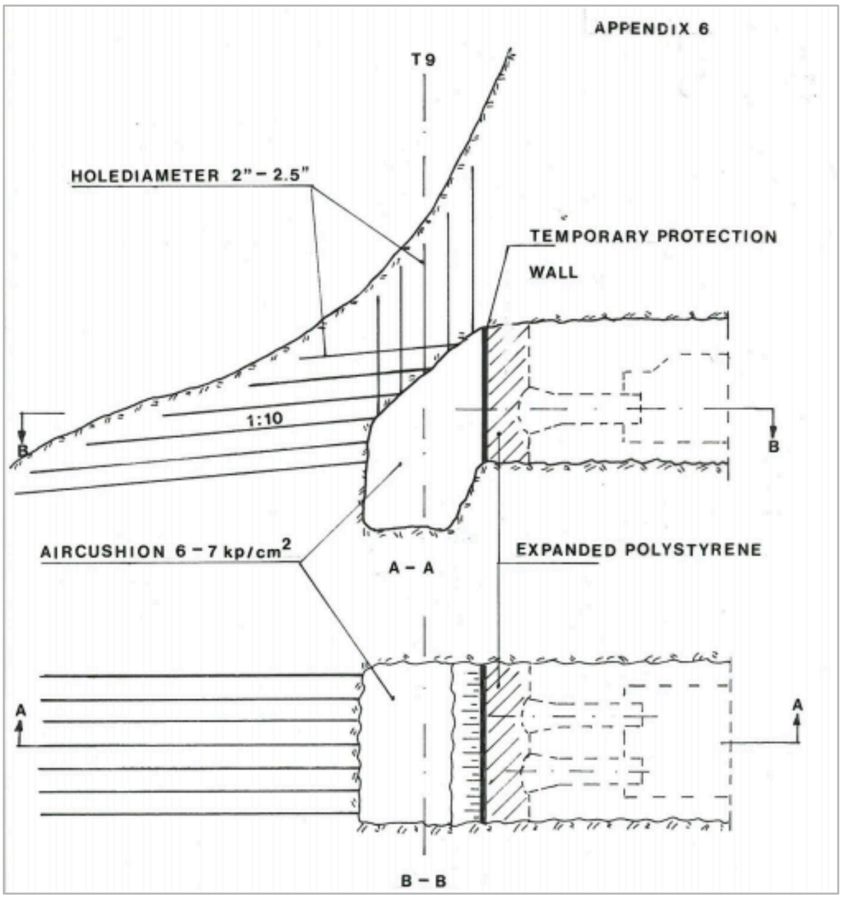

Fig. C.5: An illustration of the drilling plan, side and top view.

Due to the extra time for the project it was possible to complete the concrete constructions in a dry tunnel and excavate the tunnel before the pull-in of the oil pipe.

To map the area in front of the tunnel plug, 60 holes was drilled horizontally and vertically. The plug had a special shape which looked more like a cutting. The reason for this shape and size of the round was to be able to insert the pipe afterwards. The longest drilled holes where up to 20 m. To withstand the possibility of inflowing water in the holes, special packers where inserted into the holes. The sequence of the detonators was closely planned to make the round blast out in a triangle, where horizontal and vertical holes where blasted in pairs.

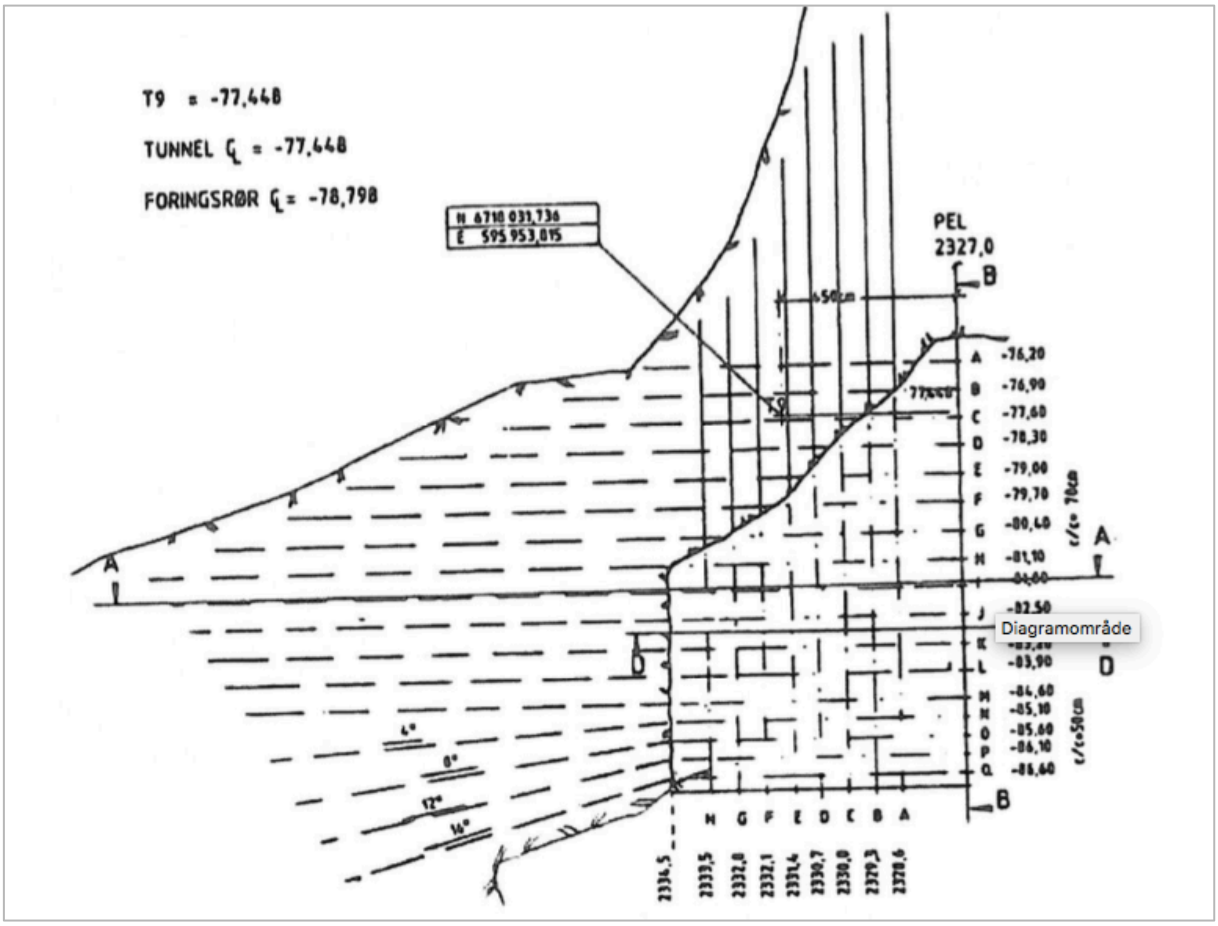

Fig. C.1: Side view, drilling pattern.

Pressure increase

The dimensioning pressure for the concrete constructions was 25 bara. The length from the concrete construction to the middle of the piercing plug was 12 m, which is considered as a very short length. It was therefore important to cover the concrete construction favorably to avoid damage on the concrete construction. The calculated maximum pressure with friction was 17.5 bara. After the blast it was measured a maximum pressure of less than 17.5 bara. No damage or leakage was observed and an inspection with a ROV showed that the opening was successful. The coverage of the concrete construction had been destroyed without leaving any damage on the construction.

For this project it was important that all errors were taken seriously and that investigation, planning risk analysis was done properly. For a project like this, any mistakes could be expensive and hazardous (Johansen) (Hoel, 1987).

D. MIDGARDSORMEN WATER SUPPLY – OPEN METHOD

Midgardsormen was an upgrade of the storm water system in Oslo, managed by the water and sewer administration in Oslo. A part of the project was to install a circular retaining shaft in soft clay (Johannesen & Grimsøen, 2011). The construction was finished in November 2010.

Calculations of the upsurge

After the piercing was complete it was done an evaluation of the blast. The gate was not exposed for any dynamic pressure of the water, and the air pressure was also low due to the 1100m between the gate and the plug. Due to the sag-curve in the tunnel the inflowing water would not develop a surge, and only rise in the shaft. The horizontal length from the gate to the plug was too long to build up a surge that could have an impact. It was assumed that the water would increase over the spillway.

Assumptions for the calculations:

Tunnel length:	1100 m
Tunnel area:	20 m^2
Tunnel volume:	25000 m^3
External water pressure:	20 m
Shaft height:	25.5 m
Shaft area:	5 m^2
Inflowing water speed:	19.8 m/s

Parameter	What	Value
C	Singular loss	1
Fs	Cross section tunnel	20
Fi	Cross section shaft	5
Fi	Cross section plug	15
ΣL/F	Length/cross section tunnel	54,08
g	Acceleration of gravity	9,81

The calculations are based on the general hydromechanics energy consideration (Bernoulli) and NTNU-model. The parameter description and calculation are described in 6.3.

n (sek)	δT	g/ΣL/F	$C\frac{Q/F_i\,\lvert Q\,/\,F_i\rvert}{2g}$	δQ	Q_n	Q	δH
0					0	0	20
1	1	0,18	0,000000	3,63	3,63	1,81	19,91
2	1	0,18	0,006708	3,61	5,42	3,62	19,73
3	1	0,18	0,026700	3,57	7,19	5,41	19,46
4	1	0,18	0,059575	3,52	8,92	7,17	19,10
5	1	0,18	0,104664	3,45	10,61	8,89	18,66
6	1	0,18	0,161044	3,35	12,24	10,57	18,13
7	1	0,18	0,227566	3,25	13,81	12,19	17,52
8	1	0,18	0,302872	3,12	15,31	13,75	16,83
9	1	0,18	0,385434	2,98	16,73	15,24	16,07
10	1	0,18	0,473585	2,83	18,07	16,66	15,24
11	1	0,18	0,565557	2,66	19,32	17,99	14,34
12	1	0,18	0,659521	2,48	20,47	19,23	13,37
13	1	0,18	0,753623	2,29	21,52	20,37	12,36
14	1	0,18	0,846030	2,09	22,46	21,41	11,29
15	1	0,18	0,934960	1,88	23,29	22,35	10,17
16	1	0,18	1,018725	1,66	24,01	23,18	9,01
17	1	0,18	1,095759	1,44	24,62	23,90	7,81
18	1	0,18	1,164648	1,21	25,11	24,50	6,59
19	1	0,18	1,224157	0,97	25,48	24,99	5,34
20	1	0,18	1,273249	0,74	25,73	25,36	4,07
21	1	0,18	1,311098	0,50	25,86	25,61	2,79
22	1	0,18	1,337107	0,26	25,87	25,74	1,50
23	1	0,18	1,350907	0,03	25,77	25,76	0,22
24	1	0,18	1,352358	-0,21	25,55	25,65	-1,07
25	1	0,18	1,341554	-0,44	25,22	25,43	-2,34
26	1	0,18	1,318803	-0,66	24,77	25,10	-3,59
27	1	0,18	1,284627	-0,88	24,22	24,66	-4,83
28	1	0,18	1,239742	-1,10	23,56	24,11	-6,03
29	1	0,18	1,185038	-1,31	22,80	23,45	-7,20
30	1	0,18	1,121565	-1,51	21,94	22,70	-8,34
31	1	0,18	1,050507	-1,70	21,00	21,85	-9,43
32	1	0,18	0,973159	-1,89	19,96	20,90	-10,48
33	1	0,18	0,890905	-2,06	18,84	19,87	-11,47
34	1	0,18	0,805189	-2,23	17,65	18,76	-12,41
35	1	0,18	0,717496	-2,38	16,38	17,57	-13,29
36	1	0,18	0,629321	-2,52	15,04	16,31	-14,10
37	1	0,18	0,542148	-2,66	13,65	14,98	-14,85
38	1	0,18	0,457428	-2,78	12,20	13,59	-15,53
39	1	0,18	0,376557	-2,89	10,70	12,15	-16,14

40	1	0,18	0,300851	-2,98	9,17	10,66	-16,67
41	1	0,18	0,231532	-3,07	7,59	9,12	-17,13
42	1	0,18	0,169710	-3,14	5,99	7,55	-17,51
43	1	0,18	0,116366	-3,20	4,36	5,96	-17,80
44	1	0,18	0,072340	-3,24	2,71	4,34	-18,02
45	1	0,18	0,038322	-3,28	1,06	2,70	-18,16
46	1	0,18	0,014837	-3,30	-0,60	1,05	-18,21
47	1	0,18	0,002247	-3,30	-2,25	-0,60	-18,18
48	1	0,18	-0,000738	-3,30	-3,90	-2,25	-18,07
49	1	0,18	-0,010323	-3,27	-5,53	-3,89	-17,87
50	1	0,18	-0,030813	-3,24	-7,12	-5,51	-17,60

- Maximum water current Q, 25 m^3/s
- Total amount of water reaches over the sill, 342.81.
- At Q = 25.43 the water will reach the sill over 2.5 m with 27 second.
- At Q = 1.05 the water will turn the directions within 46 seconds, at δH -18.21.

The water will reach the highest level in the first surge. The calculations show that the maximum water flow up the shaft will be 25 m^3/s. This will give a velocity of 25/5 = 5 m/s.

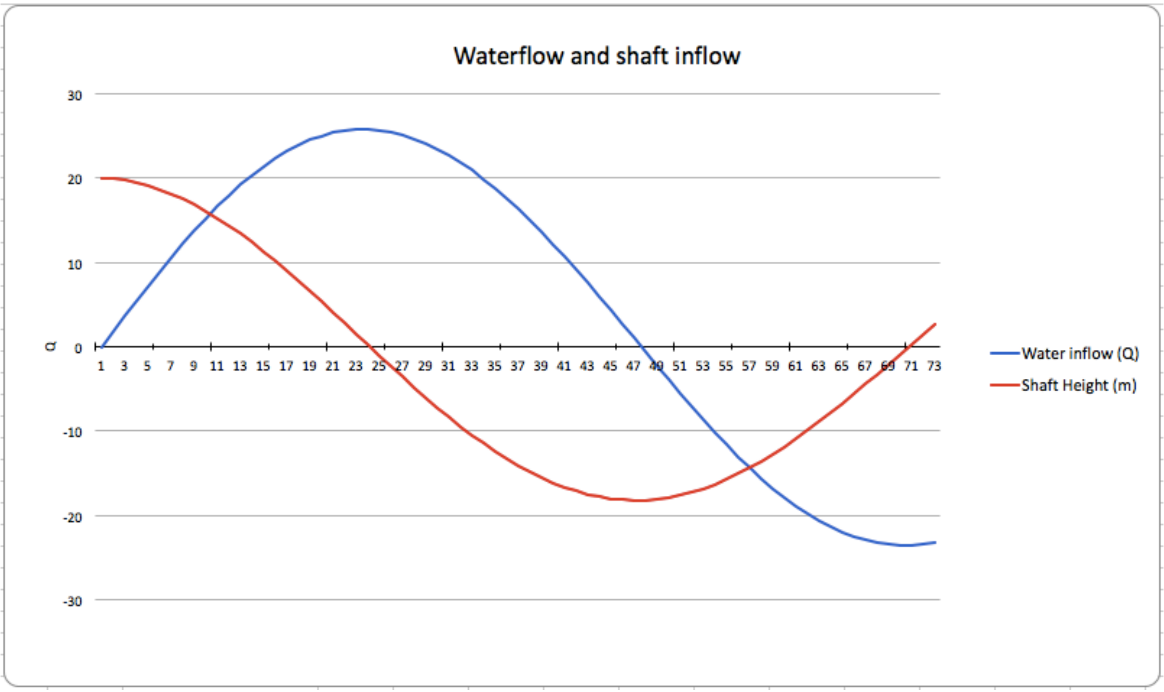

Figure shows the water inflow Q and the water level in the shaft (m) in ratio to seconds.

This velocity is not big enough for the water to behave as a fountain out of the shaft but could reach to 1 meter over the shaft opening (Olsen, Hugås, & Rømcke, 2012).

E. TUNNELLING TOWARD THE PLUG - LYNGSVATN

Situation

The project is called Lysebotn 2 and is in Lysebotn in Forsand community, Rogaland. The power plant is designed to replace the old power plant and will give an increase of 180 GWh sustainable energy (Lyse, 2013).

The project had three tunnel piercings, one in lake Lyngsvatn, another in lake Strandvatn and the last one in Lysefjorden. For this appendix the tunnel piercing at Lyngsvatn is described.

Type of tunnel piercing:	Closed
Installed power:	180 GWh
Construction period:	2012-2018
Water level:	647.78 m a.s.l
Air pressure in air pocket:	2.0 bar
External water pressure:	2.5 bar

The tunnel excavation method and adjustments toward the plug is separated into 3 phases in the following description.

Phase 1: - From the gate area to the end of the tunnel

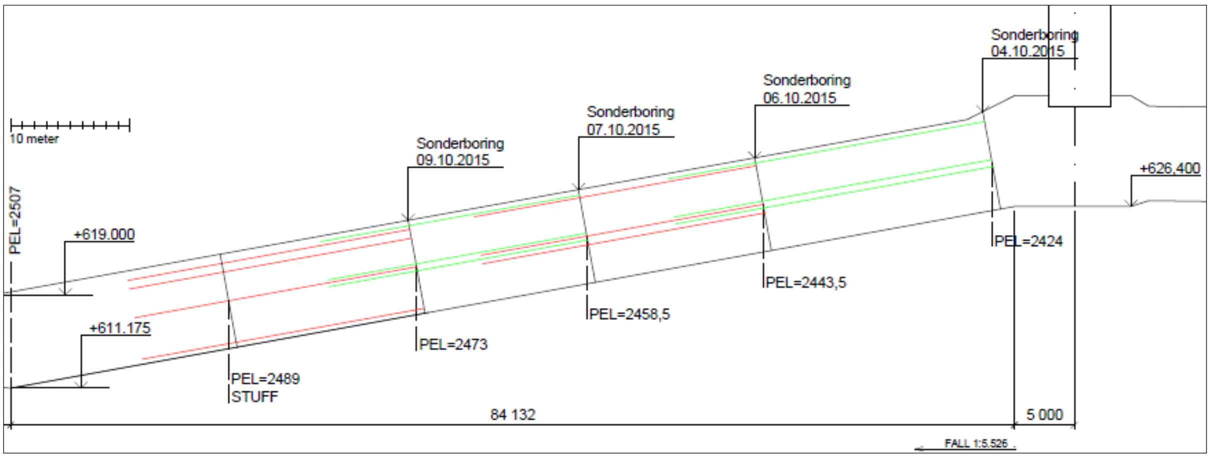

Fig. E.1: An illustration of the first phase of the excavation (Implenia AS).

Probe-drilling

- Probe-drilling for every 3rd round (15m pull).
- Length variation from 22 m to 27 m.
- Good geology. Only necessary with 3 probe-holes at station 2473.

Grouting

- Only used if larger leakages from the probe-drilling.

Phase 2: - From station 2473 to 2518.5

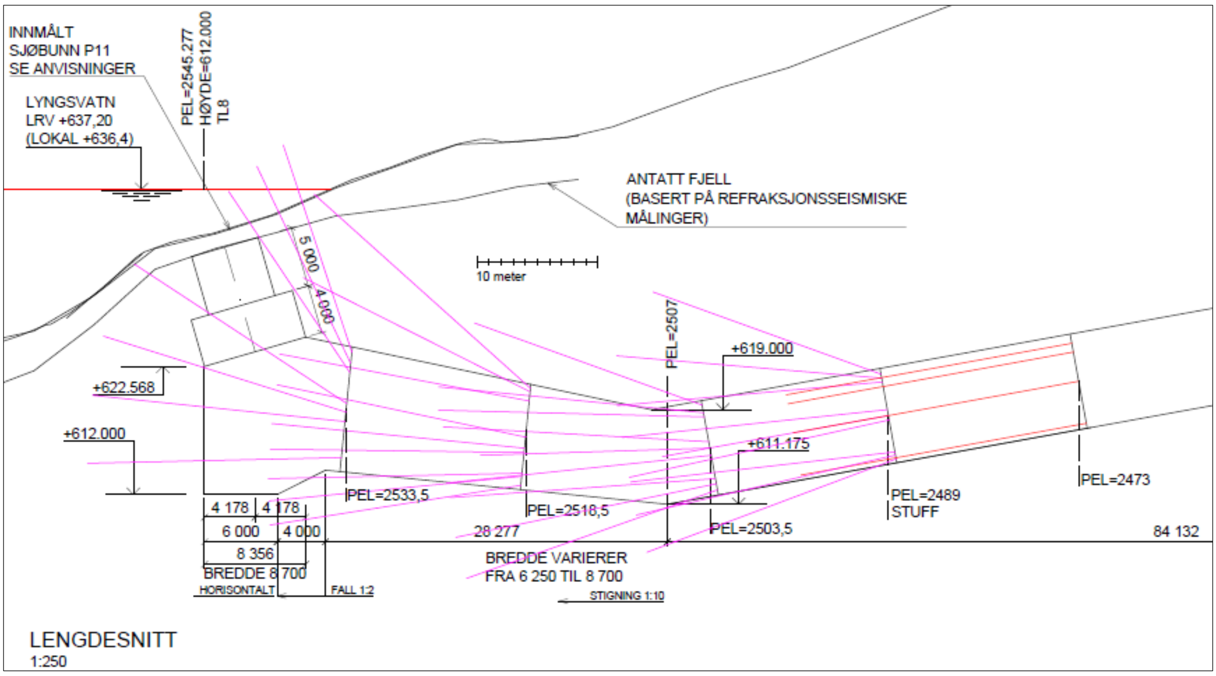

Fig. E.2: An illustration of the second phase of the excavation (Implenia AS).

Probe-drilling

- Due possibility of surface-parallel water-conducting cracks appearing ahead of the tunnel, 9 probe-holes for every 15 m is set.
- The probe-holes are placed in the face to cover the front and the contour.
- 7 m overlap = 22 m long probe-holes.
- A 19 m long hole is drilled from the centreline.
- As an extra control of the remaining distance to the reservoir a probe-hole of 20 m is drilled with an incline of 30° into the hanging wall.

Grouting

Assumed permeability of injected rock mass is 2-3 Lugeon.

New criteria due to 50 m up to the water surface. Grouting when the leakage out of a hole is:

$$Q > 3 \text{ l/min from one of the 9 probe-holes}$$

Or:

$$Q > 5 \text{ l/min total leakage from 3 of the holes}$$

Phase 3: - The last 26.5m to the bottom of the collecting pit

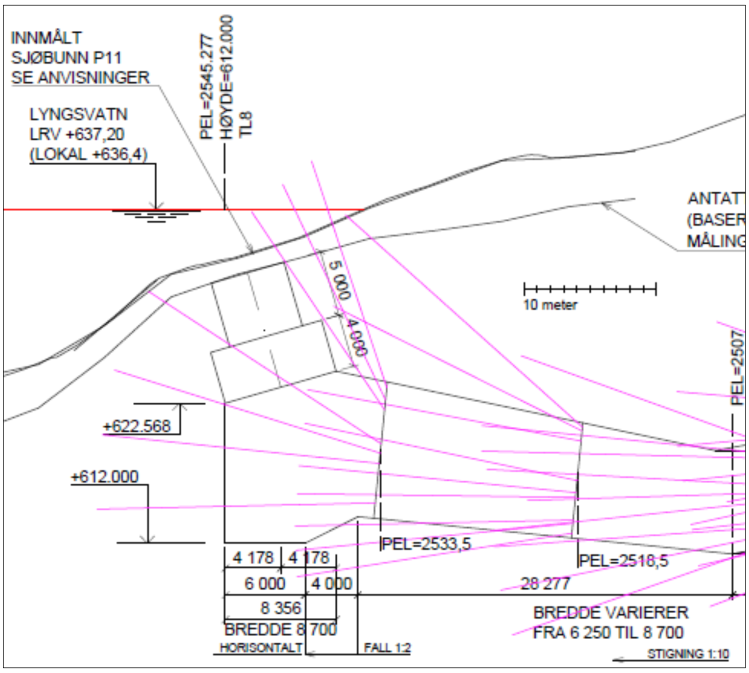

Fig. E.3: An illustration of the last phase of the excavation (Implenia AS).

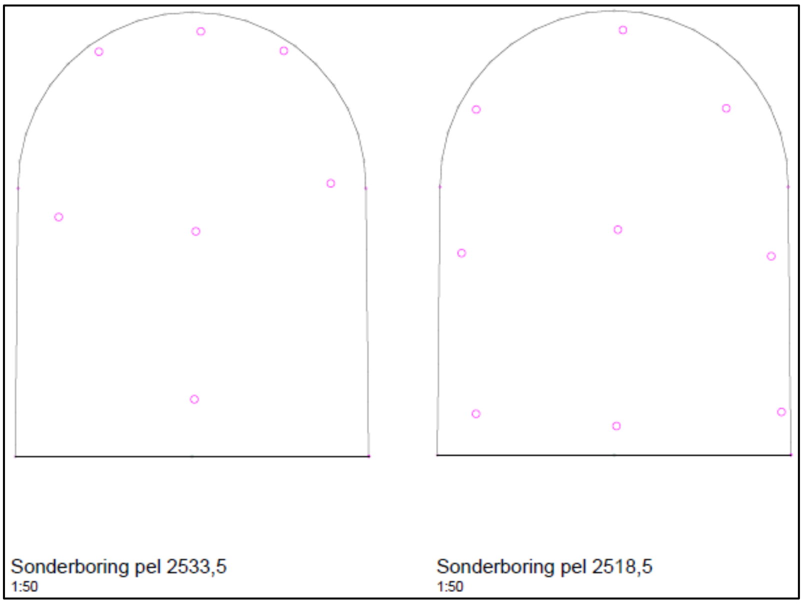

Fig. E.4: An illustration of the probedrilling of the third phase (Implenia AS).

Grouting

The criteria for this area is:

Q > 2.5 l/min from one of the 15 m long probe-holes

Or:

Q > 10 l/min total leakage from all the 15 m long holes

The piercing area

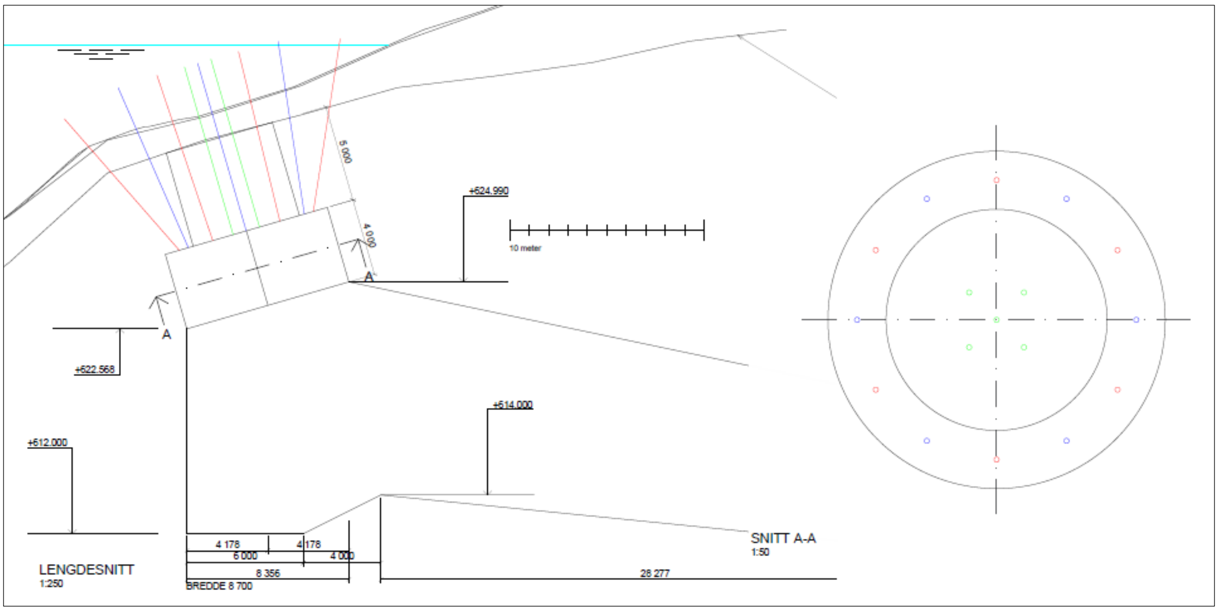

Fig. E.5: An illustration of the probe drilling for the breakthrough (Implenia AS).

Probe-drilling

After the collection pit is made, control holes toward the reservoir are drilled to map the area around the piercing area. These probe holes will penetrate into the reservoir.
The plan for the probe-drilling is shown above:

- 6 holes are drilled into the contour of the shaft (red marking). These are drilled 4 m outside the piercing area.
- Another 6 holes (blue marking) are drilled with 2 m outside the contour.
- Minimum 5 holes (green marking) are placed evenly distributed parallel with the piercing direction.

Dredging is done to prevent unnecessary sediments from flowing into the tunnel. A crane with a grab will be used to move the sediments from the piercing area, because the water surface is not supposed to be higher than 660-665 meters above sea level, in May/June.

When the bore holes have pierced through the reservoir they must be drilled another 1.5 m further into the reservoir, to make sure it is the reservoir that is hit. The piercing holes shall be plugged in the end of the hole and grouted with cement mortar. With these 17 boreholes it is now possible to map the sea bottom area more detailed. If there appears any unexpected water leakage or weak rock mass from the drilling, more probe holes should be drilled.

Grouting

- 4-8 holes are drilled in the contour of the piercing are, all the way until only 3 m remains from the rock surface, with a 10° angle form the centre line of the piercing shaft.
- Only grouting in holes with leakage > 5 l/min.
- With > 15 l/min extra grouting holes on each side of the leaking bore hole is recommended. The length will vary from 4 to 8 m.
- After the first grouting fan has harden, another fan of 4 - 8 bore holes should be drilled, with 1 - 2 m remaining to rock surface. Still only grouting for holes with > 5 l/min.

If there appears any large water inflow while drilling, it should be stopped an injected right away, before further drilling is done. Hence, the grouting rig should be at the tunnel face at all time while probe-drilling.

When grouting this close to the rock surface hydraulic cracking of the rock mass in the area is normal. Therefore, it is essential to lower the grouting pressure:

3 - 4 m from the rock surface = max. 10 bars
1 – 2 m from the rock surface = max. 5 bars

It is very important that the process is checked at all time and that the grouting pressure is stopped as soon as a sudden pressure loss occurs or an uncontrollable situation of the injection inflow. Dry holes and holes with leakage less that 1 l/min pr. 8 m borehole is filled with cement mortar.

Rock support

For the piercing area some consideration of special risk elements is connected to:

Work safety

- The risk of downfall from the face while probe-drilling, round-drilling, loading, and coupling.

Risk of downfall that leads to damage

- Loading or coupling in the piercing round
- Light cable
- Measuring cables

Risk of downfall in the tunnel after the tunnel piercing is will reduce the efficiency of the intake. Rock support for this area are bolts and fibre-reinforced shotcrete is applied at the walls, in the shaft up to the plug, and on the plug area. The reason for also applying it on the plug area is to make it easier to see and perform the loading and coupling.

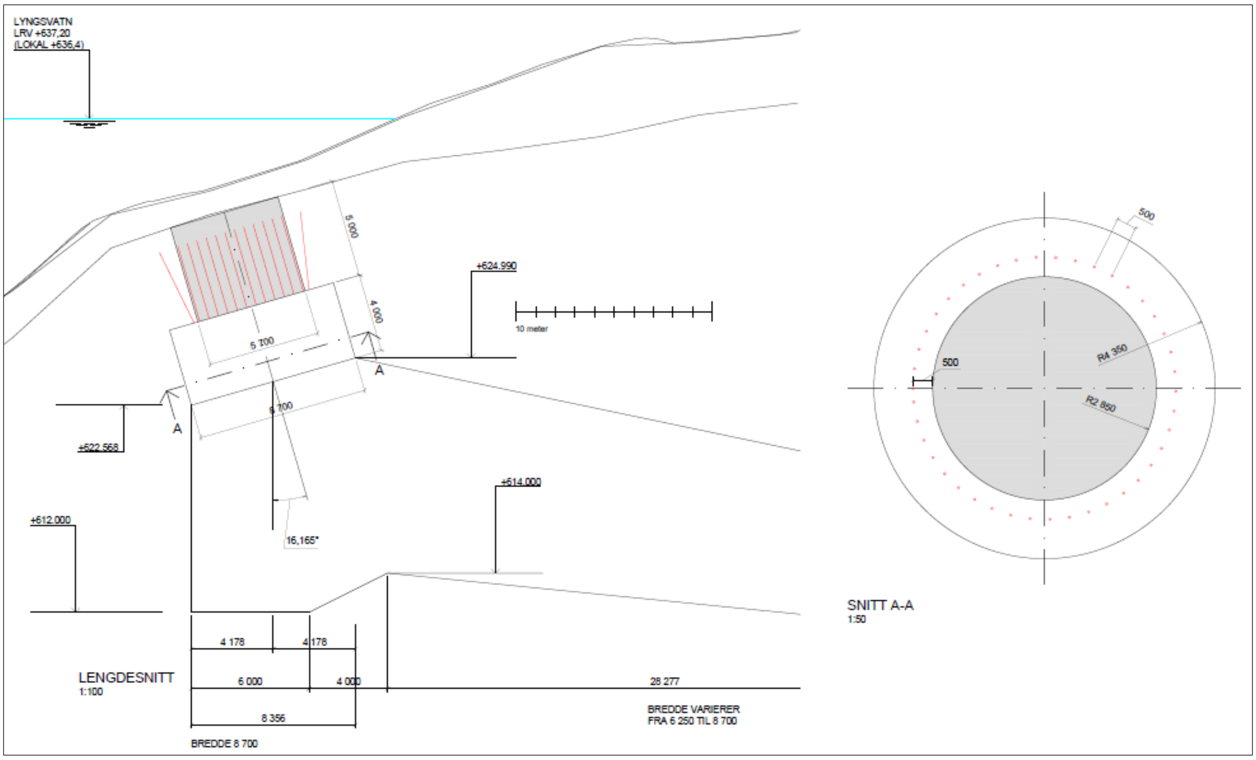

Fig. E.6: An illustration of spiling and bolts of the last round (Implenia AS).

Drilling pattern

Normally the boreholes in this area are drilled with 0.3 – 0.5 m from the rock surface. With an uneven surface, the chances of drilling the holes too far is high, which will lead to leakage into the tunnel. If this occurs a solution is to either shudder the entire hole and then drill a new one, or to plug the hole with a packer if possible. It is also important that the inflowing water is so low that it is safe to load the round.

Drilling

Boreholes, 48 mm: 76 holes

Large holes, 102 mm: 9 holes

The length of the holes varies, due to uneven surface. But in average ca 4.5 m.

Contour	29
Second contour	17
Cut holes	12
Large holes	9
Bench holes	18
Total holes	**85**

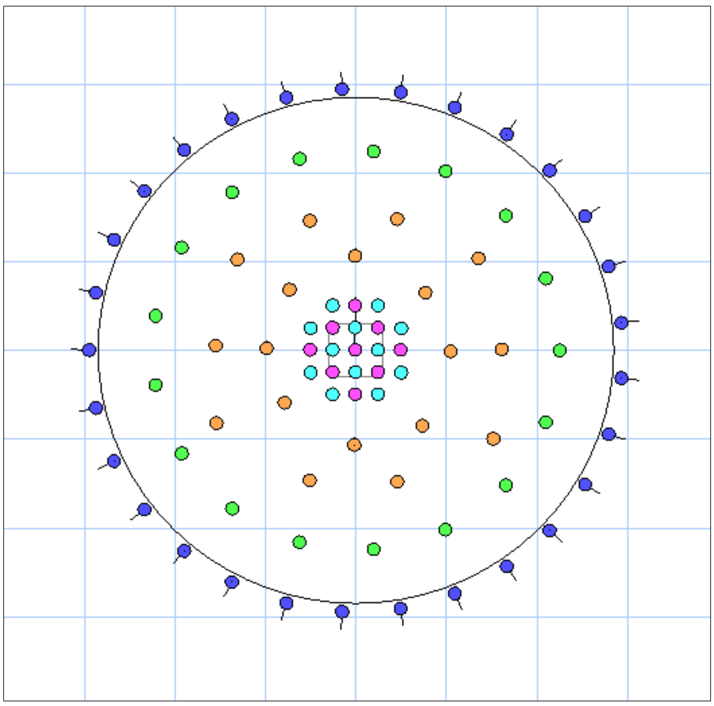

Fig. E.7: Drilling plan for the piercing plug (Implenia AS).

Loading

- All the holes are loaded with unitronic600.
- Two detonators are used in each hole, one in the end and the other in the middle. For holes longer than 6 m, an 3rd detonator is inserted 1.5 m into the holes to minimize the risk of unsuccessful blasting.
- The holes are loaded with Ricord 5, a 5 gram detonating cord, that is placed along the hole.
- Explosives:

Cut:	Eurodyn – 32x1100
Bench and contour:	Eurodyn – 39x1100

- In the end the holes are propped with a detent spring before the tree plug is installed. The tree plug will seal the hole, not only to keep the loading in place, but also to prevent water leakage.

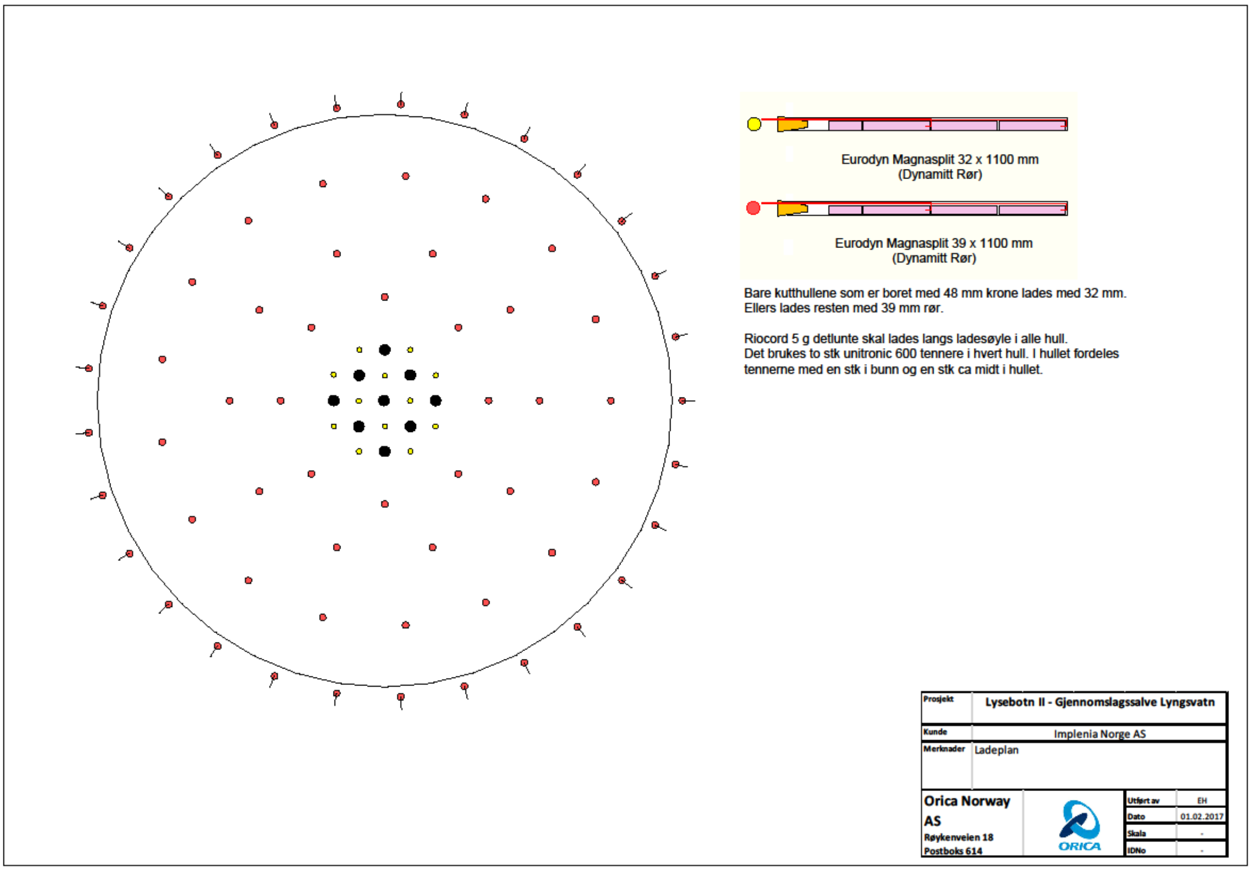

Fig. E.8: Loading plan for the last round (Implenia AS).

Detonators

Electronic detonators were used for the first time in this underwater tunnel piercing. The results turned out much more precise.

Type of hole	Numbers of holes	Average drilling length (m)	Average charge length (m)	Detonators 6 m	Detonators 9 m	Explosive dimension
Contour	30	5.1	4.6	1	1	39 mm
2. Contour	18	5	4.5	1	1	39 mm
Stross	17	5.5	5	1	1	39 mm
Cut	12	6	5.5	1	1	32 mm

Charge meter 32 mm		**66**
Charge meter 39 mm		**305.55**
Numbers of pipe (one pipe = 1,1 m)	32 mm	**60**
	39 mm	**278**
Number of boxes	32 mm	**3**
	39 mm	**22**
Amount kg	32 mm	**69.6**
	39 mm	**486.1**
	SUM	**555.7**
Amount fm3 (Average holelength = 5.3 m)		**132.5**
Specific charging		**4.19**

Coupling

- Each hole will be loaded and coupled at the same time.
- All couplings are inserted with fat to keep the durability and to protect it from damage like leakage over time.
- The connecting of the blasting wire and the coupling wire is water resistant. The cables are securely fastened to the tunnel wall, all the way to the gate. Here the wires are pulled through an opening in the gate and pulled up the shaft.
- Another pipe that is installed is the air filling. It is important that all the wires and pipes are protected from damage.

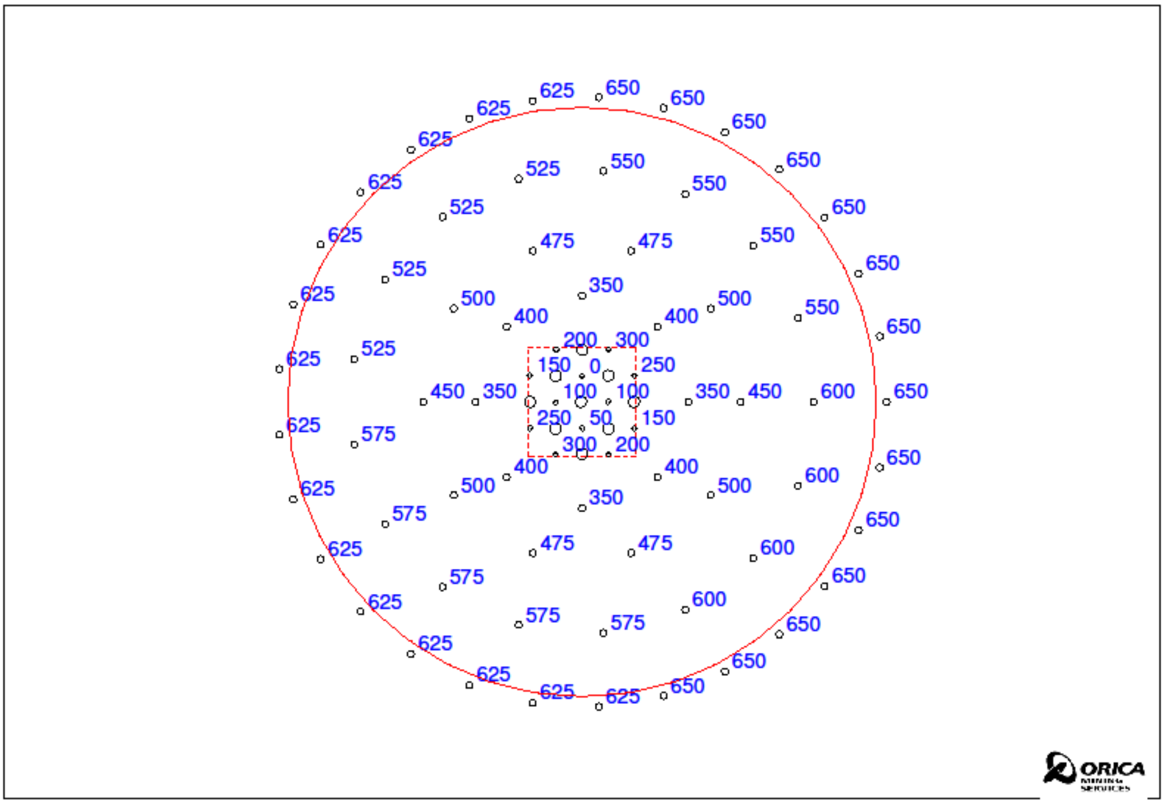

Fig. E.9: Couplin plan for the last round (Implenia AS).

Water-filling

It is estimated that the water level will be at kt. 648, when the tunnel piercing is planned to be pierced. The distance between the gate and the piercing area is 130 m.

There are 3 cables that are fastened from the piercing area to the gate.

1. Blasting wire (2.0 x 2.5 mm orange firing cable)
2. Wire to measure the water level in the air pocket.
3. Wire for gate pressure measurement (signal cable for the measurement equipment)

The air in the air pocket is inserted through a cast-in pipe in the gate construction. The dry side of the gate is connected to the compressor, and at the wet side the air pipe is coupled. This air pipe is installed in the air pocket in a way that it will not damage the coupling and loading work. The air pressure in the air pocket is then measured by a manometer, which is installed in front of the plug. When the desired level of air is achieved, the valve can be closed from the dry side of the gate.

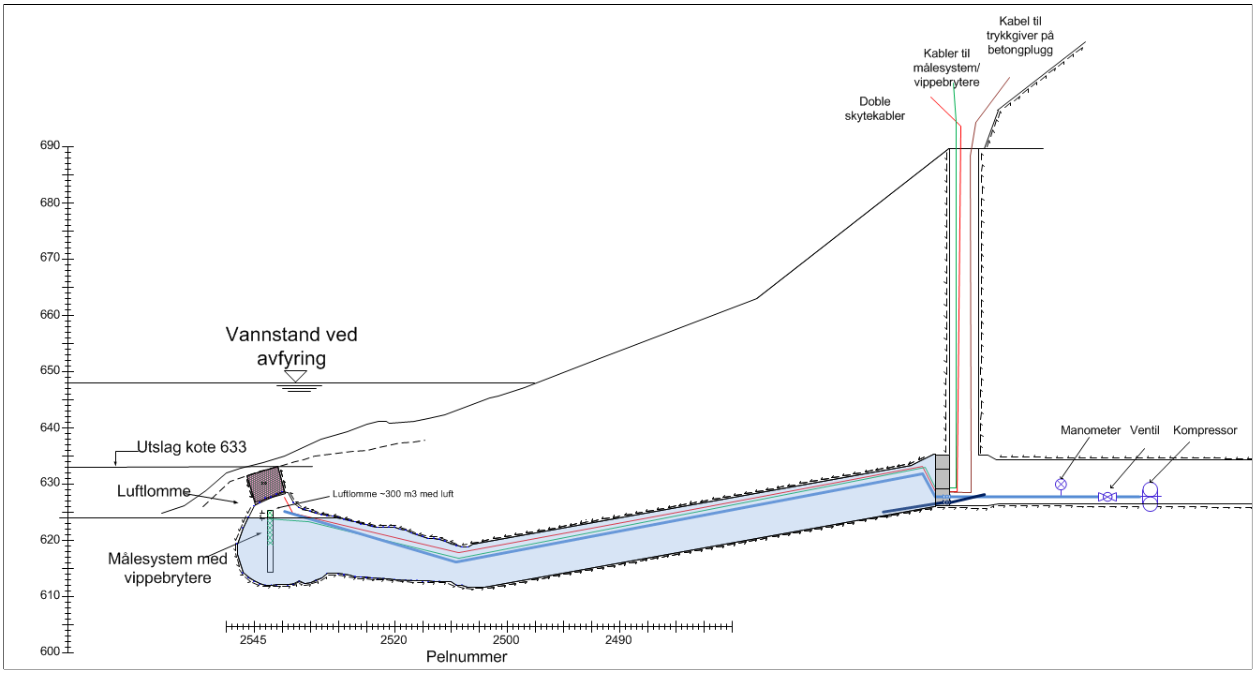

Fig. E.10: An illustration the coupling in the tunnel, side view (Implenia AS).

Air pocket

The instrument that is used to measure the water pressure at the gate is a P12 High Pressure Hydrophone. This is installed at the centre of the gate constructions lower part. The toggle switch and is installed at kt. 623 in the air pocket.

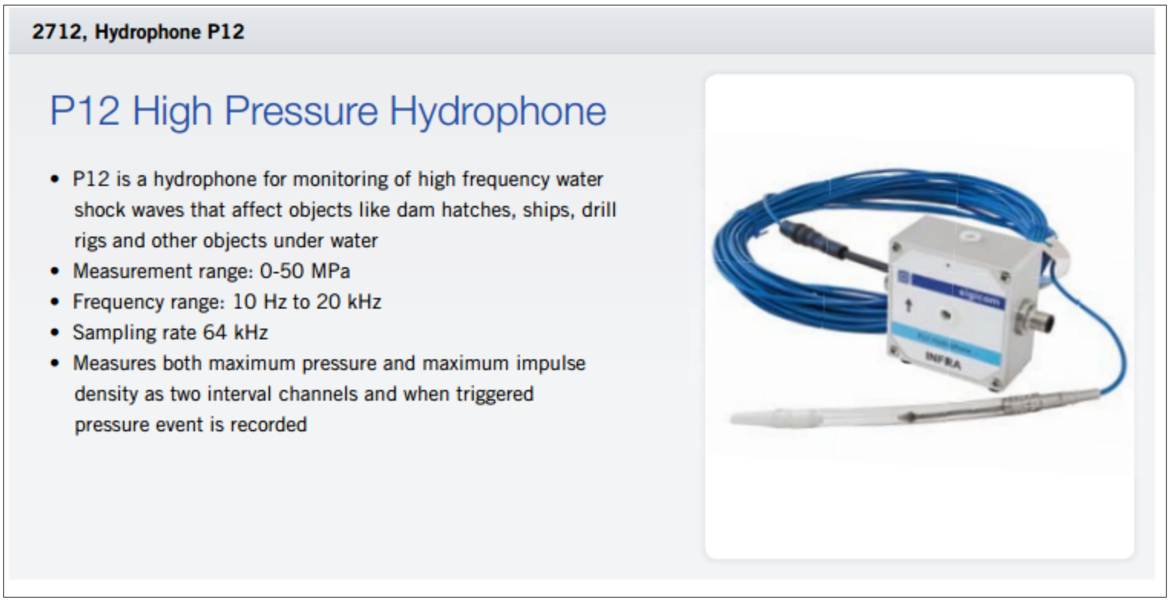

Fig. E.11: Type of measuring equipment.

Water-filling

- Pump: 60 m^3/hour.
- Time: 5 days.
- Measured with carburettor float, which is installed at several heights.
- Avoid water from reaching the charging and loading of the plug.

Results from the measurement:

Grade elevation (m.o.h)	Volume airpocket (m^3)
619	1433
620	1126
621	858
622	631
623	448
624	297

F. PRESSURE CALCULATION OF TUNNEL PIERCING AT LYNGSVATN

This is the calculation for tunnel piercing situation in Lyngsvatn at the project Lysebotn 2. In this calculation it is used Solviks calculation model for closed tunnel piercing, with neglected energy loss. This calculation uses the values of the measured results. See chapter 7.2.2. for further calculation description.

Amount of dynamite, G	556 m^3/kg
External water pressure, P_S	2.6 bara
Dimensioning gate pressure	6.8 bara
Amount of gas development into the air pocket	50% (experienced)
Measured amount of pre-compression before blasting, P_1	2.2 bara
Air volume in air pocket, V_1 (kt.623)	450 m^3 (estimated)
Volume explosive gas, V_g	$0.8 \bullet 556 = 444.8\ m^3$
Atmospheric pressure, P_a	1.0 bara

Two different pressure ratios were calculated:

1. Pressure increase caused by the gas pressure increase in the air pocket from the explosive, after the blasting (P_2)
2. Total pressure increase caused by the inflowing external water (P_{max})

The importance for this calculation is to find the right balance between reducing the pressure from the inflowing water and not damaging the gate if the pressure is too high. For the calculation it is therefore taken into account that the gas development is very variable, and it is therefore important to include all possible amounts of gas developments. From experience it is for this calculation used 50% gas development.

For the maximum pressure with 50% gas development P_2 is:

$$P_2 = \left(\frac{V_g * E_g}{V_1} + P_1^{\frac{1}{\kappa}}\right)^{\kappa} = \left(\frac{444.8 * 50\%}{450} + 2.2^{\frac{1}{1.4}}\right)^{1.4} = 3.113\ bara$$

First find the f(x):

$$f(x) = 2\left(\frac{P_s}{P_2}\right)^{0.081} = 2\left(\frac{2.6}{3.113}\right)^{0.081} = 1.971$$

$$P_{max} = \left(\frac{P_s}{P_2}\right)^{f(x)} \cdot P_2 = \left(\frac{2.6}{3.113}\right)^{1.971} \cdot 3.113 = 3.08\ bara$$

The calculation for the graph

Gas development:

	100 %		50 %		25 %		0 %
P1	P2	P1	P2	P1	P2	P1	P2
1	2,617694	1	1,754613	1	1,362283	1	1
2	3,8702	2	2,891522	2	2,434067	2	2
3	5,051783	3	3,987984	3	3,483969	3	3
4	6,197655	4	5,064638	4	4,52336	4	4
5	7,321306	5	6,129162	5	5,556379	5	5
6	8,429548	6	7,185366	6	6,585058	6	6

After-compression:

P1	P2 100%	Pmax 100%	P2 50%	Pmax 50%	P2 25%	Pmax 25%	P2 0%	Pmax 0%
0,2	1,45195	4,92547	0,74579	11,8247	0,44839	25,8144	0,2	110,503
0,4	1,77755	3,89454	1,01955	7,68409	0,68955	13,2500	0,4	31,1929
0,6	2,07214	3,28990	1,27350	5,77885	0,91892	8,83222	0,6	16,3087
0,8	2,35023	2,88108	1,51727	4,67459	1,14242	6,62706	0,8	10,7007
1	2,61769	2,58244	1,75461	3,95204	1,36228	5,31930	1	7,88374
1,2	2,87757	2,35310	1,98745	3,44178	1,57963	4,45880	1,2	6,22544
1,4	3,13173	2,17059	2,21693	3,06191	1,79513	3,85168	1,4	5,14596
1,6	3,38136	2,02144	2,44379	2,76792	2,00919	3,40133	1,6	4,39279
1,8	3,62731	1,89695	2,66853	2,53350	2,22210	3,05442	1,8	3,83999
2	3,87020	1,79132	2,89152	2,34213	2,43406	2,77920	2	3,41831
2,2	4,11048	1,70037	3,11303	2,18286	2,64523	2,55566	2,2	3,08674
2,4	4,34852	1,62119	3,3332	2,04820	2,85572	2,37053	2,4	2,81960
2,6	4,58460	1,55155	3,5524	1,93280	3,06563	2,21474	2,6	2,6
2,8	4,81896	1,48978	3,77064	1,83277	3,27502	2,08182	2,8	2,41642
3	5,05178	1,43457	3,98798	1,74520	3,48396	1,96710	3	2,26077

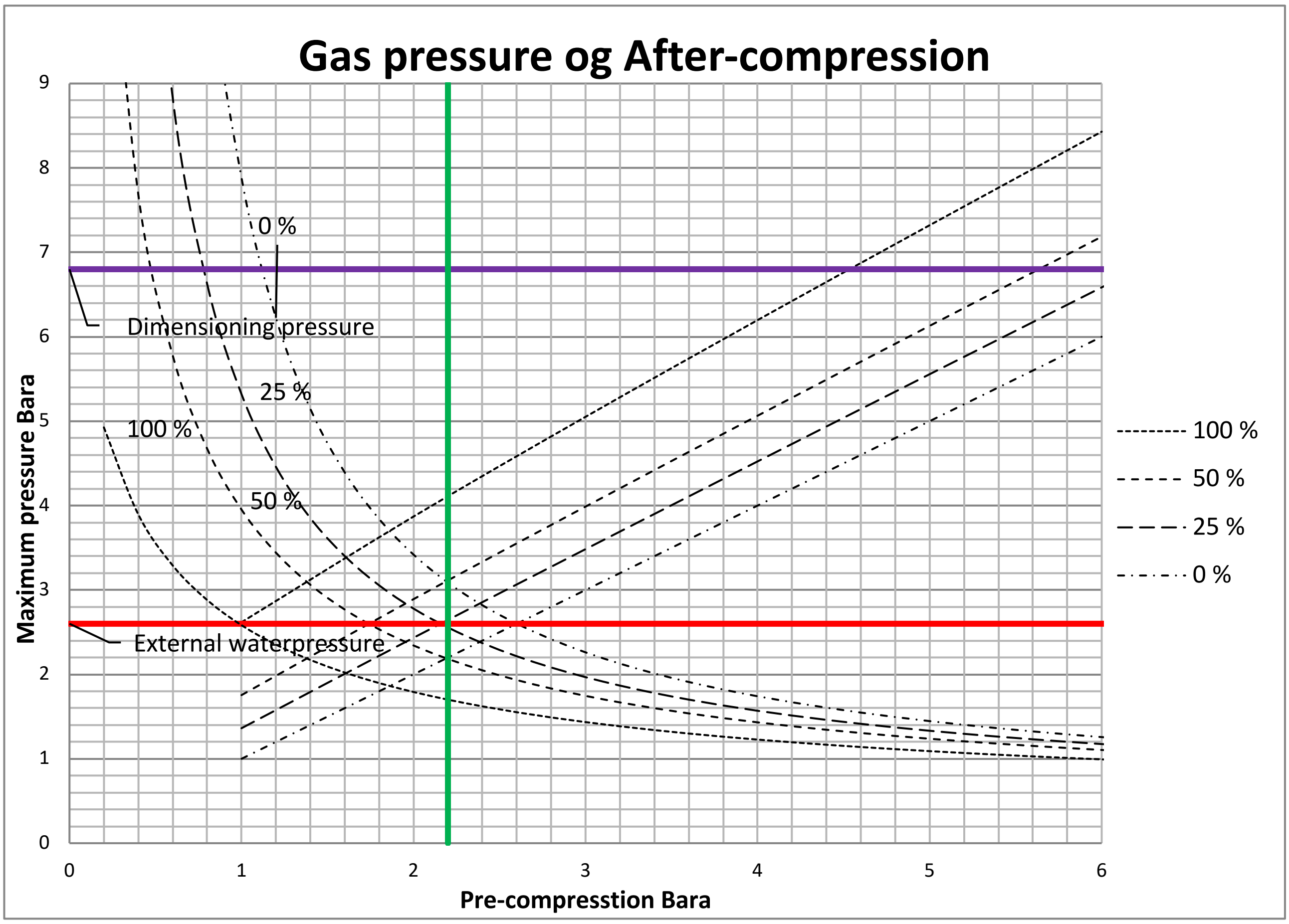

Fig. F.1: Result from the calculation, green line shows the pre-coompression (Orica AS).

Results for Lyngsvatn

The tunnel piercing at Lyngsvatn was calculated and analysed. The data from the calculations of the piercings are as follows:

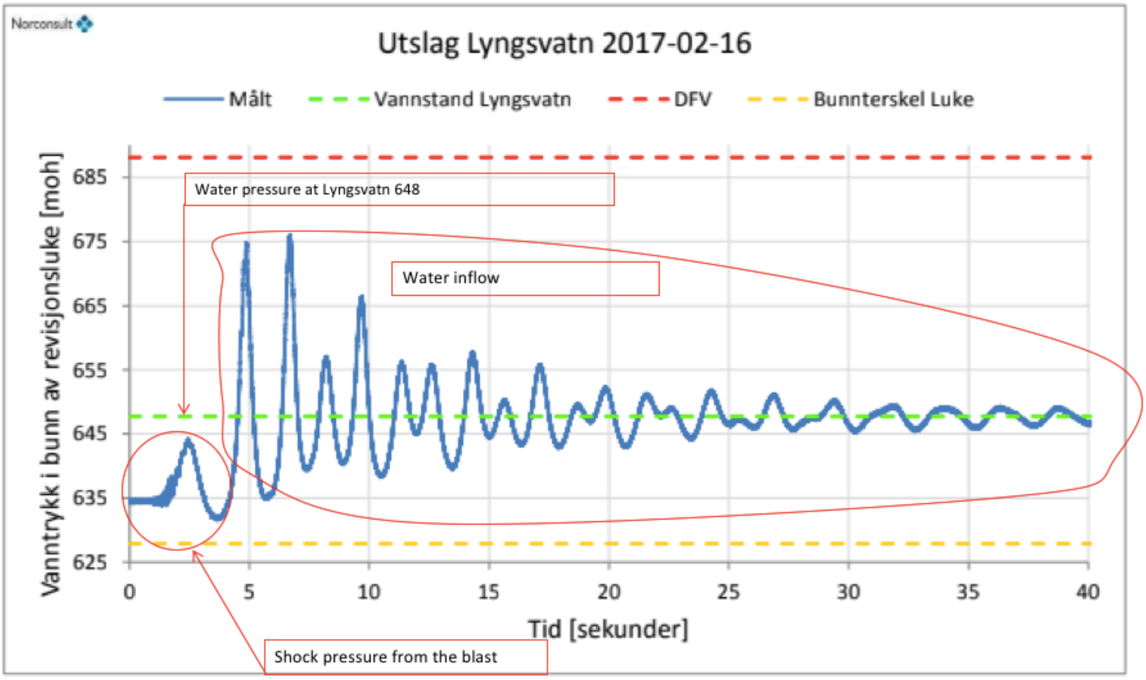

Fig. F.2: The measured result from the breakthrough blast (Norconsult).

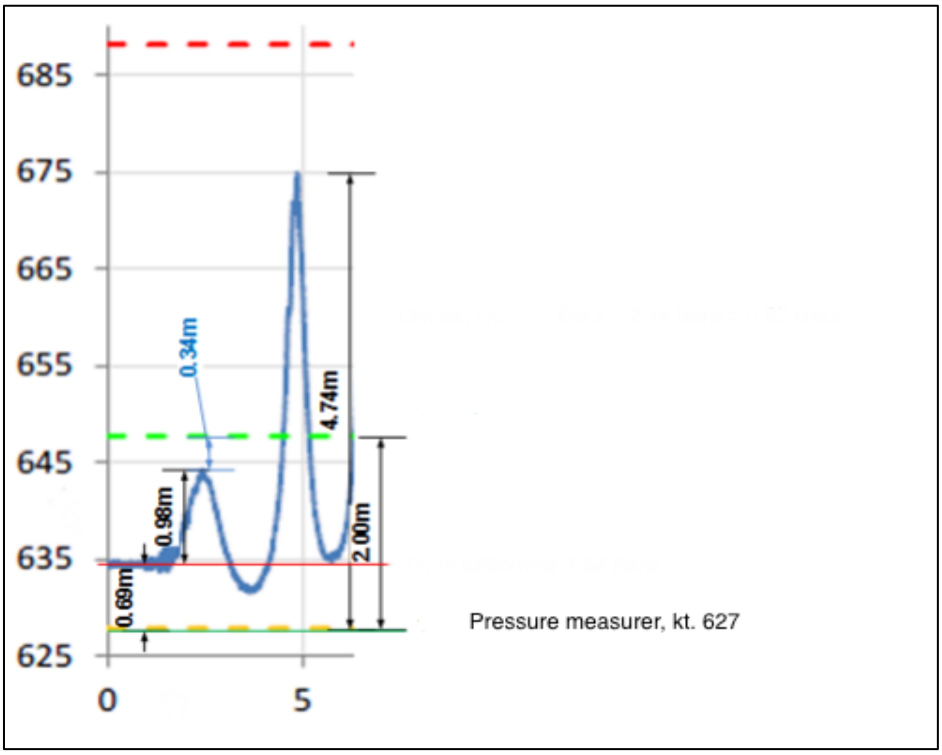

Fig. F.3: Measurement of the results in F.2.

Results from the measurement:

Starting pressure is 635-623 = 12 mVs (2,2 bara).

Pressure increase from the blast (gas development) is 645-635 = 10 mVs (2 bara).

Pressure increase from the water inflow is 675-635 = 40 mVs (5 bara).

Total pressure increase from the blast 2,2 + 0,98 = 3,18 bara.

Total pressure increase from the water inflow: 1,7 bara (0,7 + 1) + 4 bara (40mVs) = 5,7 bara

From the measurements it is possible to see that the P_{max} for this calculation is wrong. The calculated pressure is P_{max} = 3.08 bara and the measured pressure is P_{max} = 5.7 bara. This indicates that the external water has been higher than expected considering the energy that the water represents. This type of calculation does not include the size of the opening/plug for the water to flow into. This does of course have an impact on the amount of water that the inflowing water has. If the opening is big and the volume in the air pocket is small, the air pocket will not have the same effect as for an opening that fits the air pocket better. If the external water is measured from the top of the plug or underneath the plug will also have a lot to say for the results. Measuring above will give a lower external pressure, and opposite for the underneath.

For the gas development the amount of 50% is an experienced number from Orica, that did fit nicely for this project.

G. EXAMPLE OF APPLICATION FOR EXPLOSIVES GAS PRESSURE CALCULATION - LYNGSVATN

The basis of the example is the piercing round at Lyngsvatn. According to the example at (tunnelling toward the plug) the amount of explosives was set to the total amount of 556 kg.

The calculation for gas pressure is the same for both open and closed system, with water-filling.

Pressure of air in the air pocket	P_1	Measured, possible to regulate	2.2 bar
Volume of air pocket	V_1	Calculated and measured	450 m^3
Kilo explosives	G	Pre-decided	556 kg
Volume of gas development	V_g	556kg • 0.8	444.8 m^3
Atmospheric pressure	P_a	Standard	1 bar
Adiabatic exponent for dry air	κ	Standard	1.4
	β	V_g/V_1	

Total pressure in the air pocket after the blast:

$$P_2 = \left(\beta + P_1^{\frac{1}{\kappa}}\right)^{\kappa} = \left(\frac{444.8m^3}{450m^3} + 2.2\ bar^{\frac{1}{1,4}}\right)^{1,4} = 3.113\ bar$$

The amount of gas development:

$$3.113\ bar - 2.2\ bar = 0,913\ bar$$

The dimensioning pressure for this gate, was given from the producer, and set to 6.8 bar.

H. BLASTING WITH SHORT ROUNDS

Towards the tunnel piercing area, the advance per round is reduced. An estimated time consumption per round cycle and weekly advance rate are made in this appendix. For the estimation it is used the reports from 2B-05 Tunnelling and 2C-05 Tunnelling, when the round is reduced with 1 m.

Round Cycle Time

Estimation of a round cycle time for a tunnel is based on the following data:

Cross section	$30m^2$
Rock blastability	Medium
Rock drillability	DRI = 49 (medium)
Rock drills	
Drillhole diameter	48 mm
Large drillholes	102 mm

For the Project Report 2B-05, the round cycle is divided into four major operations:

1 Drilling, charging, blasting
2 Ventilation
3 Loading and hauling
4 Scaling

Time consumption per round cycle and weekly advance rate:

	Parameter	Unit						
Drilled round length	l_b	m	5	4	3	2,1	1,6	1
Tunnel cross section	A_s	m^2	30					
Skill level			High					
Blastability	SPR		Medium					
Drillhole diameter	d_h	mm	48					
Number of drillholes basic round length	N_b		59					
Driled length	l_h	cm	500	400	300	210	160	100
Correction for drilled length	k_{bl}		1	0,95	0,93	0,91	0,89	0,87
Number of holes exclusive large drillholes	N_h		65	56	55	54	53	51
Diameter large drillholes	d_g	mm	102					
Number of large drillholes	N_g		3	2	2	1	0	0
Type of drilling hammers			AC-COP 1838					
Number of driling hammers	N_m		3					
Drillability	DRI		48					
Penetration rate 48 mm drillhole	v_b	cm/min	218					
Correction penetration rate for dh in relation to 48 mm	k_{hv}	**%**	100					
Penetration rate charged holes	v_h	cm/min	218					
Correction penetration rate for dg in relation to 48 mm	k_{gv}	%	44					
Penetration rate dg	v_g	cm/min	95,92					
Drilling time	T_h	min	45,1	36,1	27,1	18,9	14,4	9,0
Drilling time large drillholes	T_g	min	5,2	2,8	2,1	0,7	0,0	0,0
Time for moving per hole	t_f	min	0,75	0,72	0,69	0,68	0,68	0,67
Time for moving	T_f	min	17,8	14,4	13,4	12,6	11,8	11,4
Unit time for rod adding	t_{st}	min	0					
Time for rod adding	T_{st}	min	0					
Rock wear quality			High					
Bit changing factor	f_k		0,04					
Unit time for bit changing	t_k	min	3					
Time for bit changing	T_k	min	14,2	9,6	0,4	4,7	3,4	2,1
Lack of simultaneousness	f_{sa}		0,11					
Extra time for lack of simultaneousness	T_{sa}	min	7,49	5,86	4,69	3,55	2,89	2,24
Necessary drilling time	T_b	min	89,76	68,75	47,72	40,53	32,50	24,70

Time consumption per round cycle and weekly advance rate:

	Parameter	Unit						
Drilled round length	l_h	m	5	4	3	2,1	1,6	1
Necessary drilling time	T_b	min	89,8	68,7	47,7	40,5	32,5	24,7
Number of charging lines			3					
Time-determining charging time for basic round length	T_{lb}	min	38	35	33	32	31	30
Correction for drillhole length	k_{ll}		1	0,91	0,82	0,73	0,64	0,55
Time-determinant charging time	T_l		38	32	27	23	20	17
Rig time drilling, charging, blasting	T_{rb}	min	13					
Incidental lost time drilling, charging, blasting	T_{tb}	min	16	13	10	9	7	6
Sum for drilling, charging, blasting		**min**	**156**	**129**	**103**	**94**	**84**	**74**
Ventilation break		**min**	**12**					
Type of loader			Cat 980G					
Transport equipment			Loading truck					
Normalised gross loading capacity	Q_l	asm³/h	78					
Factor of overbreak	f_o		1,23					
Advance per round	p_r	%	90					
Actual volume per round	v_r	asm³	166	133	100	70	53	33
Loading time per round	T_{lt}	min	128	102	77	54	41	26
Rig time loading and hauling	T_{rl}	min	12,5					
Incidental lost time loading and hauling	T_{tl}	min	16	13	10	7	6	4
Sum loading and hauling		**min**	**156**	**127**	**99**	**73**	**59**	**30**
Scaling time, use of scaling jumbo	$\mathbf{T_r}$	**min**	**35**					
Correction for drilled length	k_{lr}		1	0,92	0,88	0,86	0,85	0,845
Scaling time		min	35	32,2	30,8	30,1	29,75	29,575
Net round cycle time	T_{nr}	**min**	**359**	**301**	**245**	**210**	**185**	**145**
Extra time for niches	T_n	min	0					
Tunnel length	L	m	3000					
Correction for tunnel length an job training effect	k_{le}		1					
Standard round cycle time	$\mathbf{T_{sr}}$	**min**	**359**	**301**	**245**	**210**	**185**	**145**
Effective working time per week	hw	hr	101					
Weekly advance rate		**m/week**	**76**	**72**	**67**	**55**	**47**	**38**

Excavation cost

The Project Report 2C-05 divides the tunnel costs into the following categories:

I. Drilling, charging, blasting and scaling
II. Loading
III. Hauling
IV. Additional work

The price level in this section is January 2005. The estimated cost index from January 2005 to January 2018 is 1.335966, based on Kompendium i anleggsteknikk, Kostnadsregning anleggsmaskiner. Chapter IV.1. It is used 2/3 of the index because the time between the index is more than 5-10 years. The Index is corrected for efficiency increase and productivity increase in this period.

The corrections are corrected with extrapolation to get an estimated value.

Total standard cost:

	Parameter	Unit						
Drilled round length		m	5	4	3	2,1	1,6	1
Tunnel cross section	A_s	m^2	30					
Tunnel length		km	3					
Excavation method			Trackless tunnelling					
Drillability	DRI		49					
Blastability	SPR		Medium					
Adit: Horizontal	k_a		1,01					
Drillhole diameter	dh	mm	48					
Total costs drilling, charging, blasting…	c_d	NOK/m	1470					
Explosives costs	c_{eb}	NOK/m	720					
Correction for drilled length	k_{le}		1,00	1,04	1,15	1,20	1,25	1,30
Proportion of dynamite	ANFO	%	5					
Correction for dynamite proportion	k_{de}		1					
Corrected explosives costs	c_e	NOK/m	720	745	828	864	900	936
Scaling costs	c_{sb}	NOK/m	155					
Correction for drilled length	k_{ls}	NOK/m	1	1,18	1,41	2,39	4,75	5,68
Correted scaling costs	c_s		**155**	**183**	**219**	**370**	**736**	**880**
Sum of drilling, explosives and scaling costs	c_{dt}	**NOK/m**	2345	2372	2408	2560	2926	3070
Loading equipment			Cat 980G - Truck, Load&haul, C7C 110m					
Loading cost	c_l	**NOK/m**	**500**					
Correction for tunnel length and job training effect	k_{le}		1					
Corrected loading costs	c_{h0}	NOK/m	500					
Type of tranport			Trackless tunnelling					
Hauling costs	c_h	NOK/m	1000					
Costs for road pavement	c_r	NOK/m	182					
Tip cost	c_{tip}	NOK/m	92					
Total hauling costs	c_{ht}	**NOK/m**	**1274**					
Ventilation	c_v	NOK/m	424					
Electrical installations	c_{el}	NOK/m	119					
Water supply	c_w	NOK/m	133					
Miscellaneous costs	c_m	NOK/m	143					
Additional costs	c_a	**NOK/m**	**819**					
Labour	c_{lab}	NOK/m	2333					
Correction for drilled length	k_{la}		1	1,06	1,2	1,34	1,48	1,62

Correction for tunnel length and job training effect	k_{La}		1					
Corrected labour costs	c_{la}	**NOK/m**	**2333**	**2473**	**2800**	**3126**	**3453**	**3779**
Cost of niches	c_n	NOK/m	150					
Sum elemental costs	c_t	**NOK/m**	**7421**	**7588**	**7951**	**8429**	**9122**	**9592**
Correction for unforeseen costs	k_u		1,1					
Correction for price level	k_p		1,335966					
TOTAL STANDARD COST excl. Rock support	c_s	**NOK/m**	**10905**	**11152**	**11684**	**12387**	**13405**	**14097**