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## A study of EVA-based sprayed membranes for waterproofing of rail and road tunnels in hard rock and cold climate

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Norwegian University of Science and Technology

Doctoral Thesis

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Thesis for the degree of Philosophiae Doctor

Trondheim, December 2015

Norwegian University of Science and Technology
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SUMMARY

The scope of the presented work has been to investigate the function and properties of a single-shell waterproof sprayed concrete lining (SCL) system, and evaluate the possible use of this system as a permanent solution for modern rail and road tunnels. The context which has been investigated is hard rock environment, exposure to groundwater under low to moderate hydrostatic pressures and freeze-thaw cycles as in seasonal Nordic climate. This study covers membrane materials based on Ethyl-Vinyl-Acetate co-polymers (EVA) in sprayable form embedded in fiber reinforced sprayed concrete constructed according to Norwegian practice for permanent rock support. This research work was started September 2011 and the last investigations were concluded in June 2015.

The work has been organized as a conceptual study by firstly identifying the main functional and exposure related issues for an SCL structure, followed by field and laboratory investigations.

SCL waterproofed with a sprayed membrane represents a continuously bonded structure from the rock surface to the lining surface. In its basic form there are no draining measures in the lining, although local or selective drainage can be included with several methods if found beneficial. The lining design which is covered in this study is a tunnel with lined walls and crown and a drained and unlined invert.

This lining design implies a conceptual model for the waterproof SCL system with the following main elements:

- Water flow into the tunnel through the drained invert
- Water migration in the rock mass and water exposure of the SCL through rock joints
- Occurrence of local water pressure at the interface between the rock mass and the lining
- The tunnel space with seasonal variations of temperature and relative air humidity
- Possible water transport through the lining structure by capillary and vapor transport mechanisms
- The lining structure with the constituent materials, moisture absorption properties and moisture condition under the given exposure from the ground water and tunnel air
- Mechanical loads on the membrane through the concrete layers on either side of the membrane

The establishment of test sites included the construction of two full scale SCL sections, instrumentation of a recently constructed rail tunnel and the construction of a lining structure for realistic in-situ cured material samples in a road tunnel. Finally, valuable results from the monitoring of an SCL test section which was constructed May-June 2014 at the Forsmark underground nuclear waste storage could be included in this study. The investigations which were carried out at the test sites include the following:

- Water pressure testing for hydraulic transmissivities of the rock mass
- Continuous in-situ monitoring of ground water pressure in the rock mass in the immediate vicinity of the SCL structure
- Sampling of tunnel linings with core drilling for investigations of the in-situ moisture content in the sprayed concrete and membrane materials

- Thermal monitoring in the rock mass, the SCL structure and the climate on the tunnel air
- Testing of in-situ tensile bonding strength of the lining at intervals after construction

A large scale SCL-structure with sprayed membrane was constructed on a granitic rock mass in a specially designed freezing laboratory. Realistic moisture conditioning of the lining structure was achieved and several types of freezing loads were simulated physically in order to conduct accelerated freeze-thaw exposure and subsequent testing of the lining structure.

The complete findings from this study are compiled to verify the main conceptual model. The main findings can be summarized as follows:

- The investigated type of SCL, represents a waterproof and vapor permeable structure which allows a certain migration of water through the lining in the form of capillary and vapor diffusion transport mechanisms
- The direct exposure of the lining structure to the ground water and exposure to the tunnel climate represent critically important boundaries for the condition of the lining and the properties of the constituent materials
- The moisture properties of the tested sprayed concrete and membrane and the exposure to groundwater from the rock mass and climatic conditions in the tunnel air, result in a characteristic moisture content profile for the concrete across the lining, with decreasing water content from the rock surface towards the lining surface
- Under the found moisture condition range of the SCL structure, the membrane exhibits high tensile bond strength, in the range of 1.1 – 1.5 MPa measured in-situ in linings.
- Accelerated cyclic freeze-thaw exposure of a large scale lining structure with -3ºC minimum temperature at the membrane location during each freezing cycle showed no detrimental reduction of tensile strength after 35 freeze/thaw cycles
- The required resistance to rupture of a bonded membrane, is related to the maximum expected movement (opening) over cracks in the concrete at the interface to the membrane. The crack bridging capacity was found to be temperature sensitive with significantly lower performance at 0ºC and lower compared to standard testing conditions at 20ºC. With membrane thickness in the range of 3-4 mm, the tested membranes exhibited sufficient elasticity to bridge crack openings of approximately 1 mm at temperatures down to – 3ºC
- Results from testing of the sprayed concrete material for freeze-thaw resistance according to commonly used test methods would theoretically disqualify the sprayed concrete for use under freezing conditions. An assessment of the realistic freezing exposure and the moisture contents in the SCL structure, together with results from a specially developed functional freeze-thaw performance test for tunnel linings suggest that sprayed concrete will not suffer any frost damage in a tunnel lining, even under severely cold climate
- Under the tested conditions, comprising low permeable jointed rock masses with typical Scandinavian hard rock characteristics, low hydraulic conductivities of the rock mass (range: $10^{-8}$ to $10^{-9}$ m/s) and groundwater pressures in the range of 680 – 780 kPa, a lining design with a waterproof and undrained lining in the walls with a drained invert will result in a certain water pressure in the rock mass in the immediate vicinity of the lining, which is significantly lower than the full hydrostatic pressure
- Under the tested conditions, no effects on the ground water pressure in the rock mass in the immediate vicinity of the lining, caused by the application of the membrane could be measured, when a primary lining of fiber reinforced sprayed concrete had already been applied

- Effects of the excavation damaged zone and possibly unsaturated conditions in the rock mass in the immediate vicinity of the lining are likely explanations for the observed water pressure drop near the lining
- Destabilizing effects in the rock mass closely surrounding the undrained lining under the tested conditions which would require an increased rock support level are very unlikely

The findings from this study indicate that the type of SCL which has been investigated can be used as a permanent lining solution for road and rail tunnels in hard rock under the following conditions:

- Low to moderate ground water pressures up to approximately 700 kPa in the host rock mass
- Fiber reinforced sprayed concrete material with certain material and surface properties
- No ground induced loads from the primary (rock support) lining on the membrane (stable rock support lining)
- Thermal exposure with a minimum temperature of -3ºC at the membrane location
- Thicknesses of concrete layers which provide a favorable moisture exposure of the membrane
- Membrane thickness of minimum 3 mm

Further work is recommended to:

- Investigate the long term effects of exposure of the SCL lining to higher ground water pressure, particularly the exposure of the membrane to pressurized water at cracks
- Investigate the effects of slow deformations and slow thermal fluctuations in order to complement the finding from short-term accelerated testing the mechanical performance parameters
- Investigate the longitudinal thermal profile of selected tunnels, combined with thermal monitoring in the lining and rock mass in areas with severe freezing for precise recommendation of areas of use for this lining system
- Improve constructability details for the application process of the membrane
- Develop improved sprayable membrane materials which are less sensitive to exposure to liquid water
- Develop specifications for excavation, support and pre-grouting of rock tunnels which increase the feasibility of the waterproof SCL system. This includes careful contour blasting, required watertightness achieved by pre-grouting and sprayed concrete mix designs and application in order to produce a suitable substrate

ACKNOWLEDGEMENTS

It would not have been possible to conduct this work without the participation and substantial help from a number of organizations and persons. I am sincerely thankful for the help, contributions and active participation from a large number of people over the 4 years this work has lasted.

I am particularly thankful to my employer BASF Construction Chemicals Europe AG and my immediate superior Arvid Dokken who firstly gave me this opportunity to do research apart from the daily work, and very essentially, as the task proved to be much more complicated and time consuming than expected, allowing me to continue and complete this work. I would particularly like to thank my colleagues from BASF Mr Kristian Storhamar, Mr Arne Gravli and Mr Jan Ivar Grinden who on numerous occasions participated in strenuous tunnel field work at all times of the day and putting up with my continuous requests to conduct the work without compromising scientific precision. In this context I would also like to thank Mr Torgeir Jensen of SINTEF Marintek who has carried out all instrumentation work and configured sampling and logging of data in field and laboratory and patiently persisted long hours, day and night in tunnels and the laboratory. I am very grateful for the strong support and collaboration from Wacker Chemie AG, particularly Mrs Christine Köster and Mr Klaus Bonin by making expertise within sprayed membrane technology available, executing a significant portion of the membrane testing and at many occasions providing immediate and substantial help through valuable discussions, clarifications and execution of additionally required tests on short notice.

The participation of the Norwegian National Rail Administration and Norwegian Public Roads Administration in the form of funding of field and laboratory investigations, as well as making the different tunnel sites available and covering the additional construction costs of the SCL sections has been decisive for the feasibility of this research. The kind flexibility and readiness at the projects E6-Dovre rail line joint project, E39 Karmsundtunnelen (T-connection), E39 Harangstunnelen and the operational crew responsible for the Gevingås rail tunnel, to assist whenever needed is greatly acknowledged. The amount and quality of the findings were enabled through the active and skilled participation of these two parties at these site locations as well as the central administrations. I would particularly like to thank Dr Mona Lindström, Mr Alf Kveen, Dr Claus Larsen, Dr Øyvind Bjøntegaard, Mr Reidar Kompen and Mrs Eva Rodum of the Norwegian Public Roads Administration, Dr Jan Magnus Østvik, Mr Brede Nermoen, Dr Ragnhild Wahl and Mrs Trine Bye Sagen of the Norwegian National Rail Administration for the numerous and very useful conversations and discussions along the way. For me this has been a highly motivating factor throughout the project.

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During the process of understanding a problem, planning, conducting the initial investigations, re-thinking ideas of how to proceed, changing of plans, adding of plans and finally evaluating the findings I have had help from a few people who contributed with very clarifying conversations and apparently simple but brilliant ideas given in the right moment which later proved to be of critical importance. The value of this cannot be overestimated. In this context I would like to thank Mr Håkon Skistad for the substantial help within thermodynamics and heat conduction assessments, Mr Anders Beitnes for creative ideas in how to conduct freezing testing and possible design options, Mr Knut Garshol for discussions on lining function and rock mass behavior and Dr Peter Schubert for providing clear and objective advice. The three last mentioned are also acknowledged for having critically reviewed several of my very long article manuscripts.

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Trondheim, 7. October 2015

Karl Gunnar Holter

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PAPERS

Paper 1: Holter KG, Nermoen B, Buvik H, Nilsen B (2013) Future trends for tunnel lining design for modern rail and road tunnels in hard rock and cold climate. In proceedings of the ITA/AITES World Tunnel Congress, Geneva, 2013

Paper 2: Holter KG (2014) Loads on sprayed waterproof tunnel linings in jointed hard rock; A study based on Norwegian cases. Rock Mechanics and Rock Engineering, Vol 47/3, DOI: 10.1007/s0603-013-0498-0

Paper 3: Holter KG, Geving S (2015) Moisture transport through sprayed concrete tunnel linings. Rock Mechanics and Rock Engineering, published online 11. March 2015, DOI: 10.1007/s00603-015-0730-1

Paper 4: Holter KG, Smeplass S, Jacobsen S (2015) Freeze-thaw resistance of sprayed concrete in tunnel linings. Materials and Structures, published online 24. August 2015, DOI: 10.1617/s111527-015-0705-4

Paper 5: Holter KG (2015) Performance of EVA-based membranes for SCL in hard rock. Rock Mechanics and Rock Engineering, published online 24. September 2015, DOI: 10.1007/s00603-015-0844-5

Paper 6: Holter KG, Christiansson R, Basnet C B (2015) Effects on ground water pressure in the immediate rock mass around partially drained SCL with bonded waterproof membrane. Manuscript submitted to Bulletin of Engineering Geology and the Environment, 16.October 2015

ABBREVIATIONS FREQUENTLY USED IN THIS THESIS

| | |
|---|---|
| COV | Coefficient Of Variation, ratio of standard deviation to mean value |
| DCS | Degree of Capillary Saturation [%]. Degree of saturation of concrete with respect to total suction porosity, equal to ratio of moisture content of a concrete specimen to the moisture content at full submerged saturation at atmospheric pressure at mass equilibrium |
| DS | Degree of Saturation [%]. Degree of saturation of concrete with respect to total porosity, equal to ratio of moisture content of a concrete specimen to the moisture content at full submerged pressure saturation at 50 bars at mass equilibrium. For a polymeric membrane material DS is the ratio of the water content of a specimen to the water content at immersion at atmospheric pressure at mass equilibrium |
| EDZ | Excavation Damaged Zone. Zone of a certain thickness in the rock mass near the excavated contour of an underground opening which in which hydromechanical and geochemical modifications induce significant changes in flow and transport properties. These changes may, for example, include one or more orders-of-magnitude increase in flow permeability |
| EVA | Ethyl-Vinyl-Acetate copolymer, which together with several mineral and cementitious components forms a sprayable membrane material for tunnel waterproofing |
| NPRA | Norwegian Public Roads Administration |
| NNRA | Norwegian National Rail Administration |
| RH | Relative air humidity [%] |
| SCL | Sprayed Concrete Lining. Permanent tunnel lining system based on fiber reinforced sprayed concrete as the structural material with alternative possible waterproofing measures which are integrated into the sprayed concrete structure. Such linings may also include rock bolts for rock reinforcement |

# 1 INTRODUCTION

## 1.1 Background

Waterproofing of sprayed concrete tunnel linings (SCL) is a critical issue for the successful use of such linings in modern rail and road tunnels. Although state-of-the-art sprayed concrete material used for ground support is literally impermeable for conductive water flow, cracks and imperfections in the concrete leak when exposed to groundwater. A waterproofing measure is therefore required. Waterproofing of an SCL structure can basically be approached in the following manners:

- A spray-applied bonded membrane within the concrete structure which main function is to prevent water to flow into the tunnel through cracks and imperfections
- A sprayed concrete layer which is designed to eliminate the effect of leaking cracks. This has been attempted by increasing the ductility of the concrete to strongly reduce the formation of shrinkage cracks (Bonin et al. 2012) or by introducing an agent in the concrete material which produces a growth of crystals in the shrinkage cracks, and hence reduces or eliminates leaking water through cracks (McGrath 1998)
- Drainage measures in the form of half pipes, stripes of geotextile or membrane sheets which are installed locally at seepage points in order to drain water down to the invert and provide a waterproofing of the lining only where the water ingress occurs (Vereina rail tunnel, Switzerland; Röthlisberger 1994)

This research work is aimed at investigating the first of the above mentioned methods for the waterproofing of an SCL structure by the use of spray-applied membranes. There exist several categories of sprayed membranes for this purpose, among them membranes based on ethyl-vinyl-acetate (EVA) co-polymers, methacrylate based membranes and polyurea based membranes (Lemke 2013). Until recently the EVA-based sprayed membranes have been the mostly used for the waterproofing of SCL. In this work EVA-based membranes from the companies BASF Construction Chemicals and Orica International Ltd have been investigated. Both these companies had more than 10 years of experience with sprayed EVA-based membranes at the start of this research project.

Spray-applied waterproofing membranes, also referred to as liquid waterproofing membranes (Maidl 2013, Lemke 2013) has seen gradually increased use in some countries over the last decade. Initially waterproofing projects of limited size and importance such as cross passages, elevator shafts and escape tunnels were realized with this method. Hence, experience with this waterproofing technology was gained in a gradual manner. Suitable areas of use and the construction process has been gradually developed and improved. This process has also included construction of test sections under conditions where this system was not successful. When promising results under favorable conditions had been demonstrated, several projects with significant importance and modern requirements for service lifetime were realized with spray-applied membranes, such as Lausanne Métro L2, Tunel de Viret, Switzerland (Bridge et al 2010), the Hindhead road tunnel, UK (Holter et al 2010), Crossrail, UK (Pickett 2013), Prague Metro Line 2, Veleslavín station, Czech Republic (Hasík et al 2015) and the Gevingås rail tunnel, Norway (Nermoen et al. 2011).

The traditional lining systems with cast-in-place concrete waterproofed with sheet membrane and the drainage and thermal insulation shield systems both represent drained lining systems. The use of a bonded waterproof membrane without any drainage measures fundamentally changes the waterproofing function of the tunnel lining compared to the traditional drained lining systems. In hard rock environment, traffic tunnels are normally constructed without waterproofing in the invert. Water ingress is controlled by reducing the hydraulic conductivity of the rock mass surrounding the tunnel by pre-injection. Hence, the seepage into the tunnel is reduced to a certain required level. This facilitates the use of a drained tunnel lining approach also at locations below groundwater level.

Several issues with the SCL and sprayed membrane lining system needed be documented in order to establish this method for permanent tunnel linings. The most important issues are:

- Establish the precise waterproofing function of the bonded membrane
- Determine type and magnitude of loads which act on the membrane
- Determine the effects on possible water saturation of the concrete material on the rock side of the membrane
- Determine the effects on rock joint water pressures and possibly resulting reduced stability of the rock mass in the immediate vicinity of the lining when using an undrained waterproof lining in the walls and crown of the tunnel below the ground water table
- Specifically in cold climate, determine the effects of cyclic freezing and thawing when using this lining system without any thermally insulating measures
- Suggest testing methods with acceptance criteria which address realistic loads, moisture and thermal exposure
- Determine possible degrading mechanisms based on the different loading scenarios, water and thermal exposure and subsequently assess durability

The objectives of this research project have been established in order to answer these questions.

## 1.2 Research objectives

The main objective of this research is to establish the properties of SCL waterproofed with EVA-based membranes for tunnels in hard rock and cold climate with typical Scandinavian lining designs. The suitability of this lining method for rail and road tunnels will be evaluated based on modern requirements for service lifetime and waterproofing function. Three secondary objectives have been established.

Secondary objective 1: Loading and exposure conditions of the lining

- Establish mechanical loading conditions for the lining and the waterproofing membrane and define realistic performance criteria for the membrane
- Determine waterproofing function and the resulting moisture transport processes and moisture exposure to the lining
- Determine realistic thermal exposure

Secondary objective 2: Mechanical performance of the membrane

- Determine mechanical behavior of membrane under realistic loading modes and climatic exposure
- Conduct accelerated tests with freeze-thaw exposure followed by mechanical tests

Secondary objective 3: Assessment of effects on ground water pressure of partially drained SCL

- Investigate the effect of the undrained property of the SCL structure on water pressure in the immediate rock mass around the lining
- Evaluate if such effects on water pressure causes any reduced stability and a resulting need for increased level of rock support
- Evaluate the effects of the ground water on the saturation, condition and properties of the lining structure

## 1.3 Scope of work

### 1.3.1 Lining design and constituent materials

The tunnel lining design considered in this work is based on the Scandinavian permanent rock support philosophy which uses fiber reinforced sprayed concrete applied with the wet-mix method and rock bolts as a permanent rock support lining (NGI 2013, NCA 2011, STA 2014). The sprayed concrete material which has been used for all test sites and sample preparation has been produced as part of ongoing regular concrete production, subject to realistic quality requirements at tunnel sites under construction. The tested membrane materials are of the EVA co-polymer category. Altogether 5 sprayed membrane products from the two participating supplier companies have been included. The construction of all lining sections at the test sites included an SCL design with sprayed concrete rock support lining, sprayed membrane and a covering layer of sprayed concrete.

### 1.3.2 Construction of SCL sections, testing and age of materials

In the very beginning of this project lining sections for in-situ testing as well as material testing with samples having sufficient age had to be constructed. Two full scale tunnel lining sections (the Karmsund and Ulvin test sites) were constructed in connection with ongoing tunnel construction. Furthermore the large scale lining structure in the freezing laboratory at SINTEF was re-established with new sprayed concrete and membrane, as well as detailed thermal instrumentation of the lining structure. In addition, the portion of the Gevingås rail tunnel with waterproof SCL, constructed 2010-2011, has been used as a test site. Hence, within the time frame of this research project the tested in-situ condition of tunnel linings covered the range from 5 to 37 months age. The material testing of sprayed concrete was deliberately conducted on hardened concrete with age in the range of 300 to 450 days. Testing of the sprayed membrane material was conducted on samples with age ranging from 3 to 14 months.

### 1.3.3 Tested conditions

The engineering geological, hydrogeological and climatic conditions which could be included in this research project are the following:

- Ground conditions which cover hard crystalline rock types and rock masses ranging from massive to densely jointed
- In-situ rock stress conditions with a high horizontal to subhorizontal major principal stress
- Hydrogeological conditions including saturated rock mass locations between 50 m and 122 m below sea level.

- Hydraulic conductivities of rock masses in the range of $10^{-6}$ m/s to $10^{-8}$ m/s
- The measured maximum background water pressures in the rock mass at the invert elevation were in the range 680 – 780 kPa
- The thermal exposure of tunnel linings considered in this project covers Nordic seasonal variations in temperature and relative humidity, including severe winter climate

### 1.3.4 Limitations of scope

Several circumstances limited the conditions which could be included in this work. The tested site conditions for the moisture content and in-situ tensile bond strength covered environment with low ground water pressures, unlikely to be higher than 200 kPa at the lining surface. Hence effects caused by high water pressures at the rock/lining interface could not be investigated.

The concrete material included in this project is limited to the wet-mix fiber reinforced sprayed concrete. For regulating layers for the membrane substrate special mortars, special concrete mix designs or dry mix shotcrete regulating layers were not tested.

Regarding constructability of the membrane-concrete structure, the application and construction details for the membrane and lining structure followed the recommendations and practice provided by each of the two suppliers. Research intended to develop or improve the application methods was not part of this research.

The tested products were proposed from the two suppliers, as well as the methodology for the application of the membrane. No work was undertaken to develop new or modify existing products as part of this research.

## 1.4 Structure of this thesis

This thesis is presented as a compilation of 4 published scientific papers, one prepared manuscript draft for a scientific paper and one published conference paper, preceded by a summarizing part. The summarizing part contains the conceptualization of this study, an overview of the executed work and an extract and compilation of the main findings presented in the papers. A synthesis and discussion is presented following the main findings. Finally further work is suggested in view of the obtained results and limitations of the conducted work in this study. The summary part of the thesis is written in a manner so that the reader will get the complete overview of the work. The detailed analyses are presented in the papers. Each of the scientific papers addresses a fundamental issue in the analysis of the function, properties of the waterproof SCL system. The conference paper addresses the process of comparing different tunnel lining methods, including the waterproof SCL, in view of modern requirements and analysis tools for the total cost-effectiveness of rail and road tunnels.

Following the definition of the main conceptual model secondary conceptual models are presented to address a fundamental part of the SCL structure or a process of fundamental importance. Each of the papers contains elements from several of the secondary conceptual models. An overview of the papers is shown in Table 1.

**Table 1.** Overview of papers submitted in this thesis

| Paper no | Authors | Type of paper, reference | Title |
|---|---|---|---|
| 1 | K G Holter, B Nermoen, H Buvik, B Nilsen | Conference paper, ITA/AITES World Tunnel Congress 2013, Geneva | Future trends for tunnel lining design for modern rail and road tunnels in hard rock and cold climate |
| 2 | K G Holter | Scientific journal paper; published in Rock Mechanics and Rock Engineering, 2014, DOI 10.1007/s00603-013-0498-0 | Loads on sprayed waterproof tunnel linings in jointed hard rock; A study based on Norwegian cases |
| 3 | K G Holter, S Geving | Scientific journal paper; published in Rock Mechanics and Rock Engineering, 2015, DOI 10.1007/s00603-015-0730-1 | Moisture transport through sprayed concrete tunnel linings |
| 4 | K G Holter, S Smeplass, S Jacobsen | Scientific journal paper; published in Materials and Structures, 2015, DOI 10.1617/s11527-015-0705-4 | Freeze-thaw resistance of sprayed concrete in tunnel linings |
| 5 | K G Holter | Scientific journal paper; published in Rock Mechanics and Rock Engineering, 2015, DOI 10.1007/s00603-015-0844-5 | Performance of EVA-based membranes for SCL in hard rock |
| 6 | K G Holter, R Christiansson, C B Basnet | Manuscript prepared for scientific journal paper in Bulletin of Engineering Geology and the Environment | Effects on ground water pressure of partially drained waterproof SCL in hard rock |

### 1.4.1 Published papers, division of work, structure and content

Paper 1: Future trends for tunnel linings and hard rock and cold climate. Authors: Holter KG, Buvik H, Nermoen B and Nilsen B. In: Proceedings of the World tunnel Congress, Geneva, 2013

The paper is written by Holter. The aim and scope of the paper was defined together with Nermoen and Buvik. Nermoen, Buvik and Nilsen provided critical review of the paper.

The main aim of paper 1 is to place the waterproof SCL in the context of the specification and design of tunnel linings subject to modern requirements. Such requirements relate to the total cost-effectiveness expressed as Life Cycle Cost (LCC) with the required service lifetime, reliability, accessability (minimum downtime) and maintainability of the tunnel lining. The structure and analysis approach of paper 1 is shown in Figure 1.

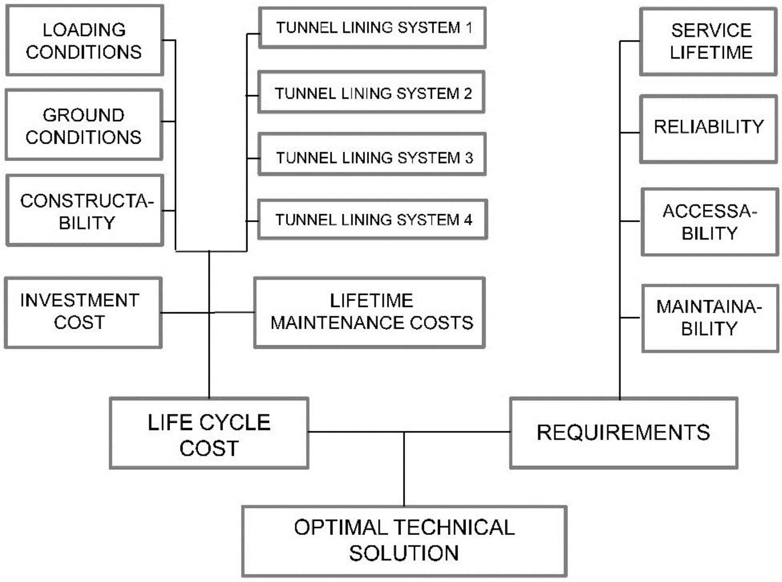



**Figure 1.** Approach of the analysis and content of paper 1.

<u>Paper 2:</u> Loads in sprayed waterproof tunnel linings. A study based on Norwegian cases. Published in: Rock Mechanics and Rock Engineering, Vol 47: 1003 – 1020, Author: Holter KG

The paper is written by Holter. The aim and scope was defined by Holter.

The main aim of paper 2 is to assess which loads which be expected to expose the lining under normal hard rock conditions. The structure and analysis approach of paper 2 is shown in Figure 2.

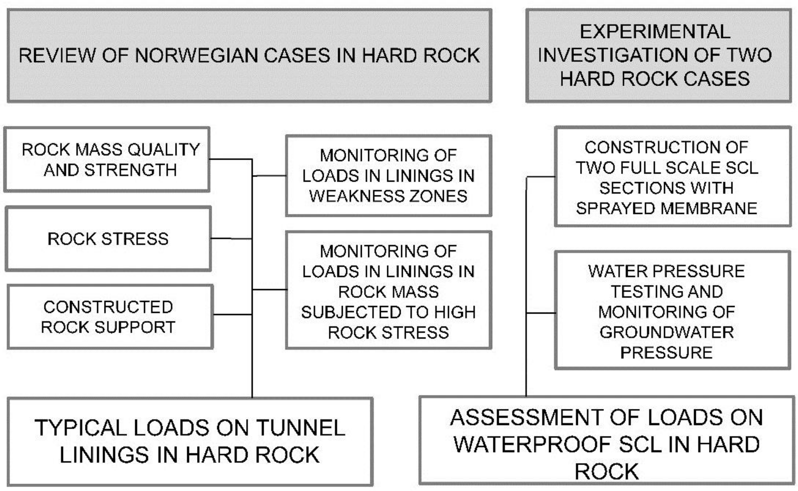



**Figure 2.** Approach of the analysis and content of paper 2.

Paper 3: Moisture transport through sprayed concrete tunnel linings. Authors: Holter KG, Geving S. Published online 11. March 2015 in: Rock Mechanics and Rock Engineering, DOI 10.1007/s00603-015-0730-1

The paper is mainly written by Holter. Holter defined the aim and scope of the paper as well as planned and executed all investigations. Some of the material testing was assigned to the concrete laboratories at SINTEF (vapor conductivity) and NPRA (desorption testing). Geving has written the section on moisture transport theory and conducted the numerical simulations for moisture transport in the software WUFI, as well as critically reviewing the manuscript. The structure and analysis approach of paper 3 is shown in Figure 3.

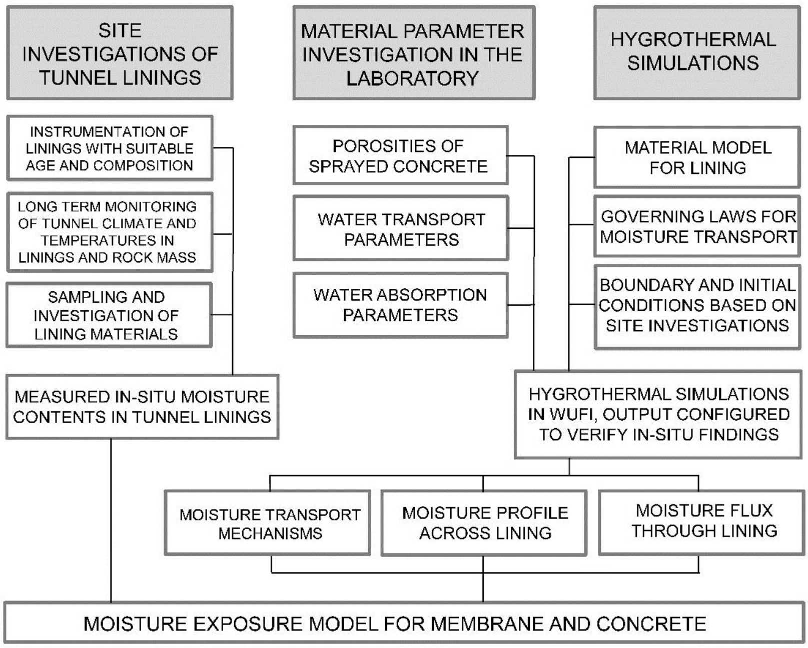



**Figure 3.** Approach of the analysis and content of paper 3

Paper 4: Freeze-thaw resistance of sprayed concrete in tunnel linings. Authors: Holter KG, Smeplass S, Jacobsen S. Published online 27. August 2015 in: Materials and Structures, DOI 10.1617/s11527-015-0705-4

The paper is written by Holter. Smeplass and Jacobsen contributed substantially with defining the aim and scope, participated in the detailed planning of the freeze-thaw tests, discussion of findings and critical review of the manuscript. Holter planned and executed all in-situ material construction work in tunnels, sampling and conditioning of specimens and executed most of the freeze-thaw testing. Master student Mrs Tandberg executed parts of the tunnel lining freeze test and the salt frost scaling test. The latter was executed in Oslo at the laboratories of the NPRA. The structure and analysis approach of paper 4 is shown in Figure 4.

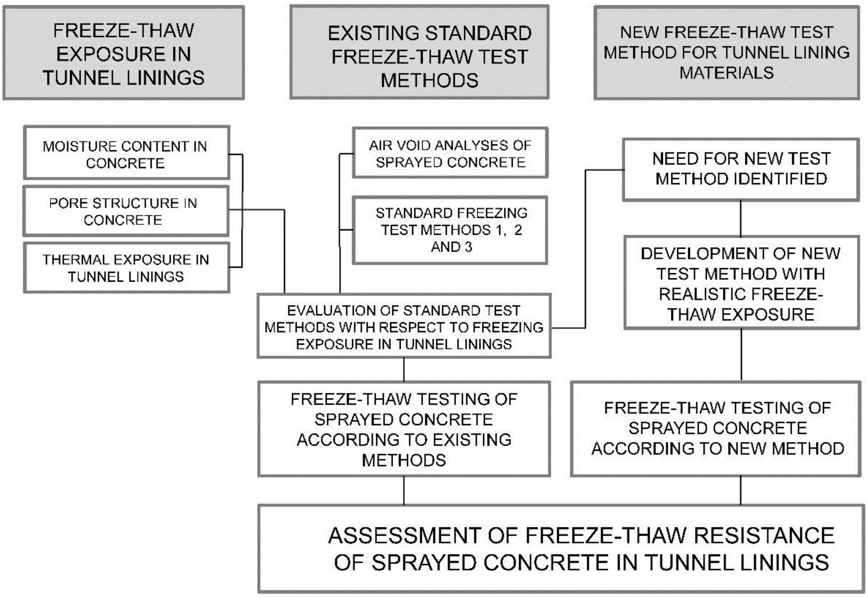



**Figure 4.** Approach of the analysis and content of paper 4

Paper 5: Performance of EVA-based membranes for SCL in hard rock. Author: Holter KG. Published online 24. September 2015 in: Rock Mechanics and Rock Engineering, DOI 10.1007/s00603-015-0844-5

The paper is written by Holter. Holter defined the aim and scope as well as planned, organized and executed the site construction work, site investigation program and most of the laboratory investigations. The students Mrs Langås and Mrs Tandberg contributed with laboratory work executing a substantial part of the testing of membrane materials at NTNU as part of their master degree work. Mr Bonin and Mrs Köster of Wacker Chemie AG contributed substantially with discussion of testing methods, testing conditions, membrane material properties as well as executing the SEM-analyses, the crack bridging testing and parts of the elongation tests. The structure and analysis approach of paper 5 is shown in Figure 5.

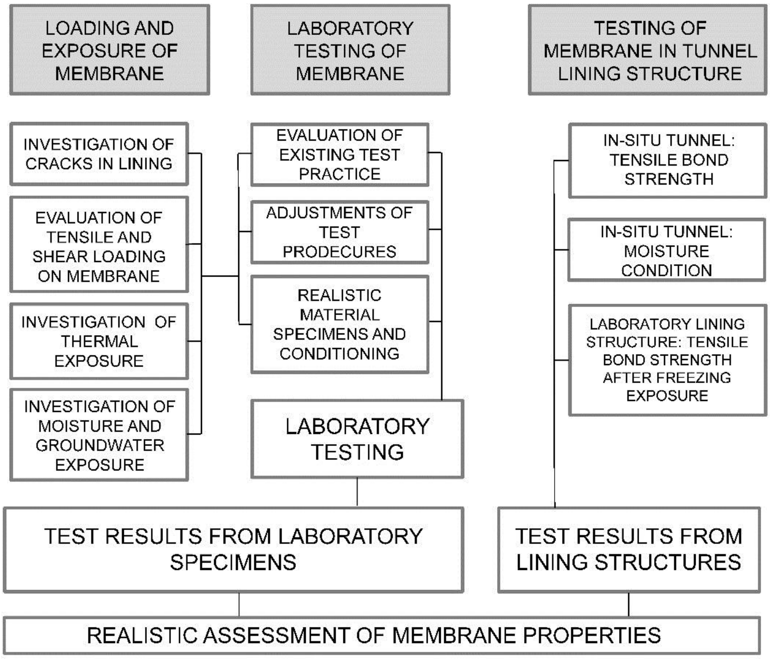



**Figure 5.** Approach of the analysis and content of paper 5

Paper 6: Effects on ground water pressure in the immediate rock mass of partially drained SCL with bonded waterproof membrane. Authors: Holter K G, Christiansson R, Basnet C B. Manuscript prepared for submittal to Bulletin of Engineering Geology and the Environment.

The paper is written by Holter. Holter defined the aim and scope as well as planned the in-situ water pressure testing and water pressure monitoring at the Karmsund and Ulvin sites. The numerical simulations in FLAC3D were conducted by Basnet. Karstein Monsen of Geoscan, Bergen, Norway conducted the simulations in UDEC BB. For the Forsmark test site Christiansson managed the construction and water pressure monitoring and Holter contributed with technical advice. The co-authors reviewed the manuscript critically. The structure and analysis approach of paper 6 is shown in Figure 6.

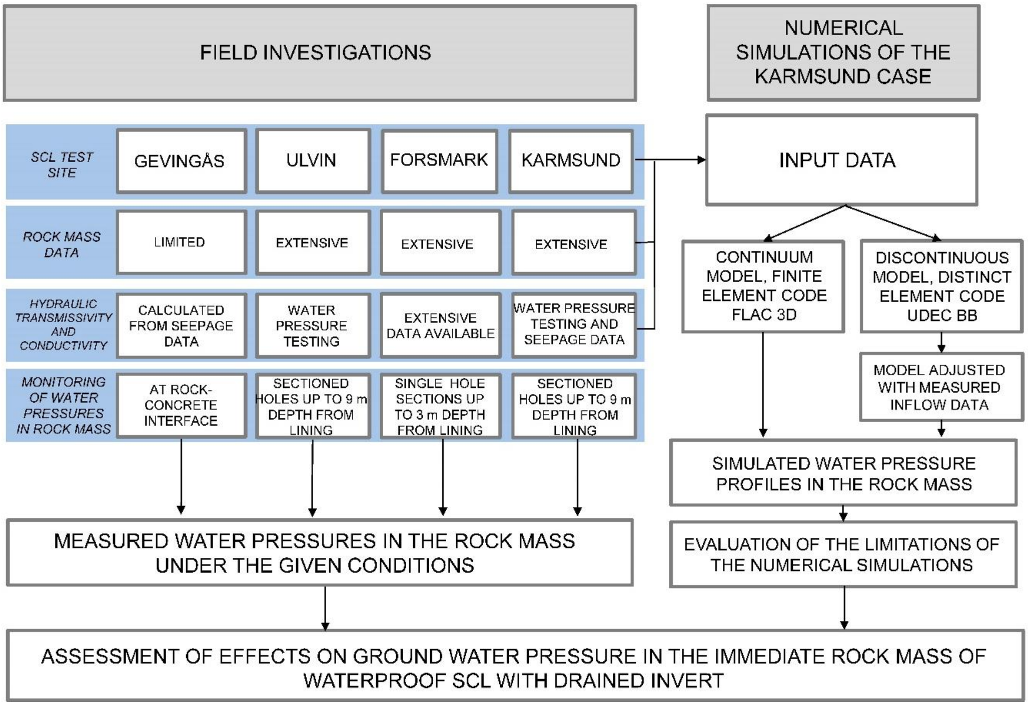



**Figure 6.** Approach of the analysis and content of paper 6

### 1.4.2 Presentation of work at conferences

During this program several conference papers were prepared and presented. This involved the compilation of preliminary findings and interpretation of these at the time of writing. This enabled several useful discussions and exchange of points of views. The conferences which were attended with a presentation during this project are shown in Table 2.

**Table 2.** Conferences where this PhD project has been presented

| Year | Conference | Topic/paper presented | Publication of paper |
|---|---|---|---|
| 2011 | NPRA, Annual Technology Conference (Teknologidagene), Trondheim, Norway | Waterproof permanent tunnel lining with sprayed concrete and sprayed membrane. Contents and layout of research project. | None |
| 2012 | Norwegian Tunnelling Conference, Oslo, Norway | Waterproof permanent tunnel lining with sprayed concrete and bonded membrane. Status for ongoing research project | In proceedings from the conference |
| 2013 | ITA/AITES World Tunnel Congress, Geneva, Switzerland | Future trends of tunnel lining design for rail and road tunnels in hard rock and cold climate | In proceedings from the conference, paper included in this thesis |
| 2013 | NPRA, Annual Technology Conference (Teknologidagene), Trondheim, Norway | Waterproof permanent tunnel lining with sprayed concrete and sprayed membrane: Function, properties and test methods | None |
| 2014 | ITA/AITES World Tunnel Congress, Iguassu Falls, Brazil | Testing of sprayed waterproofing membranes for single shell sprayed concrete tunnel linings in hard rock | In proceedings from the conference |
| 2014 | Concrete Innovation Conference, Oslo, Norway | Assessment of freezing and thawing damage in waterproof sprayed concrete tunnel linings | In proceedings from the conference |
| 2014 | International conference of wet-mix sprayed concrete, Sandefjord, Norway | Testing of sprayed water proofing membranes for single shell sprayed concrete linings in hard rock | In proceedings from the conference |
| 2015 | ITA/AITES World Tunnel Congress, Dubrovnik, Croatia | Testing of properties and constructability considerations of EVA-based sprayed membranes for waterproofing of tunnels | In proceedings from the conference |
| 2015 | NPRA, Annual Technology Conference (Teknologidagene), Trondheim, Norway | Findings from research project 2011-2015: Waterproof tunnel linings based sprayed concrete and sprayed concrete | None |

## 2 TECHNICAL BACKGROUND

### 2.1 Main requirements for service lifetime, serviceability and cost-effectiveness

Due to several recent cases with costly rehabilitation and maintenance of relatively new rail and road tunnels, both rail and road administrations in Norway are reconsidering the lining methodology for modern tunnels. A decision process for the approval of different tunnel lining systems for different tunnel categories which involves the use modern analysis tools such as LCC (Life Cycle Cost) and RAMS (Reliability, Accessability, Maintainability and Safety) is now the preferred approach. (NPRA 2012)

### 2.2 Currently used lining systems for rail and road tunnels in hard rock and cold climate

The currently used lining method for rail and road tunnels in hard rock in Norway consists of a rock reinforcement lining based on sprayed concrete and rock bolts and a separate inner drainage and thermal insulation shield system. The rock reinforcement lining is normally designed according to the Q-system (NGI 2013). The inner shield lining system is designed to provide drip protection in the traffic area, provide thermal insulation, resist mechanical loads as well as meeting the esthetic requirements (NNRA 2012, NPRA 2012). The two main systems currently in use are:

- Shield structure with polyethylene (PE) foam sheets with thickness normally 50 mm, covered with 60 mm fiber reinforced sprayed concrete
- Light concrete segment structure with sheet membrane
- A combination of the light concrete segments (for walls) and the PE-foam sheets (springline and crown)

Both systems are constructed as a suspended structure mounted on bolts in a regular grid pattern. Hence, with this lining method there will be an air gap between the rock support sprayed concrete surface and the inner lining which can vary between 0.3 to approximately 1 m. Both rail and road administrations in Norway require manual/visual inspections of the rock support surface at certain time intervals to be conducted behind the inner lining structure.

The drainage and thermal insulation lining system is illustrated in Figures 7 and 8.

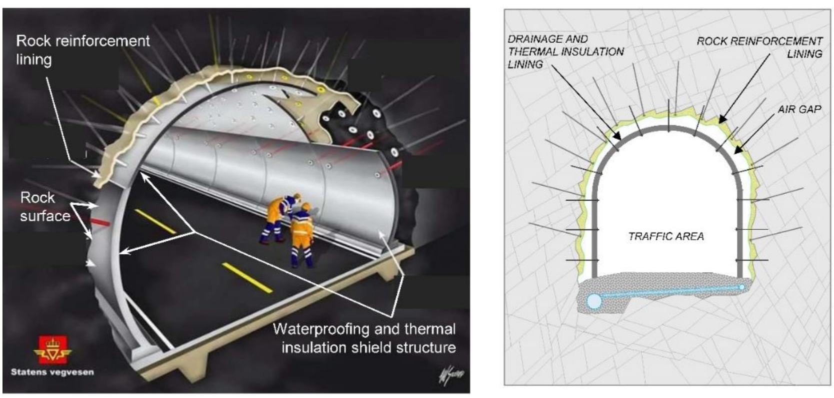



**Figure 7.** Conceptual drawings illustrating the drainage and frost insulation lining system traditionally used in Norwegian rail and road tunnels, Left: 3D image of lining system for a road tunnel (NPRA 2012). Right: vertical section for a rail tunnel

**Figure 8.** Photos of interior of complete drainage and thermal insulation lining system. Left: The Bærum rail tunnel (completed 2010) with PE-foam-sheets covered with sprayed concrete. Right: The E6 Eidsvoll motorway tunnel (completed 2010) with light concrete segment lining. (courtesy: Ådne Homleid, Byggeindustrien)

Analyses carried out by the NPRA for future modern tunnels (NPRA 2012) concluded that these lining systems could have an expected service lifetime of approximately 50 years. For modern high traffic tunnels, both the NNRA and NPRA (NPRA 2012, NNRA 2015) are therefore suggesting the use of the traditional central European lining system with cast-in-place concrete and sheet membrane waterproofing for a design service lifetime of 100 years.

## 2.3 Cast concrete lining system adopted for rail and road tunnels in hard rock

Cast concrete linings for rail and road tunnels are widely used in European countries outside Scandinavia. For the use in hard rock environment the study conducted by the NPRA "Modern Road Tunnels 2008-2011" suggests the use of the cast concrete lining system with sheet membrane as a pure waterproofing measure for tunnels with high traffic density (NPRA 2012). The permanent stability of the rock mass shall be granted by the rock reinforcement lining as the current practice (NGI 2013) suggests. Hence, a cast concrete lining would need to have only the minimum possible thickness to serve the purpose of keeping the sheet waterproofing membrane in place. Currently a lining structure with a minimum thickness of 250 mm of unreinforced cast-in-place concrete with sheet membrane waterproofing is proposed (NPRA 2012), illustrated in Figure 9. In Norway, this proposal has so far only been realized by the NNRA during the construction of the double track rail tunnels, shown in Figure 10 at the E6-Dovre Rail line joint project (Havik 2012).

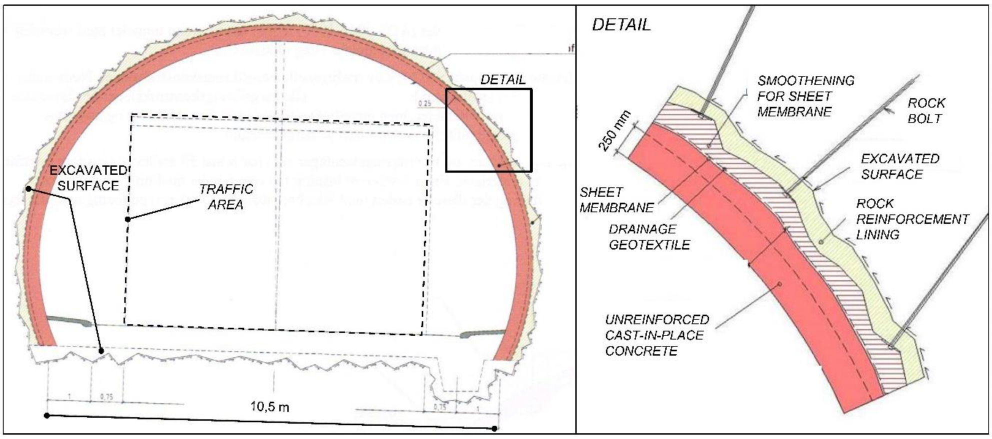



**Figure 9.** Drawings showing a possible future technical solution for a dual carriageway road tunnel with cast concrete lining. Left: complete theoretical lining section for a dual carriageway tunnel. Right: lining detail (after NPRA 2012).

**Figure 10.** Photos of the cast concrete lining system during the construction of the Ulvin rail tunnel at the E6-Dovre rail line joint project, Norway, 2015. Left: installation of sheet membrane waterproofing. Right: interior of completed tunnel lining

## 2.4 Alternative innovative lining system: waterproof SCL

Tunnel linings based on waterproof SCL were introduced to the tunneling industry around 2000, and has seen increased use in some countries (Switzerland, UK, Czech Republic, Italy and Norway) over the last decade.

The basic idea with spray-applied waterproofing in an SCL context in hard rock, is to utilize the rock reinforcement lining as the final lining structure, and include an elastic, bonded membrane embedded within the sprayed concrete structure which provides the required waterproofing. The construction process of the waterproofing is a sequence of operations which includes surface preparation, temporary handling of water ingress points, application of the membrane and the application of the final inner lining (ITA/AITES 2013, Holter & Foord 2015). Examples of spray application of the waterproofing membrane is shown in Figure 11. Figure 12 shows the robotic application of sprayed concrete

**Figure 11.** Construction of waterproof SCL. Left: Manual spray-application of membrane at the Ulvin test site (2011). Right: Robotic spray-application of membrane during the construction of the Hindhead motorway tunnels, UK (2009)

**Figure 12.** Construction of waterproof SCL, application of final inner layer of sprayed concrete. Test section with SCL in the Bærum rail tunnel, Norway (2009)

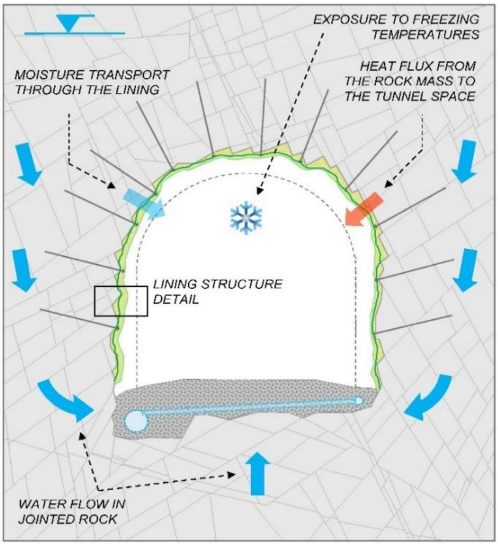
**Figure 13.** Photos of completed SCL with sprayed EVA based membrane. Left: The Gevingås rail tunnel, Norway (2011). Right: completed lining for a section of the twin track Holmestrand rail tunnel, Norway (2015).

# 3 RESEARCH METHODOLOGY

## 3.1 Conceptualisation and approach to task

Evaluating the properties of the waterproof SCL system is a multidisciplinary task which involves rock engineering, hydrogeology, concrete technology, waterproofing technology and building physics (Figure 14, right). The working method of conceptualizing the main task was chosen as the approach. In this way the main task is broken down in modules of secondary research tasks. The main reason for following this approach is to increase the clarity of each task, identify separate working modules and structure the work. For this purpose a main conceptual model is presented, in which the main task is subdivided in several modules. Each of these modules represent important issues in the understanding of the function and properties of the waterproof SCL system, and form more detailed secondary conceptual models. Thus, the conducted investigations are oriented towards verifying a detailed conceptual model for each of the special items. A synthesis of these findings constitute the verification of the main conceptual model

The main conceptual model contains the rock mass with the underground opening, the rock support including the tunnel lining, the exposure to groundwater and exposure to the climate in the tunnel space, and is illustrated in Figure 14, left. The main items are summarized in Table 3.



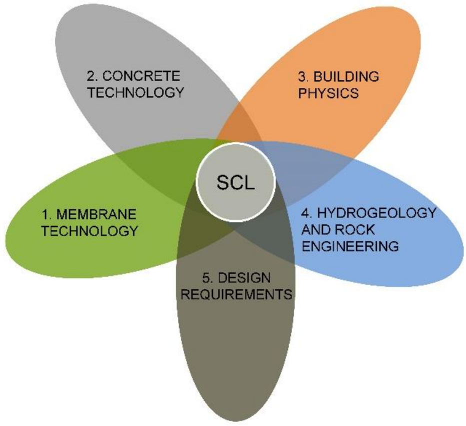



**Figure 14**. Left: Main conceptual model with configuration of lining and processes which influence the condition of the lining. Right. Conceptual diagram illustrating the multidisciplinary character of the case waterproof SCL

**Table 3**. Conceptual model, main elements

| Main element in model | Condition, processes | Described in paper |
|---|---|---|
| Lining structure, with constituent materials | Details of sprayed concrete<br>Details of membrane | 3<br>4<br>5 |
| Mechanical loads which act on the membrane | Cracking of concrete, shear and tensile loads on membrane, moisture content at given exposure | 3<br>5 |
| Effect of undrained waterproofing in walls and crown. Loads on lining | Water pressure gradient causes flow of water towards tunnel invert on rock joints. Increased joint water pressure in immediate rock mass | 2<br>6 |
| Moisture transport through the tunnel lining | Saturation of the sprayed concrete from the rock side and exposure to air at the lining surface causes capillary transport and vapor diffusion through lining | 3 |
| Freezing exposure of lining and effects of freeze-thaw on the constituent lining materials | Heat flux from rock mass towards lining surface which gives a certain minimum possible temperature at the location of the membrane at a certain thermal exposure at the lining surface | 4<br>5 |

## 3.2 Secondary conceptual models

### 3.2.1 Tunnel lining structure: constituent materials

The SCL structure considered in this study consists of a multilayered bonded structure with fiber reinforced sprayed concrete with water/binder ratios in the range of 0.44-0.47. All sprayed concrete layers have fiber reinforcement (steel or structural polypropylene). One or two layers of sprayed concrete is normally applied for the rock support, and a separate layer will be required as a smoothening layer. The thicknesses of the sprayed concrete is given by the required rock support level. The minimum required thickness of the sprayed concrete for rock support is 80 mm (NCA 2011, NPRA 2012). The membrane considered in this study has a minimum thickness of 3 mm. The inner lining sprayed concrete has a minimum required thickness of 60 mm, corresponding to the required cover of waterproofing sheets. Rock bolts are required to be fully grouted and completely embedded in the sprayed concrete before the membrane is applied. The SCL structure is shown in Figure 15.

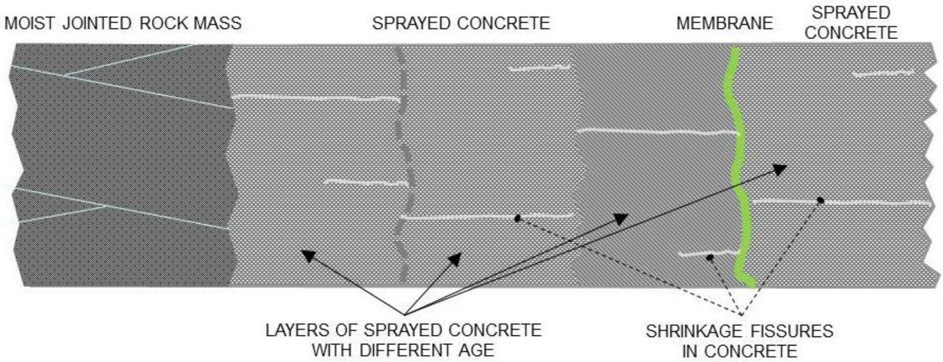



**Figure 15**. Conceptual diagram with section of SCL lining structure with main materials

### 3.2.2 Mechanical loading of lining

The main assumption made in this study is that the primary rock support lining is stable and hence, further deformations caused by rock mechanical loads are not expected. This will normally be the case, since the waterproofing and final lining presumably will be constructed several months after excavation of the tunnel and construction of the rock support lining. The loads on the membrane found to be important in this study are the deformations which occur across cracks in the concrete and thermal and shrinkage induced shear deformations.

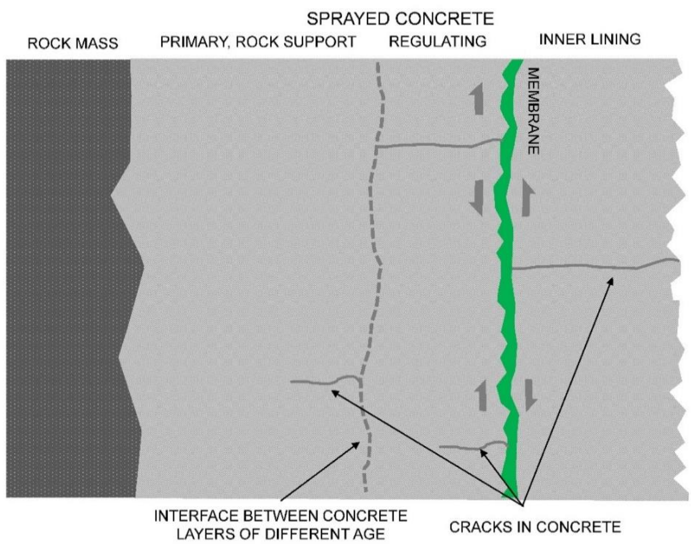



**Figure 16.** Conceptual drawing showing type of deformations in the concrete which can expose and load the membrane

### 3.2.3 Effects of undrained tunnel lining in walls and crown

The undrained property of the SCL structure in the walls and the crown of the tunnel will direct the flow of groundwater towards the invert of the tunnel. This can possibly lead to a higher groundwater pressure in the surrounding rock mass and at the rock/concrete interface compared to a case with a drained lining surface. This is illustrated in Figure 17.

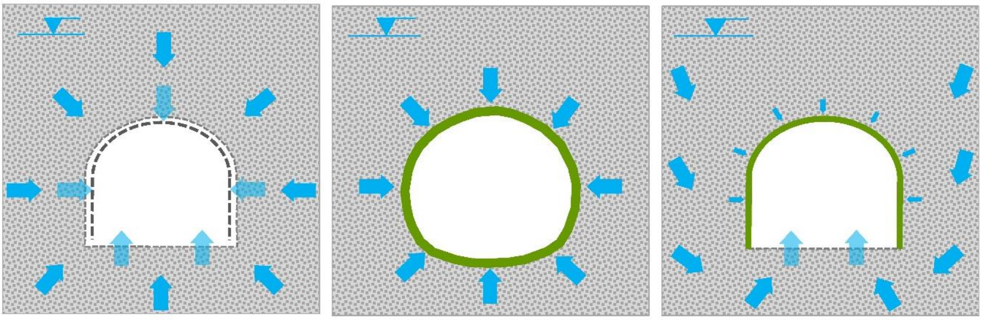


**Figure 17.** Conceptual vertical sections of three main lining configurations with respect to waterproofing. Left: Rock support lining without waterproofing with separate drained waterproof (and often thermally insulating) shield structure. Middle: Waterproof lining in entire tunnel perimeter. Fully tanked lining structure exposed to full hydrostatic pressure. Right: Partially drained tunnel with waterproof undrained lining in the walls and crown, and drained invert

### 3.2.4 Moisture exposure and moisture transport

The continuous and bonded property of the different layers in the lining resulting from the spray application, implies that the interfaces between the materials are perfect hygric contacts. The unilateral exposure to groundwater on the rock side of the lining will result in an absorption and migration of water into the lining materials. Cracks and imperfections in the concrete materials will most likely be saturated with water in a long term perspective. Exposure to dry air on the lining surface will result in vapor pressure gradients, and hence create a certain moisture transport through the lining. A model of the lining structure with water exposure processes is shown in Figure 18.

The moisture condition of the membrane and the concrete materials are important for the assessment of the possible degrading mechanisms which can occur. Such degrading mechanisms could possibly include softening of the membrane due to water exposure and risk of delamination, pore pressure in the concrete on the rock side of the membrane, water pressure in fissures and cracks in the concrete which expose the membrane.

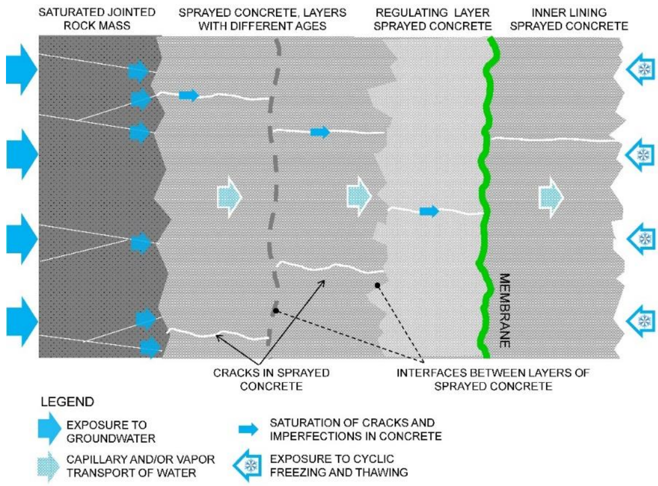



**Figure 18.** Conceptual drawing showing a model of the SCL structure with constituent materials and water exposure

### 3.2.5 Freezing exposure

The freezing exposure of a tunnel lining will depend on several factors, which can be divided into two main issues:

- The longitudinal climatic profile of the tunnel with typical fluctuations, governed by the outdoor climate, the ventilation characteristics of the tunnel and the traffic type in the tunnel
- The heat conduction from the rock mass through the tunnel lining to the tunnel space

The longitudinal thermal profile will be unique for each tunnel case and will be governed by the ventilation characteristics, the outdoor climate, the length of the tunnel and the temperature of the rock mass. With a prevailing ventilation direction, warm air during summer and cold air during winter will penetrate significantly longer into the tunnel from one side with respect to the other side, as shown in Figure 19. This is confirmed by a detailed study carried out in two Swedish rail tunnels (STA 2012a, STA2012b).

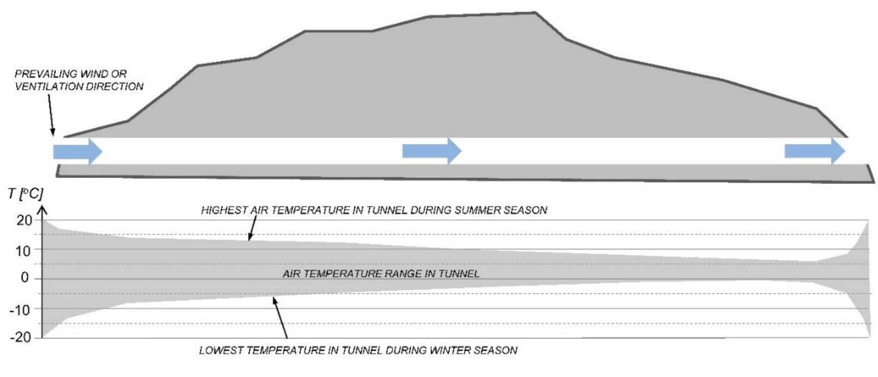



**Figure 19.** Longitudinal section of a tunnel with a certain length and indication of range of air temperatures in the tunnel space with a prevailing wind direction. Compiled from Swedish studies (STA2012a, STA2012b)

A conceptual model for the thermal exposure of a tunnel lining is shown in Figure 20. In this study thermal measurements in the freezing range in the lining could be done in the large scale freezing laboratory and at the Ulvin test site. Details are given in Paper 5, Section 4.4.

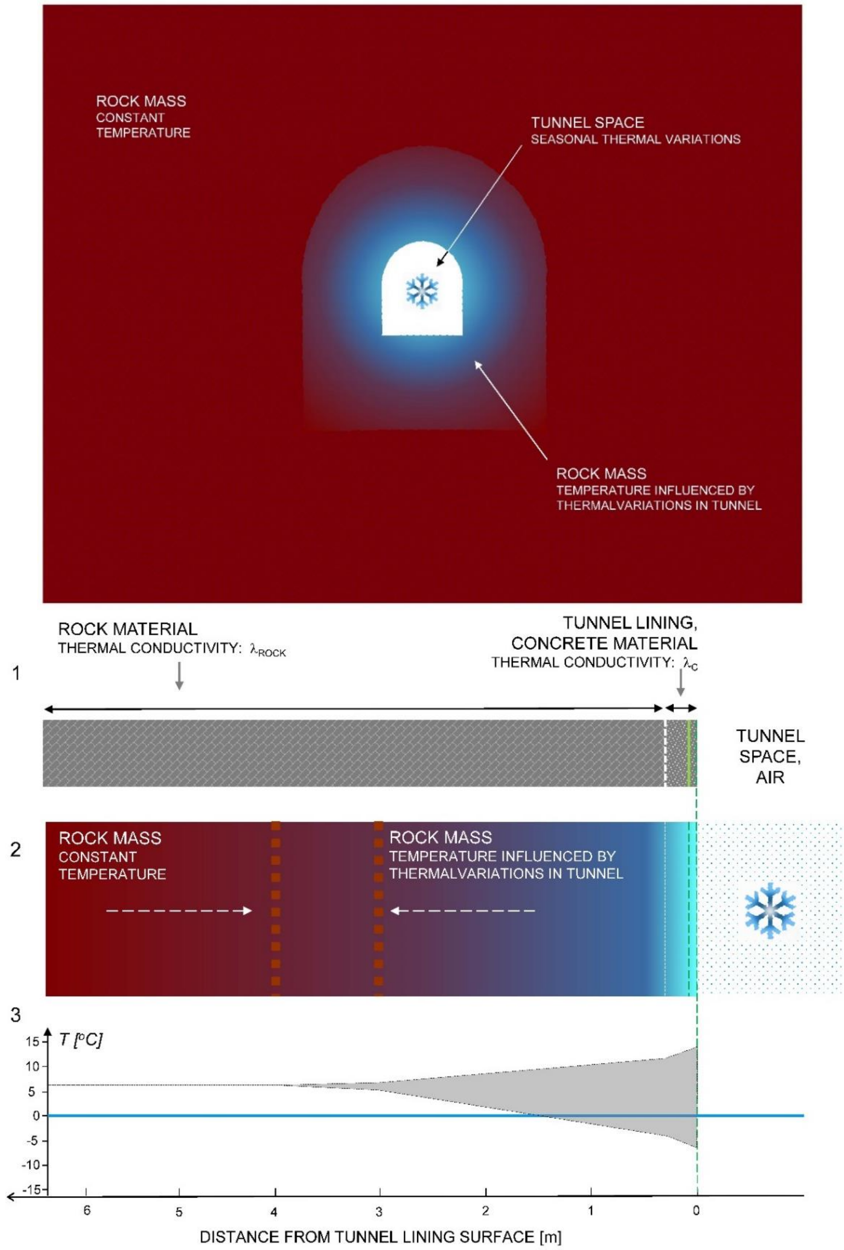



**Figure 20.** Conceptual model for thermal exposure of the tunnel lining. Top: Cross section illustrating the rock mass and zone influenced by thermal fluctuations. Bottom: Profile along a radial axis from the tunnel surface with indication of (1) material types, (2) visualization of cooling from the lining surface and (3) the range of temperatures along the radial axis with a certain thermal exposure on the lining surface.

# 4 EXECUTED WORK

## 4.1 Main goal, purpose

The main intention of the work was to construct several full scale tunnel lining sections and subsequently verify the properties of the lining system through monitoring, investigations of in-situ condition and strength combined with detailed laboratory investigations of material properties.

The conceptual models described in Section 3 have been subject to continuous development and refining during the project. Hence, the finally executed work was a result of a process where initial findings lead to the final executed work in order to verify the final updated conceptual models.

The executed work can be divided in the following six main items:

- Construction work at selected test sites and establishment of full scale or large scale SCL sections and installation of different types of instrumentation
- Construction of a large scale SCL section on a homogenous rock mass in a freezing laboratory
- Field investigation work at the test sites with mapping of engineering geological conditions, monitoring of water pressure, temperatures, relative humidities, as well as in-situ freeze exposure tests, strength testing and extraction of realistic material samples for laboratory testing
- Laboratory testing of sprayed concrete material for water transport and moisture absorption processes, porosity and freeze-thaw resistance
- Laboratory testing of sprayed membrane material for mechanical performance parameters, moisture absorption and moisture transport processes
- Numerical simulations of selected tasks in order to verify the findings from field and laboratory investigations and make further assessments

The main content of these work modules are presented in the following sections, 4.2 – 4.6.

## 4.2 Field work – construction of test sections and instrumentation

In order to conduct field investigations with realistic conditions, two full scale lining section were constructed as early as possible at two different tunnel sites, the Karmsund and Ulvin tunnel sites (Figure 21) These projects were under construction at the time. Hence, construction materials and infrastructure was available for spayed concrete and constructing the lining under realistic conditions. Furthermore, a portion of the Gevingås rail tunnel, completed in 2011 of which 1.85 km of totally 4 km had been constructed with SCL waterproofed with sprayed EVA based membrane, was made available as a test site. Thus, a lining structure with age up to approximately 3 years could be included in this project. In order to have sufficient quantity of material specimens of realistic type and curing history available for laboratory testing, a small test field in the Harangen road tunnel, which was under construction at the time, was established.

In order to conduct precise freezing testing of the lining structure, the freezing laboratory facility, originally constructed in 2009 (SINTEF 2011, Nermoen et al. 2011), was re-established for this study with new sprayed concrete lining, 4 different membrane products and thermal instrumentation in both concrete layers on either side of the membrane. An additional feature was included, with 36 holes of 20 mm diameter drilled through the lining, slightly dipping, in order to add water to the concrete at the rock-concrete interface. In this way exposure to groundwater by capillary absorption was simulated. This is explained in Section 4.4.

The location of the test sites is shown in figure 21.

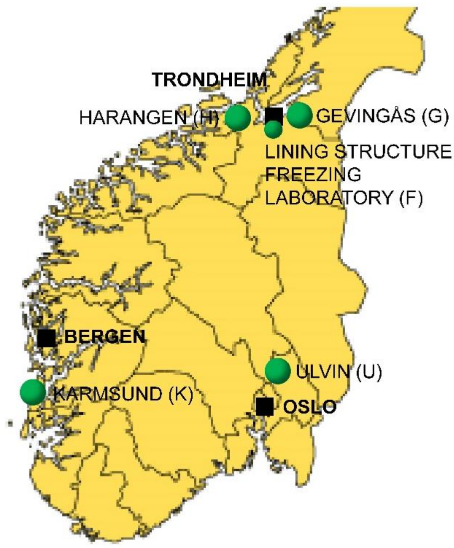



**Figure 21.** Locations of test sites

The main features of the test sites are illustrated in Figure 22. The layout and main features of freezing laboratory with lining structure is shown in Figure 23. The conducted construction and site installation work is shown in Table 4.

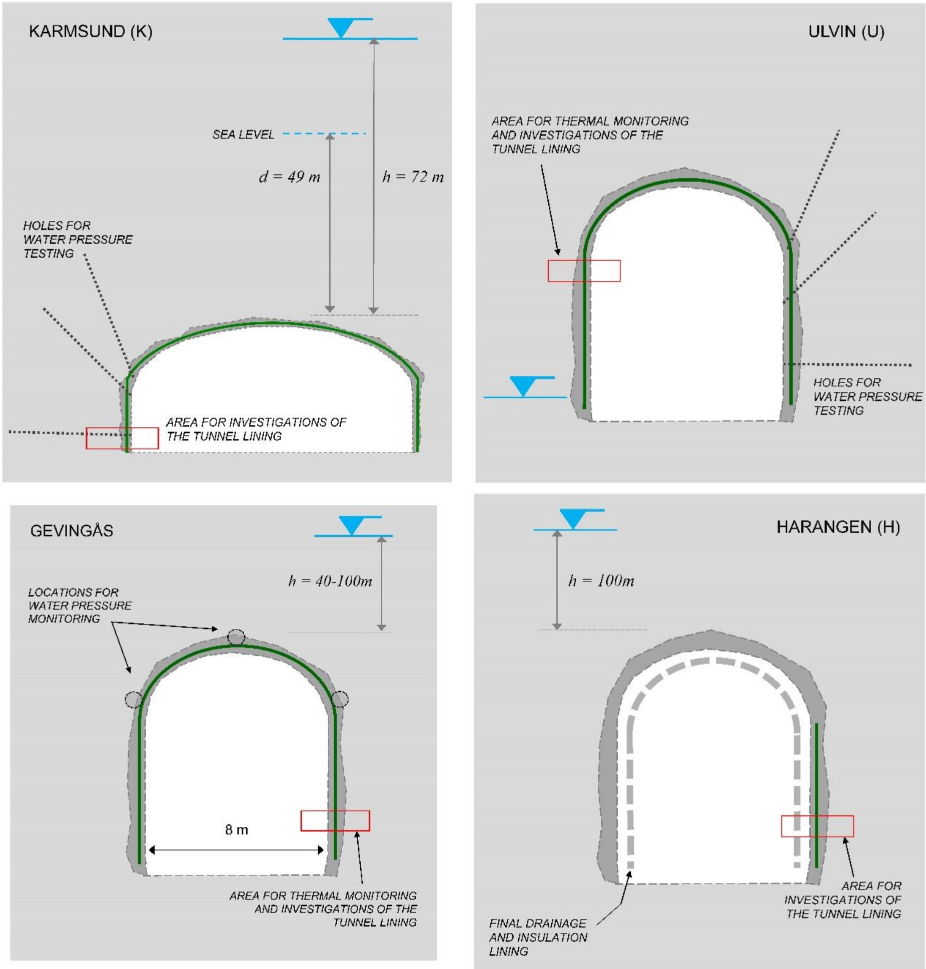



**Figure 22.** Conceptual cross sections of the four tunnel test sites, illustrating layout of lining structure, hydrogeological context and location of executed investigations

**Table 4.** Executed construction and instrumentation work

| Site | Executed work | Time (year/month) | Main purposes |
|---|---|---|---|
| Karmsund subsea tunnel (K), ventilation cavern | Construction of complete lining system in 20 linear meter. Installation of water pressure sensors in three drill holes | 2011 / 09-11 | 1. Investigate any effects of rock joint water pressure when using an undrained waterproof lining in walls and crown, but leaving the invert drained |
| Ulvin South access tunnel (U) | Construction of complete lining system in 60 linear meters. Installation of water pressure sensors in three drill holes | 2011 / 11-12 (construction, water pressure monitoring) 2012 /12 (thermal instrumentation) | 1. Investigate any effects of rock joint water pressure when using an undrained waterproof lining in walls and crown, but leaving the invert drained 2. Determine condition of lining and provide basis for assessments of durability |
| Gevingås rail tunnel (G) | Installation of thermal instrumentation in 4 drill holes in rock mass and tunnel lining with lengths up to 2 m | 2012 / 12 | 1. Obtain data for tunnel climate and thermal conditions in rock mass and tunnel lining: basis for hygrothermal simulation 2. Determine condition of lining and provide basis for assessments of durability |
| Harangen road tunnel (H) | Construction of complete lining system in a 5 m by 3 m on the tunnel wall | 2013 / 02-03 | 1. Production of material samples for laboratory testing with realistic history, production method and in-situ curing |
| Large scale lining structure freezing laboratory (F) | Demolition of old lining system from initial testing (2010). Reconstruction of lining complete system including 4 different membrane products. Installation of thermal instrumentation in lining structure and core drilling of through rock mass for water exposure of lining | 2012 / 06 - 09 | 1. Large scale simulation of tunnel lining with moisture condition, different thermal exposure settings 2. Basis for assessments of durability |

## 4.3 Field work: monitoring and in-situ testing

The main purpose of the monitoring and site testing was to obtain as much data as possible in order to substantiate the conceptual models. Results from the field investigations formed a basis to execute laboratory testing under realistic conditions, as well as making assessments on the validity of laboratory results with respect to the

- Obtain field data for verification of the conceptual model
- Obtain basis for realistic conditions for laboratory testing
- Assess validity of laboratory results
- Assess realistic boundary conditions and input parameters for numerical simulations

An overview of the executing site testing is shown in Table 5.

**Table 5.** Executed investigations and monitoring and at the tunnel sites

| Site | Executed work | Time | Purpose |
|---|---|---|---|
| Karmsund subsea tunnel, ventilation cavern (K) | 1.Water pressure testing in 0.5m sections in 3 radially core drilled holes with 9 m length<br>2. Monitoring of water pressure in the rock mass in drill hole sections<br>3. Video of interior of core holes<br>4. Investigations of core material from the rock mass<br>▪ joint roughness coefficient JRC and joint compressive strength JCS (Bandis-Barton index parameters)<br>▪ samples for measurement of Young modulus and uniaxial compressive strength¨<br>▪ rock joints: location and angle<br>5. Mapping of rock joints in lower walls<br>6. Recording of wet spots through final lining surface<br>7. Investigations of core material from the tunnel lining | 1. 2011/10<br>2. 2011/10 – 2012/5<br>3. 2011/10<br>4. 2012-2013<br>5. 2011/10<br>6. 2012/05<br>7. 2013 | 1. Measure hydraulic transmissivity of sections in the rock mass<br>2. Measure water pressure situation in immediate rockmass around lining, and any changes over time<br>3, 4 and 5. Obtain rock joint data for test site<br>6. Assess final state of tunnel lining |
| Ulvin access tunnel (U) | 1. Water pressure testing in 0.5m sections in 3 radially core drilled holes with 9 m length<br>2. Monitoring of water pressure in the rock mass in drill hole sections<br>3. Video of interior of core holes<br>4. Investigations of core material from the rock mass<br>▪ rock joints: location and angle<br>▪ measurement of thermal conductivity of rock material<br>5. Mapping of rock joints in lower walls and in some sections at the tunnel face<br>6. Recording of remaining wet-spots in the tunnel lining<br>7. Thermal monitoring in tunnel lining and rock mass in 4 holes up to 2 m depth from lining surface<br>8. Full scale freezing exposure experiment of entire portion of SCL test section with 36 hours of forced ventilation with cold air from outside<br>9. Investigations of the in-situ condition of the tunnel lining at different ages<br>▪ Moisture content profiles of concrete<br>▪ Moisture content in membrane material<br>10. Measurements of in-situ tensile bond strength at different ages | 1. 2011/12<br>2. 2011/12 2012/06<br>3. 2011/12<br>4. 2012-2013<br>5. 2011/12<br>6. 2013/12<br>7. 2012/12-2013/06<br>8. 2013/01<br>9. 2013/12-2015/08<br>10. 2014/04 – 2015/08 | 1. Measure hydraulic transmissivity of sections in the rock mass<br>2. Measure water pressure situation in immediate rockmass around lining, and any changes over time<br>3, 4 and 5. Obtain rock joint data for test site<br>6. Assess final state of tunnel lining<br>7.Verify thermal condition of lining and rock mass during freezing exposure<br>8. Obtain data for a full scale field freezing test.<br>9. Determine in-situ moisture condition of lining, and any changes over time<br>10. Determine in-situ tensile strength and any change over time |
| Gevingås rail tunnel (G) | 1. Investigations of the in-situ condition of the tunnel lining at different ages (24, 36, and 50 months)<br>▪ Moisture content profiles of concrete<br>▪ Moisture content in membrane material<br>2. Measurements of in-situ tensile bond strength at different ages<br>3. Thermal monitoring in tunnel lining and rock mass in 4 holes up to 2 m depth from lining surface<br>4. Measurements of temperature and relative humidity in tunnel air | 1. 2013/04, 2014/04, 2015/08<br>2. 2014/04, 2015/08<br>3. 2012/12 – continuously<br>4. 2012/12 - continuously | 1. Verify in-situ moisture content of lining, and any changes over time<br>2. Determine in-situ tensile strength and any change over time<br>3. Verify thermal condition of lining with seasonal variations. Input for hygrothermal simulation<br>4. Verify climate in tunnel space with seasonal variations. Input for hygrothermal simulations |
| Harangen road tunnel (H) | 1. Extraction of samples of sprayed concrete and complete lining structure by core drilling<br>2. Investigations of the in-situ condition of the tunnel lining at 5 months age<br>▪ Moisture content profiles of concrete<br>▪ Moisture content in membrane material | 1. 2013/08<br>2. 2013/08 | 1. Prepare specimens of realistic sprayed concrete material for freeze-thaw laboratory testing<br>2. Verify moisture condition of lining at 5 months age |

## 4.4 Freezing laboratory with large scale lining structure on rock mass

The large scale laboratory for physical simulations of freezing loads and subsequent testing of the lining was originally constructed 2009-2010 for freezing load assessment prior to the construction of the Gevingås rail tunnel. For the purpose of this study, the rock mass and the two rooms with thermal insulation and freezing/refrigeration units could be re-utilized. The old lining structure was demolished and a new lining structure based on fiber reinforced sprayed concrete and EVA-based membranes was constructed in June-August 2012. Four different membrane products were applied, each covering an 800 mm wide vertical field on the wall. The substrate of fiber reinforced sprayed concrete was floated without adding water, in order to produce an even and homogenous surface for all membranes.

The new and important feature which was added to the model for this study, was the exposure of the lining structure to water through 20 mm diameter drillholes with the intention of achieving a moisture content in the lining materials which was comparable to the in-situ condition found in the tunnel sections. The holes were drilled with a slight dipping angle towards the lining in order have the holes constantly filled with water and give exposure with atmospheric pressure. The lining structure was conditioned at high relative air humidities by keeping the tunnel room constantly wet at ambient temperature (20-23ºC) and closed for 13 months. The sequence of activities which then followed is shown in Table 6. By adding water in the drillholes an increase of the water content in the lining structure was observed, and the found moisture contents were in the same range as the finding from the in-situ investigations of moisture content in the tested tunnel sections. This moisture conditioning of the lining structure enabled the further execution of freeze-thaw testing of the lining under realistic moisture exposure. The rock mass surface with the location of the drillholes for water exposure is shown in Figure 23 (left). The four tested membrane products applied on the rock wall are shown in Figure 23, right. The laboratory lining structure is shown in Figure 24.


**Figure 23.** Photos of the freezing laboratory during the construction phase of the lining structure. Left: Rock mass surface after demolition of previous lining. The holes for water exposure are indicated. Right: Surface of lining structure with 4 different membrane products prior to the application of the inner lining of fiber reinforced sprayed concrete.

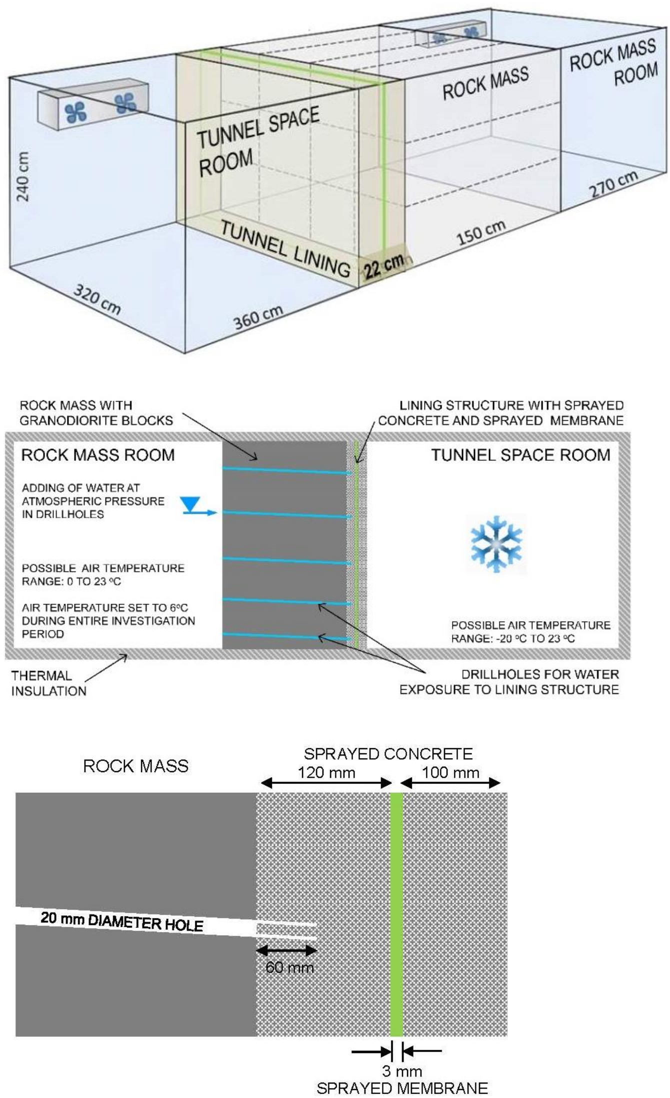



**Figure 24.** Conceptual drawings of the freezing laboratory with large scale lining structure constructed on a rock mass of granodiorite blocks. Top: 3D view with layout of rock mass with lining structure. Middle: longitudinal section through the model. Bottom: Detail of the lining structure with layer thicknesses and a hole for simulation water exposure

**Table 6.** Executed investigations in the large scale lining structure in the freezing laboratory

| Executed work | Time | Purpose |
|---|---|---|
| 1. Adding of water to the lining structure through drillholes in the rock to simulate exposure to ground water | 1. 2013/09-2015/08 | 1. Condition the lining structure to realistic moisture contents |
| 2. Freeze-thaw exposure of the lining structure with 35 cycles with -3ºC minimum temperature in each cycle at the location of the membrane | 2. 2014/02-05 | 2. Accelerated cyclic freeze-thaw exposure of lining, with moderate freezing load |
| 3. Freeze-thaw exposure of the lining structure with 20 cycles with -7ºC minimum temperature in each cycle at the location of the membrane | 3. 2014/05-08 | 3. Accelerated cyclic freeze-thaw exposure of lining, with severe freezing load |
| 4. Isothermic freezing exposure of the lining structure with - 7ºC air temperature in the tunnel space for 40 days | 4. 2014/09-10 | 4. Simulate steady state conditions in the model and measure thermal profile |
| 5. Moisture transport test. Uniform temperature of 6ºC held in entire model. Possibly constant RH 80% held in tunnel room. Measurement of water added through drill holes | 5. 2014/10 – 2015/08 | 5. Measure moisture transport though lining structure with controlled boundary conditions |
| 6. Investigations of the in-situ condition of the lining structure after complete curing of the concrete, after 5 months water saturation, after each of the cyclic freeze-thaw exposure levels and after the final moisture transport test<br>▪ Moisture content profiles of concrete<br>▪ Moisture content in membrane material | 6. 2013/09, 2014/01, 2014/05, 2014/08. 2015/08 | 6. Verify moisture condition of lining structure during the different tests |
| 7. Measurements of in-situ tensile bond strength at different stages in the history of the lining structure (after freezing exposure) | 7. 2014/01, 2014/05, 2014/08. | 7. Determine any change of strength after freezing exposure |

## 4.5 Laboratory investigations of sprayed concrete material

The main purpose of the laboratory testing of the sprayed concrete material was to find out if freezing damage can be expected with the given moisture contents and freeze-thaw exposure characteristics. Material properties pertaining to the actual material constructed in tunnel linings and realistic exposure to moisture and freezing were emphasized. Such material parameters relate to porosity characteristics, water absorption, permeability and resistance to freeze-thaw damage with the realistic exposure. An overview of the executed laboratory investigations of sprayed concrete material is shown in Table 7.

The exposure severity of the commonly used test methods for freeze-thaw resistance of concrete (salt frost scaling and rapid freeze/thaw) was recognized, and a need to test the sprayed concrete with more realistic parameters for the tunnel exposure case was identified. The main reason for this is to avoid to reject materials which will fail according to a very severe test procedure with unrealistic conditions, but still might be suitable and exhibit durability under the conditions in a tunnel lining. The development and pilot testing of this test method was carried out in conjunction with a master thesis work (Tandberg 2014), and presented at Concrete Innovation Conference CIC in Oslo June 2014 (Holter et al. 2014)

**Table 7.** Executed laboratory testing of sprayed concrete material from tunnel sites

| Investigation | | Laboratory, location | Concrete from site[1] | Age of concrete at testing [days] | Time | Number of specimens | Purpose |
|---|---|---|---|---|---|---|---|
| Porosity and moisture content | Optical air void analyses ASTM C457 | NTNU | H | 400 | 2014/06 | 5 | Determine air void parameters for frost resistance |
| | Suction porosity, air voids and moisture content, according to the PF method | NTNU | G | 360 -1100 | 2012-2014 | 80 | Determine degrees of water saturation, porosity characteristics for frost resistance |
| | | | U | 180 - 800 | 2012-2014 | 79 | |
| | | | H | 150 | 2013 | 39 | |
| | | | F | 400 - 650 | 2012-2014 | 49 | |
| Standard frost resistance tests | Salt frost scaling, CEN/TS 12390-9 | NPRA[2] | H | 300 | 2014/02 | 8 | Determine resistance salt scaling |
| | Resistance to rapid freeze thaw, ASTM C666, procedure A | SINTEF[3] | H | 300 | 2014/02 | 5 | Determine resistance to internal frost damage during rapid freeze-thaw exposure |
| | Freezing induced dilation, ASTM C681 | NTNU | H | 350 | 2014/05 | 4 | Determine freezing induced dilation at high degree of capillary saturation |
| Frost resistance under tunnel lining conditions | Functional performance freezing test, developed at NTNU | NTNU | H | 250-300 | 2013-2014 | 8 | Determine frost resistance under realistic exposure |
| Water vapor permeability | Nordtest NT Build 369 | SINTEF[3] | H<br>G | H: 300<br>G: 1000 | 2013/10-2014/01 | H: 5<br>G: 6 | Moisture transport analysis |
| Hydraulic conductivity | SOP | LPM, Switzerland | H<br>G<br>U | H: 300<br>G: 1050<br>U: 750 | 2013/12-2014/03 | H: 10<br>G: 10<br>U: 15 | Moisture transport analysis |
| Thermal conductivity and specific heat capacity | | NTNU | G<br>U | G: 800<br>U: 570 | 2014 | G: 2<br>U: 2 | Heat flux and thermal exposure analysis |
| Unidirectional water absorption | SINTEF, 1988 | NTNU | H | 250-300 | 2014 | 14 | Moisture transport analysis |

[1] Abbreviations for site locations are explained in Figure 21.
[2] Norwegian Public Roads Administration, central laboratories, Oslo
[3] SINTEF Building and Infrastructure, Trondheim

### 4.6 Laboratory testing of membrane material

The laboratory testing of the membrane material was aimed at measuring material parameters according to realistic loading and exposure scenarios, based on the field investigations. Initially 4 membrane products from the two suppliers were included. Later a fifth membrane product was also delivered to this study. An overview of the tested products is shown in Table 8.

**Table 8.** Tested membrane products in this study

| Product designation in this study | Supplier | Product Name | Lot no / production date | Polymeric content[3] [weight %] |
|---|---|---|---|---|
| M1 | BASF[1] | Masterseal 345 | 56 30 04 / 2009 | 73.7 |
| M2 | BASF | MEYCO TSL865 | 11-28-081 / 08.2011 | 54.6 |
| M3 | Orica[2] | Tekflex DS-T | 301 / 22.05.2012 | 62.4 |
| M4 | Orica | Tekflex DS-M | 303 / 08.11.2011 | 48.3 |
| M5 | Orica | Tekflex DS-W | 001 / 20.05.2013 | 71.2 |

[1] BASF Construction Chemicals Europe AG

[2] Orica International Ltd (Minova International Ltd at the commencement of this project)

[3] Measured with thermo-gravimetric analysis (TGA) up to 800ºC with Argon-6 as test gas (Laboratory: SINTEF Materials and Chemistry)

Based on poor findings for elasticity and tensile bond strength for the membranes M2, M3 and M4 early in this study, these three products were not included in the complete test program. The complete test program for the membrane testing is shown in Table 9.

**Table 9.** Executed tests for membrane properties

| Issue tested, parameter | Test method, applicable standard | Test location, laboratory | Tested membrane products | Thermal exposure range | Moisture condition modes | Number of specimens |
|---|---|---|---|---|---|---|
| Tensile bond strength | EN-ISO 4624 | NTNU | M1<br>M2<br>M3<br>M4<br>M5 | 20 to -7ºC | ▪ Dry<br>▪ Conditioned by immersion at atmospheric pressure | M1 : 35<br>M2 : 7<br>M3: 11<br>M4 : 7<br>M5: 27 |
| Shear deformability and strength | Direct shear testing with special adopted procedure | NTNU | M1<br>M5 | 20ºC only | Conditioned by immersion at atmospheric pressure | M1: 5<br>M5: 4 |
| Elasticity | DIN 53504 | BASF [1]<br>Orica [2]<br>Wacker [3] | M1<br>M2<br>M3<br>M4<br>M5 | 23 to -12ºC | Moisture conditioned at RH 50 and 95% | M1: 41<br>M2 : 5<br>M3 : 5<br>M4 : 5<br>M5 : 41 |
| Crack bridging | DIN EN 1062-7 | Wacker [3] | M1<br>M5 | 23 to -3ºC | Moisture conditioning at RH 50 and 95% | M1:<br>M5: |
| Watertightness | CEN/TS 12390-8 | Hagerbach Test Gallery, Switzerland | M1<br>M5 | 20ºC | n.a. | M1 : 3<br>M5 : 6 |
| Polymeric content | Thermo-gravimetric analysis | SINTEF [4] | M1<br>M2<br>M3<br>M4<br>M5 | n.a. | n.a. | One sample per product |
| Water vapor permeability | NS-EN ISO 12572 | SINTEF [5] | M1<br>M5 | 20ºC | Wet cup method with humidity range RH 50 to 94% | M1: 6<br>M5: 9 |
| Unidirectional water absorption rate | PF-Method (SINTEF 1988) | NTNU | M1 | 20ºC | Specimens conditioned at RH 50% | 6 |
| Water content in hygroscopic range (desorption isotherms) | - | NPRA [6] | M1<br>M5 | 20ºC | n.a. | M1:25<br>M5:25 |
| Concrete – membrane interface characteristics | SEM-analysis | Wacker [2] | M1 | n.a. | n.a. | M1: 2 |

[1] BASF Construction Chemicals Europe AG, Kaisten, Switzerland
[2] Orica International Ltd (at the time of testing: Minova International Ltd), Siemianovice Slaskie, Poland
[3] Wacker Chemie AG, Burghausen, Germany
[4] SINTEF Materials and Chemistry, Trondheim
[5] SINTEF Building and Infrastructure, Trondheim
[6] Norwegian Public Roads Administration, central laboratories, Oslo

## 4.7 Numerical simulations

Numerical simulations were carried out for two different purposes in this study:

- Assess moisture transport through SCL with EVA based membrane and explain observed moisture condition in the investigated tunnel linings
- Assess trends in ground water pressure in the immediate rock mass of an undrained SCL structure and investigate of the ground water flow mechanisms represented in the numerical codes and possible explain the measured ground water pressures

The executed numerical simulations are shown in Table 10.

**Table 10**. Executed numerical simulations

| Main task | Software | Type of simulation | Purpose |
|---|---|---|---|
| Hygrothermal, simulations of moisture transport through tunnel lining | WUFI | Simultaneous heat and moisture flux through building materials | Quantify moisture transport by vapor and capillary conduction through the lining. Assess effects of such moisture transport |
| Rock joint water pressure around partially drained SCL | UDEC BB | Distinct element simulation of rock mass. Discontinuous rock mass model | Assessment of possible effect of undrained SCL on rock joint water pressure with mapped rock joint parameters |
| Rock mass pore pressure around partially drained SCL | $FLAC^{3D}$ | Finite element | Assessment of possible effect of undrained SCL on pore pressure in rock mass using a simplified continuum model |

# 5 MAIN FINDINGS

## 5.1 Overview

The main results from the executed investigations are presented in this section as shown in Table 11. The publication of these results is indicated with the assigned numbering of the papers, included in this thesis.

**Table 11.** Overview, presentation of main findings

| Section in thesis | Issue | Details published in paper |
|---|---|---|
| 5.2 | Measured and simulated ground water pressures in the immediate rock mass | 2, 6 |
| 5.3 | Loading and exposure conditions on the lining structure and the membrane | 2, 3, 5, 6 |
| 5.4 | Sprayed concrete: main data and measured porosities | 3, 4 |
| 5.5 | Moisture absorption and permeability properties of sprayed concrete and sprayed membrane | 3, 5 |
| 5.6 | Moisture content in sprayed concrete and sprayed membrane in tunnel linings | 3, 5 |
| 5.7 | Moisture transport mechanisms through sprayed concrete linings | 3 |
| 5.8 | Freeze-thaw resistance of the sprayed concrete material | 4 |
| 5.9 | Mechanical properties and performance of the tested membranes under found loading and exposure conditions | 5 |

## 5.2 Measured and simulated ground water pressures

The main aim of the water pressure investigations was to study the effects of an undrained and waterproof lining structure, when constructing the lining in the walls and crown and leaving the invert drained. Initially the test sites Karmsund and Ulvin were planned to provide the basis for this study. The conditions at the Ulvin site with a drained and unsaturated rock mass made this site unsuitable for assessments of effects of ground water pressures acting on the lining. Some field data from the Gevingås site were added. An additional SCL test section with monitoring of groundwater pressure in one of the access tunnels to the Forsmark nuclear waste storage in Sweden was constructed May-June 2014 by the Swedish Nuclear Fuel and Waste Management Company in conjunction with this study. The water pressure measurement data from the Forsmark test site were made available for this study.

### 5.2.1 Conditions at test sites for investigations of ground water pressure

An overview of the constructed sites with the conducted test is shown in Table 12. The conditions at the four test sites for the water pressure study are presented in Table 13.

**Table 12.** Constructed SCL sections for the investigations and conducted tests

| Site | Lining section | Conducted tests |
|---|---|---|
| Karmsund road tunnel (subsea) | 24 linear m of ventilation cavern | Water pressure testing in sectioned holes with four sections in each hole. Water pressure monitoring over 8 months in sectioned 9 m long holes |
| Ulvin access tunnel | 60 linear m of construction adit | Water pressure testing in sectioned holes with four sections in each hole. Water pressure monitoring over 8 months in sectioned 9 m long holes |
| Gevingås | 1.85 km of 4.2 km constructed with SCL method | Water pressure monitoring at the rock/concrete interface over a period of 4 months in one profile at a location with higher seepage than average in the tunnel |
| Forsmark nuclear storage site, (subsea) | 15 linear m of a service tunnel | Water pressure monitoring in single sections of holes up to 3 m length |

**Table 13.** Main Engineering geological and rock mechanical conditions for the four test sites for water pressure investigations

| Parameter | Karmsund | Forsmark | Ulvin | Gevingås |
|---|---|---|---|---|
| Rock overburden [m] | 130 | 100 | 55 | 40-50 |
| Distance below ground water table [m] | 75 | 122 | 0 | 30-40 |
| Major principal rock stress, value, dip angle [MPa], [°] | 8.1 (8) | 7.6 (0) | 7.1 (4) | Not measured |
| Rock type | Massive granitic gneiss | Medium to fine grained granite to granodiorite | Banded amphibolitic gneiss | Dark mica schist |
| Uniaxial compressive strength of intact rock [MPa] | 137-238 | 139 - 280 | 148 – 285 | Not measured |
| Young's modulus of intact rock [GPa] | 64 – 73 | 66 – 105 | 43 - 49 | Not measured |
| Number of joint sets / joint spacings [m] | 2 + random 0.7 – 1 | 2 + random<br>0.1 – 0.5 | 3<br>0.2 - 0 .5 | 2 -3<br>0.2 – 0.8 |
| Number of joints per m drillcore | 1 – 3 | 6-9 | 1 - 7 | Not measured |
| Rock mass quality Q, range and typical value (in brackets) | 6 – 66 (23) | 5 – 25 (11) | 5 – 12 (8) | 3 -17 (5) |
| Estimated average hydraulic conductivity of rock mass [m/s] | $10^{-8}$ to $10^{-9}$ | $10^{-8}$ to $10^{-9}$ | $10^{-7}$ to $10^{-8}$ | $10^{-7}$ |
| Calculated hydraulic transmissivities of rock joints [$m^2$/s] | $10^{-8}$ to $10^{-9}$ | | | |

5.2.2 Results from pressure monitoring

The pressure monitoring at the test sites Karmsund and Forsmark were both conducted in rock masses with hydraulic saturation with background ground water pressures in the range of 680 – 750 kPa. The test at Karmsund had to be terminated approximately 10 months after commencement of the measurements. Hence, long term effects could not be observed. The results from the Karmsund site (Figure 25) show consistent trends for all holes with pressure reduction towards the undrained lining. The background ground water pressure at the invert level is assumed to be approximately 780 kPa. The measured pressures in hole 2 and 3 closest to the lining, are noteworthy. The likely explanation for this is an unsaturated situation locally close to the tunnel lining. This is unlikely to be a long term condition, since the rock mass is under long term hydraulic saturation and air and vapor will tend to migrate through the lining.

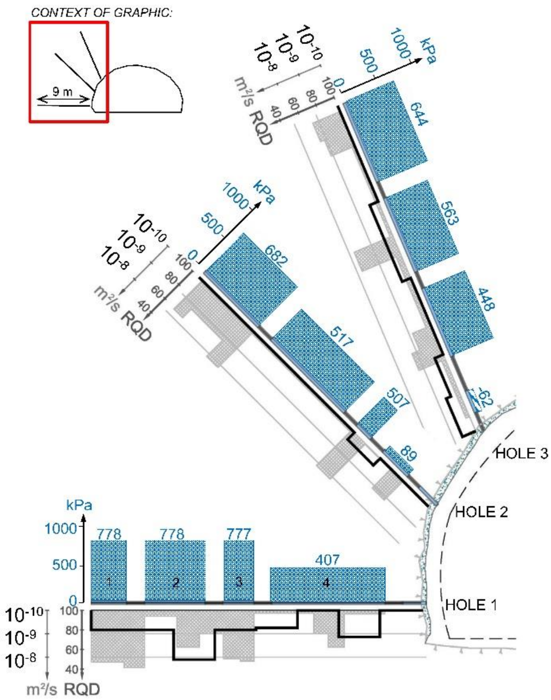



**Figure 25.** Results from the in-situ water pressure monitoring and water pressure tests conducted at the Karmsund site, with measured pressures in hole sections after 4 months after installation, calculated hydraulic transmissivities for hole sections with length 0.5 m and rock jointing given as RQD. The length of the measurement holes is 9 m

The measured ground water pressures at Forsmark exhibit to important features:

- A trend with higher water pressure for longer holes, indicating lower pressures closer to the tunnel lining
- Significant differences in measured pressure in holes with the approximately same lengths within relative short distances

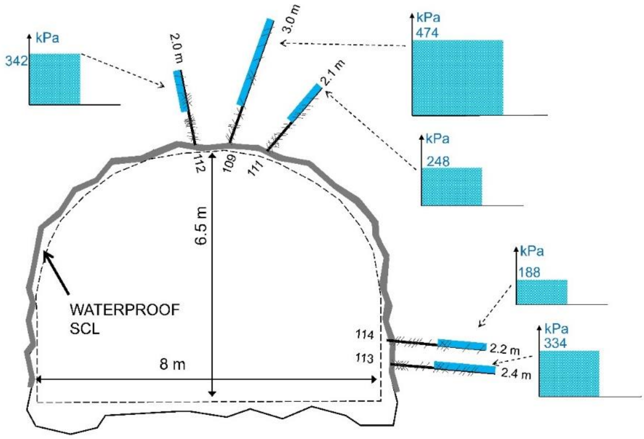



**Figure 26.** Measured water pressures at the Forsmark site in single bore hole sections (indicated in blue) after 6 months after installation. The background water pressure at the invert level of the tunnel was measured to approximately 700 kPa

### 5.2.3 Results from numerical simulations – comparison with measurements

For the Karmsund case the discontinuum model UDEC BB and continuum model FLAC$^{3D}$ simulations both predict a trend of increasing water pressures in the immediate rock mass at decreasing distance from the lining the first 1-2 m from the lining. At 3-4 m distance, the discontinuous model predicts water pressures in good agreement with the measured. Neither of the models were designed to take into account any possible effects of the EDZ or the vapor transport through the lining (vapor transport explained in Section 5.7 in the thesis).

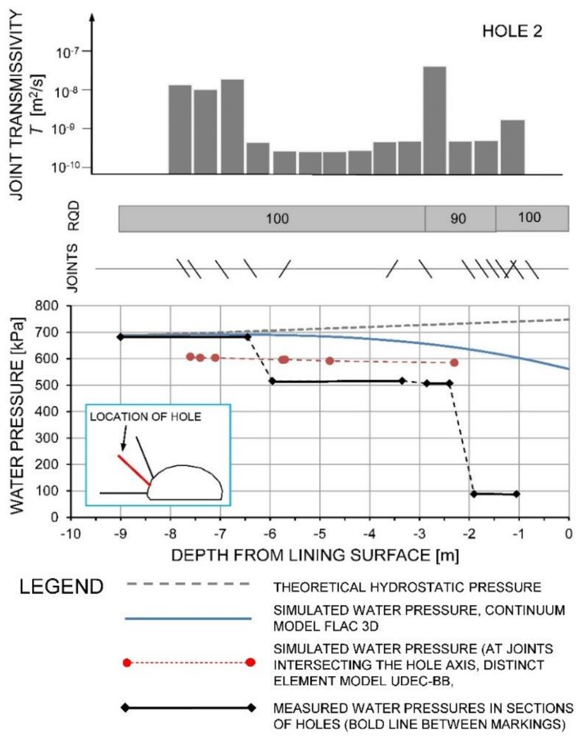



**Figure 27**. Compilation of rock joint transmissivity, rock joint data and measured and simulated ground water pressures in an investigation hole at the Karmsund test site

## 5.3 Loading and exposure conditions of the lining structure and the membrane

The loading conditions were studied in order to plan the laboratory testing and define realistic acceptance criteria.

The results from the study of loading and exposure conditions are summarized in Table 14.

**Table 14.** Main loads and exposure conditions for SCL in hard rock and cold climate

| Load type | Relevant value / size | Implication for laboratory testing |
|---|---|---|
| Rock mechanical loads | None. Only local loads | None |
| Groundwater pressure induced loads | Very unlikely for the investigated cases. Not considered | None for cases with low or no hydrostatic pressure |
| Dynamic loads from traffic area | 10 kPa amplitude of air pressure (pressure + suction loads) | None |
| Tensile loads | Gravity induced load from inner lining: 2 kPa | Not realistic requirement |
| Maximum crack width in concrete and thermal opening/closing | Typical crack width range: 0.1 – 0.3 mm. Maximum crack width 0.8 mm<br>Thermally induced opening and closure : 0.6 mm | Testing of elasticity under relevant temperatures and moisture contents required<br>Crack bridging performance at 1 mm crack width proposed |
| Shear deformation along interfaces | 0.5 – 0.6 mm / m | 1 mm shear deformation within linear shear elasticity behavior |
| Moisture exposure | 15-18 % moisture content range in the membrane material for membranes M1 and M5 | Pre-conditioning of membrane to relevant moisture content |
| Thermal exposure | Possible temperature range +15 to -6 °C at membrane location in tunnel lining | Testing at realistic temperatures |

## 5.4 Sprayed concrete: material composition and measured porosities

The sprayed concrete tested in this program was produced at tunnel sites under construction, and followed the current quality requirements for rock support sprayed concrete in road and rail tunnels (NCA 2011). Hence, this study covers state-of-the art sprayed concrete applied with the wet-mix method.

Sprayed concrete is compacted through the kinetic energy and impact on the rock wall during spraying and undergoes a transition from a liquid state to solid state in a few seconds. Cast concrete, on the contrary, remains liquid long enough for a controlled compaction through vibration to take place. Hence the properties pertaining to the hardened sprayed concrete on the rock wall need to be understood in order to make assessments of porosity and permeability characteristics, and related material behavior for moisture transport and freeze-thaw resistance.

The material testing of sprayed concrete in this study has been conducted following the established practice regarding sampling and test methods currently used in Norway. This was realized through a collaboration with the concrete laboratory at SINTEF Building and Infrastructures in Trondheim, the concrete group at the Department of Structural Engineering at NTNU and the Tunnel and Concrete section at the Norwegian Public Roads Administration.

The tunnel linings at the test sites were sampled by core drilling and both sprayed concrete and membrane materials were testes. The main used test method for the sprayed concrete was the PF Method (STF 1986, SINTEF 1988) from which the in-situ moisture condition, suction porosities and macro porosities (air void volumes) were obtained. The adopted method is explained in detail in paper 3. Tunnel linings from four of the test sites were sampled at several age. This provides a substantial database on in-situ pore characteristics and moisture condition of the sprayed concrete. A compilation of the main results from the field investigations of the sprayed concrete is shown in Table 15.

**Table 15.** Concrete composition data and measured porosities obtained with the PF-method for sprayed concrete linings from four different tunnel linings

| Concrete mix design / site | Concrete mix data | | Measured porosity data for concrete[1] | | | | PF Value | | Age at testing [days] | Number of specimens |
|---|---|---|---|---|---|---|---|---|---|---|
| | Binder[2] content [$kg/m^3$]<br>Fiber reinforcement [$kg/m^3$, % by volume] | Water/binder ratio[3] | Suction porosity, $p_{suc}$ [%][4] | | Macro porosity, $p_{air}$ [%][5] | | Mean | COV [%] | | |
| | | | Mean | COV [%] | Mean | COV [%] | | | | |
| Ulvin access tunnel (U) [6] | CEM II A-V 42.5: 391<br>CEM I 42.5 : 92<br>Micro silica fume: 26<br>Steel fiber: 35, 0.5 | 0.45 | 19.1<br>(18.7) | 7 | 4.2 | 25 | 0.18 | 24 | 180 - 850 | 78 |
| Gevingås rail tunnel (G) [6] | CEM II A-V 42.5: 513<br>Micro silica fume: 21<br>PP-fiber[8] : 7, 0.8 | 0.44 | 20.8<br>(19.3) | 12 | 4.6 | 19 | 0.18 | 18 | 360 - 1100 | 80 |
| Harangen road tunnel (H)[7] | CEM II A-V 42.5 : 502<br>Micro silica fume: 25<br>PP-fiber: 9, 1.0 | 0.46 | 21.1<br>(20.2) | 4 | 4.5 | 10 | 0.17 | 9 | 150 | 27 |
| Laboratory lining structure (F) [7] | CEM II A-V 42.5: 489<br>Micro silica fume: 26<br>PP-fiber: 7, 0.8 | 0.47 | 20.9<br>(20.5) | 8 | 4.9 | 13 | 0.18 | 14 | 150 - 540 | 49 |

[1] Calculated suction porosities according to classic concrete theory are shown in brackets

[2] Cement materials: CEM II A-V 42.5: Norcem Standard FA (fly ash cement), CEM I 42.5: Norcem Industrisement

[3] Including water added with the set accelerator during the spray application. Accounting for equivalent binder content: cement + 2·microsilica

[4] Pores which are saturated by storage under water at atmospheric pressure for 7 days

[5] Pores which are additionally saturated after storage under water at 50 bars pressure for 3 days

[6] Nominal concrete composition obtained from the batching plant log

[7] Actual fresh concrete composition measured at the batching plant

[8] PP: Structural polypropylene (macro) fibers

5.5 Moisture properties of sprayed concrete and sprayed membrane

The three properties pertaining to water absorption and water content of the concrete and membrane materials are illustrated in Figure 28.

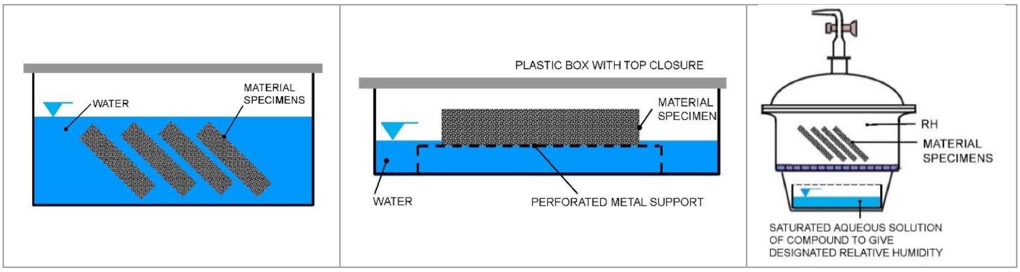



**Figure 28.** Conceptual diagram showing the three moisture properties which were tested for membrane and sprayed concrete materials. Left: water content at immersion at atmospheric pressure. Middle: Unidirectional water absorption (sorptivity). Right: Moisture retaining property at different air humidities obtained by isothermic desorption (representation as desorption isotherms)

The water content at immersion at atmospheric pressure can be regarded as the maximum water content the material can contain. For concrete this relates to the saturation of the capillary and gel pores, commonly referred to the suction porosity. The mean values for maximum water uptake at immersion for sprayed concrete and sprayed membranes M1 and M5 are shown in Table 16.

**Table 16.** Water contents at immersion at atmospheric pressure for sprayed concrete and membrane materials

| Material | Water content at saturation at immersion [1, 2] [kg/m$^3$] | Saturated density [2] [kg/m$^3$] | Ratio water content at immersion / dry material weight [%] |
|---|---|---|---|
| Sprayed concrete U | 191 | 2254 | 9.2 |
| Sprayed concrete G | 208 | 2239 | 10.2 |
| Sprayed concrete H | 211 | 2210 | 10.6 |
| Sprayed concrete F | 209 | 2225 | 10.4 |
| Membrane M1 | 368 | 1253 | 41.6 |
| Membrane M5 | 242 | 1127 | 27.3 |

[1] Saturation at immersion at atmospheric pressure means DCS = 100 % for the concrete and DS = 100% for the membrane materials

[2] Mean value for all tested specimens

The unidirectional absorption rate (sorptivity) expresses the speed of capillary absorption in concrete. For membrane material the water absorption is related to the water uptake in the polymeric structure of the material and possible molecular water uptake. The concrete specimens were dried at 60$^o$C for 7 days and membrane specimens had been stored at ambient temperature (20-23$^o$C) at RH 50-60% for approximately 1 year. The results of this test is normally presented as water uptake per area unit as a function of square root of time. A compilation of the measured values for sorptivity for sprayed concrete and sprayed membrane M1 is shown in Figure 29.

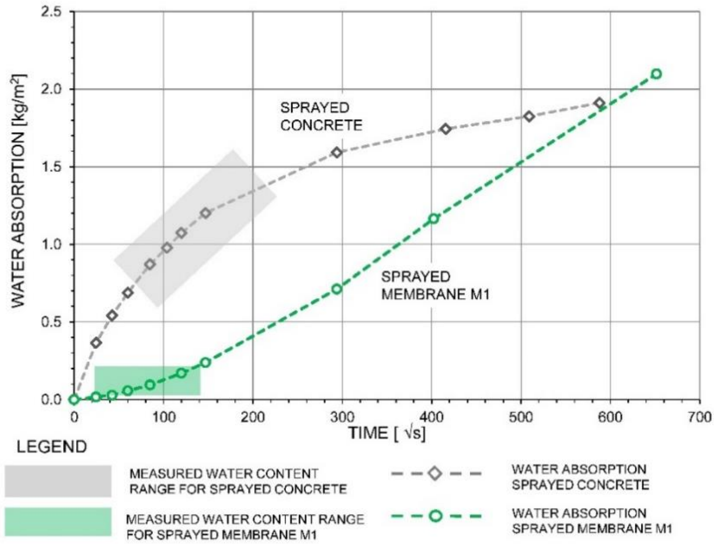



**Figure 29.** Results from unidirectional water absorption tests of sprayed concrete and sprayed membrane M1

The moisture retaining capacity expresses how much water a material will contain at equilibrium at a certain relative humidity. This was investigated by taking material specimens which were completely saturated at immersion and placing them in closed vessels (exsiccators) with a certain RH at constant temperature until mass equilibrium was reached. The result of this test is normally represented as a graph with moisture content (or degree of saturation) versus RH, commonly called a desorption isotherm. Desorption isotherms for sprayed concrete and sprayed membranes M1 and M5 are shown in Figure 30.

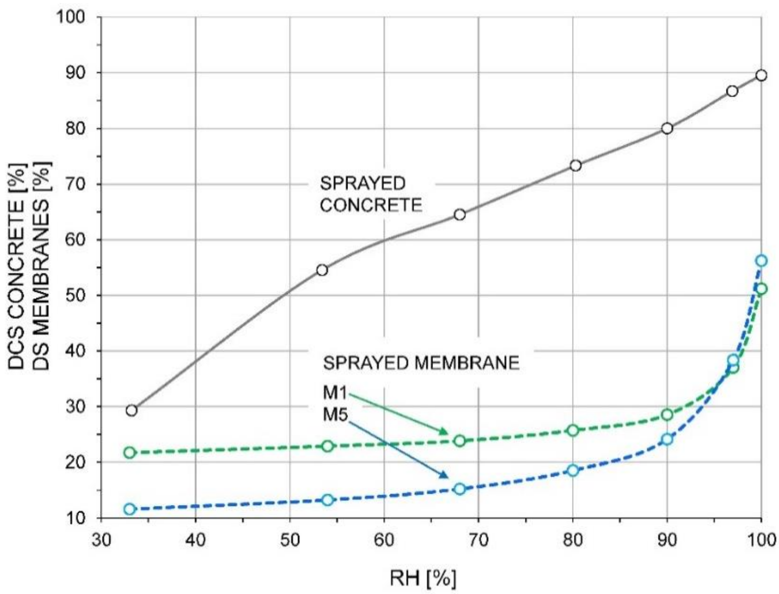



**Figure 30.** Desorption isotherms for sprayed concrete (from site H) and sprayed membrane obtained at 25ºC for different values of RH. Values are shown as degree of saturation at immersion (DCS for concrete, DS for membrane) versus RH

Both water and vapor permeability properties were investigated for sprayed concrete. For sprayed membrane the vapor permeabilities were investigated in this study. The water tightness of the membranes has been verified through external tests at the Swiss Federal Laboratories for Materials and Research EMPA and the Hagerbach Test Gallery VSH (EMPA 2002, VSH 2013, VSH 2015a, VSH 2015b).

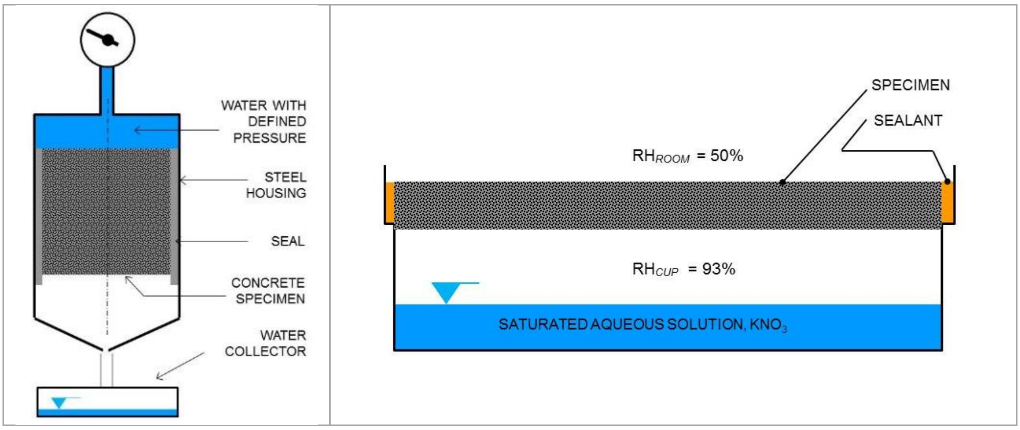



**Figure 31.** Conceptual drawings showing testing setup for hydraulic conductivity (conductive water permeability) and water vapor conductivity

Hydraulic conductivity of sprayed concrete was tested with a test set-up shown in Figure 31, left, which aims to reproduce the parameters in Darcy's law. Altogether 35 specimens from three different sprayed concretes obtained from in-situ tunnel linings were tested. The results, shown in Table 17, indicate that the intact sprayed concrete material has a very low permeability. For the specimens showing no measurable flow, the lower limit of measurement of water flow for the test setup was used.

**Table 17.** Compilation of test results for hydraulic conductivity $k_w$ of sprayed concrete specimens

| Site | Specimen | Measured $k_w$ [m/s] | Duration of pressure test [hours] |
|---|---|---|---|
| Ulvin (U) | U1-3 | $3.3·10^{-11}$ | 94 |
| | U1-5 | $3.6·10^{-12}$ | 94 |
| | U1-10 | $3.5·10^{-12}$ | 94 |
| | 12 specimens | No measurable water flow through specimens, $k_w < 5·10^{-14}$ m/s | 264-268 |
| Harangen (H) | H3 | $9.6·10^{-12}$ | 94 |
| | 9 specimens | No measurable water flow through specimens, $k_w < 5·10^{-14}$ m/s | 210-268 |
| Gevingås (G) | 10 specimens | No measurable water flow through specimens, $k_w < 5·10^{-14}$ m/s | 264-268 |

The vapor permeabilities were measured using the wet-cup method, illustrated in Figure 31, right. The results are shown in Table 18. The notable observation from this test is that the sprayed concrete and the tested sprayed membranes M1 and M5 exhibit vapor permabilities which are almost similar, or within the same scatter range.

**Table 18.** Measured vapor permeabilities for sprayed concrete and sprayed membrane

| Material, product | Measured vapor permeability | | Remark |
|---|---|---|---|
| | Mean [kg/m·s·Pa] | COV (%), number of specimens | |
| Sprayed concrete, Gevingås (G) | $0.74 \cdot 10^{-12}$ | 23 (6) | Fibre reinforcement with structural polypropylene, 6 kg/m$^3$ (0.7 % by volume) |
| Sprayed concrete, Harangen (H) | $2.25 \cdot 10^{-12}$ | 3 (5) | Fibre reinforcement with structural polypropylene, 9 kg/m$^3$ (1% by volume) |
| Membrane M1 | $0.87 \cdot 10^{-12}$ | 10 (5) | Sprayed |
| Membrane M5 | $1.01 \cdot 10^{-12}$ | 6 (5) | Sprayed |

The results from the moisture property and permeability testing can be summarized as follows:

- Intact materials of both sprayed concrete and the sprayed membranes M1 and M5 are practically impermeable when exposed to water at 500 kPa pressure
- Both materials sprayed concrete and membranes M1 and M5 can absorb significant amounts of water at immersion, but the membranes exhibit much higher water uptake at immersion, relative to its dry weight, compared to sprayed concrete
- Sprayed concrete exhibits higher sorptivity (unidirectional water absorption) than membrane (only membrane M1 tested)
- Sprayed concrete is a much more hygroscopic material than the sprayed membranes M1 and M5
- Sprayed concrete and membranes M1 and M5 exhibit water vapor permeabilities of the same magnitude

## 5.6 Moisture content in sprayed concrete and sprayed membrane

The moisture content of the sprayed concrete material in tunnel linings was investigated systematically for the two test sites Ulvin and Gevingås as well as the lining structure in the freezing laboratory. The found moisture contents in the tunnel linings were represented as moisture content profiles showing the DCS for the concrete obtained by splitting the core samples in pieces with approximately 40 mm length (Figure 32, top). The results show several consistent features. The lining at the Ulvin site, which was subject to precise management of the sprayed concrete application, shows a clear trend can be observed with a DCS close to 100% at the rock/concrete interface, and a gradient with decreasing DCS from the rock mass to the lining surface. This trend can also be observed for the lining at Gevingås, but with larger scatter, and a less distinct gradient with a drop in DCS.

The in-situ condition of the membrane material in the tunnel linings was investigated parallel to the concrete material by extracting pieces of the membrane from the core samples. The moisture content of the membrane could be tested for several of the linings at two intervals, with ages up to 37 months within this project. The results are shown in Figure 33. There are two noteworthy observations: firstly the relatively low water content in-situ in the membrane material, and secondly a trend in which the moisture content over time seems to reach 12 – 14 % . This corresponds to a degree of saturation of approximately 31%. The measured moisture contents of the membrane and concrete material in the tunnel linings are consistent with the laboratory results of moisture content at two materials at equilibrium at RH approximately 95-97%, which indicates a DCS of the concrete in the range of 85-90%.

The feasibility of realistic testing of the large scale laboratory lining structure depended on achieving a moisture content situation in the lining which was representative for the in-situ tunnel linings. The effect of the moisture conditioning which was undertaken by physically simulating water exposure by adding water to the lining through the rock mass is illustrated in Figure 34. The achieved moisture condition of the laboratory lining after 5 months of water exposure is very close to the measured moisture content of the Gevingås lining at 37 months age (Figure 32, right).

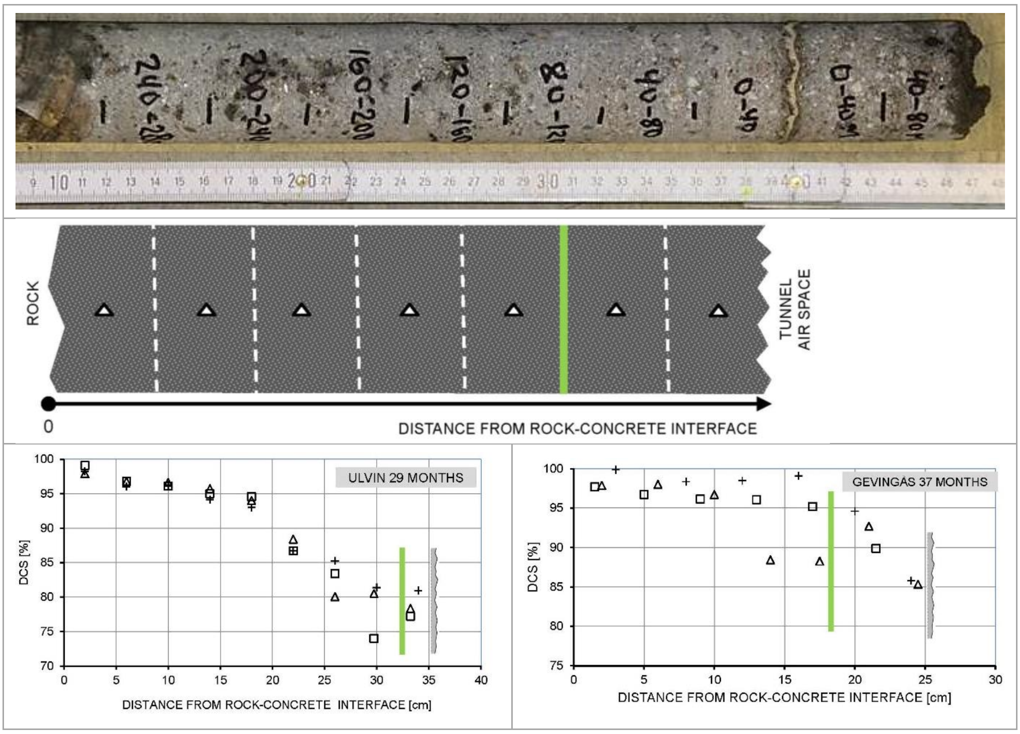



**Figure 32**. Measured moisture content of the sprayed concrete material in from the two sites Ulvin at age 29 months (bottom, left) and Gevingås at age 37 months (bottom, right) represented as DCS versus the distance from the rock concrete interface. Top images: Sectioning of core by mechanical splitting and assignment of data along the core. The membrane is indicated as the green line. Same symbols indicate the average moisture content over certain section of one core sample

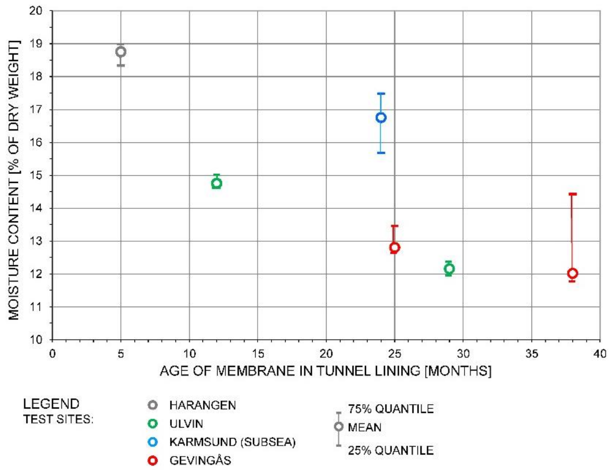
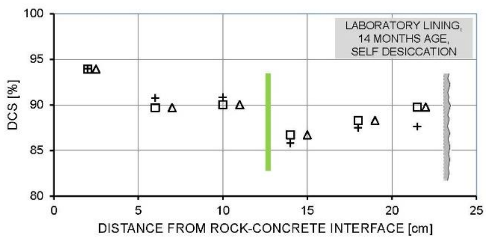



**Figure 33**. Measured moisture content in membrane material (M1) extracted from tunnel linings



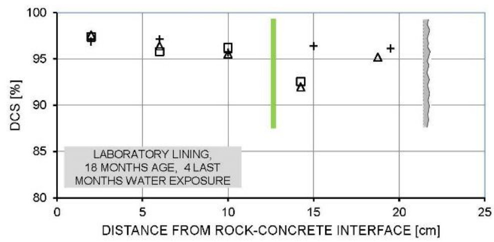




**Figure 34.** Measured moisture content of the sprayed concrete material in the lining structure in the freezing laboratory. Left: before moisture exposure. Right: after 5 months of adding water through drillholes in the rock mass (Section 4.4, Figures 23 and 24)

## 5.7 Moisture transport through sprayed concrete linings

The measured moisture contents exhibit a gradient from almost complete capillary saturation at the rock-concrete interface towards the lining surface, where significantly lower moisture contents have been consistently measured. It was assumed that this could be explained by moisture transport through the lining, in spite of the observed completely dry tunnel lining surfaces at the test locations. A hygrothermal simulation in the building physics software WUFI was executed in order to possibly explain this observation.

The numerical simulation requires a simplification of the material model and the boundary conditions. For this purpose, the rock-concrete interface was assumed to be kept constantly wet and the conditions at the tunnel lining surface were implemented in the numerical model with a climatic exposure with temperature and relative air humidity, illustrated in Figure 35, left. The material parameters required for the numerical model were all measured.

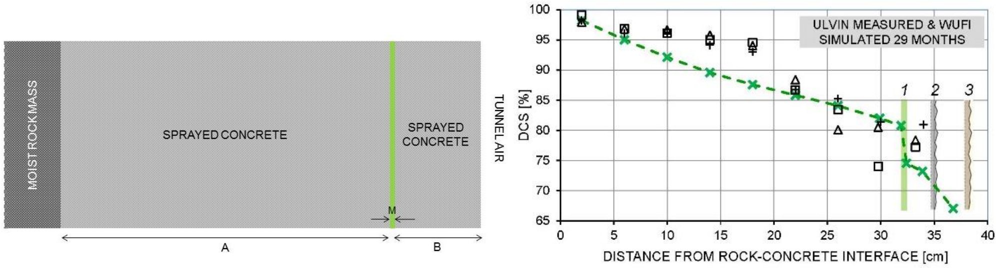



**Figure 35.** Numerical simulations of moisture transport through SCL in the software WUFI. Left: simplified material model with boundary conditions. Right: comparison of simulated (dotted green line) and measured moisture contents for the Ulvin test site at age 29 months. The location of the membrane is indicated with green line (1). The range of the real location of the lining surface for the field results is indicated with (2) and (3)

The hygrothermal simulations produce moisture content profiles which show the main features of the measured moisture profiles (Figure 35, right). The numerical simulation does not account for the exposure of liquid water through cracks and imperfections. Hence, the higher measured values in the part of the lining closest to the rock, can be explained by a likely higher exposure to water through the cracks in the first layer (approximately 15 cm thickness) of sprayed concrete. The numerical model also quantifies the moisture transport through the lining with the given conditions.

## 5.8 Freeze-thaw resistance of the sprayed concrete material in SCL

Freeze-thaw resistance of the sprayed concrete was assessed using three direct freezing testing methods, as well as indirect assessments by air void analyses, illustrated in Figure 36. The two most commonly used direct freezing testing methods in Scandinavia (resistance to salt scaling) and North America (resistance to rapid freeze-thawing and resistance to salt scaling) were selected. In addition testing of freezing induced dilation at a certain degree of capillary saturation (following a not renewed standard test) was executed in order to document the effect of one freezing event with realistic, but worst case, water content in the sprayed concrete material.

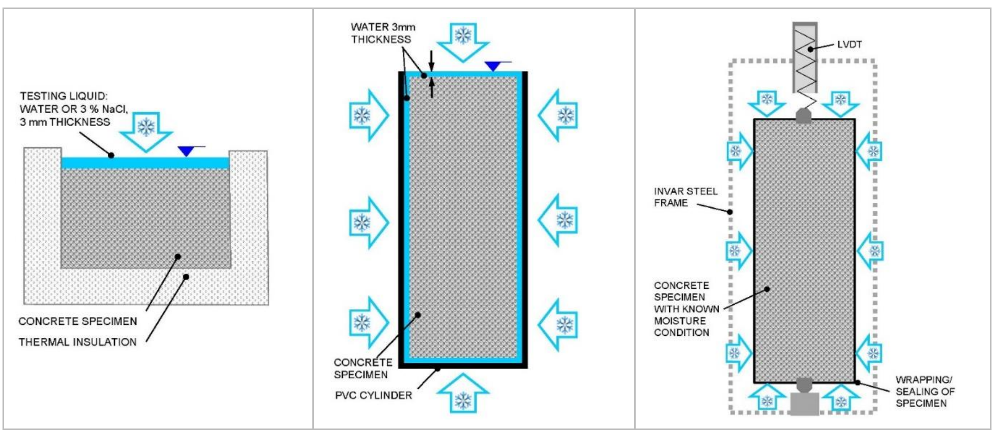



**Figure 36.** Conceptual drawings showing freezing exposure modes for the three executed direct freezing tests. Left: Resistance to salt scaling (CEN-TS 12390-9). Middle: Resistance to rapid freezing and thawing (ASTM C666 procedure A). Right: Measurement of freezing induced dilation (ASTM C671-86)

Air void parameters can give indication of freeze-thaw resistance of concrete. Sufficient air void volume with a favorable spacing between the air voids is used as a freeze-thaw resistance quality parameter. The PF method suggests a certain ratio of the air void volume to the suction porosity as an indicator of freeze thaw resistance.  The application method of sprayed concrete produces a different air void structure than for cast concrete, and apparently with air void volume within a certain range. This is visualized in Figure 37.  Several studies find that the adding of air entrainment agent does not influence the air void volume for sprayed concrete as it does for cast concrete. The measured air void parameters for sprayed concrete as shown in Table 19.

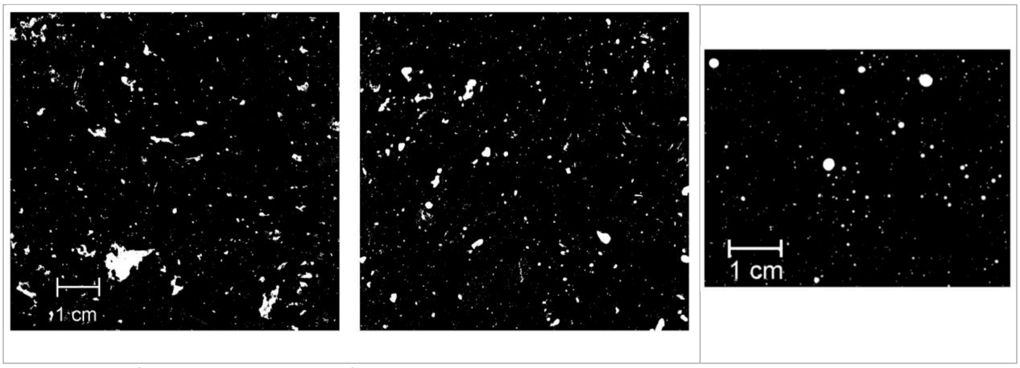



**Figure 37.** Left and center: images of air voids in hardened sprayed concrete (type H) obtained by an optical contrast enhancing procedure. No air entrainment agent has been added to this concrete. The two photos are to scale. Right: Image of spherical air voids produced by the adding of air entrainment agent in cast mortar (right photo: Fonseca & Scherer 2014)

**Table 19.** Results from air void analyses of sprayed concrete material

| Method | Parameter | Mean | COV [%] | Minimum - maximum | Number of specimens |
|---|---|---|---|---|---|
| Optical image analysis/ ASTM C457 | Air void volume [%] | 5.7 | 21.4 | 2.9 – 6.1 | 5 |
| | Powers' spacing factor [mm] | 0.27 | 35.1 | 0.22 – 0.46 | 5 |
| | Paste/air ratio | 7.2 | 41.4 | 6.6 – 14.5 | 5 |
| | Specific air void surface [$mm^2/mm^3$] | 20.2 | 19.8 | 13.6 – 24.3 | 5 |
| PF Method | Air void volume, $p_{air}$ [%] | 4.5 | 9.8 | 4.0 – 6.0 | 27 |

The main findings from the freeze-thaw resistance testing of sprayed concrete according to standard methods can be summarized as:

- The standard test methods for freeze-thaw resistance salt scaling and rapid freezing/thawing represent very severe exposure conditions compared to realistic and worst case exposure conditions in a tunnel lining
- Sprayed concrete fails according to these tests
- The freezing dilation tests do not show any deleterious strains in the concrete material following conditioning with 9 months immersion in water
- The measured porosity characteristics, air void content, spacing factor and Protective pore Factor (PF) indicate a certain freeze-thaw resistance.

### 5.8.1 New functional freeze-thaw performance test for tunnel linings

A new procedure was developed in order to execute testing of freeze-thaw resistance under realistic exposure conditions which the tunnel lining represents. The new test is a functional performance test which aims to simulate a realistic freeze-thaw exposure mode with conservative exposure parameters.

The test is laid out as a two-step procedure:

- Freeze-thaw exposure with pre-determined number of cycles of core specimens which have been pre-conditioned to a certain moisture content
- After the cyclic freeze-thaw exposure, the specimen is tested for damage

The set-up for the freeze-thaw exposure part of the procedure is shown in Figure 38. Further details of this method is given in Paper 4, Section 5.

The new test shows promising results. Cyclic freeze-thaw thaw exposure could be precisely controlled and was successfully reproduced. Testing of freezing damage was done by measuring resonant frequency on disc-shaped specimens which were cut from the core sample which was exposed to cyclic freezing/thawing in the first part of the procedure.

The experience with this method is still limited since only 8 core specimens have been completely tested with this method, of which all passed the test. Further testing of different materials, particularly materials which are likely to fail the test, is required to precisely define acceptance criteria and refine the testing parameters.

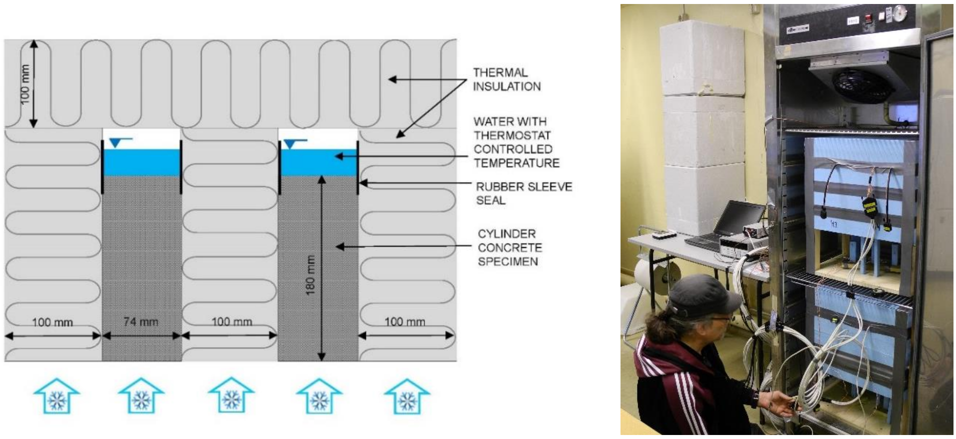



**Figure 38.** Functional performance freeze-thaw test for tunnel lining conditions. Left: Drawing with vertical section through the apparatus and indication of freezing exposure from below. Right: Testing in progress with two testing assemblies placed in freezing cabinet for controlled thermal exposure

The investigation of freeze-thaw resistance of sprayed concrete in tunnel linings shows that testing according to the most commonly used standards will give the result "not frost resistant". Testing with the freezing-dilation procedure (ASTM C671) and the new developed test in this study, applying conservative conditions regarding moisture content, cooling rates and thermal gradients, the main finding of this investigation is that the sprayed concrete is not prone to freeze-thaw damage in a tunnel lining. The results of the freeze-thaw resistance investigation of the sprayed concrete is summarized in Table 20.

**Table 20.** Main results from the investigation of freeze-thaw resistance of sprayed concrete

| Test | | Result | Conclusion |
|---|---|---|---|
| Type of test | Applicable standard | | |
| Resistance to rapid freezing and thawing | ASTM C666 procedure A | Severe damage at 97 of 300 required freeze-thaw cycles | Failed |
| Resistance to salt frost scaling | CEN-TS 12390-9 | Severe scaling with 3% NaCl as test liquid. Insignificant scaling with distilled water as test liquid | Failed |
| Freezing induced dilation | ASTM C671-86 | Very low measured freezing induced dilation with specimens conditioned to DCS > 97% | Passed |
| Pore Protection Factor | SINTEF (1988) | 0.17 – 0.19 | Partially passed |
| Air void spacing Powers' spacing factor | ASTM C457 (enhanced image procedure) | 0.22 – 0.49 mm | Partially passed |
| Air void volume | ASTM C457<br>SINTEF 1988 | 3 – 6 %<br>4 – 6 % | Partially passed |
| Tunnel lining performance freezing test | Developed during this work | No measureable damage after 50 freeze-thaw cycles | Passed |

## 5.9 Mechanical properties of the tested membranes

The detailed testing program was laid out according to the loading model. The initial performance test for membranes was a standard elasticity test, in order to establish basic elasticity properties and roughly assess the suitability of the purpose of waterproofing leaking cracks in a sprayed concrete lining. The applied test procedure for this purpose is the standard elasticity test for rubber materials DIN53504. For sprayed EVA membrane specimens this test is however vulnerable to large scatter due to variations in specimen thickness, curing conditions, temperature and moisture conditioning. The results from the series shown in Figure 39 refer to membrane specimens which were produced by spraying within a few hours at the same location and been subject to an identical handling, storage and conditioning prior to testing. The test results show that membrane products with low polymeric contents exhibit low elasticity.

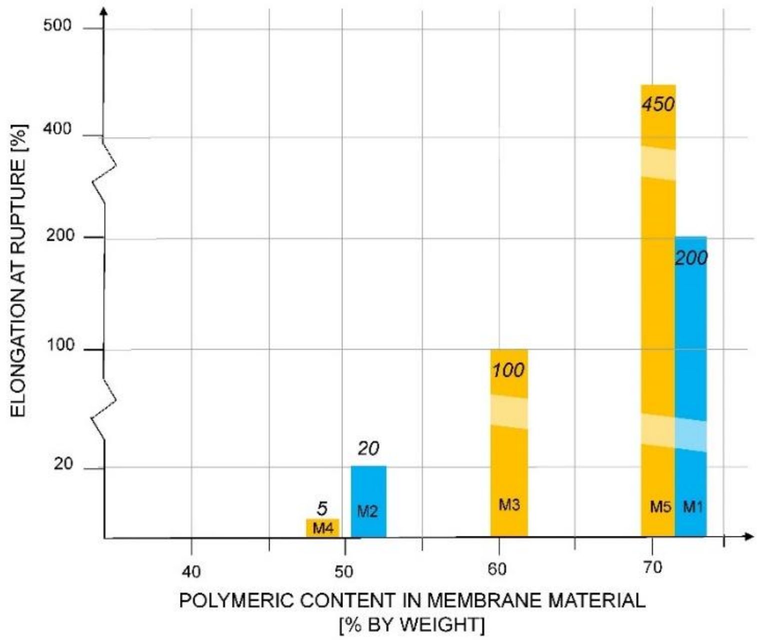
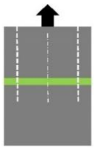



**Figure 39.** Measured elongation performance for five different sprayable membrane products according to DIN 53504 versus polymeric content measured by thermo-gravimetric analysis. The two colors indicate the two different suppliers of membrane products

The results from the testing of the main properties are shown in the compilation in Table 21. The acceptance criteria, shown in Table 22, are in accordance with the loading model (explained in the thesis Section 5.3, Table 14 and in Paper 5, Sections 4.6, 6.4, 6.5 and 6.6)

The results for the tensile testing on membranes show values lower than 0.5 MPa for the membranes M2, M3 and M4. Together with the results from the elasticity testing, showing low values for elasticity for these three membranes (Figure 39), and the poor in-situ waterproofing performance of membrane M4 (shown in Section 5.10) it was decided to discontinue the testing of the membranes M2, M3 and M4. The membranes M1 and M5 were continued for the complete test program.

**Table 21.** Compilation of main test results for sprayed membranes

| Performed test | | Specimens, testing conditions | Tested membrane products, main results |
|---|---|---|---|
| Pull test, tensile strength [MPa] |  | Testing of core specimens. Several types of pre-treatment:<br>▪ Dry<br>▪ Immersion<br>▪ Frozen/thawed | Laboratory results:<br><br>Dry / Moist / Frozen/thawn<br>M1 1.1-1.6 / 0.3-1.15 / 0.3-0.85<br>M2 – / 0.25-0.7 / 0.1-0.55<br>M3 0.2-0.4 / 0.3-0.45 / 0.2 – 0.4<br>M4 – / 0.25-0.4 / 0.15-0.5 |
| |  | In-situ test method for lining structure<br>▪ In-situ tunnel<br>▪ Large scale lining structure<br>▪ Slabs of lining structure<br>▪ Different freezing exposure | Lining structure test results:<br>M1, tunnel test sites: 1.1- 1.6<br>M1, after 35 cycles to -3ºC [1] 1.1-1.2<br>M1, additional 20 cycles to -7 ºC [2] 0.4-0.7<br><br>Lining structure slabs<br><br>Dry / Moist / Frozen/thawn<br>M1 1.25-1.3 / 0.6-1.1 / –<br>M5 0.58-0.85 / 0.55-0.7 / 0.4-0.6 |
| Shear testing |  | Direct shear testing on core specimens in large scale shear box<br>▪ Pre-treatment of specimens by immersion | Shear deformation at initial rupture [mm] / Shear stiffness [MPa/m] / Peak shear stress [MPa]<br><br>M1 1 / 350 / 0.75<br>M5 1 / 297 / 0.45 |
| Crack bridging |  | Crack bridging (w/t –ratio [3] at different temperatures) | Testing temperature [ºC] 23 / 0 / -3 / -8<br><br>M1 1.2 / 0.8 / 0.6 / 0.7<br>M5 1.6 / 1.4 / 0.4 / 0.7 |

[1] Minimum temperature -3ºC and maximum temperature 7ºC at the membrane location in the lining structure during freeze-thaw cycles each lasting 48 hours
[2] After the initial 35 freeze-thaw cycles to -3ºC, additional freezing to -7ºC was executed
[3] w/t-ratio = ratio of crack width at rupture w to membrane thickness t

1. **Table 22.** Proposed acceptance criteria for mechanical parameters for bonded membranes [1]

| Parameter | Test procedure | Acceptance criterion | Remark |
|---|---|---|---|
| Elasticity | DIN 53504 | 150% | Testing at 20ºC , initial qualification of product |
| Tensile bond strength | In-situ pull test | 0.5 MPa | No significant deterioration due to or age or freezing exposure |
| Shear elasticity | Direct shear | 1 mm shear deformation without signs of initial failure | Normal load 0.5 MPa |
| Crack bridging | Dynamic loading, controllable crack width | w/t = 1 at 20ºC<br>w/t = 0.5 at -3ºC [2] | Minimum membrane thickness in the range of 3 to 4 mm |

[1] General remark: all test specimens to be tested with realistic moisture content. Moisture content to be tested and reported with measured result
[2] or testing at lowest exposure temperature which is expected at the location of the membrane in the lining

## 5.10 Results and experiences from constructed test sections

The construction of the test sections with SCL followed the recommendations from the suppliers of the membrane products. The target result for the watertightness of the lining was a completely dry lining surface. The resulting condition of the lining surface in terms of watertightness for the test sites is summarized in Table 23. The locations for water pressure testing and lining sampling with core drilling were dry. The typical initial seepage condition for the test sites and typically remaining wet spots before any local post injection works is shown in Figure 40.

The applied construction methodology consistently left some minor damp spots in the test sites. This was also the experience during the construction of the Gevingås rail tunnel 2010-2011. For this project nearly all damp spots were removed by local injections, leaving the major portion of the tunnel completely dry. For the test sections Ulvin and Karmsund no injections were carried out, which caused some remaining damp spots.

The experience with the construction method suggested by the suppliers at the time of construction is that under favorable conditions with a minor extent of drip spots through the primary lining, a completely dry result is possible with a minor effort of point injections through the final lining. For areas with denser drips, the effort of wet spot handling in form of temporary drainage works and the amount of remaining seepage points which require injection may be higher. Experiences from the recent construction of the Holmestrand rail tunnel (2014-15) with a large number of dense drip spots through the primary lining show that a completely dry result is feasible with a significant effort of wet spot treatment (drainage and injection). The final result of this case is shown in Section 2.4, Figure 13 right. For ground conditions which exhibit only minor seepage, in the form of a few drips, a completely dry result is feasible with minor effort in the form of temporary drainage works and local injections through the final lining. The Gevingås rail tunnel represents such a case, and is shown in Section 2.4, Figure 13 left.

**Table 23.** Achieved watertightness for the test sites for SCL with bonded membrane

| Site, membrane tested | Condition at testing area before application | Condition at testing location at site | Total result for test site | Tunnel length covered | Injections for removal of remaining wet spots |
|---|---|---|---|---|---|
| Ulvin, M1 | 50-60 dense drip spots | Completely dry | 7 minor damp spots | 30 linear meter tunnel | no |
| Ulvin, M4 | 50-60 dense drip spots | Large number of seepage points | 7 drip spots, 43 damp/moist spots | 30 linear meter tunnel | no |
| Karmsund, M1 | 32 scattered drip spots | Completely dry | 9 damp spots | 24 linear meter cavern | no |
| Gevingås, M1, tunnel lining investigation | Scattered drip spots | Completely dry | 1-2 damp spots per 100 m | >500 linear meter tunnel | yes |
| Gevingås, M1, water pressure monitoring | Dense strong drip spots | Completely dry | 3 damp spots | 30 linear meter tunnel | yes |
| Forsmark, M1 | Scattered drip spots | Completely dry in crown, damp spots in wall | 4 damp spots | 20 linear meter tunnel | yes |


**Figure 40.** Constructability issues for SCL with bonded membrane experienced at test sites. Upper photos: seepage condition through the rock support lining before applying the membrane, left: Ulvin; right: Forsmark. Lower photos: typical experienced leaks through inner lining prior to any post injection works, left: Ulvin M1 damp spot in crack, right: Forsmark, leak around rock bolt

Favorable results, with minor efforts of post-injection, are achieved when the sprayed concrete substrate is smooth facilitating the application of a continuous membrane minimal risk of leaving spots with insufficient membrane coverage.

The current state of working methodology for the tested sprayed membranes show that the application technology is sensitive to several conditions in a tunnel. This can be overcome by strict and extensive quality control procedures. A further development of membrane products less sensitive to moisture in the substrate and improved working methodology which is less sensitive to variable workmanship is required.

## 5.11 Health and safety: monitoring of dust during membrane application

During the spray application of the membranes M1 and M4 at the Ulvin test site, dust measurements were carried out at three different working positions for the application crew. The reason for conducting such measurements was to assess the working hazards related to the dry spraying of the membrane products, particularly the risk of excessive dust concentrations. The spray application of the membranes for this test site was done by manual handheld application, which represents a more severe working hazard exposure than e.g. spraying with a robot manipulator.

The working situation for the spray application works is shown in Figure 41. The ventilation situation was unfavorable with the location of the ventilation duct approximately 30 m behind the location of the pump. This produced a very turbulent air flow situation around the test site. An air flow speed at the invert level of approximately 0.3 m/s was recorded.

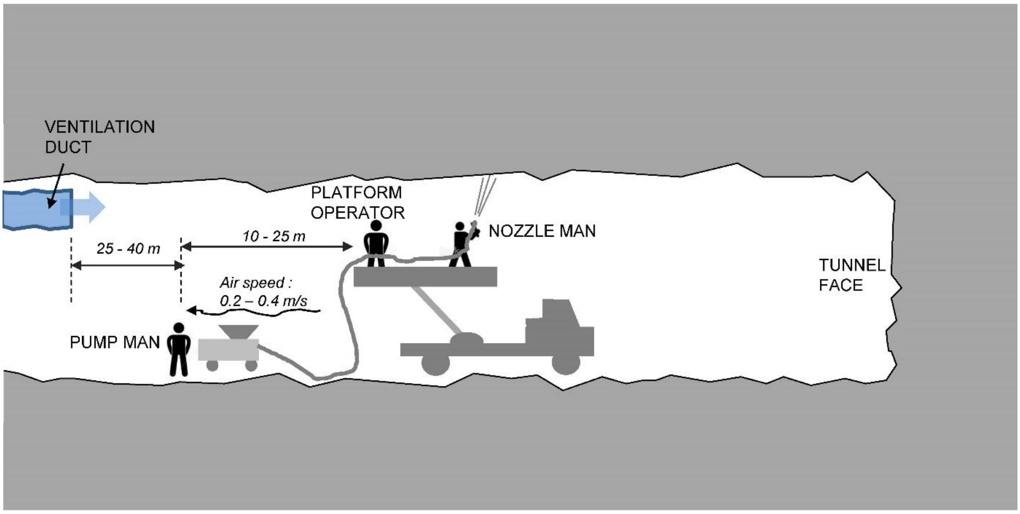



**Figure 41. Set**-up and organization of membrane spray-application and measurements of dust at three different working positions

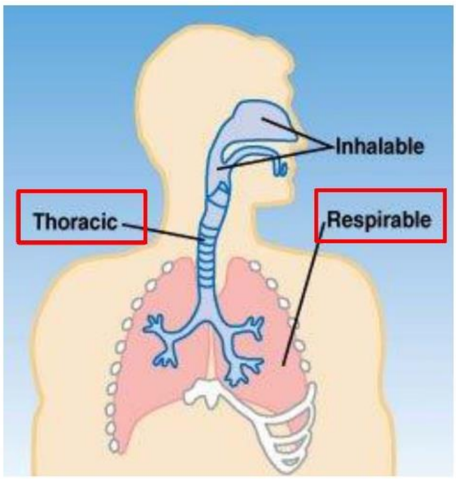



**Figure 42.** Measured types of dust

Respirable and thoracic dust (explanation in Figure 42) was measured for the pump man, the nozzle man and the platform operator. The results are shown in Table 24. The values for respirable dust are high with the mean values slightly below the norm of 5 mg/m$^3$. However some of the maximum recorded values are above the norm. For thoracic dust there is no administrative norm. The recorded thoracic dust concentrations are however high.

**Table 24.** Results from dust measurements during the membrane spray-application at the Ulvin test site

| Working situation | Number of measurements | Thoracic dust ($mg/m^3$) | | | | Respirable dust ($mg/m^3$) | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Average | Mean | Min. | Max. | Average | Mean | Min. | Max. |
| Nozzle man | 4 | 4.26 | 9.00 | 4.23 | 15.78 | 2.05 | 1.79 | 0.55 | 4.09 |
| Platform operator | 4 | 9.71 | 8.44 | 0.95 | 21.03 | 4.15 | 3.48 | 0.58 | 9.06 |
| Pump man | 4 | 9.50 | 9.00 | 4.23 | 15.78 | 3.97 | 3.54 | 1.10 | 7.70 |

Although the ventilation situation experienced at this test site was unfavorable and can be improved for larger applications, the measured dust concentrations suggest that this spraying method, product handling and equipment setup gives a too high exposure to dust.

## 6 SYNTHESIS OF FINDINGS AND DISCUSSION

### 6.1 Verification of main conceptual model

The continuously bonded property of the SCL structure from the rock surface through all the constituent layers, means that there is a mechanical and hygric continuity through the entire lining structure. Consequently, the exposure to the ground water at the concrete-rock interface is a critical boundary condition for the lining system. The lining will be exposed to water along conductive sections of rock joints. Capillary absorption through the concrete material and conductive flow in cracks in the concrete where such cracks intersect the conductive rock joints will be the sources of water exposure to the primary sprayed concrete lining. The moisture condition and moisture transport processes are summarized in Figure 43.

The SCL structure with a bonded EVA based membrane in principle represents a concrete structure with a certain type of barrier to transport water. In this case the membrane constitutes a barrier to conductive water flow, whereas it poses no barrier to vapor transport. The moisture transport property of the lining structure is found to have a critically important effect on the moisture condition of the different parts of the lining. The main effect of the moisture transport property is that major parts of the lining will remain unsaturated and unable to transmit a hydraulic pore pressure. A general hydraulic pore pressure acting from the concrete material on the membrane surface can therefore be ruled out.

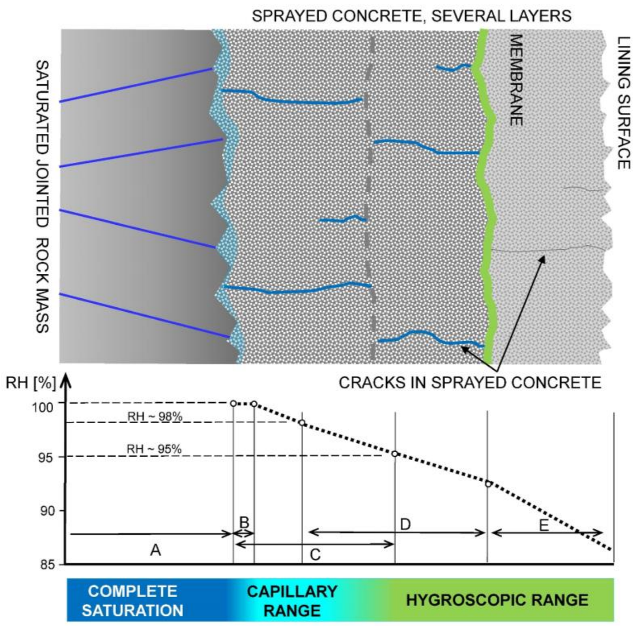



**Figure 43**. Dominant water transport mechanisms in the tunnel lining

Explanation to the water transport mechanisms in Figure 43:

A: Saturated conductive flow on rock joints

B: Capillary and possibly saturated conductive flow in concrete, conductive flow on cracks in concrete

C: Capillary flow and vapor transport in concrete, conductive flow on cracks in concrete

D: Vapor transport in concrete, conductive flow on cracks in concrete

E: Vapor transport in concrete and membrane

Groundwater pressure, although low, will lead to an increased saturation of the cracks and imperfections in the sprayed concrete in the primary lining, and therefore lead to a slightly higher degree of saturation of the concrete material, than in cases which have no groundwater pressure. The field investigation results and the measured moisture absorption and moisture transport properties of the sprayed concrete and membrane materials suggest that even under relatively high degrees of capillary saturation of the sprayed concrete (in the order of 95%), the membrane will exhibit sufficiently low in-situ moisture content to maintain high tensile bond strengths, sufficient elasticity and shear deformability.

The external mechanical loads on the lining, including effects on the immediate rock mass, caused by the undrained situation in the walls and the crown have been found to be negligible under the investigated conditions with hydrostatic pressures up to approximately 800 kPa and hard crystalline rock conditions.

The mechanical loads acting on the membrane directly are found to be shear deformations between the concrete layers on either side of the membrane which may cause elongation of the membrane over cracks, as well as shear strains in the membrane. The expected shear deformation can possibly lead to shear strains in the range of 0.5 - 1 mm/m and crack openings in the range of 0.2 to 0.8 mm.

No clear model for the tensile loading of a certain magnitude of the membrane could be established based on this study. It is therefore impossible to define an accurate criterion for required tensile strength based on a clearly estimated load. In order to consider the SCL structure having a mechanical integrity the minimum required tensile bonding strength for the other parts of the lining is proposed. The tensile bond requirement between sprayed concrete and the rock surface used in Norway and Sweden is 0.5 MPa. ITA/AITES 2013 proposes 0.5 MPa a minimum required tensile bond strength for the membrane. In this study no results indicate that this should be reconsidered.

The thermal exposure at the lining surface in the tunnel will be unique for each tunnel case. The minimum temperatures at the membrane location in the lining with a given outdoor climate will depend on the distance to the portal and the ventilation characteristics of the tunnel. The possible area of use of the SCL system under a cold climate, needs to be considered based on the longitudinal profile with the lowest possible temperatures.

The maximum groundwater pressure exposure to the linings under the tested rock mass conditions with hydrostatic pressures in the range of 680 to 780 kPa is likely to be in the range of 200-300 kPa at the rock/lining interface. A likely cause of the reduced groundwater pressure near the lining interface is the effect of the EDZ with a significantly increased hydraulic conductivity. For rock masses not exhibiting an EDZ with a significantly higher hydraulic conductivity, the numerical models suggest an increase in water pressure in the rock mass at decreasing distance to the lining. For the linings which could be tested for moisture content and in-situ tensile strength, only the lining at the Gevingås test site can be assumed to have been exposed to a certain water pressure, most likely in the order of 200 kPa.

## 6.2 Conditions for the moisture exposure of the membrane

The performance of EVA based membranes have been found to be sensitive to moisture exposure. The found moisture contents in the linings and the moisture transport mechanisms represent the basis for the favorable system behavior of the lining. The moisture properties of both concrete and membrane materials are therefore critical for this finding. Any significant changes to the vapor conductivity of the materials and possible access to liquid water through the concrete material may change the entire performance of the lining. The high vapor permeability of the membrane allows for moisture transport through the membrane, and the low hydraulic conductivity of the intact concrete material prevents a general exposure to liquid water on the concrete/membrane interfaces. The effect of the vapor permeability of the membrane is illustrated in Figure 44.

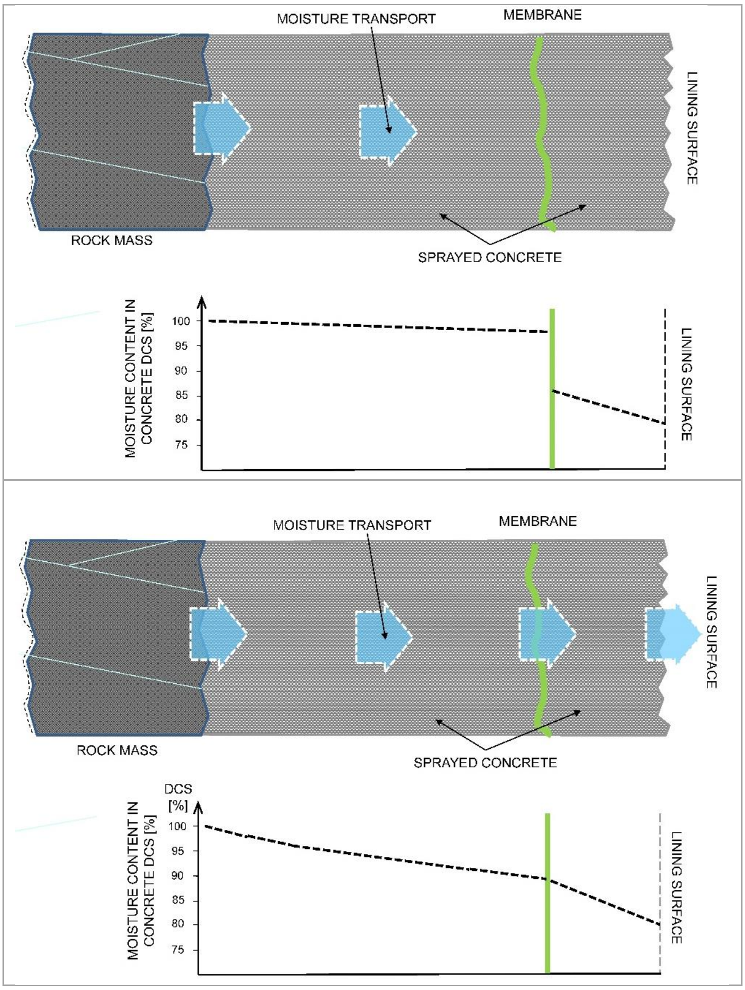



**Figure 44**. Interpreted main effect of a vapor permeable bonded membrane avoiding a long term saturation of the membrane material itself and the concrete material on the rock side of the membrane. Top: bonded membrane with low vapor permeability acting as barrier to vapor transport. Bottom: vapor permeable membrane which results in a long term unsaturated condition of the concrete material on the rock side of the membrane.

A membrane with low vapor conductivity will tend to lead to a long term saturation of the concrete on the rock side of the membrane. A membrane with a vapor conductivity of the same magnitude as the sprayed concrete will avoid a long term saturation of the concrete behind the membrane. A sprayed concrete material or any other constructed material exhibiting high hydraulic conductivity which interfaces the membrane, would expose the membrane to liquid water. This would possibly cause a water absorption which is higher than the favorable range and may potentially lead to a softening of the membrane and a reduced tensile strength. For this reason the moisture properties of the materials used in the substrate need to be fully understood in order to provide favorable moisture exposure.

## 6.3 Considerations regarding durability

The moisture condition of the lining materials is found to be a decisive issue for the lining properties. The moisture condition in the lining structure may change by introducing different constituent materials, which can lead to a different response to the water exposure or a substantial change of the water exposure to the lining. The consequences of any substantial changes need to be evaluated.

The found moisture condition of the entire lining system with the tested materials and conditions (Figure 34, Section 6.1) indicates that the membrane will be located in a favorable situation and be "protected" from degrading mechanisms such as delamination due to pore pressure in the concrete, softening of the membrane and reduction of tensile bond strength due to saturation of the membrane.

The problem which freezing exposure can possibly create for such linings is that of freezing damage to the lining structure caused by the formation of ice from the water contained within the lining. Formation of ice at the lining surface poses no potential problem with this lining system with a dry lining surface. The study has been aimed at detecting if the observed water contents in the concrete and membrane materials can possibly lead to freezing damage. The membrane material exhibits elongation and crack bridging capacity which is temperature sensitive. For this reason the thermal exposure should avoid being below **-** 3°C.

Results from the freeze-thaw testing of the concrete material and the membrane-concrete interfaces indicate good freeze-thaw resistance. This is probably due to the unsaturated condition of the two materials which allows formation of ice to take place without creating damage.

These considerations relate to the condition of the two materials in a continuous composite structure and do not take into account all possible effects of water exposure through the cracks. Cracks in the concrete will intersect the membrane in lines with thickness corresponding to the crack width and expose the membrane directly to water at these locations, to the extent that the concrete cracks are fed with groundwater from the conductive rock joints. The distance between the cracks which could be visually detected was found to be in the range of 0.2 – 1.5 m, with 0.7 to 1 m as the most represented distance. During the core drilling for sampling no cracks on inside of the membrane could be found. None of the in-situ tensile tests, which were all drilled at a perfectly intact and dry lining surface, showed any low results for tensile strength.

An important issue to consider, is how a high water pressure exposure to the lining could possibly lead to any degrading mechanisms. This is discussed in Section 6.7.

The series of crack bridging tests conducted with high water content in the membrane, showed high elasticity at crack opening, even at -3°C. This indicates that the high water content in the membrane at the crack locations will function as a softener and increased crack bridging capacity also at low temperature.

In this study no analysis of the effect of geochemical exposure was included. Under conditions with chemically aggressive groundwater possible chemical degradation needs to be investigated. The EVA based membranes have been reported to have a high resistance to inorganic chemical exposure, but poor resistance to exposure to hydrocarbons.

A precise assessment of service lifetime is impossible to make based on the findings in this study.

## 6.4 Effects of sprayed concrete material quality in the lining

The long term durability of the SCL system very likely depends on creating favorable moisture exposure conditions which are illustrated in Figure 43. Irregularities and imperfections in the lining will to some extent compromise this and are impossible to avoid, but can be reduced to a certain extent. Such irregularities are:

- Cracks in the concrete
- Irregularities in the form of large holes or voids or concrete with higher porosity due to the sprayed application process (covering of rebound, poor nozzle angles, occurrence of spraying "shades" during high excavation surface roughness/irregularities)
- Use of materials in the regulating layers which, in spite of producing a good finish, can result in higher cracking and porosity and hence, a higher occurrence of water at the membrane/concrete interface

Such irregularities can be addressed by measures such as:

- higher fiber dosage in the sprayed concrete to achieve better distribution of cracks and smaller individual cracks
- low water/binder ratios in the sprayed concrete
- improved concrete spray-application to produce a more regular surface and reduce irregularities caused by poor application

## 6.5 Constructability issues for the membrane

With the state-of-art of the sprayed membrane technology at the time of writing, there are several constructability issues which need to be addressed in order to facilitate the construction of the SCL system. Several of these issues relate to conditions of the tunnel with its excavated surface, water ingress and sprayed concrete primary lining. These issues are discussed in section 6.6. This thesis shows that the membrane material in constructed in-situ condition in the lining shows favorable system properties. In this study it was not possible to investigate and develop the spray application details for the membrane materials. Today's membrane application technology makes it feasible, but under several circumstances difficult to construct the complete SCL system successfully. The application methodology needs improvements in the following main fields:

- Improved spraying methodology to achieve consistent membrane thickness and material quality
- Improved spraying methodology to achieve better working conditions from a health and safety perspective

- Improved control procedures to verify the applied coverage and thickness of the membrane
- Membrane materials which are less sensitive to direct water exposure in fresh (wet) condition during and immediately after the spray application

## 6.6 Implications for tunnel lining design and specification

The testing of the lining structure in this study was unable to address all the imperfections and irregularities. Favorable system properties have been substantiated under ideal conditions. Providing favorable conditions for the lining system to function according the established properties is a consequence of these findings. Such conditions relate to the factors which maintain a low direct water exposure of the membrane and minimize the loads on the membrane. Construction of the SCL system with bonded membrane will benefit from specifications of the excavation and support of the tunnel which are aimed at producing favorable conditions. This is illustrated in Figure 45. The parts of a specification for the construction of a tunnel which can be aimed at improving the feasibility of the SCL with bonded membrane are the following:

- Pre-grouting of the rock mass to reduce the hydraulic conductivity and hence, reduce the number and magnitude of water ingress points
- A stricter requirement for the pre-grouting in the upper part of the surrounding rock mass (indicated as $k_1$ in Figure 45) giving a higher tightness than in the rock mass below the invert (indicated as $k_2$ )
- Contour blasting with strict requirements for the contour quality
- Rock bolt holes to be sealed (no seepage) before installation and complete grouting
- Rock bolts to be completely covered/embedded in the sprayed concrete in the primary lining
- Sprayed concrete mix designs and application details aimed at producing a surface with low roughness and minimal occurrence of inferior areas such as void areas with high porosity
- Specification of sprayed concrete layer thicknesses of the primary lining in order to produce favorable conditions for the membrane as illustrated in Figure 43

The afore mentioned points all represent existing technology which can be realized by developing the specifications, acceptance criteria, quality control procedures and allocation of the contractual responsibility for the result of the application.

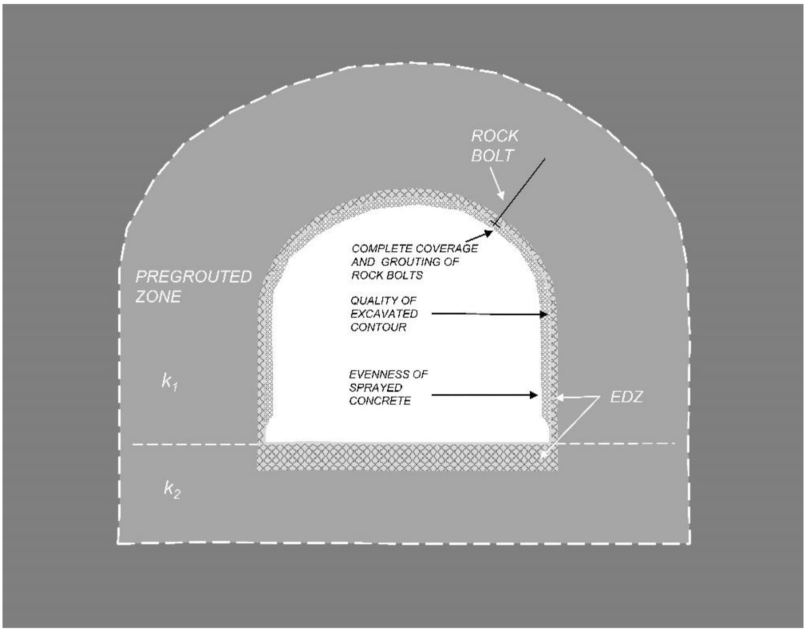



**Figure 45**. Conceptual drawing showing parts of the tunnel construction process which can be optimized to improve the conditions for the SCL system

6.7 Effects of water pressure in the rock mass close to the lining,

The material model for the lining stated in this thesis with cracks in the primary lining, postulates that pressurized water can occur on these cracks. An exposure to high groundwater pressure can possibly lead to a saturation of more cracks and fissures, in addition to exposure to water pressure on these cracks. The effects of such exposure were unable to be investigated in this study, and is therefore presented as a conceptual model in this section, Figures 46 and 47. For cases with higher water pressure than this study includes, this model should be substantiated. A longterm exposure to higher water pressures can possibly have the following effects:

- Higher degree of saturation of the concrete material near the rock/concrete interface
- Higher degree of saturation of the concrete material at the immediate vicinity of the cracks
- Exposure at the crack-membrane interfaces to water with pressure

The latter point is illustrated in Fig. 47.

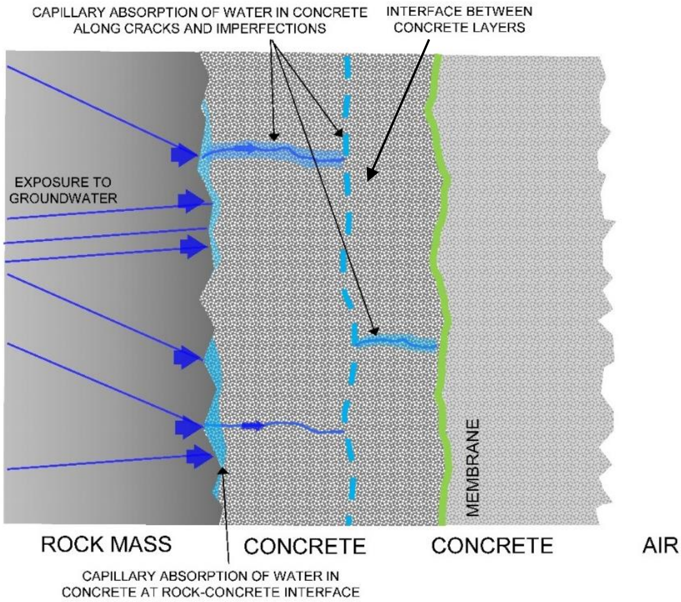



**Figure 46.** Conceptual drawing of section of lining with water exposure paths

A possible water pressure induced debonding can be hypothesized based on the model for water pressure exposure through saturated cracks. Such a mechanism could possibly initiate as a debonding failure at the interfaces and progressively develop. The conceptual model for this failure mechanism, illustrated in Fig. 47, contains the following main issues:

- Water pressure (1) in the crack acting on the concrete surfaces and the membrane
- The swelling (2) of the membrane material under direct unconfined exposure to liquid water
- Failure mechanisms (3), either cohesive (shear) failure in the membrane material or adhesive (debonding) failure at the interfaces between membrane and concrete

The verification of this conceptual model will require measurements of the swelling properties of the membrane, including swelling pressure in confinement and unrestricted swelling.

Under the tested conditions, with exposure to low groundwater pressure, no effects of debonding could be observed. An effect of a slightly higher degree of saturation of the concrete layers close to the rock mass, than the moisture transport model based on capillary and vapor transport predicts, was observed for both the Gevingås and the Ulvin test sites (Sections 5.6 and 5.7 in the thesis, Figures 33 and 35; Paper 3, Section 6.7)

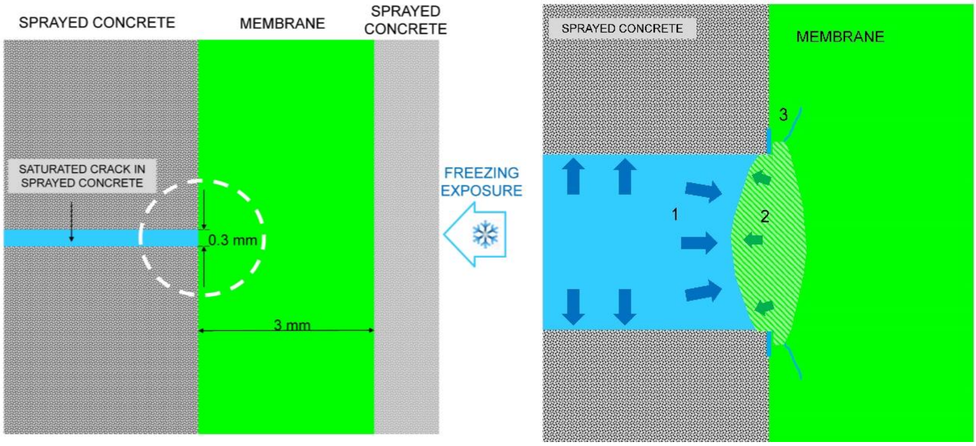



**Figure 47**. Conceptual drawings illustrating the problem of a pressurized wet crack. Left: geometry and dimensions of problem. Right: Detail with some features of the problem. 1: Water pressure acting on membrane and concrete. 2: Swelling of membrane when exposed directly to liquid water. 3: Possible failure mechanisms: delamination at the concrete interface and cohesive failure in the membrane material

# 7 CONCLUSIONS

The single shell tunnel lining system based on fiber reinforced sprayed concrete and EVA-based sprayed membranes has been investigated in the field and laboratory. The ground conditions and lining design context covered by this study were hard rock ground conditions with a rock support lining in the walls and crown of the tunnel, with a drained invert.

The investigated lining system, without any constructed drainage measures has been found to exhibit the following main system properties:

- The lining is a continuously bonded structure from the rock surface to the lining surface in which the interfaces between layers of the constituent materials are continuous hygric and mechanical contacts
- The water exposure at the rock/concrete interface and the exposure to the tunnel climate at the lining surface constitute critically important boundaries for loading, moisture transport and thermal exposure to the lining system
- With the investigated type of sprayed concrete and membranes, the lining system is waterproof and vapor permeable
- The function of a sprayed membrane in a bonded SCL structure has been found to be a barrier to liquid water at cracks in the concrete while providing significant vapor permeability in the continuous contact to sprayed concrete
- The mechanisms of capillary and vapor transport through the lining structure explain the measured moisture condition in the investigated lining structures
- The found moisture properties and in-situ moisture condition of the sprayed concrete and sprayed membrane are decisive for the in-situ mechanical performance of the membrane

The investigations of the possible loading of the membrane under the scope of this study show that the membrane needs to exhibit the following:

- Crack bridging capacity with crack width opening up to approximately 1 mm
- Shear deformability up to approximately 1 mm without sign of initial rupture of the membrane
- Tensile bonding strength of 0.5 MPa at the membrane/concrete interfaces in order to maintain a monolithic mechanical structural performance from the rock to the lining surface
- Once cured, exhibit designed properties within a temperature range -3 to 15$^{o}$C at the membrane location
- Favorable moisture properties in bonded contact with sprayed concrete

The testing of the membranes show the following:

- Polymeric content of the membrane product above approximately 70% seems to be required for sufficient elasticity performance
- The water vapor conductivity of the membranes is within the range of water vapor conductivity of sprayed concrete
- The sprayed membrane material is significantly less hygroscopic than sprayed concrete
- The tested membranes with polymeric content over 71% show high and sufficient crack bridging capacities at 20$^{o}$C
- The elongation performance, in the form of crack bridging capacity, has been found to be temperature sensitive, with significantly lower crack bridging capacity at temperature from 0$^{o}$C and lower.

- With sufficient membrane thickness (>3 mm) bridging of cracks up to 1 mm opening has been found possible down to approximately -3$^o$C with realistic moisture contents
- In-situ testing of the lining in tunnels indicates tensile bond strengths of the membrane/concrete structure in the range of 1.1 to 1.5 MPa
- Testing of the tensile bond strength on a large scale lining structure in an accelerated freeze/thaw exposure setting with minimum temperature -3$^o$C at the membrane location suggest a slight reduction of strength caused by this freeze/thaw exposure after 35 cycles. In a tunnel lining, with slow cooling rates and the found moisture content at the interfaces of the concrete, the membrane will very likely be freeze/thaw resistant with a minimum temperature of -3$^o$C at the membrane location
- Testing of shear deformability in a short term test procedure of specimens of lining structure at 20$^o$C indicates linear shear elasticity up to approximately 1 mm shear deformation for the tested membranes at realistic moisture contents

The sprayed concrete material covered by this study is wet-mix fiber reinforced sprayed concrete according to the current Norwegian practice for permanent rock support linings. This sprayed concrete methodology gives the following main data for hardened intact concrete material with age in the range of 300-400 days in-situ in the lining:

- Suction porosities in the range of 19 to 21 %
- Air void volumes (air porosity, macro porosity) in the range of 4 to 6 %
- Extremely low hydraulic conductivity, $< 5 \cdot 10^{-14}$ m/s
- Young's modulus in the order of 26-27 GPa
- Uniaxial compressive strength in the range of 68-75 MPa

Testing of freeze/thaw resistance and assessments of moisture content and thermal exposure of the sprayed concrete in tunnel linings suggest that freeze/thaw damage of the sprayed concrete material in an SCL structure is very unlikely to occur even under severe cold climate.

The investigation of the effects on ground water pressure caused by an undrained SCL structure with waterproof bonded membrane indicate the following:

- In saturated rock masses in hard rock with hydrostatic pressures in the order of 600-800 kPa and low hydraulic conductivities (in the order of $10^{-9}$ to $10^{-8}$ m/s) the measured ground water pressures in the surrounding rock mass of the lining have been consistently recorded to be lower than the hydrostatic
- The recordings show a trend of decrease of ground water pressure at decreasing distance towards the lining surface
- Assessments which only take into account the average hydraulic conductivity of the rock mass or the hydraulic transmissivity of the rock joins are unable to explain such a decrease in ground water pressure near the lining surface
- The likely explanation for the observed trend with decreasing ground water pressure near the lining is the effect of the EDZ (substantiated by another recent study), which can exhibit several orders-of-magnitude increase in hydraulic conductivities in the rock mass in the immediate vicinity of the excavated contour
- An EDZ with a significantly higher hydraulic conductivity is likely to be a critical condition for the feasibility of an SCL with undrained walls and crown in rock masses with low hydraulic conductivities under hydrostatic pressure

Although no detailed investigations of the EDZ could be conducted in this study, the effect of the EDZ is important and should be included as a system property for the SCL when constructing this lining system in rock masses below the groundwater level.

The findings in this study indicate that the SCL with bonded membrane exhibits favorable system properties due to the interaction between the concrete and membrane materials and between the lining and the rock mass. When the parts of the lining are constructed within the found range of material properties and under the conditions of this study, this lining system is likely to meet modern requirements.

Although technical feasibility and cost-effectiveness has been demonstrated on a number of tunnel projects, the current state of this technology still exhibits several shortcomings and challenges which require further improvements. The main shortcomings are:

- Sensitivity to imperfections in the lining materials, particularly caused by irregularities in the sprayed concrete substrate, which in turn requires strict quality control
- Application methodology of the membrane, lack of robustness for typical tunnel construction conditions
- Membrane product which, in freshly applied condition, is too sensitive to water exposure

The main issues to further develop the SCL method with bonded waterproof membrane as a technically feasible, cost-effective and durable tunnel lining system can be summarized in the following main points:

- Improvement of constructability details for the sprayed concrete substrate and membrane application processes in order to reduce sensitivity to variation workmanship
- Development of specifications and requirements which cover the processes of the excavation and support of the tunnel in order to provide favorable conditions for the construction, long term condition and durability of the lining. The main issues are pre-grouting of the rock mass, contour quality during excavation, mix-design and application of the sprayed concrete in the rock support lining

Despite the current shortcomings of this lining system, it has already proven to be a valuable and cost-effective method to waterproof a sprayed concrete lining in several situations.

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# Paper 1

*Title:*

Future trends for tunnel lining design for modern rail and road tunnels in hard rock and cold climate

*Authors:*

Karl Gunnar Holter

Harald Buvik

Brede Nermoen

Bjørn Nilsen

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*G. Anagnostou & H. Ehrbar (eds)*

# Future trends for tunnel lining design for modern rail and road tunnels in hard rock and cold climate

K. G. Holter[(1)], H. Buvik[(2)], B. Nermoen[(3)], B. Nilsen [(1)]

[(1)]*Dept. of Geology and Mineral Resources Engineering, Norwegian University of Science and Technology, Trondheim, Norway*

[(2)]*Norwegian Public Roads Administration, Geo- and Tunnel Technology Section, Oslo, Norway*

[(3)]*Norwegian National Rail Administration, Infrastructure Division Region North, Trondheim, Norway*

ABSTRACT: The design of Norwegian rail and road tunnel linings is currently being reconsidered in order to meet modern functional requirements for service life time, maximum allowed down time and total cost effectiveness. This paper reviews the current design practice and gives an overview over some suggested and possible future technical solutions for such tunnel linings. Specific issues which are different for rail and road tunnel linings are also reviewed. The use of modern analysis tools for decisions on technical solutions and cost optimizing are described. Until recently rail and road tunnels have been designed with a tunnel lining system consisting of a permanent rock reinforcement lining according the sprayed concrete and rock bolt method, and an inner insulation and waterproofing shield system. The rock reinforcement lining has been designed according to the Q-system which recognizes fiber reinforced sprayed concrete and rock bolts as permanent and long term durable elements of the tunnel lining. The traditionally employed inner shield system has had two main functions; namely the waterproofing and drainage and the thermal insulation to avoid formation of ice. Additionally for road tunnels the esthetic design of the traffic area is important for safety reasons. Recent experiences with operational costs, maintenance and need for refurbishment of relatively new tunnels have revealed that current design practice of rail and road tunnels does not meet modern functional requirements for the desired service lifetime and required maintenance level. The modern analysis tools RAMS (reliability, availability, maintainability and safety) and LCCA (life cycle cost analyses) are suggested to establish the detailed decision basis for technical solutions for tunnel linings. Based on such analyses cost-effective technical solutions for tunnel linings according to modern functional requirements can be achieved. Possible future technical solutions for tunnel linings for high speed rail or highways with dense traffic are largely based on European experiences and consist of cast-in-place or segmental concrete linings.

## 1 Introduction

Keeping the construction costs low has traditionally been considered the main issue in the total cost-effectiveness of a new rail or road tunnel project in Norway. The costs related to maintenance and refurbishment has only to a limited extent been considered in the planning, decision and design process. In most cases one has accepted significant and frequent time slots with closure due to required maintenance. The vast portion of Norwegian tunnels has therefore been constructed with a tunnel lining system which has had a low investment cost, but also a limited service lifetime in a number of cases.

Modern road and rail infrastructure requires tunnels to be placed in increasingly more difficult ground conditions and sensitive environment. Requirements for maximum down time and service lifetime are changing in a more demanding direction.

## 2 Background – current technical solutions

Norwegian rail and road tunnels are currently being designed with a functionally divided tunnel lining system. The rock reinforcement lining is designed with sprayed concrete and rock bolts to provide permanent stability of the rock mass. The design of this lining is carried out according to the Q-system (Barton et al. 1974). This procedure is also referred to as the Norwegian Method of Tunneling NMT (Barton et al. 1994). An important feature is that the rock reinforcement lining is not waterproof. Hence, the tunnel structures are designed as globally drained structures. Water seepage control is handled with the pre-grouting method (Garshol 2003). The pre-grouting method essentially utilizes a systematic pressure grouting of cementitious and mineral grouts ahead of the advancing tunnel face. Todays practice enables hard rock tunnel pre-grouting to achieve water ingress rates down to 1-2 litres per 100 linear m tunnel per minute (Hognestad et al. 2005). With rock overburdens in the range of 10-100 m this implies hydraulic conductivities after pre-grouting of the rock mass in the range of $10^{-8}$ to $10^{-9}$ m/s. The remaining seepage has been allowed to enter into the tunnel. This implies a global drainage of the immediate rock mass around the tunnel. The globally drained tunnel structure has been a fundamental principle of Norwegian rail and road tunnel construction. Since 1982 a large number of subsea road tunnels in rock have been successfully designed and constructed according to this principle (Nilsen and Henning 2009). This technical solution for tunnel linings implies the need for an inner lining structure which collects and drains the water down to the invert. In areas exposed to freezing thermal insulation to prevent formation of ice is an important issue. For road tunnels the inner lining has also been designed to obtain a proper esthetic design of the traffic area. Examples of the traditionally employed tunnel lining systems are shown in figures 1 and 2.

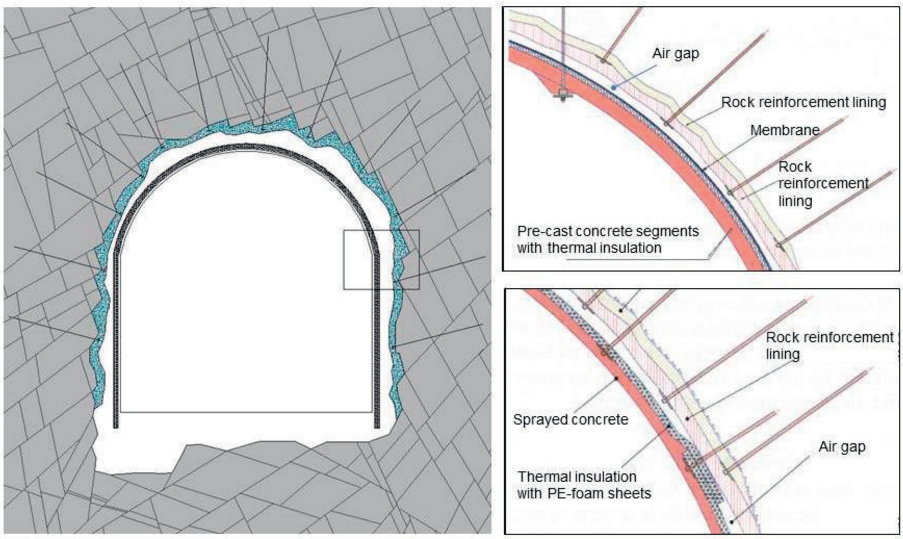


**Figure 1. Layout of traditional Norwegian tunnel lining system with two options. A: shield system with thermally insulating pre-cast concrete elements. B: shield system with PE sheets (after NPRA 2012).**

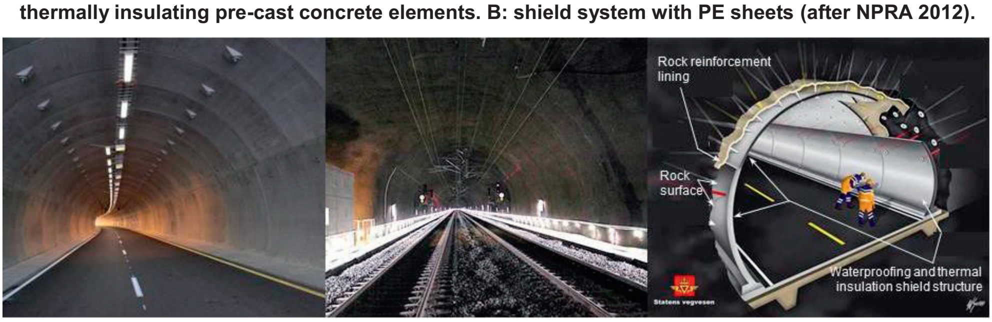


**Figure 2. Recently constructed tunnel lining systems with drainage and thermal insulation shield structures as shown in fig 1. Left: Concrete segment shield structure for a highway tunnel. Middle: PE-foam shield structure in a high speed rail tunnel. Right: 3D image of the concrete segment and PE foam lining system highway tunnels (left and middle photos: Ådne Homleid/Byggeindustrien).**

# 3 Functional requirements for modern rail and road tunnels in Norway

## 3.1 Current practice

The design principle of the globally drained tunnel structure (Nilsen and Henning 2009, NPRA 2012) implies that a certain amount of water is allowed to seep through the rock mass into the tunnel. The maximum allowed amounts of water seepage into the tunnel is subject to analyses of the hydrogeological balance and is specified in most cases as a given quantity of water per time unit per linear meter tunnel. This represents the functional requirement for the pre-grouting works, which take place at the tunnel face concurrently with the excavation and the rock reinforcement works.

The Norwegian norms for both rail (NNRA, 2012) and road (NPRA 2006 and 2010, NNRA 2012) address the rock support on one hand and the waterproofing and thermal insulation on the other hand. This essentially represents the functional division of the tunnel lining into the two following parts:

- A structural part, reinforcing the exposed rock surface and the immediate rock mass
- An inner operational part, providing an environment which suits the function of the tunnel

For the rock reinforcement part of the tunnel lining, very little can be found in the Norwegian documents which can be considered to be functional requirements. The only clearly stated functional requirement for the rock reinforcement part of the tunnel lining is that all elements of the lining shall be considered permanent and have a durability which is at least equal to the service life of the tunnel. However, clear requirements are given for the operational part with waterproofing and thermal insulation shielding for both rail and road tunnel linings in the above cited norms. For rail and road tunnels these requirements have a number of similarities, but also some important differences.

### 3.1.1 Road tunnels

For road tunnel linings the functional details are typically given as performance requirements like:

- Water drained down to invert without freezing
- Thermal insulation in the freezing zone designed according to the frost amount F for one winter for the given location, in which F is the accumulated number of hours multiplied with the number of $^{o}C$ below 0 $^{o}C$
- Service life 50 years

### 3.1.2 Rail tunnels

For rail tunnel linings the main specific differences are:

- All tunnel lining surfaces to be waterproofed
- In most cases a stricter requirement on maximum allowed down time, mainly due to the lack of possibilities for partly closure and lack of detour possibilities
- No required esthetic interior design
- Larger aerodynamic loads on the tunnel lining
- Chemical deterioration due to chloride negligible

The probability of collision events within the service lifetime of a rail tunnel is also very much lower than for a road tunnel.

## 3.2 Trends for modern functional requirements

Modern functional requirements are being elaborated for both rail and road tunnels in order to meet modern demands. Such functional requirements are categorized in the following main items:

- Service Lifetime
- Maximum allowed downtime during operation
- Maintainability
- Geometrical evenness of tunnel contour

### 3.2.1 Road tunnels

For road tunnels one has chosen an approach to differ between higher and lower traffic densities. For tunnels with high traffic density one would require a very high service lifetime for the lining system. For tunnels with lower traffic densities one would accept lining systems with lower service lifetime, but also a significantly lower investment cost. The downtime imposed by planned maintenance and refurbishment directly influences the downtime for the tunnel. The definition of acceptable downtime is influenced by the possible detour options around a certain tunnel and the possible time which the tunnel can be closed for maintenance or operated with partial capacity or limited traffic density. For tunnel linings in modern road tunnels, one can summarize the following important issues:

- For high traffic density , AADT > 4000, service life time 100 years *(AADT = annual average daily traffic movements)*
- For medium and low traffic densities, AADT < 4000, service life time 50 years
- Esthetic design of traffic area to suit modern demands
- Thermal insulation with respect to design freezing loads
- Traffic area designed for likely accident and collision scenarios within the service lifetime
- Chemical durability against chlorides in lower walls
- Only fire resistant materials in tunnel lining for high traffic density tunnels
- Defined maximum allowed down time, “white hours” for maintenance and repair

### 3.2.2 Rail tunnels

For rail tunnels one has so far not defined a strictly required service lifetime. The main approach is to require a service lifetime of 100 years for the entire lining system in all new rail tunnels. However, one would allow some adjustments in special cases. Today’s practice to a large extent involves project specific approaches for important technical solutions. Therefore one important goal is to elaborate a set of guidelines at a superior level for all new rail tunnels.

Modern rail lines designed for high speed imply much more rigid alignment curvatures than in the past. The extent of very long tunnels will be higher than for the existing rail network.

For tunnel linings in modern rail tunnels one can summarize the following important issues:

- Service lifetime 100 years, but adjustments possible
- Any amounts of freezing F will imply a frost resistant lining system
- Esthetic design not of critical importance
- Definition of maximum downtime. Detour generally not possible

## 4 Modern analysis tools for decisions of technical solutions for tunnel linings

Both rail and road administrations have started to use modern analysis tools for decisions on tunnel lining design. Such analysis tools comprise RAMS (Reliability, Availability, Maintenance and Safety) and LCCA (Life Cycle Cost Analyses). The aim is to use these tools in a systematic manner for the final decisions on technical solutions. The following three main issues are considered critical for the choice of technical solutions for tunnel linings for both rail and road tunnels. These are:

- Safety
- Minimal downtime
- Total cost effectiveness

Safety in this context relates to serious events such as collision, derailment, collapse and fire.

The minimum downtime for a traffic tunnel depends on how reliable it is for unplanned failure events and planned maintenance which results in closure. Furthermore, the possibility to carry out maintenance within restricted time slots will influence the down time. The maximum required downtime for a certain tunnel is given based on the traffic density and the importance of the road or rail connection in question, as well as the possibility for temporary detour. For a given technical solution for tunnel linings one therefore needs to analyze if the technical solution will influence the downtime. For this purpose one uses the RAMS analysis tool. As a basis for a RAMS analysis one needs to know or define the required maximum downtime for the tunnel. The detailed analysis addresses the functional requirements for the structure in question as input and gives an expected maximum downtime as a result. This resulting downtime from the analysis should then match the required maximum downtime (figure 3). If the resulting downtime is unsatisfactory, one needs to consider special efforts in order to meet the initially defined requirement.

A RAMS analysis will not give a complete picture regarding the most cost-effective technical solution. For this reason LCCA should be carried out in conjunction with RAMS analyses. LCCA is a process of evaluating the economic performance of a structure over its entire life. LCCA balances initial monetary investment with the long-term expense of owning and operating the structure (Stanford University, 2005). When these two analyses are carried out with good input data, one obtains better and more objective decisions as to which technical solution for tunnel linings is the most suitable in each case.

The contents of the RAMS and LCC analysis tools are graphically shown in figure 3.

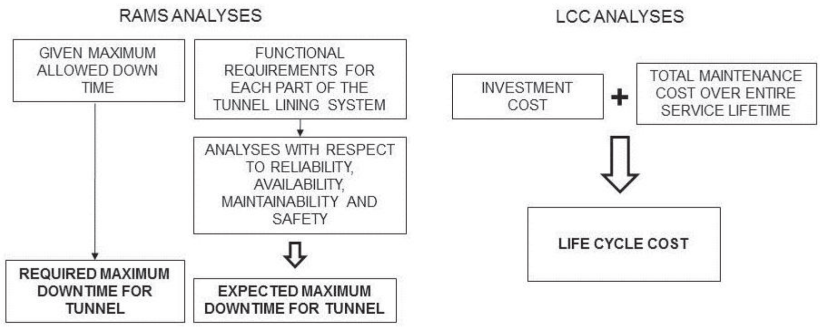


**Figure 3. Schematic representation of RAMS analyses and LCC analyses**

# 5 Trends for future technical solutions for rail and road tunnel linings

Both rail and road administrations in Norway are adopting the central European approach by designing tunnel linings with either cast-in-place or segmental concrete structures. The main reason for this is the expected high service lifetime and low need for maintenance for such tunnel lining systems compared to the performance of the tunnel lining systems which have traditionally been in use. A lowest possible down time is very important for tunnels for major rail and highway portions. Hence, tunnel linings which require a minimum of maintenance are suggested for such projects. For road connections with less importance, one has chosen to accept the latest developed versions of the existing drainage and insulation shield lining systems. For rail tunnels one has so far suggested one main approach, with possibility for project specific adjustments.

## 5.1 Rock reinforcement and water control design

For both road and rail tunnels, the proven and established rock reinforcement method and water control philosophy with the pre-grouting method is suggested for the future geomechanical and hydrogeological design (NPRA 2012). For geomechanical stability of the tunnel lining this implies that sprayed concrete and rock bolting will still be the main rock mass reinforcement lining method (Barton et a. 1974, Barton et al. 1994, Norwegian Concrete Association 2011). Hence, the possible use of cast-in-place concrete tunnel linings with sheet membrane waterproofing has an esthetic and waterproofing function only and is not considered to have a structural or geomechanical function (NPRA 2012). The groundwater control philosophy with pre-grouting with strictly evaluated allowed water ingress amounts will still be the main approach. The main concept is to allow controlled and maximum defined amounts of water ingress. This water control philosophy implies the tunnel structure being a globally drained structure, in which no loads imposed by water pressure act on any parts of the tunnel lining.

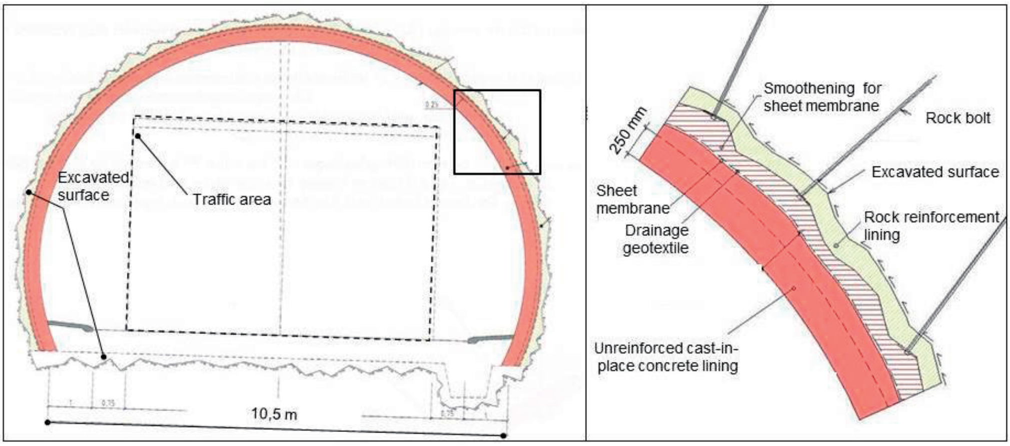


**Figure 4. Suggested design with cast-in-place tunnel lining for modern two-lane highway tunnels in hard rock in Norway. Left: full cross section. Right: detail of lining structure (NPRA, 2012)**

### 5.2 Trends for design of permanent linings in road tunnels

Two and three lane highway tunnels in hard rock in Norway will also in the future very likely be excavated by the drill-and-blast method. For major highway connections one recommends the cast-in-place tunnel lining system as shown in figure 4 for tunnels with annual average daily traffic movements (AADT) more than 4000. The lining system is adopted in a globally drained context, using a geotextile fleece for drainage, a sheet membrane for waterproofing and leaving the invert unwaterproofed.

For road connections with less traffic density one will accept the shield systems as shown in figure 1. With the recent technical improvement of these tunnel lining systems one can realistically require a service lifetime of 50 years.

### 5.3 Trends for design of permanent linings in TBM excavated rail tunnels

The suggested future design of tunnel linings in rail tunnels largely follow the considerations made for road tunnels. However for rail tunnels over a certain length, TBM excavation is likely to be a realistic cost-effective alternative to drill-and-blast excavation. The planned extension and modernization of the rail network in Norway, including the High Speed Rail network (HSR) will involve construction of long rail tunnels. Several design options for tunnel linings are currently being considered for new long rail tunnels in Norway. The design and construction, as well as service life time and maintenance considerations for these lining types is well proven in the Alp countries (Strappler et al. 2012).

Figure 5 shows two main design options for TBM excavated tunnels in hard rock which can be adopted in Norway. In both cases a shielded TBM would be employed and a segmental concrete lining is installed. Case A (left) shows a gasket sealed segmental concrete lining, which implies a completely waterproof and undrained tunnel lining structure. This lining type needs to be designed for the full hydrostatic groundwater pressure.

Case B (right) shows a segmental concrete lining and an inner cast-in-place concrete lining with sheet waterproofing membrane and drainage. This system implies global drainage of the tunnel lining structure, without any hydrostatic pressure. The outer segmental concrete lining is designed for geomechanical loads only, exclusive of water pressure. The case B as shown in figure 5 can also be constructed with a lining with fibre reinforced sprayed concrete rather than the segmental lining, hence employing an open gripper TBM. An inner lining with the cast-in-place concrete and sheet waterproofing system can then be constructed subsequently. This would be a technically feasible and cost-effective option in prevailing hard rock conditions, when the short term stability of the excavated tunnel surface is favourable.

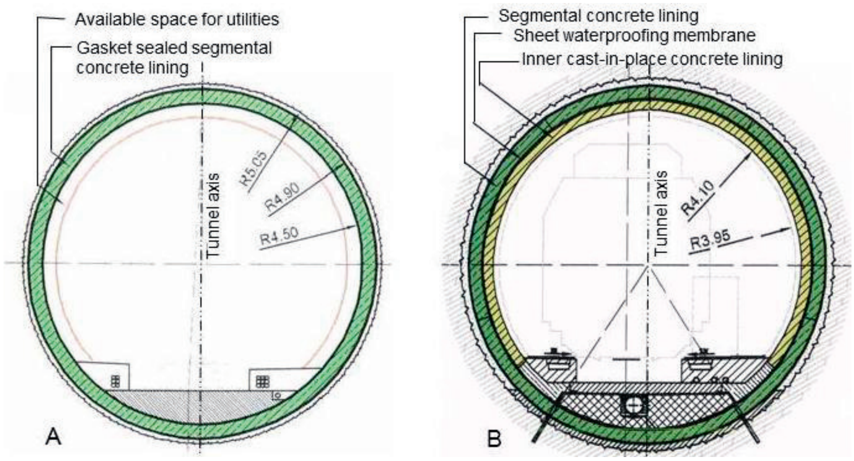


**Figure 5. Two design options for tunnel linings in TBM excavated rail tunnels in rock. A: Single-shell undrained gasket sealed segmental concrete lining. B: Double-shell drained lining with outer segment concrete lining and inner cast-in-place concrete lining. (After Strappler et al. 2012)**

Such an approach was realized during the construction of the Steg-Raron portions of the Lötschberg base tunnel (Classen et al. 2003) as well as the Gotthard Base Tunnel, both in Switzerland.

## 5.4 Innovative design of permanent linings in traffic tunnels with the sprayed concrete and bonded membrane lining system

An innovative tunnel lining system with sprayed concrete and bonded waterproof membrane is currently subject to detailed research for possible use in modern rail and road tunnels in Norway. This system has already been successfully used on several rail and road tunnel projects in central Europe (Holter et al. 2010) and most recently in Norway for a significant section of the recently constructed single-track 4 km long Gevingås rail tunnel near Trondheim (Nermoen et al. 2011). The design process for the tunnel lining system included a RAMS and LCC analysis (DNV, 2010), in which the total cost-effectiveness of this tunnel lining method was verified.

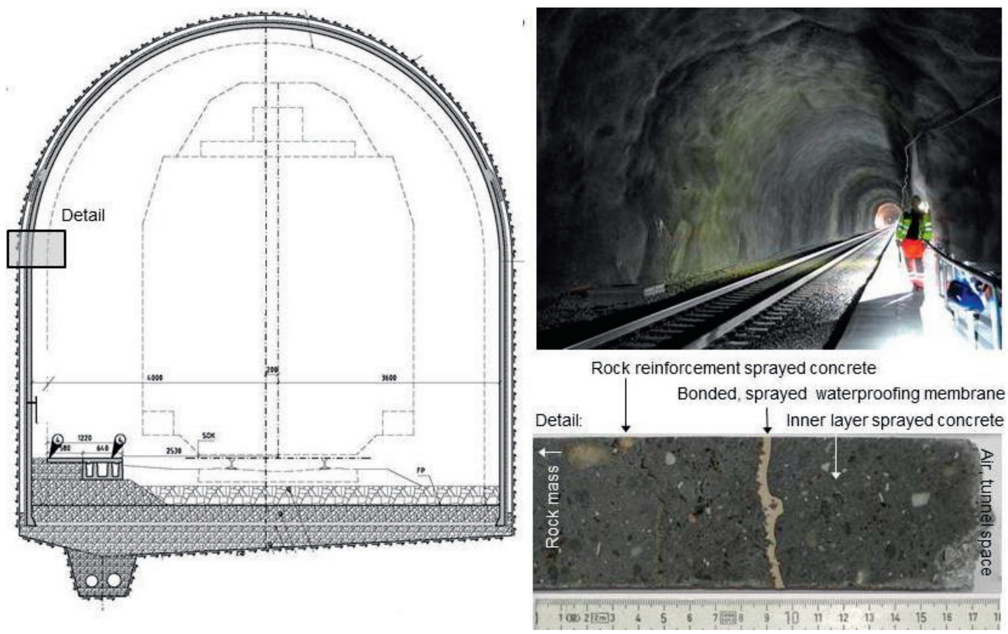


**Figure 6. Innovative option for traffic tunnels in rock with sprayed concrete and bonded waterproofing membrane. Design and finished lining from the recently constructed Gevingås rail tunnel in Norway**

Example of a tunnel lining layout for a rail tunnel using the sprayed concrete and bonded membrane method, with system detail and example of finished tunnel lining in a modern rail tunnel, is shown in figure 6. This tunnel lining method has proven to have a high maintainability without requiring long periods of down time.

## 6 Conclusions

The currently employed tunnel lining systems for rail and road tunnels in Norway are found to have too short service lifetime and too high down time. Both rail and road administrations are elaborating functional requirements to suit the real and specific needs for modern rail and road tunnels. Alternative modern tunnel lining systems with cast-in-place concrete for drill-and-blast excavated tunnels and segmental concrete linings for TBM excavated tunnels are being planned. For drill-and-blast excavated tunnels the existing practice for the rock reinforcement lining and pre-grouting method for water control will be continued. The modern design decision tools RAMS and LCC analyses will be implemented in a systematic manner in order to obtain the best technical solutions for tunnel linings in the future.

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# Paper 2

*Title:*

Loads on sprayed waterproof tunnel linings in jointed hard rock: A study based on Norwegian cases

*Author:*

Karl Gunnar Holter

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# Paper 3

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*Authors:*

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Stig Geving

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# Paper 4

*Title:*

Freeze-thaw resistance of sprayed concrete in tunnel linings

*Authors:*

Karl Gunnar Holter

Sverre Smeplass

Stefan Jacobsen

*Published in:*

Materials and Structures

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Rock Mech Rock Eng
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ORIGINAL PAPER

# Performance of EVA-Based Membranes for SCL in Hard Rock

**Karl Gunnar Holter**[1]

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**Abstract** The bonded property of multi-layered sprayed concrete tunnel linings (SCL) waterproofed with sprayed membranes means that the constituent materials will be exposed to the groundwater without any draining or mechanically separating measures. Moisture properties of the sprayed concrete and membrane materials are therefore important in order to establish the system properties of such linings. Ethyl-vinyl-acetate based sprayed membranes exhibit high water absorption potential under direct exposure to water, but are found to be significantly less hygroscopic and exhibit lower sorptivity (water absorption rate) than sprayed concrete. This material behavior explains the relatively dry in situ condition of the membrane that was observed. Measured in situ moisture content levels of the membrane material in tunnel linings have been found to vary within the range of 30–40 % of the maximum water absorption potential, and show a decreasing trend over the first 4 years after construction has been completed. A model for the mechanical loading, moisture condition and thermal exposure of the membrane and the resulting realistic parameters to be tested is presented. Laboratory testing methods for the membrane materials are evaluated considering possible loads, moisture and freezing exposure. Material testing of membrane materials was conducted with preconditioning to realistic moisture contents and under different temperature conditions including relevant freezing temperatures for tunnel linings. The main effects of the in situ moisture condition of the tested membrane materials are favorable tensile strengths in the range of 1.1–1.5 MPa and low risk of freeze–thaw damage. The crack bridging capacity of the tested membranes is found to be sensitive to temperature. With membrane thicknesses in the range of 3–4 mm, crack bridging capacity up to 4–6 mm opening of the crack width at 23 °C and approximately 1 mm opening at −3 °C was measured for the tested membranes. No significant reduction of the tensile bond strength could be demonstrated after 35 freeze–thaw cycles with −3 °C minimum temperature at the membrane location in the lining. Further work is required to verify the performance of the SCL system under exposure to high hydrostatic pressures and the effects of long term mechanical exposure.

**Keywords** SCL · Sprayed concrete · Loading conditions · Sprayed waterproof membrane · EVA based membrane · Testing · Durability

✉ Karl Gunnar Holter
karl-gunnar.holter@ntnu.no

[1] Department of Geology and Mineral Resources Engineering, Norwegian University of Science and Technology, Sem Sælands vei 1, 7491 Trondheim, Norway

**Abbreviations, definitions and terms**

| | |
|---|---|
| SCL | Sprayed concrete lining. Permanent tunnel lining system based on fiber-reinforced sprayed concrete as the structural material with different possible waterproofing measures which are integrated into the sprayed concrete structure. Such linings may also include rock bolts for rock reinforcement |
| EVA-based sprayed waterproofing membrane | Ethyl-vinyl-acetate copolymer material used in the category of sprayed waterproofing membranes referred to in this paper |

| | |
|---|---|
| DCS | Degree of capillary saturation (%). Degree of saturation of concrete with respect to total suction porosity, equal to the ratio of water content of a given concrete specimen to its water content at saturation at immersion at atmospheric pressure at mass equilibrium |
| RH | Relative air humidity (%) |
| COV | Coefficient of variance, ratio of standard deviation to mean value |

## 1 Introduction

In hard rock environment in the Scandinavian countries permanent rock support linings are widely constructed with fiber-reinforced concrete and rock bolts (NGI 2013; NCA 2011; STA 2011). The final waterproofing and thermal insulation has normally been resolved by constructing a separate suspended shield structure (NPRA 2012; STA 2011). Modern requirements for service lifetime, serviceability and maintainability have raised concerns with the use of these shield lining systems. Cast-in-place concrete lining waterproofed with sheet membranes or pre-cast concrete segment linings for rail and road tunnels have therefore been proposed as the future technical solution in rail and road tunnels subjected to high traffic density (NPRA 2012; Holter et al. 2013).

SCL with sprayed ethyl-vinyl-acetate (EVA)-based membranes are being considered as a possible technical solution under certain conditions as an alternative to the well established lining systems. The main benefit would be the reduced total lining thickness since the rock support lining based on sprayed concrete can be utilized as part of the final lining, and large concrete thicknesses can be avoided. Although such linings with spray applied membrane have been constructed for approximately a decade and have seen increased use in some countries, the main properties and function have yet to be fully understood.

SCL waterproofed with a sprayed membrane represents a continuously bonded multi-layered structure from the rock mass to the tunnel lining surface. The bonded property of the lining structure implies that the constituent materials of the lining will be exposed to the groundwater without any constructed draining or mechanically dividing measures. The construction process of spray-application produces continuous and bonded interfaces, which also can be assumed to be perfect hygric contacts between the different layers. The moisture properties of the membrane material and the concrete on either side of the membrane, as well as the exposure to any groundwater in the immediate rock mass will influence the moisture condition of the materials in the tunnel lining.

A research project in Norway has been carried out in order to assess the suitability of this lining system for modern rail and road tunnels. An important part of this research has been to conduct site and laboratory investigations in order to establish the function and properties of such linings. This work contains several main modules which have required detailed studies. The investigation of the in situ moisture condition and possible moisture transport mechanisms through sprayed concrete tunnel linings are published in Holter and Geving (2015) which forms the basis for moisture exposure during laboratory testing. The freeze–thaw resistance of the sprayed concrete in tunnel linings under realistic moisture contents and thermal exposure has also been investigated and will be published in a separate paper.

The scope of the present investigation is to study the properties of the membrane material, particularly the loading conditions for the membrane, evaluate testing methods and conduct testing of important parameters under varying climatic conditions. A conceptual model for the tunnel lining is presented in order to define the main items, its properties and the important processes for the waterproof SCL system. The in situ exposure conditions for the membrane will be substantiated from field investigations. The context which is considered in our study is a hard rock environment in which the primary rock support structure is considered stable and has no imposed ground induced loads or deformations on the bonded membrane and inner lining.

This study refers to EVA-based membranes with products from two different suppliers. The study contains the following main elements:

- Definition of material model based on the layout of the tunnel lining.
- Model for different loading scenarios of the membrane.
- Field investigations: moisture content, thermal exposure and in situ tensile bond strength.
- Evaluation of laboratory test methods for membranes in a SCL context.
- Laboratory investigations of hygroscopic properties of the concrete and membrane materials.
- Laboratory investigations of mechanical properties of the membrane material.
- Analyses of results.

The first findings of this study were presented at the World Tunnel Congress 2014 (Holter et al. 2014). Findings from additional field and laboratory investigations have been included. The recommendations of the ITAtech Design Guidance for

Sprayed Membranes (ITA/AITES 2013) compiled by Dimmock (2014) will form the basis for the evaluation of test methods for membranes. Adjustments to these test methods will be discussed and made based on the loading model and the findings from the thermal and moisture analyses.

Possible degradation processes and long term durability under relevant mechanical loading and climatic exposure, as well as recommendations regarding testing details and acceptance criteria will be discussed based on the results.

## 2 Conceptual Model for Tunnel

Current SCL designs and the application methodology for concrete and membrane materials form the basis for the conceptual model. The bonded and thus undrained interfaces of the multi-layered structure result in moisture transport processes governed by the hygroscopic properties of the constituent materials. For hard rock tunnels the tunnel lining structure consists of a primary lining based on fiber-reinforced sprayed concrete and rock bolts. In poor ground conditions in a hard rock environment reinforced sprayed concrete ribs are frequently used for permanent ground support (Grimstad et al. 2008; Mao et al. 2011). In order to produce a suitable substrate for the application of the membrane, a regulating layer of sprayed concrete is normally required. For these investigations the regulating layer has been applied using wet-mix fiber-reinforced sprayed concrete. Dry-mix sprayed concrete or mortar for use in the substrate for the membrane has been excluded from this investigation. The conceptual model for the waterproof SCL is shown in Table 1 and Figs. 1 and 2.

Fiber-reinforced sprayed concrete is the structural material in an SCL structure. The sprayed concrete mix designs investigated in this study all represent state-of-the-art developments in mix designs and robotic application technology. Four different sites with different mix designs were included in this study. However, the basic mix designs differ only slightly from one another. The mix design of the sprayed concrete used for detailed material investigations in our study is shown in Table 2. The range of the contents of the different components is also given.

All together five membrane products nominated M1 to M5 have been included. For the field investigations of the lining structure only M1 has been analyzed so far.

## 3 Mechanical Loading, Moisture and Freezing

### 3.1 Mechanical Loading

#### *3.1.1 Loads from the Rock Mass*

A study of the possible loading of rock support linings based on sprayed concrete and rock bolts from the groundwater and rock mass has been undertaken (Holter 2014). The current practice with the use of rock mass classification according to the Q-system (NGI 2013) normally ensures rock stability with a high factor of safety. From this study it is concluded that in hard rock environment the stresses and loads which occur in tunnel linings are negligible in most cases. Even in severe weakness zones significant loading of the tunnel lining structure does not normally take place, other than local loads (Mao et al. 2011; Grimstad et al. 2008; compilation by Holter 2014). Still, special design of the rock support is undertaken for severe weakness zones.

**Table 1** Main items in the conceptual model for waterproof SCL

| Main item in conceptual model | Situation, condition | Processes |
|---|---|---|
| Tunnel lining structure with sprayed concrete and membrane in walls and crown | Bonded undrained contacts from rock surface through all materials<br>No thermally insulating materials in lining<br>Concrete of different ages | Moisture transport through lining structure<br>Differential shrinkage, of concrete on either side of membrane<br>Thermal and shrinkage induced movement of cracks in concrete<br>Exposure to water at cracks<br>Exposure to movement at cracks caused by gravitation, shear displacement, stress release, swelling |
| Rock mass below GW table | Saturated jointed rock material<br>Exposure to GW at the rock-concrete interface | Local saturation of concrete material at rock-concrete interface<br>Water flow on joints into tunnel through invert |
| Tunnel space | Climate in tunnel<br>Seasonal variations in temperature and relative humidity | Exposure of lining surface to tunnel climate<br>Heat flux from rock mass to tunnel space<br>Cyclic freezing and thawing of lining<br>Change of properties of membrane and concrete |

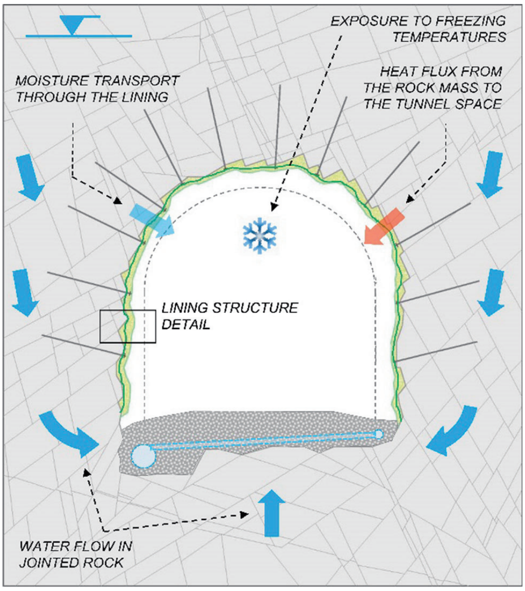



**Fig. 1** Main elements in the conceptual model for a tunnel with permanent SCL based on fiber-reinforced sprayed concrete, sprayed waterproofing membrane and rock bolts constructed in hard rock. Detail is shown in Fig. 2

### *3.1.2 Ground Water Induced Loads*

The waterproof SCL lining system represents an undrained structure. Hence, possible water pressures acting on the tunnel lining need to be considered. A study including monitoring of groundwater pressures around sprayed concrete tunnel linings with drained inverts has been conducted (Holter 2014). These results indicate water pressures lower than the hydrostatic pressure in the immediate vicinity of the tunnel lining.

Any occurrence of unfavorable ground water pressures in the immediate rock mass needs to be considered in the rock support design as well as evaluating the need for drainage measures where this is feasible. Ground water under a certain pressure can possibly saturate cracks and imperfections in the sprayed concrete in the primary lining. Under such circumstances a wet-crack situation with ground water pressure exposing the membrane locally can be hypothesized. The investigated sites in this study had a water pressure near the lining of maximum 2 bars. No deterioration of the lining structure was detected at any of the test sites. However, the wet crack problem at higher hydrostatic pressures cannot be assessed in detail from this study.

### *3.1.3 Loads from the Weight of the Tunnel Lining*

The gravity induced stresses in the tunnel lining caused by the weight of the tunnel linings represent a constant static load. By considering a thickness of the inner layer sprayed concrete of 100 mm, and assuming that the concrete lining is "hanging" on the substrate a gravity induced tensile stress of 2 kPa in the center of the tunnel crown can be calculated.

### *3.1.4 Dynamic Loads from the Traffic Area of the Tunnel*

Rail and road tunnels are exposed to fluctuations in air pressure caused by traffic. Highway and high speed rail

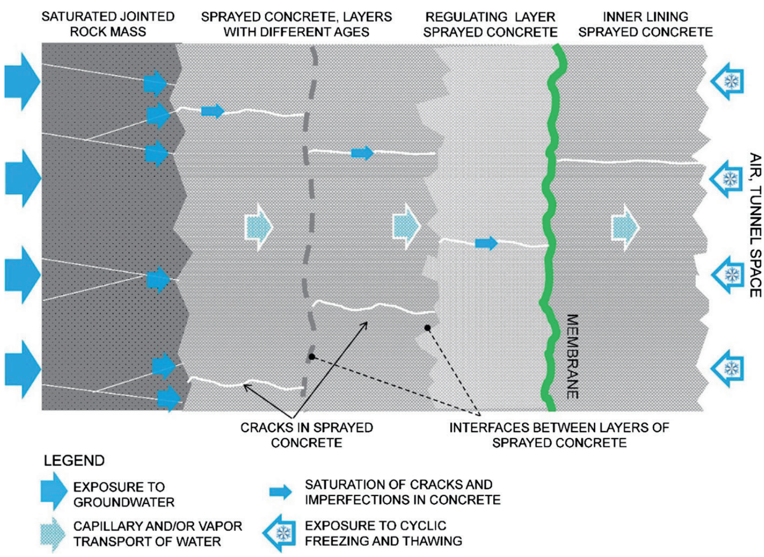



**Fig. 2** Detail of waterproof SCL. Conceptual model with section of the lining structure with the constituent materials, moisture transport processes and exposure to freezing

**Table 2** Sprayed concrete mix design for the Harangen road tunnel, and range for other sprayed concrete mixes for tunnel sites investigated in this study

| Component | Quantity | Range for investigated concrete mixes from the other test sites |
|---|---|---|
| Cement CEM II A-V 42.5 | 502 kg/m$^3$ | 488–513 kg/m$^3$ |
| 17–18 % fly ash content | | |
| Micro silica fume | 25 kg/m$^3$ | 21–26 kg/m$^3$ |
| Water added with base mix | 240 kg/m$^3$ | 202–245 kg/m$^3$ |
| Water added with accelerator at spraying nozzle | 17 kg/m$^3$ | 17–20 kg/m$^3$ |
| Water/binder-ratio[a] | 0.45 | 0.44–0.47 |
| Aggregate 0–8 mm | 1497 kg/m$^3$ | 1497–1588 kg/m$^3$, fractions used 0–4, 0–8, 0–10 mm |
| Superplasticizer | 7 kg/m$^3$ | 7–10 kg/m$^3$, different suppliers |
| Fiber reinforcement, structural polypropylene (PP) | 9 kg/m$^3$ | 5–9 kg/m$^3$ (PP) 0.5–1 % by volume |
| | 1 % by volume | 40 kg/m$^3$ (steel) 0.6 % by volume |
| Binder paste content | 0.43 m$^3$/m$^3$ | 0.41–0.44 |

[a] Considering equivalent binder content: weight of cement + two times weight of micro silica

tunnels in Norway have design requirements for expected maximum dynamic loads and number of loading events throughout the service lifetime. Current design requirements for rail and road tunnels (NPRA 2006; NNRA 2012; STA 2014) are shown in Table 3.

For a tensile loading consideration, the values for air pressure changes shown in Table 3 are considered changes in tensile stress. For a high speed double track rail tunnel a single design event for traffic induced air pressure change in the tunnel imposes tensile stresses with a factor five times higher than the calculated static gravity induced load from the tunnel lining. However, the dynamic air pressure induced loads are approximately a factor 100 times lower than measured in situ tensile bond strengths of the membrane-concrete interfaces. It is therefore considered very unlikely that this dynamic loading represents a dynamic fatigue scenario for a bonded SCL structure.

#### *3.1.5 Deformations of the Membrane Over Cracks in the Concrete and Shear Deformations Along the Concrete-Membrane Interfaces*

Deformations can occur in the sprayed concrete lining caused by the differential shrinkage of the concrete with different age on either side of the membrane, as well as thermally induced contraction of the concrete material due to fluctuations in the temperature. Such deformations are illustrated in Fig. 3. Each layer of concrete will exhibit a set of shrinkage cracks which will normally not persist across layers with different age. The membrane represents a deformable and ductile material, which is designed to bridge the cracks in the concrete. The two concrete layers, one on either side of the membrane may be applied with a time gap of several weeks or up to several months. From a load consideration perspective, the full shrinkage potential from the covering layer of concrete is assumed. Shrinkage properties of sprayed concrete has been subject to a recent Swedish study (BeFo 2014; Bryne et al. 2014a). Free (unrestrained) shrinkage of fiber-reinforced sprayed concrete after approximately 120 days was found to be in the range of 0.045–0.055 %, or 0.45–0.55 mm per m on laboratory sprayed slab specimens subjected to norm climate conditions (storage at RH 50 % and 20 °C, following 7 days of initial curing under water). In a restrained context such as bonding to rock as well as the unilateral exposure to moisture on the rock side and drying on the air side, precise assessments of shrinkage are difficult to make. Effects of surface drying may cause high shrinkage locally at the concrete surface. The shrinkage will result in the cracking of the concrete material. The use of fiber

**Table 3** Dynamic loads in modern tunnels given as sudden change in air pressure per design traffic event

| Tunnel type | Design speed (km/h) | Air pressure change per event (kPa) | Number of events in service lifetime | Time interval between each event, range |
|---|---|---|---|---|
| Highway, double carriageway | 140[a] | 1.5 | $5 \times 10^7$ | 20–60 s |
| Rail, double track | 250 | 10 | $1 \times 10^7$ | 2–5 min |
| Rail, single track | 250 | 8 | $1 \times 10^7$ | 2–5 min |

[a] 20 km/h higher than legal speed limit

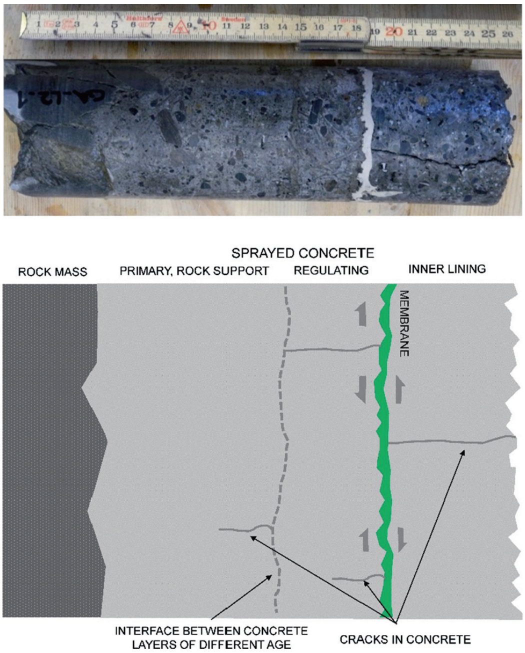



**Fig. 3** Shear deformation and elongation at cracks of the membrane in the lining structure. *Top* photo showing persistent shrinkage crack in the secondary lining being bridged by the membrane. *Bottom* model for cracks and shear deformation

reinforcement and the restraint caused by the bond to the membrane will have some crack width reduction effect. Measurements of crack widths in the concrete lining has been conducted (Sect. 4.5 in this paper) in order to substantiate typical crack widths.

### 3.2 Exposure to Moisture

The continuously bonded property of the waterproof SCL system implies that there is an exposure to the groundwater at the interface between the rock mass and the sprayed concrete. Both the constituent materials concrete and membrane exhibit capillary and hygroscopic properties. Thus, the in situ moisture content of the lining materials and its effect on the mechanical properties need to be accounted for. Moisture properties of the lining materials are shown in Sect. 6.1 in this paper. The measured in situ moisture content in the investigated tunnel linings is shown in Sects. 4.2 and 4.3.

### 3.3 Consideration of the Monolithic Character of the Lining

The mechanical performance of a continuously bonded SCL depends on the performance of the weakest element in the lining structure. A tunnel lining based on two layers of sprayed concrete separated by a bonded membrane should ideally be considered as one structure for the entire lining thickness. For this reason, the tensile bonding strength of the membrane-concrete interfaces should not be significantly lower than the tensile bonding strength between the rock surface and the sprayed concrete. Tensile bonding strengths for sprayed concrete interfaces to the rock substrate vary highly depending on rock type and the type of surface, as well as the application and material details of the sprayed concrete. Measured values for tensile bonding strength for the interface between sprayed concrete and rock vary between 0.2 and 1.8 MPa (NCA 2011; BeFo 2014; Bryne et al. 2014b). The gravity induced tensile stresses would be approximately a factor of 100 times lower than the lowest recorded tensile bond strength of concrete against rock. For this reason it is reasonable to propose an acceptance criterion for tensile bonding strength for the membrane which is in the magnitude of relevant tensile bonding strength between rock and sprayed concrete. Norwegian and Swedish standards propose 0.5 MPa as a minimum required tensile bond strength between rock and sprayed concrete. The ITAtech Design Guidance (ITA/AITES 2013) for sprayed membranes proposes an acceptance criterion of 0.5 MPa for tensile bonding strength.

### 3.4 Exposure to Freezing

The basic SCL design in our study has no insulating layers to avoid freezing exposure. The aim of this study is to determine the possible damage or reduction in performance caused by realistic freezing exposure. Given a membrane thickness of 3–4 mm, the thermal conductivity of the concrete material in secondary lining will be decisive for the thermal exposure. Each tunnel will represent an individual case with respect to freezing exposure based on the rock mass temperature, the winter climate, the ventilation of the tunnel and the location in the tunnel considered.

## 4 Field Investigations

### 4.1 Overview, Goal

The main goal of the field investigations was to substantiate as much as possible the loading conditions for the membrane (moisture, thermal and crack situation), as well as carrying out in situ measurements of the tensile bonding strength of the interfaces between the membrane and the concrete. We have included the investigations carried out on the large scale laboratory lining structure as part of the field investigations in this paper since this investigation context has proven to cover comparable conditions to in situ tunnel. The field investigations were carried out in the period 2012–2014. Table 4 shows an overview of the conducted field investigations with locations and main purposes. The 4 locations and type of test sites are described in further detail in Holter and Geving (2015).

### 4.2 Moisture Content of the Tunnel Lining Structure

A detailed study of the moisture content and moisture transport mechanisms in waterproof SCL sections has been reported by Holter and Geving (2015). The findings of this study serve as an important basis for the analysis of the performance of the membrane described in this paper. The field investigations of the moisture content were carried out in three different tunnel sites at yearly intervals with ages up to 4 years. Several consistent observations were made during these investigations. The main features are:

- High degree of capillary saturation (DCS) of the concrete material, close to 100 %, at the rock-concrete interface.
- A gradient with degreasing DCS towards the lining surface.
- Depending on the lining thickness, the DCS of sprayed concrete on either side of the membrane is found to vary between 80 and 95 %. For primary (rock support) lining thickness of approximately 150 mm the DCS of the concrete at the membrane was found to be around 95 %.

The found moisture condition of the investigated linings can be explained by moisture transport processes from common building physics principles. Further details are given in Holter and Geving (2015).

### 4.3 Moisture Content in the Membrane Material

In addition to the investigations of the concrete material in the tunnel linings, also the membrane material was analyzed. Immediately after splitting of the cores, samples of the membrane material were removed from the concrete and tested for moisture content. Membrane samples from four different tunnel lining locations have been taken from 5 up to 38 months after construction. The measured values for moisture content, given as weight of water in % of dry weight of the membrane material are shown in Fig. 4. Dry weight of the membrane refers to weight after drying of 3–4 mm thick specimens at 105 °C for a minimum of 2 days.

A trend with decreasing moisture content in the membrane material with increasing time after construction can be observed, in spite of high degrees of capillary saturation of the concrete on either side of the membrane. These data refer to four different hard rock tunnel projects indicated with the different colors. The tunnel linings in all four cases were constructed with drained inverts and waterproof undrained SCL in the walls and crown. The Karmsund case is a subsea road tunnel located at approximately 70 m below the groundwater table. Hence a complete saturation under hydrostatic pressure of the rock mass, and higher saturation of the imperfections of the concrete is likely to have taken place.

### 4.4 Thermal Exposure to Tunnel Linings

The rock and concrete materials exhibit thermal conductivities which govern the temperature profile form the rock mass to the lining surface under a given thermal exposure in the tunnel space. In this study, monitoring of temperatures under freezing exposure at full scale conditions,

**Table 4** Overview of conducted field investigations for sprayed waterproofing membranes

| Investigation | Location, test site | Main purpose of investigation |
|---|---|---|
| Moisture exposure and in situ moisture content, development over time | Gevingås<br>Harangen<br>Ulvin<br>Karmsund | Basis for moisture conditioning during laboratory testing. Basis for assessment of degradation mechanisms |
| Mapping of cracks in sprayed concrete linings | Gevingås | Obtain realistic crack data for sprayed concrete |
| In situ tensile bonding strength | Ulvin<br>Gevingås<br>Laboratory lining structure | Tensile bonding strength under real exposure |
| Freezing exposure parameters | Ulvin<br>Laboratory lining structure | Thermal profile through lining during severe freezing exposure |

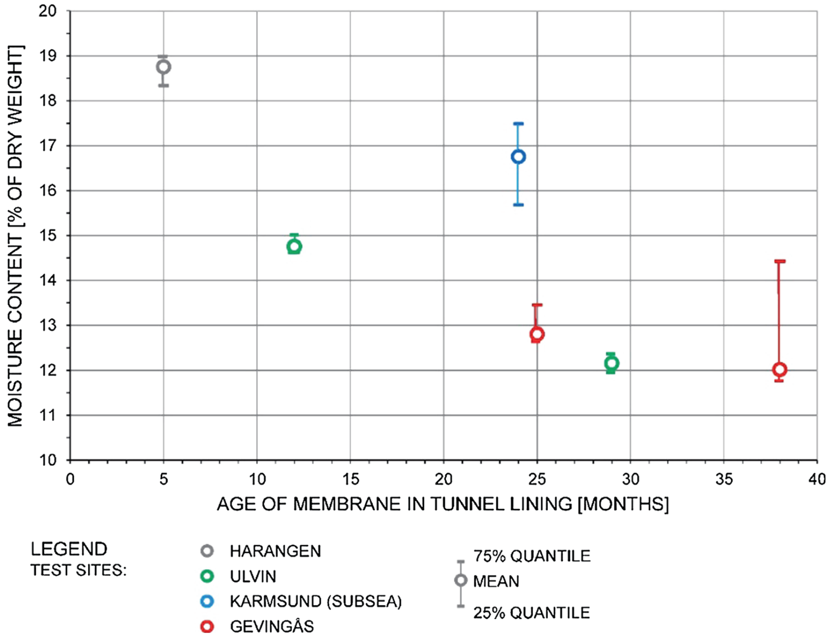



**Fig. 4** Measured development of in situ moisture content in sprayed membrane material in tunnel linings

measurements of thermal conductivities of rock and concrete materials and thermal calculations were carried out. Thermal conductivities for sprayed concrete and intact rock were measured in a separate study (NTNU 2013). Some of the findings are shown in Table 5.

Thermal monitoring with freezing exposure was carried out in the full scale lining section at the Ulvin test site and the lining structure in the freezing laboratory. The main goal of this monitoring was to measure temperature profiles under realistic conditions. The temperature at the location of the membrane can then be assessed. The two monitoring cases are explained in Fig. 5.

The two test sites for thermal monitoring have the following main characteristics:

Case 1: Test site Ulvin, an access tunnel under construction with a test field of 90 linear m with SCL, with lining thickness 300 mm and membrane location at 150 mm from lining surface. The ventilation at the monitoring location was arranged with a gate in the tunnel so that air with a constant temperature of approximately 2 °C from the tunnel face (located more than 2 km in rock mass with constant temperature) could be alternated with cold air from outside. In this way an

**Table 5** Measured thermal conductivities

| Material | Density (kg/m$^3$) | Thermal conductivity (W/m K) | COV |
|---|---|---|---|
| Rock, dark gneiss (Ulvin site) | 2616 | 2.95 | 0.3–0.5 % |
| Rock, granodiorite (Trondhjemite, freezing laboratory) | 2657 | 2.77 | 0.2 % |
| Sprayed concrete, Ulvin site, steel fiber[a], dry[b] | 2138 | 1.64 | 0.5–1 % |
| Sprayed concrete, Ulvin site, steel fiber, saturated[c] | 2214 | 1.85 | 0.2–0.5 % |
| Sprayed concrete, Gevingås site, PP-fiber[d], dry | 2211 | 1.65 | 0.5–1 % |
| Sprayed concrete, Gevingås site, PP-fiber, saturated | 2281 | 1.85 | 2–3 % |

[a] Steel fiber, dosage 35 kg/m$^3$, 0.5 % by volume

[b] DCS 70 %

[c] DCS 100 %

[d] Structural polypropylene fiber, dosage 7 kg/m$^3$, 0.8 % by volume

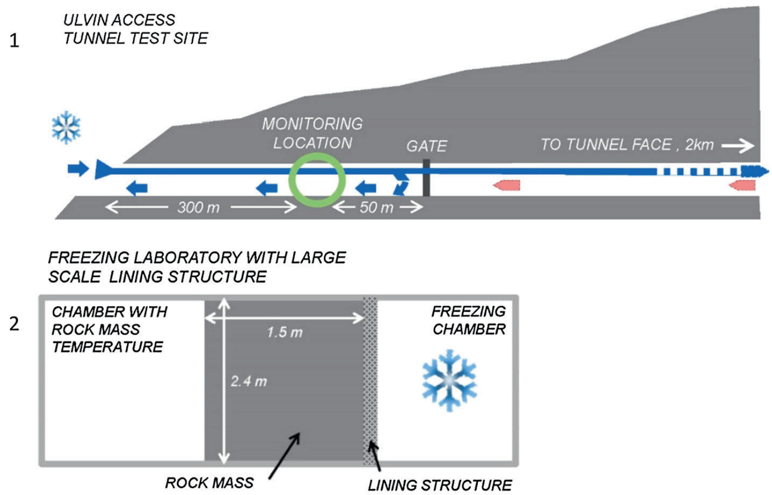



**Fig. 5** Longitudinal vertical sections with configuration of the Ulvin test site for in situ measurements (case 1) and the laboratory lining structure (case 2) for controlled thermal exposure

exposure to cold air at approximately 50 m distance from the portal could be applied experimentally in full scale. The field test at the Ulvin site could only be carried out in a short period of time for one severe freezing cycle over 36 h and hence, provided no information of long term freezing exposure.

Case 2: Laboratory test facility with large scale lining structure constructed on a rock mass of homogenous granodiorite blocks with lining thickness 240 mm with membrane location at 110 mm. Controlled freezing exposure was applied to the lining surface, simulating different freezing scenarios. Exposure modes included cyclic loads for accelerated freeze–thaw testing of the lining materials, and as isothermic exposure in order to simulate the effect of long term cooling of the lining.

Findings from an investigation conducted in the Glödberget rail tunnel in north Sweden indicate that low air temperatures can penetrate far into the tunnel (STA 2012). The first 200–300 m from the portal can be exposed to air temperatures in the range of −15 to −20 °C in severe cases. The design of the tunnel lining for thermal insulation in portions with such severe exposure need to be evaluated in each single case based on local climate conditions and ventilation of the tunnel under operation during winter season.

The field test at the Ulvin site was arranged to produce a cooling of the tunnel lining by running the ventilation at approximately 1 m/s air flow with cold air from outside. Temperatures in the tunnel air at the test location in the range of −7 to −9 °C were achieved. After 36 h the test had to be terminated due to the tunnel construction cycle. A profile of the tunnel lining with the measured temperatures after 36 h is shown in Fig. 6. The calculated temperatures at steady state conditions with −7 and −9 °C in the tunnel space are indicated. The measurements indicate a background temperature of the rock mass at the location of the tunnel of approximately 7 °C.

A large scale simulation of isothermic freezing exposure with −6 °C in the tunnel space was carried out on the lining structure in the freezing laboratory (case 2, Fig. 5). This exposure was held constantly for 30 days with continuous thermal monitoring. The results are shown in Fig. 7. The measured temperature profiles after 36 h and 17 days together with a calculated temperature profile at steady state are indicated. The measured values refer to three different sets of sensors in the lining—rock mass structure, and hence exhibit a slight scatter due to precision of location.

Based on the conducted freezing exposure tests, and calculation of temperatures at steady state conditions, the minimum temperature exposure at the membrane at given lining thicknesses can be assessed, Table 6.

### 4.5 Mapping of Cracks in the Sprayed Concrete

A mapping of cracks was carried out in the Gevingås rail tunnel on the 2nd August 2013, after an extended period of warm weather with maximum outdoor temperatures in the range of 25–30 °C. The temperature of the tunnel lining at 10 mm depth measured during the mapping of the cracks was 12 °C. Approximately 210 cracks were mapped and marked using a concrete crack width gauge (Fig. 8) in a systematic manner in order to re-record the same cracks later. Hence, the crack mapping was repeated at the exact same location in February 2014 when the temperature was 6 °C at 10 mm depth.

The measured crack widths are shown in Fig. 9. Crack widths ranging from 0.05 to 0.2 mm account for 78 % of

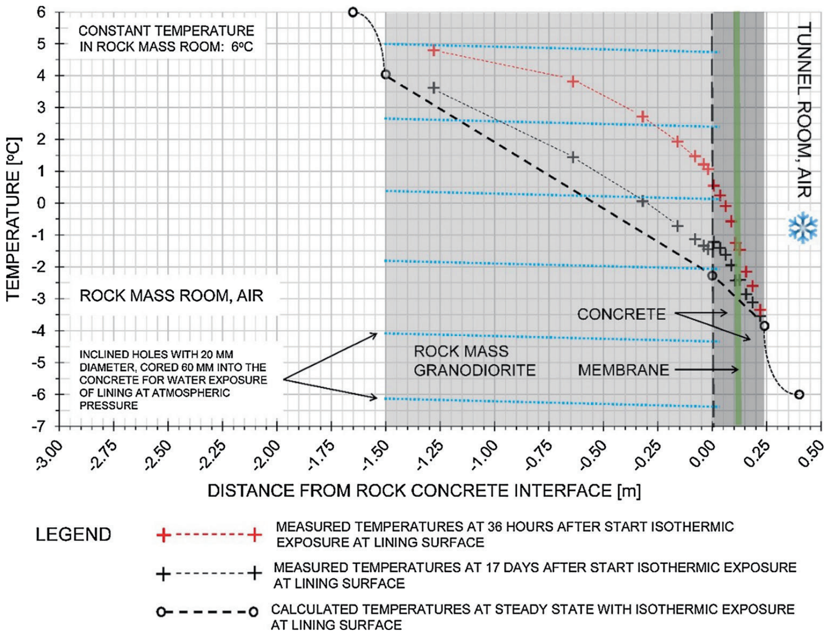



**Fig. 6** Large scale laboratory simulation: measured and calculated temperature profiles through rock mass and lining structure

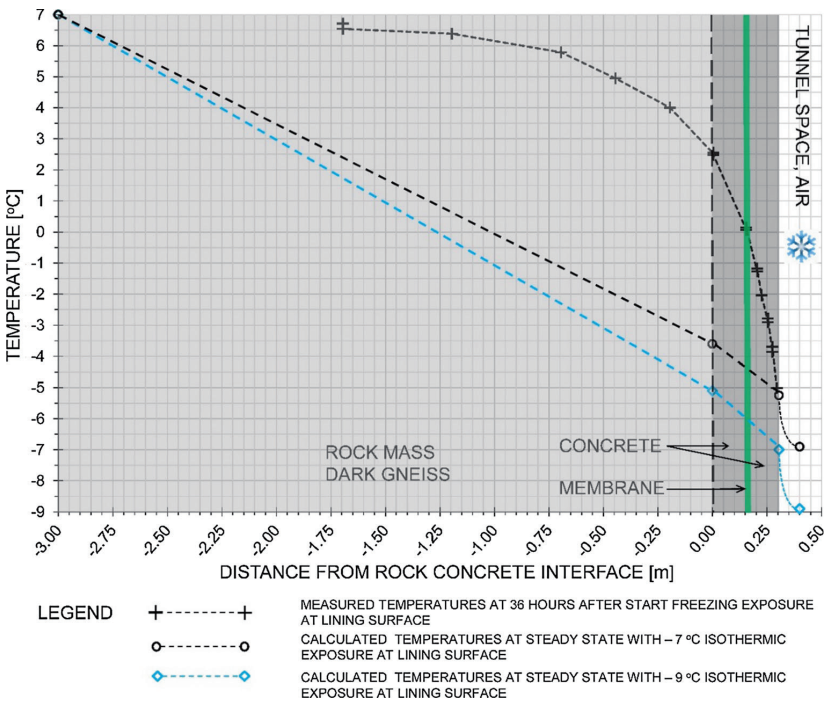



**Fig. 7** In situ tunnel, Ulvin test site: measured and calculated temperature profiles through rock mass and lining structure

the recorded cracks for the measurements done in August 2013. The crack measurements in February 2014 show an increase in crack width, and a larger scatter of the recordings. The most represented crack width for the measurements conducted in February is approximately 0.3–0.35 mm. Thus, an average increase in crack width of

**Table 6** Temperatures at the location of the membrane in an SCL structure based on measurements and thermal calculations at steady state

| Air temperature in tunnel space (°C) | Thickness of covering layer of sprayed concrete over membrane (mm) | Temperature at membrane (°C) |
|---|---|---|
| −6 | 110 | −3.5 |
| −7 | 150 | −4.5 |
| −9 | 150 | −6 |

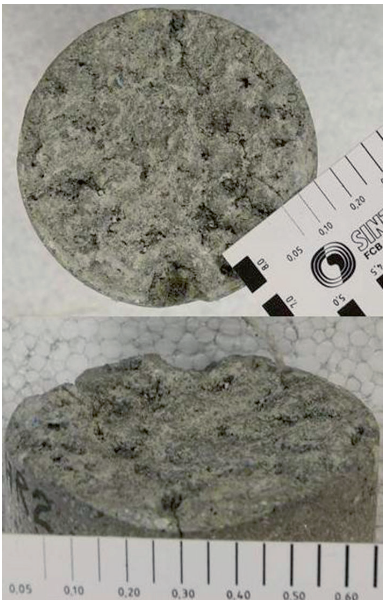



**Fig. 8** Example of recordings of cracks in a sprayed concrete surface using a concrete crack measurement gauge

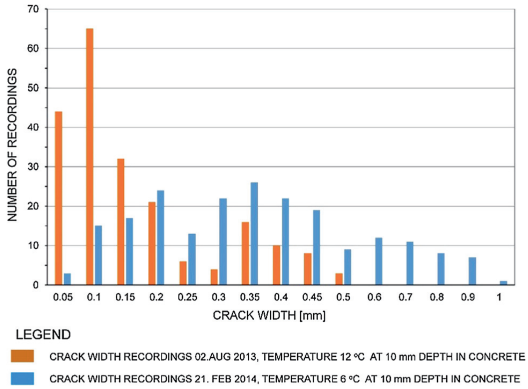



**Fig. 9** Measured crack widths in the sprayed concrete lining surface at the same location in August 2013 and February 2014

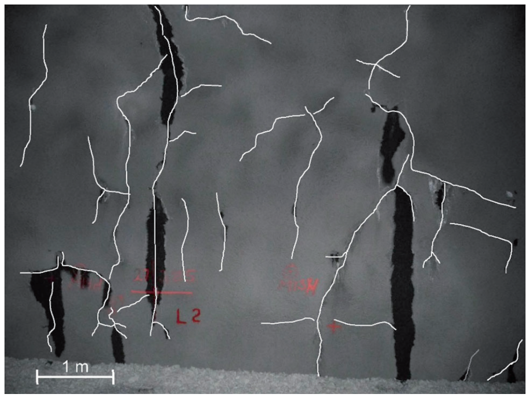



**Fig. 10** Surface observations of cracks in sprayed concrete

approximately 0.2 mm with a temperature decrease of 6 °C is observed. A typical crack pattern was obtained by observing the sprayed concrete lining surface in an area which exhibited leaks and showed mineral stains from leaks through wet cracks. This is shown in Fig. 10. Visible crack distances vary from approximately 0.2 m up to approximately 1.5 m. The most represented crack distance is in the range of 0.7–1 m.

### 4.6 Summary of Field Investigations, Verification of Loading Model

The loads which expose the membrane considered in this study are summarized in Table 7.

## 5 Evaluation of Laboratory Test Methods

The main purpose of the laboratory test methods is to conduct material testing of the membrane under realistic loading and climatic exposure. There are several standardized test procedures for building materials which may be used for membrane materials. The most updated compilation of suggested tests is given in ITAtech Design Guidance for Spray Applied Waterproofing Membranes (ITA/AITES 2013). However this guidance has no loading models, neither any guidelines for mechanical, thermal nor moisture exposure testing of the membrane material. In this section the loading model (Sect. 3) and findings from the field investigations (Sect. 4) will be used to substantiate details in the laboratory test methods and relevant acceptance criteria.

**Table 7** Compilation of loads on the membrane considered in this study

| Load type | Relevant value/size | Implication for laboratory testing |
|---|---|---|
| Rock mechanical loads | None. Only local loads | None |
| Groundwater pressure induced loads | Very unlikely for the investigated cases. Not considered | None for cases with low or no hydrostatic pressure |
| Dynamic loads from traffic area | 10 kPa amplitude of air pressure (pressure + suction loads) | None |
| Tensile loads | Gravity induced load from inner lining: 2 kPa | Not realistic requirement |
| Maximum crack width in concrete and thermal opening/closing | Typical crack width range: 0.1–0.3 mm, maximum 0.8 mm | Testing of elasticity under relevant temperatures and moisture contents required |
| | Thermally induced opening and closure : 0.6 mm | Crack bridging performance at 1 mm crack width proposed |
| Shear deformation along interfaces | 0.5–0.6 mm/m | 1 mm shear deformation within linear shear elasticity behavior |
| Moisture exposure | 15–18 % moisture content range in the membrane material | Pre-conditioning of membrane to relevant moisture content |
| Thermal exposure | Possible temperature range +15 to −6 °C at membrane location in tunnel lining | Testing at realistic temperatures |

## 5.1 Moisture Properties of Lining Materials

Due to the hygric continuity of the lining structure and the direct exposure to groundwater, the moisture properties of the lining materials need to be included in order to substantiate the realistic moisture condition for testing. Recent reported testing of membranes for waterproof SCL (Su et al. 2013; Su and Bloodworth 2014; Nakashima et al. 2015) have not included the moisture condition and moisture properties of the constituent materials in the lining. Testing of moisture properties of membrane and concrete materials have yet to be included in guidance for design and testing of spray applied membranes. Standard test methods for sorptivity and moisture content at equilibrium commonly used for concrete are adopted in this study. Thus, comparison to findings from other studies of concrete is possible.

## 5.2 Elasticity and Crack Bridging Properties of the Membrane

Preventing water flow through the lining by the bridging of cracks is the main waterproofing function of the membrane. Testing of the membrane's elasticity can be done by a pure elastic test or by a functional test of the resistance to rupture over a discontinuity in the substrate. Rupture of the membrane over a crack with increasing width is found to be the main failure mechanism in the loading model (Sect. 3). Hence, the crack bridging test as proposed in the ITAtech guidance (ITA/AITES 2013) guidance directly addresses a relevant failure mode. A pure elasticity test does not account for the bonding of the membrane to the substrate. It is difficult to quantify a requirement in terms of pure elasticity which translates to the required crack bridging capacity. However, the elasticity test is simple and can give an indication of the elasticity of the membrane material in order to reject unsuitable materials without conducting costly testing.

### *5.2.1 Elasticity Testing According to DIN 53504*

This is a simple test conducted on specimens with standard dimensions which are stretched to failure while measuring tensile deformation and tensile force. Standard dog bone shaped specimens, shown in Fig. 11, have normally been used for this purpose. The sensitivity of EVA-based membranes to moisture content means that details regarding storage and conditioning as well test procedure for such membranes needs to include details regarding humidity and temperature. Sprayed specimens are preferred to molded specimens in order to test realistic membrane material. However, sprayed specimens are more difficult to produce with even thicknesses for the purpose of reproducing consistent standard dimensions for laboratory testing.

### *5.2.2 Crack Bridging Performance*

The proposed test method for crack bridging performance according to ITA/AITES (ITA/AITES 2013) is a static crack bridging test and is designed for the testing of coating materials on exterior surfaces of masonry and concrete (DIN EN 1062-7:2004). With this test method one can basically test only one crack width, although it would be possible to include a few increments in the crack width

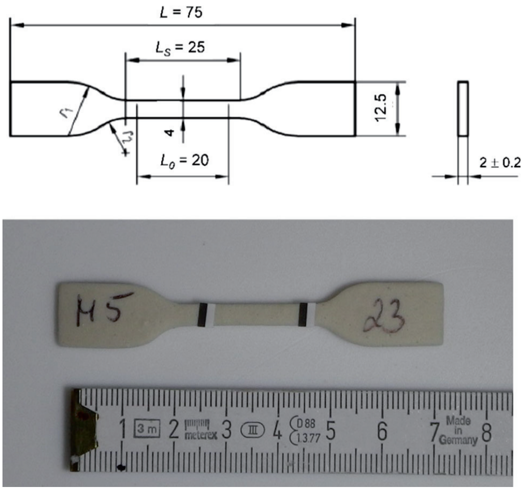



**Fig. 11** Specimen for elasticity test according to DIN 53504. *Top* dimensions of the S2-type dog bone shaped specimen used for this purpose, with figures in mm. *Bottom* photo of specimen with markings of the length $L_0$ area for precise elongation measurement with video-extensometer

before the maximum crack width, given by the geometry of the test jig, is reached. The main features of this test method are shown in Fig. 12.

An adjusted crack bridging test has been considered in order to apply a more controllable opening of the crack and hence enable a precise determination of the crack width at rupture. This adjusted test method has many similarities to the dynamic tensile test described in DIN EN 1062 Annex C4. The adopted procedure is shown in Figs. 13 and 14.

### 5.3 Tensile Bond Strength

Testing of tensile bond strength, also referred to as pull-off strength or adhesion, of sprayed membranes has been conducted by pulling the membrane off the substrate (Ozturk and Tannant 2010). This procedure uses a disc shaped plate mounted on an elevator bolt which is glued to an over-cored section of the membrane and subsequently pulled in a controlled manner. This method can prove useful for a temporary test of the membrane's tensile bond strength before the inner lining concrete is applied. Testing of tensile bonding strength of the membrane in the lining structure as proposed by ITA/AITES (2013) is a standard pull-off test for adhesion including the entire lining structure, according to EN ISO 4624 section 9 (2003). The principle of the test is shown in Fig. 15. However, the wet core drilling, the risk of applying unfavorable bending and tensile loads during the core extraction, inconsistent

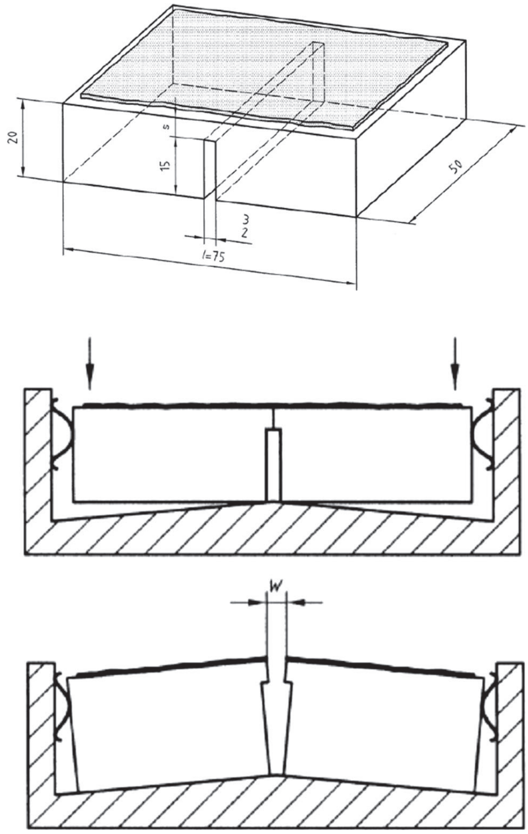



**Fig. 12** Conceptual illustrations showing the static crack bridging test procedure according to DIN EN 1062-7 Annex C1, proposed by ITA/AITES 2013

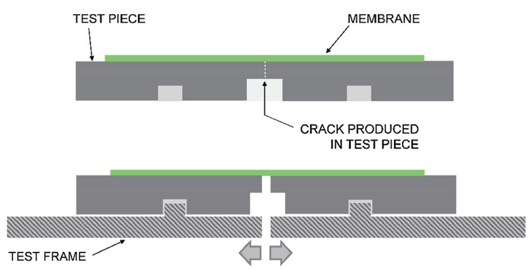



**Fig. 13** Conceptual illustrations showing the loading mode during the adopted version of the dynamic crack bridging test

moisture conditioning and details in the test setup might influence the results significantly.

A procedure to measure tensile bond strength without extracting core samples was adopted. The main purpose of this procedure was to test the tensile bond strength under as

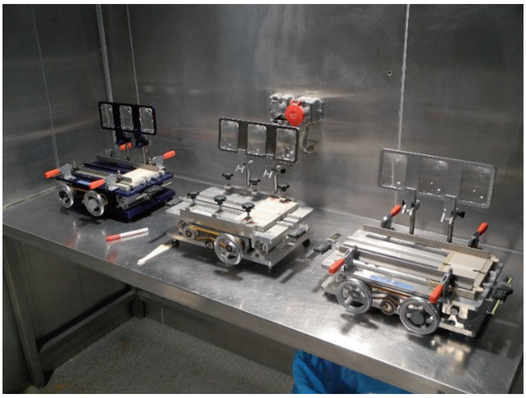


**Fig. 14** Crack bridging testing with adopted procedure in progress in climate chamber. (Courtesy by Wacker Chemie AG)

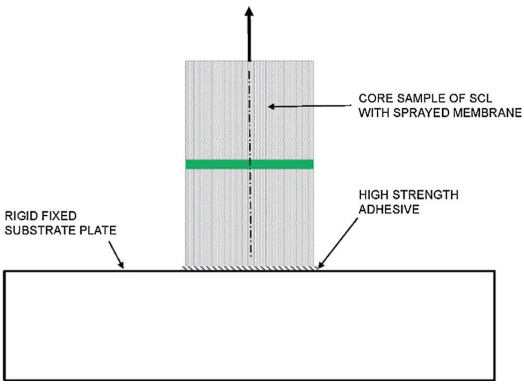



**Fig. 15** Testing of tensile bonding strength according to EN-ISO 4624

realistic conditions as possible. The procedure was laid out as an in situ test in which the test specimen consisted of an over-cored part of the lining structure. The layout of the pull-off details were arranged in order to achieve a perfect axial alignment to the core specimen. The adopted test is shown in Fig. 16. The testing device used in this investigation could only record maximum tensile strength.

### 5.4 Shear Performance of the Membrane-Concrete Interfaces

Direct shear testing is not proposed by ITA/AITES (2013). Direct shear testing is included in this study in order to establish the membrane's ability to perform under shear deformation which can occur between the substrate and inner lining concrete layers. Previous direct shear testing of such membrane concrete interfaces has been reported by


**Fig. 16** Adopted procedure for in situ measurements of tensile bonding strength. *Top* conceptual diagram showing the layout of the test. *Middle* Preparation with over-coring with grinding of the test specimen. *Bottom* in situ specimen after testing

BASF (TU Graz 2008), Su et al. (2013) and Su and Bloodworth (2014). These investigations refer to EVA based sprayed membranes in which the specimens were tested in dry state without any pre-conditioning to relevant moisture content. A study of the composite action of EVA-based membranes for SCL was carried out by Nakashima et al. (2015). However, this study also considers a lining structure and the membrane material in dry condition.

For our study a large scale shear box with constant normal load was available. Controlling the normal load in order to apply constant normal deformation or constant normal stiffness was not possible in our study. The test procedure and moisture conditioning of the specimens is explained in Sect. 6.6 together with the obtained results.

## 6 Conducted Laboratory Investigations

The laboratory investigation program was based on the conceptual model, the results from the field investigations and the evaluation of testing methods. The main goal of the laboratory investigations was to verify the conceptual model, establish detailed performance properties of the lining, as well as providing a basis for the acceptance of a membrane product under certain conditions.

### 6.1 Moisture Properties of Sprayed Concrete and Membrane Materials

#### *6.1.1 Specimens*

The specimens for the sprayed concrete testing of the moisture properties were all obtained from the tunnel lining from the same site and the same location, the Harangen test site (details in Table 1). Hence, the specimens represent the same concrete in terms of age, curing history, mix design and spray application. The membrane specimens were all produced from sprayed sheets, and tested after complete curing for approximately 18 months. The investigation of moisture properties in our study covers water content after immersion at atmospheric pressure, sorptivity (water absorption rate) and moisture bearing capacity in the hygroscopic range. These are illustrated in Fig. 17.

#### *6.1.2 Water Content at Immersion at Atmospheric Pressure*

Membrane specimens obtained by core drilling specimens from sprayed panels as well as sprayed membrane specimens from spray sheet panels were tested. Results are shown in Fig. 18 and Table 8. For the membranes M1 and M5 the maximum water uptake potential is found to be approximately 42 and 30 % of the dry weight of the material. The water content at immersion for concrete at atmospheric pressure is assumed to be equal to the complete saturation of the suction porosity of the concrete.

#### *6.1.3 Water Absorption Rate (Sorptivity)*

Sorptivity expresses the water absorption rate under unilateral and unidirectional water exposure, and was investigated by Holter and Geving (2015). A compilation of the results for concrete and membrane M1 is shown in Fig. 19. The measured water absorption rate of the two materials exhibit a significant contrast within the moisture content range which is found in tunnel linings.

#### *6.1.4 Moisture Content at Equilibrium in the Hygroscopic Range*

The investigation of the moisture content at equilibrium for concrete was presented by Holter and Geving (2015). Testing of the membranes M1 and M5 with moisture content at equilibrium obtained by isothermic desorption has been added in this study. The desorption isotherms show moisture content represented as degree of saturation at immersion for the materials versus relative humidity. A compilation of the results is shown in Fig. 20. The difference in behavior when

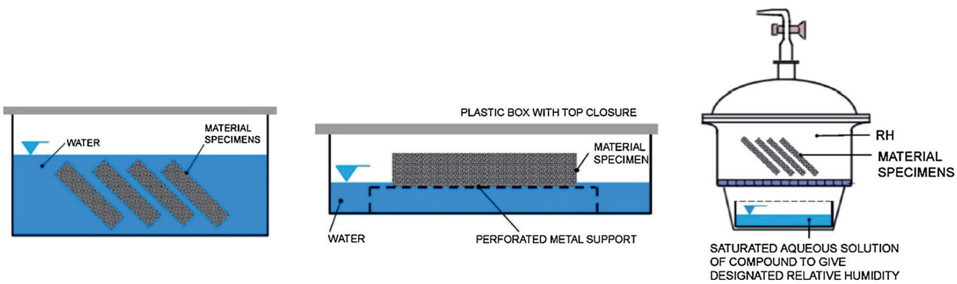



**Fig. 17** Sketches of the three water absorption modes which have been tested in the laboratory. *Left* water content at complete immersion. *Middle* sorptivity (water absorption rate) at unidirectional water exposure. *Right* moisture bearing capacity at equilibrium in the hygroscopic range

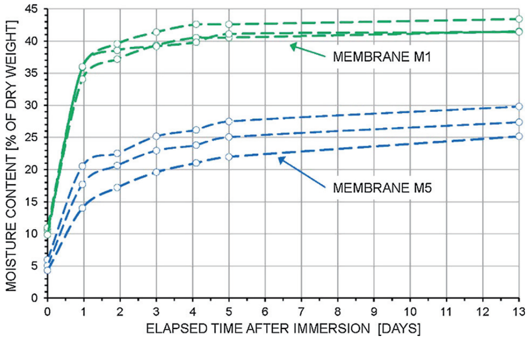



**Fig. 18** Moisture content of membranes measured after complete immersion at atmospheric pressure for sprayed specimens of membranes M1 and M5

**Table 8** Moisture contents at immersion for two series of specimens for membranes M1 and M5

| Membrane product | Moisture content at immersion (weight % of dry weight) | |
|---|---|---|
| | Specimens obtained from lining structure slabs (Fig. 19) | Specimens from sprayed membrane sheets |
| M1 | 41.5 | 42.4 |
| M5 | 27.4 | 30.3 |

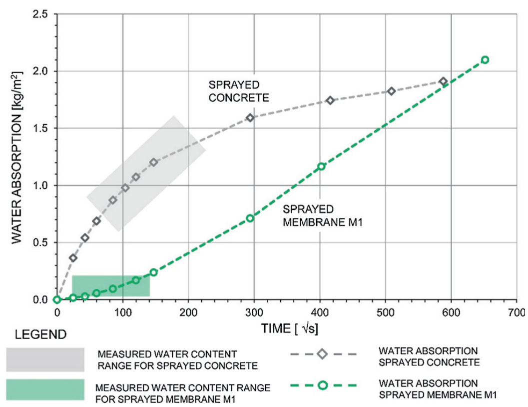



**Fig. 19** Water absorption rate (sorptivity) of sprayed concrete and sprayed membrane (M1) represented as water absorption versus square root of time (compiled from Holter and Geving 2015)

taking the material from immersion to RH 100 % is noteworthy. Concrete loses approximately 10 % of its moisture content, whereas the membranes lose approximately 50 %

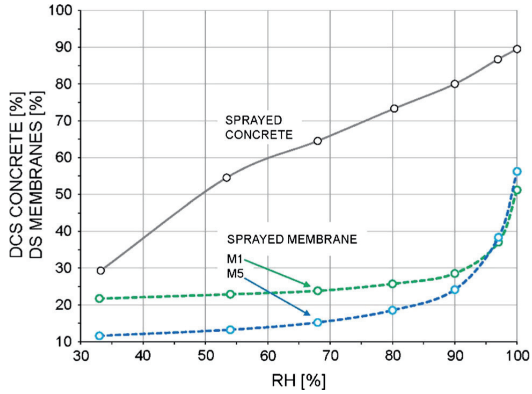



**Fig. 20** Desorption isotherms obtained at 25 °C for sprayed concrete and sprayed membrane, showing the moisture content represented as degrees of saturation at equilibrium versus different values for RH

**Table 9** Measured linear thermal expansion coefficient for steel fiber-reinforced sprayed concrete

| Parameter | Mean (m/m K) | COV (%) | Temperature interval (°C) | Number of specimens |
|---|---|---|---|---|
| Thermal expansion coefficient | $1.27 \times 10^{-5}$ | 1.6 | 3–19 | 3 |

of their moisture content. The desorption isotherms (Fig. 20) show that sprayed concrete is a much more hygroscopic material than the membrane material.

## 6.2 Thermal Expansion of Sprayed Concrete

Linear thermal expansion was measured on cut prism samples of sprayed concrete with dimensions 70 × 70 by 280 mm in different temperature intervals from 3 to 34 °C. The measured values for the temperature interval 3–19 °C are found to be the most relevant and are shown in Table 9. The mix design of the sprayed concrete is shown in Table 2, Sect. 2.

## 6.3 Elasticity of the Membrane Material, DIN 53504

### *6.3.1 Specimens, Conditioning and Testing Temperature*

Specimens from both sprayed and molded sheets of membrane were prepared. Five membrane products were tested in three different test series, in three different laboratories. Altogether five membrane products were tested. Conditions during testing covered humidity conditioning at RH 50 and 95 % and at specific temperatures 23, 0, −3, −8

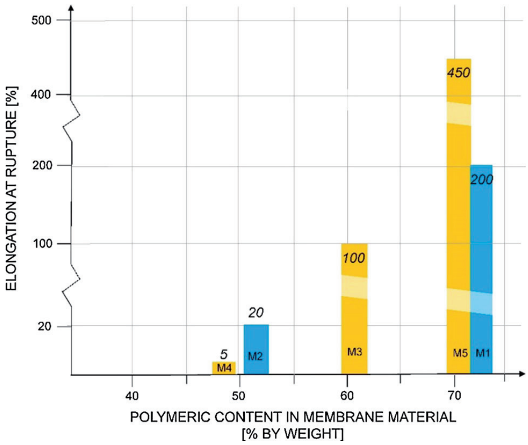



**Fig. 21** Elongation performance of sprayed membrane samples versus polymeric content in the base (powder) membrane products M1–M5 measured by thermo-gravimetric analysis (TGA) with argon as test gas. The elongation was measured in one single test series (laboratory 1) with specimens which had undergone identical treatment from application to testing

and −12 °C. A test of the polymeric content elasticity which could be related to the elasticity performance of the membranes was also carried out.

Our findings using this test method show that it is difficult to obtain consistent results when comparing different test series. The main limitations were:

- Difficulty in preparation (spray application) of specimens with even thicknesses required for reproducible laboratory tests.
- The thickness and evenness of the membrane specimens influences the result significantly.
- Different storage conditions after application and the precise conditioning details influence the absolute measurement values.
- Different interpretation of the test standard regarding testing details.

### *6.3.2 Findings*

For the first elongation test, the polymeric content of the membrane products was tested using a thermo-gravimetric analysis with Argon as test gas in the test vessel. The membrane material was heated to 800 °C. Hence, it was possible to record the weight loss due to pure evaporation of the components, considered to be the pure organic polymeric content. The results are shown in Fig. 21. The two colors indicate the two different suppliers of the membrane products.

The results of the initial test are illustrated in Table 10. Cyclic freezing and thawing in air has the effect of a slight increase on the elongation performance. When thawing under water between each freezing cycle membrane M1 shows approximately 50 % reduction in elongation performance, whereas the other membranes are unaffected by the freeze–thaw exposure.

The main findings from the conducted elongation testing can be summarized as follows:

- Within the same test series, a consistent trend of significantly decreasing elasticity with decreasing temperature has been observed.
- A relatively large scatter is caused by varying thicknesses of the membrane within the same specimen as well as specimens with different thicknesses.
- Conditioning at RH95 % gives higher measured elasticities compared to specimens conditioned at RH50 %.
- Elasticity mainly increases with increasing polymeric content (shown in Table 11).

## 6.4 Crack Bridging

### *6.4.1 Specimens*

Specimens for crack bridging testing were produced by applying the membrane material on pre-fabricated test pieces of porous artificial sandstone (Fig. 22). Both spray-applied and molded membrane specimens were prepared. Three series of specimens, shown in Table 12, were

**Table 10** Initial test series of elongation carried out in Laboratory 1

| Membrane (all sprayed) | Measured elongation at failure, mean (%) | | |
|---|---|---|---|
| | No freezing, storage at 23 °C RH 95 % | 6 cycles[a], freezing to −20 °C in air, thawing at 23 °C RH 95 % | 6 cycles[a], freezing to −20 °C in air, thawing at 23 °C immersed in water |
| M1 | 194 | 212 | 94 |
| M2 | 20 | 22 | 18 |
| M3 | 131 | 135 | 131 |
| M4 | 4.3 | 4.9 | 6.2 |
| M5 | 438 | 457 | 408 |

[a] Freezing 24 h, thawing 24 h

**Table 11** Measured values for elongation for membrane according to DIN 53504 for two test series

| Test location | Membrane sprayed/molded | Pre-conditioning | Measured elongation (strain) at failure, mean (%) | | | | |
|---|---|---|---|---|---|---|---|
| | | | 23 °C | 0 °C | −3 °C | −8 °C | −12 °C |
| Laboratory 2 | M1 sprayed | RH 95 % | 45 | | 38 | 10 | 5 |
| | M5 sprayed | | 685 | | 89 | 20 | 8 |
| Laboratory 3 | M1 sprayed | RH 50 % | 20 | 9 | 6 | 0.6 | |
| | M5 sprayed | RH 50 % | 242 | 56 | 14 | 1.4 | |
| | M1 molded | RH 50 % | | 71 | 31 | | |
| | M1 molded | RH 95 % | 242 | 76 | 55 | | |
| | M5 molded | RH 50 % | | 21 | 9 | | |
| | M5 molded | RH 95 % | 446 | 40 | 13 | | |

prepared in order to cover a range of temperatures and moisture contents.

### 6.4.2 Procedure

The test method stated in DIN EN 1062 annex C4 dynamic tensile test, with the modifications developed by Wacker Chemie AG, described in Sect. 5.2.2 in this paper was followed. The crack width was increased in increments of 0.2 mm every 5 min. Immediately before a crack width increase was applied to the specimen, the membrane surface was visually inspected and deemed intact or ruptured. Any sign of initial rupture was interpreted as membrane failure (Fig. 23).

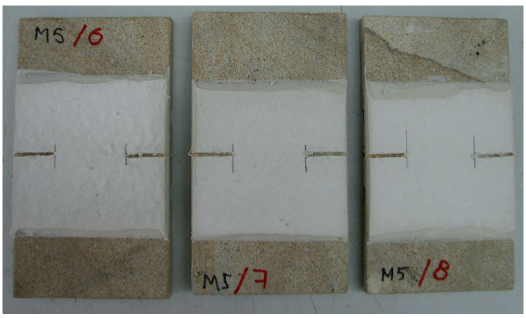



**Fig. 22** Specimens for crack bridging test with membrane applied on the surface of test pieces. The dimensions of the test pieces are 100 mm × 200 mm

### 6.4.3 Results

The results of test series 1 and 2 are shown in Fig. 24. For the specimens with spray applied membrane (series 1) values are shown as mean with 25 and 75 % percentiles. For the molded specimens (series 2) only average values are shown, since there were too few satisfactory readings to obtain statistical data. The specimens had slightly differing membrane thicknesses. Therefore the value for rupture was given as the ratio between crack width at rupture and the membrane thickness which varied from 1.9 to 5.5 mm. The variation of the membrane thicknesses was largest for the specimens with spray applied membrane. The results (Fig. 24) show that the crack bridging capacity decreases with decreasing temperature with the given conditioning of the specimens. Both membrane products M1 and M5 were found to bridge cracks with an aperture in the range of 0.4–0.8 times the thickness of the membrane at −8 °C. At

**Table 12** Matrix for conducted testing of crack-bridging

| Test series number | Membrane, sprayed/molded | Conditioned at RH (%) | Temperature at testing, number of specimens tested | | | |
|---|---|---|---|---|---|---|
| | | | 23 °C | 0 °C | −3 °C | −8 °C |
| 1 | M1 sprayed | 95 | 3 | 2 | 3 | 3 |
| | M5 sprayed | 95 | 3 | 3 | 3 | 3 |
| 2 | M1 molded | 50 | | 3 | 3 | |
| | M5 molded | 50 | | 3 | 2 | |
| 3 | M1 molded | 95[a] | | 3 | 3 | |
| | M5 molded | 95[a] | | 3 | 3 | |

[a] Cured in RH 95 % for 28 days immediately after molding prior to testing

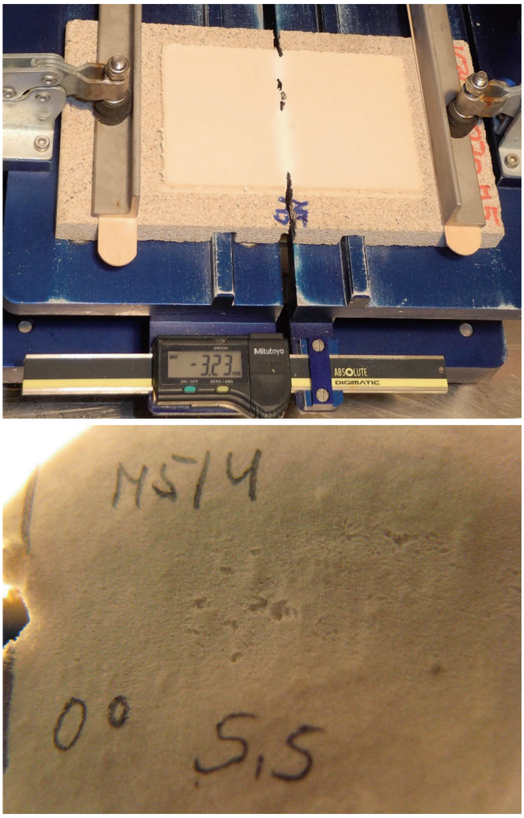



**Fig. 23** Crack bridging testing in progress. *Top* specimen in testing apparatus for precise measurement of crack aperture at rupture. *Bottom* definition of rupture with visible initial damage of the membrane

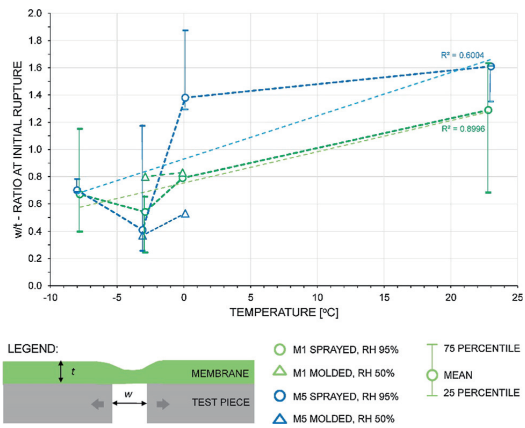



**Fig. 24** Results from crack-bridging testing for membranes M1 and M5 conducted at different temperatures and pre-conditioned at RH 50 and 95 %

23 °C the two membranes M1 and M5 were to bridge cracks with an aperture in the range of 1.3–1.6 times the thickness of the membrane. Series 3, tested only at 0 and −3 °C with molded specimens cured and pre-conditioned at RH 95 % for 28 days after application, did not exhibit any rupture at 11 mm which is the maximum crack width which the machine could produce. This indicates that the immediate curing after application at RH95 % leaves sufficient water in the membrane to act as a "softener" with resulting high elasticity.

## 6.5 Direct Shear Tests, Shear Bond Strength

### 6.5.1 Specimens

The specimens were produced from square panels using poured concrete with typical sprayed concrete mix design and spray applied membrane. In this way regular interfaces were achieved. The panels were stored under water for 5 months before specimens with 74 mm diameter were core drilled. The core specimens underwent another 30 days of storage under water. Throughout the storage under water a 40 mm wide strip strong tape was applied around the core completely covering the membrane and protecting it from direct water exposure. In this way the membrane only received exposure to water through the concrete pores.

### 6.5.2 Procedure

After water storage the specimens were prepared for shear testing by mounting them in a steel frame assembly. The process of preparing a series of three specimens in the test assembly and conducting the shear tests could be undertaken in 1 day. The assembly of the specimens is shown in Fig. 25 and the steps in the procedure are shown in Fig. 26.

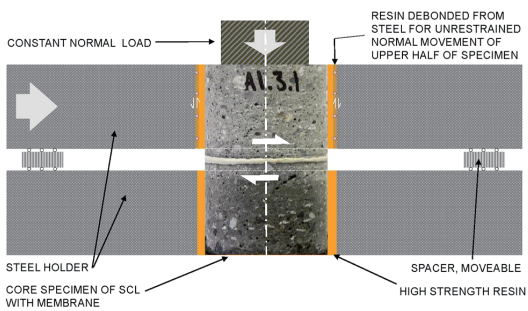
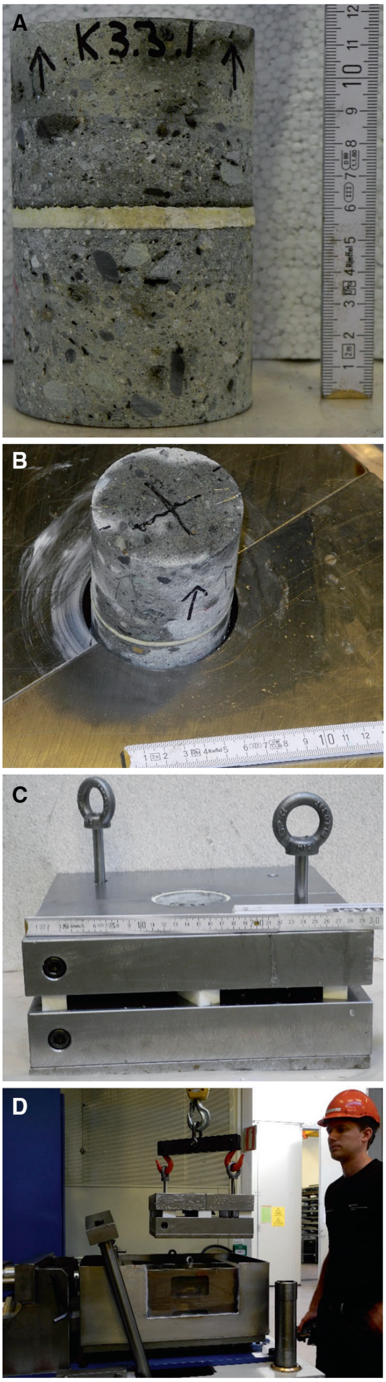



**Fig. 25** Conceptual diagram showing a section of the assembly with core specimen mounted in the steel frame used for direct shear testing




◀**Fig. 26** Direct shear testing of SCL structure. **a** Typical core specimen with 74 mm diameter. **b** Specimens placed in steel holder ready for fixing with high strength resin. **c** Specimen in complete assembly ready for testing. **d** Placement of test assembly into shear box

**Table 13** Conditions for shear testing

| Parameter | Condition |
|---|---|
| Shear loading | Constant displacement, 0.5 mm/min |
| Normal loading | Constant normal load, 0.45 MPa |
| Measured parameters during test | Shear displacement, normal displacement, shear load |
| Age of specimen at testing | 180 days |

Details regarding the testing procedure are shown in Table 13.

### *6.5.3 Results*

The results are presented as shear-stress versus shear displacement diagrams, providing the following information:

- Peak shear stress for the specimen.
- Shear displacement at peak stress.
- Maximum shear displacement within linear elastic behavior.
- Shear stiffness during linear elastic behavior.

The results for membrane M1 are shown in Figs. 27 and 28 and for membrane M5 in Figs. 29 and 30. A compilation of the recorded data is shown in Table 14.

Both membranes exhibit almost the same behavior within the initial deformation, showing linear shear elasticity up to approximately 1 mm shear deformation. Membrane M1 exhibits a slightly higher shear stress at this point, corresponding to the higher shear stiffness K1 compared to M5. M1 exhibits a clear bonding (adhesive) failure (Fig. 28) with peak shear stresses in the range of 0.55–0.85 MPa, after approximately 3–4 mm shear deformation. After the initial zone of shear elasticity, the two membranes have very different behavior. Membrane M1 exhibits increasing strain softening behavior and membrane M5 exhibits a bi-linear behavior with increased displacement. In the latter phase a lower shear stiffness can be observed. After approximately 7–8 mm horizontal displacement membrane M5 exhibits almost

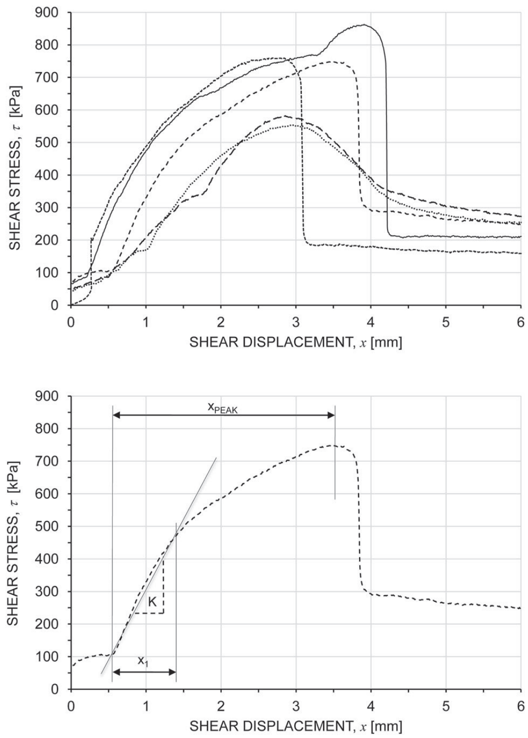



**Fig. 27** Results from shear testing for membrane M1 represented as shear stress versus shear displacement. *Top* diagram showing results for all five specimens. *Bottom* results for one specimen with recorded parameters, shown in Table 16

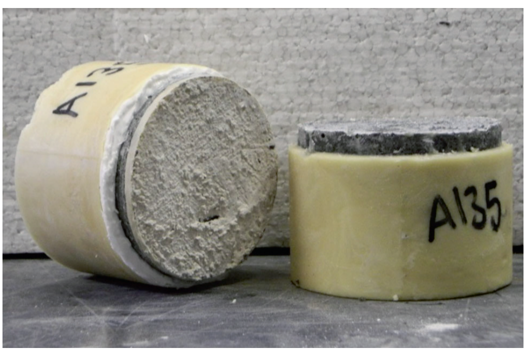



**Fig. 28** Specimen of membrane M1 after shear testing exhibiting a debonding (adhesive) failure between membrane and the substrate concrete

perfect plasticity and reaches a peak shear stress of approximately 0.45–0.5 MPa. No clear failure could be observed during the shear testing with the M5

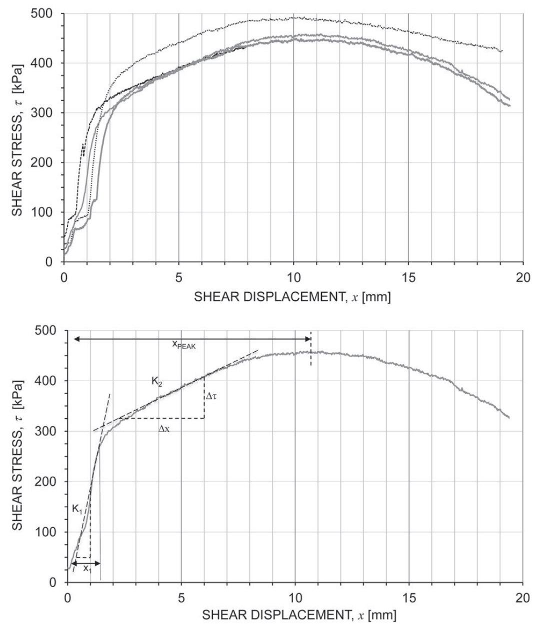



**Fig. 29** Results from shear testing for membrane M5 represented as shear stress versus shear displacement. *Top* diagram showing results for all five specimens. *Bottom* results for one specimen with recorded parameters, shown in Table 15

**Fig. 30** Specimen of membrane M5 after shear testing exhibiting shear (cohesive) failure in the membrane material

specimens. The tests for M5 were terminated after 19 mm of shear deformation. When removing the specimens from the test assembly, failure in the membrane could be observed since the upper and lower part of the specimen could easily be separated (Fig. 30).

**Table 14** Compilation of results from direct shear testing of membranes M1 and M5

| Membrane (number of specimens) | Shear displacement $x$: mean, mm (COV, %) | Shear displacement $x_{PEAK}$: mean, mm (COV, %) | Shear stiffness K1: mean, MPa/m (COV, %) | Shear stiffness K2: mean, MPa/m (COV, %) | Peak shear stress: mean, kPa (COV, %) | Membrane thickness range, mm | Measured moisture content in membrane, % (COV) |
|---|---|---|---|---|---|---|---|
| M1 (5) | 1.0 (33) | 2.5 (25) | 350 (20) | n.a. | 745 (16) | 3.5–4 | 15.7 (10) |
| M5 (4) | 1.1 (18) | 9.0 (7) | 297 (20) | 19 (9) | 450 (5) | 4–6 | 14.5 (12) |

## 6.6 Tensile Strength of Membrane-Concrete Interface

### 6.6.1 Specimens

Testing of tensile strength of the membrane-concrete interface was carried out in 4 different series (explained in Table 15), including laboratory tensile testing of 74 mm diameter core specimens drilled from panels, pull testing on panels with lining structure and in situ pull testing from full scale tunnel linings. The moisture content of the concrete and membrane materials of the specimens was measured whenever possible. The testing of the linings at the Gevingås and Ulvin tunnel sites was conducted parallel to the moisture condition sampling and testing. For the large scale laboratory lining structure (Fig. 7, Sect. 4.4) the moisture content of the sprayed concrete and membrane which was achieved after 6 months of moisture conditioning was found to be very close to the moisture content measured in tunnels (Holter and Geving 2015). Hence, the laboratory lining structure could be used for controlled freeze–thaw testing with realistic moisture content.

### 6.6.2 Procedure

Both testing procedures described in Sect. 5.3 in this paper were followed. Test series 1 was conducted with the original method stated in EN-ISO 4624 on core specimens which were moisture conditioned by immersion for a minimum of 14 days and subsequently tested in a laboratory tensile pull machine. Test series 2 was conducted with the in situ tensile test method on slabs cut from slabs of lining structure. Prior to testing, the slabs received different pre-treatment types with moisture exposure at immersion and cyclic freezing and thawing. Series 3 and 4 comprise the tensile testing which was done on full scale lining structures, either in situ in tunnels or on the large scale

**Table 15** Matrix for the different test series for tensile strength

| Test series | Type of lining structure for specimens | Testing method | Membrane products tested | Conditioning and exposure of lining structure or specimen | Age at testing |
|---|---|---|---|---|---|
| 1 | Sprayed panels 600 mm × 600 mm | Drilled core specimens, tested in pull test machine | M1<br>M2<br>M3<br>M4 | Dry<br>Saturated by immersion for 14 days<br>Frozen/thawed | 12–14 months |
| 2 | Sprayed panels 600 mm × 600mm | Laboratory<br>In situ pull tests conducted on panels | M1<br>M5 | Dry<br>Saturated by immersion for 60 days<br>Frozen/thawed 6 times, −20/+20 °C<br>Testing in frozen and thawed condition after freeze–thaw exposure | 18–19 months |
| 3 | Full scale lining in tunnel (Gevingås and Ulvin test sites) | In situ pull tests in tunnel lining | M1 | In situ moisture exposure in rock mass | Ulvin: 29 months<br>Gevingås: 37 months |
| 4 | Laboratory lining structure on rock mass with water exposure and freezing | In situ pull tests in lining structure | M1 | Moist without freezing exposure<br>Moist frozen-thawed to −3 at membrane, tested after 20 and 35 cycles<br>In frozen condition after 35 cycles | 19–25 months |

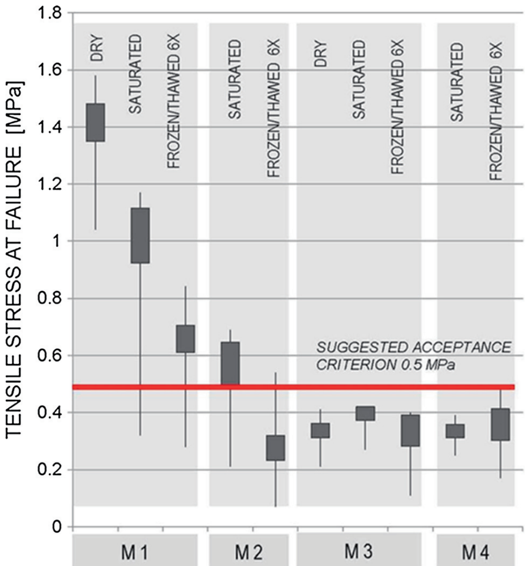



**Fig. 31** Results from tensile bond tests for test series 1 (Table 15), drill core specimens with diameter 74 mm. Membrane numbers M1–M4 are explained in Sect. 6.3.2

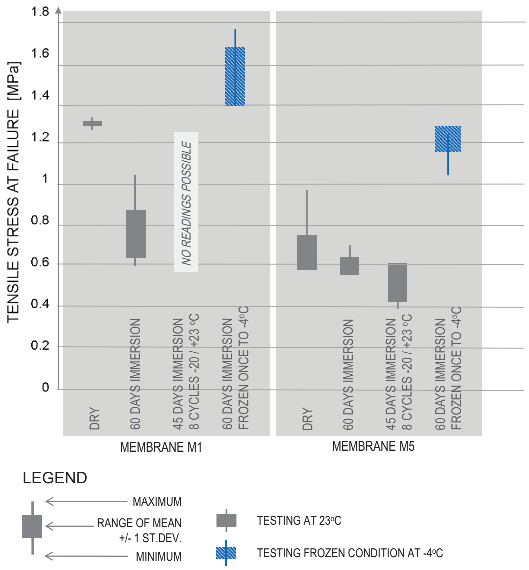



**Fig. 32** Results from tensile bond tests for test series 2 (Table 15), in situ pull test method

laboratory lining structure. The adopted in situ tensile test method described in Sect. 5.3 was used for this purpose.

### 6.6.3 Results

The results for the first test series, which included the membranes M1, M2, M3 and M4 are shown in Fig. 31. With this testing method membrane M1 exhibits a range of strength reduction between 1.1–1.5 MPa tensile bonding strength (comparing dry specimens to saturated) and frozen/thawed specimens showed even more reduced tensile strengths. The scatter in measured tensile strength using this test method is relatively high. The membranes M2, M3 and M4 exhibit relatively low tensile strengths close to or below the recommended requirement of 0.5 MPa tensile strength.

The membrane M5 was introduced as a substitute for M1 and M2. M3 was discontinued for further testing. The results for series 2 which only includes membranes M1 and M5 are shown in Fig. 32. This testing context shows that saturation through 60 days immersion of the entire slab gives a significant reduction of the tensile strength for M1 and a slight reduction of tensile strength for M5 compared to dry specimens. Eight freeze–thaw cycles to −20 °C result in a reduction of tensile strength from approximately 0.7–0.5 MPa for M5. For M1 no readings were possible after the freeze–thaw cycles due to jamming of the drilling equipment at the membrane. For both membranes the tensile strength measured in frozen condition was significantly higher than for the measured strengths in dry or saturated condition.

Series 3 and 4 (Table 15) represent in situ readings conducted with horizontal drilling on tunnel walls in different lining sections ranging from complete tunnel to large scale lining structure in a laboratory (Fig. 7). Exposure to moisture took place through the substrate sprayed concrete. Results from the tunnel test sites exhibit high tensile strengths in the range 1.1–1.6 MPa, with 1.3 MPa as the mean value (Fig. 33, left part). Two readings could be made at a wet crack (defect) in the inner lining, at which 0.8 MPa tensile bond strength was measured. Since realistic moisture contents were achieved in the lining structure at the SINTEF freezing laboratory, the effect of cyclic freezing and thawing on tensile bond strength could be measured (Fig. 33, right part). An initial tensile bond strength of 1.4 MPa at realistic moisture content was measured. After 20 and 35 freeze–thaw cycles with −3 °C minimum temperature at the membrane during each cycle, a slight reduction to respectively 1.15 and 1.1 MPa tensile strength could be measured. A tensile strength in the range of 1.1 to 1.3 MPa was measured in frozen condition at −3 °C at the membrane. An additional freeze–thaw exposure with 20 cycles with a minimum temperature of −7 °C at the membrane was conducted after the first 35 cycles to −3 °C. Tensile strengths ranging from 0.4 to 0.7 MPa were measured after this exposure. Difficulty in

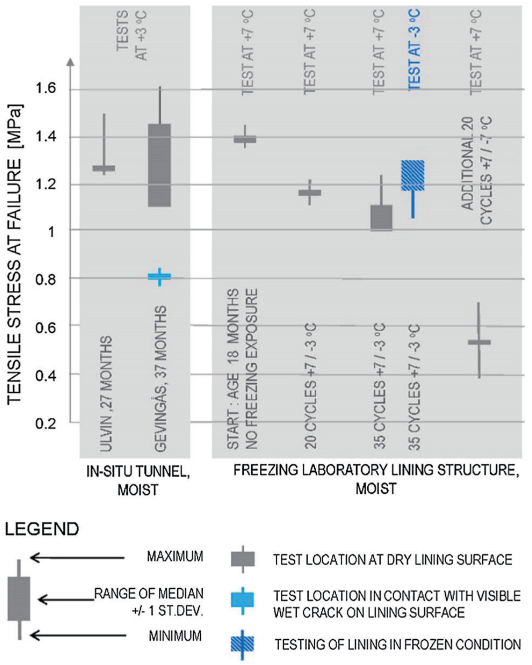



**Fig. 33** Results from tensile bond tests for membrane M1, test series 3 and 4 (Table 15), in situ pull test method

conducting the pull tests was experienced during the last series at −7 °C due to damage in the outer part of the concrete lining.

### 6.7 Microscope Analyses of the Interfaces Between Concrete and Membrane

Scanning electron microscope (SEM) analysis of the interfaces of the membrane-concrete structure was conducted on specimens obtained from slabs which had been constructed with realistic application methods of both materials. The main purpose of this analysis was to study any visual characteristics or significant differences between the two interfaces, illustrated in Fig. 34 which could be of importance for the interpretation of the tensile bond and shear strength test results.

The interface on which the membrane has been applied on the substrate concrete (interface 1), shown in Fig. 35 exhibits a sharp contrast between membrane material and concrete material. Membrane material can be seen filling the irregularities of the sprayed concrete surface. The two materials exhibit distinct phases with no visible transition zone.

The interface on which the secondary lining concrete has been applied onto the membrane (interface 2) shown in

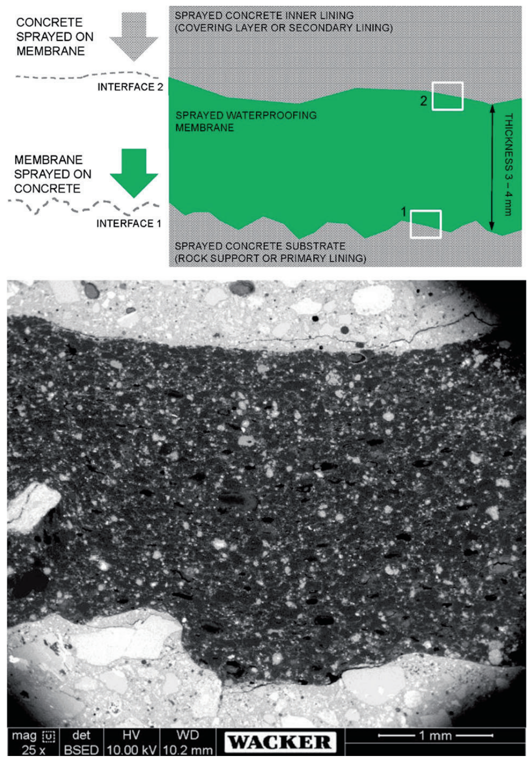



**Fig. 34** Principle sketch and photo of the two interfaces between membrane and sprayed concrete. The interfaces 1 and 2 are shown respectively in Figs. 35 and 36 (courtesy by Wacker Chemie AG)

Fig. 36 exhibits a different morphology than interface 1. A transition zone of approximately 15–25 µm thickness with visible effects of the impact of the sprayed concrete on the membrane can be clearly seen. This transition zone consists of a mineral phase with visible needle shaped crystals which separates the membrane material from the sprayed concrete material. With spectral analysis the mineral substance at the interface was found to be mainly composed of calcium carbonate $CaCO_3$.

## 7 Discussion of Results

### 7.1 General

The testing of deformability and mechanical strength in this study contain accelerated or short term tests with main aim of simulating a loading scenario which takes place in the tunnel lining. The loading scenarios caused by thermally induced deformations considered in the loading

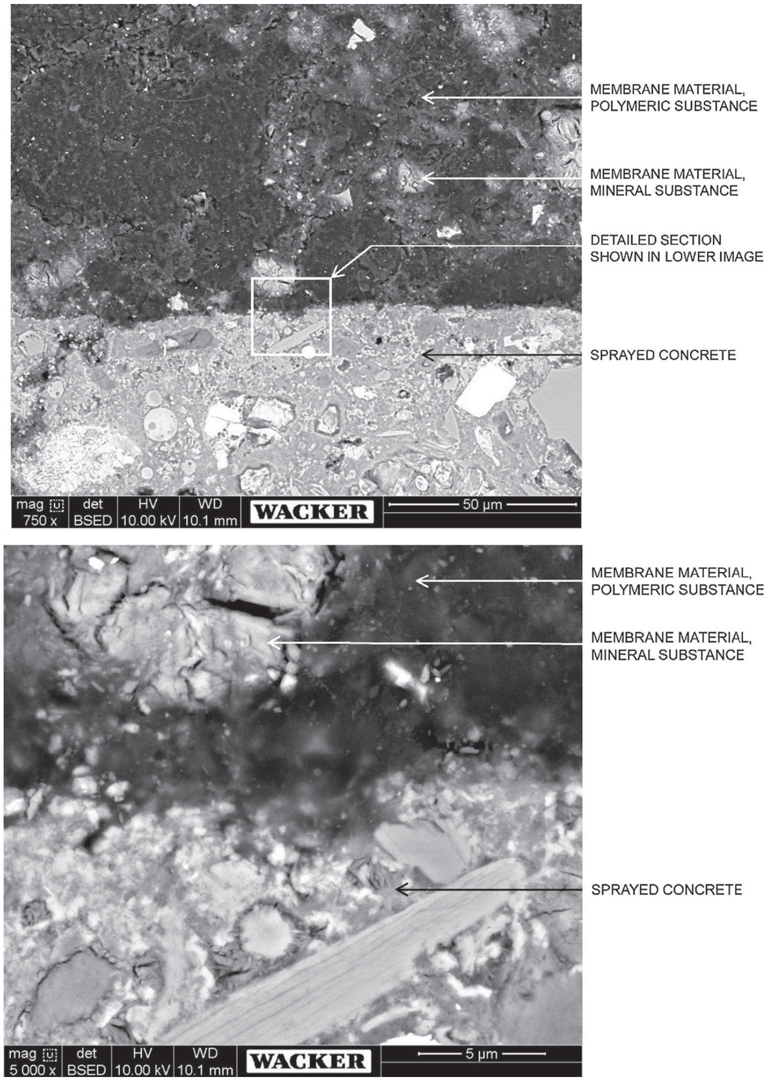



**Fig. 35** Images obtained by SEM-microscopy of interface 1, with membrane spray applied onto a primary lining (rock support) sprayed concrete substrate. *Top* 750× enlargement. *Bottom* 5000× enlargement (courtesy by Wacker Chemie AG)

model take place over several months. Possible effects of small and very slow deformations such as healing of the material, creep or reduction of strength have not been taken into account for short term tests.

### 7.2 Effect of Moisture Conditions of Specimens

Establishing realistic moisture condition of specimens for laboratory testing of this category of membranes is important and difficult. A test result should always be reported and evaluated with respect to its moisture condition. The results from in situ tensile bond strengths (with realistic moisture exposure) compared to specimens which are moisture conditioned by immersion, show that immersion very likely represents a too severe exposure to water, and gives lower strength values than realistic values. A consequence of this is that a complete testing program for membranes should include the construction of a full scale lining section in order to verify properties under realistic conditions in addition to findings from laboratory tests.

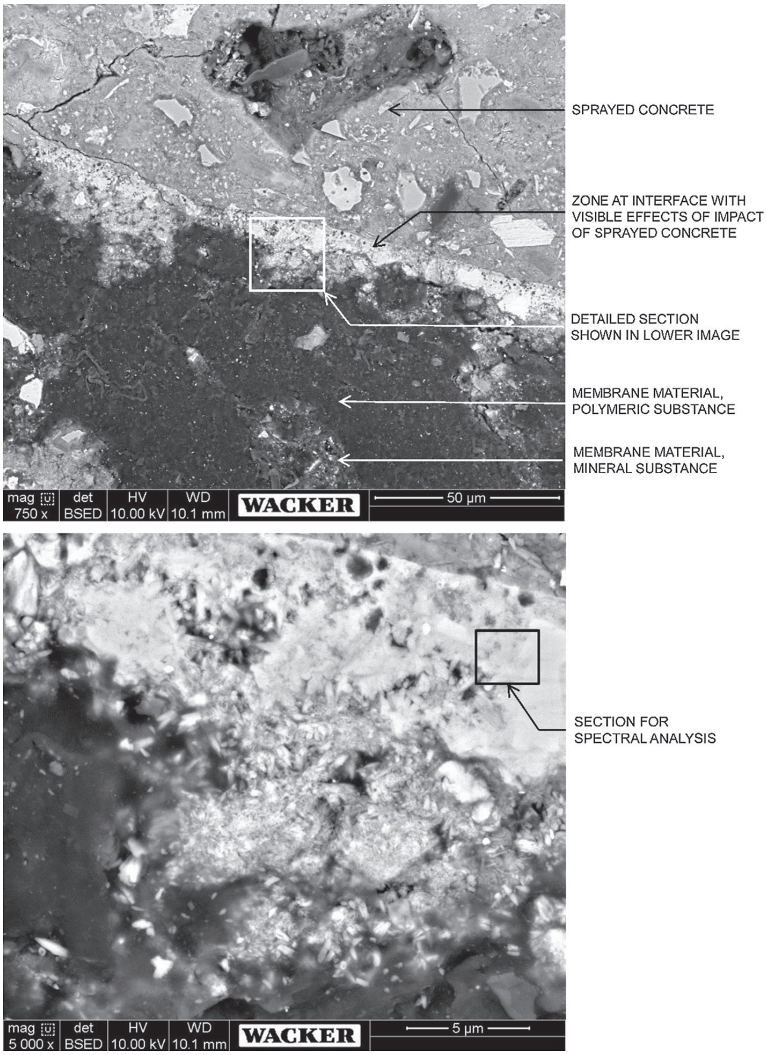



**Fig. 36** Images obtained by SEM-scanning of interface 2, with sprayed concrete applied onto the sprayed membrane surface. *Top* 750× enlargement. *Bottom* 5000× enlargement (courtesy by Wacker Chemie AG)

### 7.3 Thermal Exposure

The measured and calculated temperature profiles in the lining structure in this study cover freezing exposure with temperatures in the tunnel air in the range of −6 to −9 °C. Such temperatures in the tunnel air will occur under severe winter climate with outside air temperatures below −10 °C over sustained periods of time (STA 2012). The rock mass temperature at shallow locations in Scandinavia is normally 6–8 °C. This was measured in two Swedish studies (STA 2012) as well as the measured rock temperatures at the Ulvin site (Fig. 6, Sect. 4.4), as well as in the Gevingås rail tunnel (Holter and Geving 2015). The precise air temperature conditions at a given location in a tunnel needs to be assessed in each single case based on the local meteorological conditions and the ventilation characteristics of the tunnel.

### 7.4 Effect of Geometry of Specimens

The interfaces between membrane and sprayed concrete in tunnel linings exhibit surfaces with a certain degree of roughness. The measured mechanical properties shear and tensile strength will be influenced on the geometry of the interfaces which in turn will cause increased scatter. Our laboratory testing of specimens with planar surfaces represents ideal and unfavorable geometrical conditions, and takes no account for effects of surface roughness. For the specimens prepared from slabs with lining structure, the concrete surfaces were prepared by floating in order to produce the same geometry for all specimens. Our test results obtained from specimens with planar surfaces cannot be directly translated to the in situ properties. The peak stresses obtained in the laboratory for tensile and shear testing are likely to be lower than what would be the case under realistic interface conditions regarding moisture and geometry. On the other hand, specimens with the realistic roughness of the sprayed concrete surfaces would have introduced scatter making the interpretation of the results difficult, as well as limiting the reproducibility of the tests. For the tensile strength, our in situ measurements are the most representative. Such in situ measurements should be included in a test program in order to obtain values from realistic surfaces in addition to simplified or idealized surfaces during laboratory testing.

### 7.5 In situ Mechanical Loading of Lining and Membrane

The construction sequence of a waterproof SCL structure normally implies that the membrane and inner lining be applied after tunnel breakthrough, or several months after excavation and construction of the primary lining. Therefore the primary lining needs to be designed to be stable and designed for any rock mechanical loads before the membrane is applied. In our study we have therefore only included loads which can be imposed to the membrane by the possible effects of the membrane itself or the inner lining sprayed concrete.

### 7.6 Elongation and Crack Bridging

Elongation performance of a sprayed membrane according to DIN53504 (2009) can only give an indicative figure for the required elasticity for a tunnel lining purpose. This elongation performance exhibits significantly higher sensitivity to lower temperatures compared to the crack bridging. Conclusions based only on elongation results for temperatures 0 and −3 °C would likely deem the membrane unsuitable for such thermal exposure. The crack bridging results show significant performance at freezing temperatures, although a decreasing performance at temperatures 0 °C and below is observed.

We have applied strain loads on the membrane within a range to be expected by the effects of shrinkage and thermal expansion. Thermal fluctuations from approximately −3 to 15 °C can be expected at the membrane location within the lining structure. Thermally induced crack opening with an average crack distance of 1 m and a thermal change of 18 °C can be calculated to be in the order of 0.2 mm based on the thermal expansion coefficient. Our in situ crack measurements suggest a 0.2 mm crack opening for a drop in temperature of 6 °C. With a total thermal change over the year of approximately 18 °C in the lining structure at the membrane, 0.6 mm crack opening can be assumed. The possible shear deformations along the membrane interfaces caused by differential shrinkage or thermal expansion are in the same order of 0.5–0.6 mm. Our suggestion to use 1 mm as a crack bridging requirement and 1 mm as a critical shear deformation magnitude is therefore likely to be on the conservative side. Our crack bridging testing takes no account for any hydrostatic exposure at the cracks. We have only included the effects of high moisture content achieved by conditioning at RH 95 %.

### 7.7 Shear Performance

Shear testing of the membrane can contain several sources of error such as the loading rate, the normal loading mode and a possible oblique membrane plane relative to the shear direction. The applied loading rate during the test in the laboratory was 0.5 mm per minute whereas an in situ shear straining of the membrane most likely would take several months. Effects of creep and self healing therefore most likely would occur. Such effects are not accounted for in our short term test. The specimens for our tests had floated sprayed concrete substrate surfaces and were moisture conditioned by immersion. This likely represents an overexposure to moisture compared to in situ conditions. Hence, our laboratory findings for peak shear stress and shear stiffness are likely to be lower than values in realistic moisture exposure conditions.

### 7.8 Tensile Bond Strength

The site testing show consistent high values for tensile bond strength (Sect. 6.6.3, Fig. 33). At testing these lining sections had a history of several thermal expansion cycles as well as the exposure to the differential shrinkage between the two concrete layers. The test results from the Ulvin site also include one complete freeze–thaw cycle to approximately −3 °C at the membrane location. The measured high values for tensile strength indicate no in situ degradation of the lining after 4 years.

The laboratory testing on core specimens possibly contains three main sources of error: the geometry of the interfaces, the moisture condition of the specimen and the alignment of the pull direction parallel to the core axis. In addition the effect of a short term test with a duration of a few minutes might fail to account for all long term effects.

Float finished concrete surfaces will normally result in a locally higher water/binder content and consequently higher porosity and possibly higher permeability. A slightly higher water exposure at the interface between membrane and concrete with a specimen with float finished surfaces compared to non-floated surfaces is therefore possible. The alignment of the pull equipment based on visual assessment will sometimes be difficult. Specimens without a perfect alignment in the testing machine might receive partial bending loads, and hence exhibit lower peak stress during the test. The in situ pull test method described in Sect. 5.3, Fig. 15, eliminates the three afore mentioned sources of error. However, with the available equipment a controlled loading speed could not be precisely applied. The effect of wet core drilling for either of the methods is unavoidable. Water exposure will soften the membrane at the core surface. When drilling in a downwards vertical direction on a slab of lining structure, the drilling water will fill the core groove and expose the membrane to water immediately before testing. When drilling horizontally in a lining structure the exposure to water will be less.

### 7.9 Performance Under Freeze–Thaw Exposure

Our investigations pertaining to freeze–thaw durability comprise tensile bonding strength, elongation and crack bridging. The findings from the tensile testing after freezing exposure to −3 °C indicate that no significant damage occurs at this temperature. The likely explanation for this is the unsaturated condition of the concrete and membrane materials. This allows the volumetric expansion during the freezing of water to buffer into air filled voids without creating damage. For temperatures lower than −3 °C at the membrane location, thermally insulating measures need to be considered.

### 7.10 Durability and Service Lifetime

Prediction of service lifetime under freezing exposure is an important question. Our testing of tensile bond strength contains accelerated freeze–thaw tests in order to simulate a slightly more severe exposure with a high number of cycles which can be related to a period of service time. The number of freeze–thaw cycles that occur per year will vary from year to year in addition to the characteristics of the location. For tensile bond we have conducted 35 cycles to −3 °C at the membrane with 48 h per cycle which resulted in only minor reduction of tensile bond strength. Effects of healing between each freeze–thaw cycle are not accounted for in such an accelerated test layout. This indicates that real exposure would be less severe than our testing, and that our findings with high tensile strength after freezing exposure is likely to be realistic, or even conservative. Only when exposing the lining structure to 20 freezing cycles to −7 °C at the membrane location following the 35 cycles at −3 °C, a significant reduction in tensile bond strength could be observed. A precise service lifetime prediction is not possible based on our results. However, when this lining system is used in tunnels with moderate freezing exposure, with a lowest temperature of −3 °C at the location of the membrane, a service life time of 100 years or more is likely.

### 7.11 Recommended and Planned Further Work

The time dependent effects of in situ moisture exposure will be investigated further with continued sampling and testing of moisture content as well as in situ tensile bonding strength at the test sites.

Our study includes cases with low hydrostatic pressures. The effects of higher hydrostatic pressures (more than approximately 2 bars at the interface between rock and concrete) cannot be substantiated based on our results. Further material testing and large scale model investigations verified by field testing in order to substantiate the detailed behavior at the water filled cracks which expose the membrane are required. Such testing should account for all relevant material properties.

The detailed shear load characteristics need to be investigated in further depth. The main issues are: effects of long term, slow loading, creep and the normal loading mode, as well as the normal stiffness (with respect to the membrane surface) of the secondary lining structure.

## 8 Conclusions

A study of the properties of sprayed membranes for SCL in hard rock has been carried out with the following main scope:

- Assessment of loading, moisture and freezing exposure conditions based on field and large scale laboratory investigations.
- Evaluation of laboratory investigation methods.
- Conducting of laboratory investigations.
- Assessment of membrane properties including performance under freeze–thaw exposure.

The main findings from this study are the following:

- The main mechanical loading mechanisms on the membrane have been found to be represented by movement over cracks in the sprayed concrete and shear straining caused by differential shrinkage and thermal expansion.

- The moisture exposure to the membrane through the interfaces with the sprayed concrete leads to an in situ moisture content corresponding to 30–40 % of the membrane's maximum water uptake potential. This has been found to be governed by the moisture properties of the membrane and concrete materials and the bonded contacts between these two materials.
- Membrane products with low polymeric content (below 70 %) exhibit low elasticity and are most likely unsuitable for tunnel waterproofing purposes in a bonded SCL context.
- Testing methods need to include details regarding moisture preconditioning and moisture exposure in order to test realistic materials and substantiate statements on in situ performance.
- A test program should include field testing in order to assess the relevance of the findings from laboratory testing.
- Testing of tensile bond strength on core or slab specimens in the laboratory which are conditioned by immersion, tend to give slightly lower measured values compared to site or large scale model testing.
- Testing of crack bridging shows decreasing performance at decreasing temperature. With 3 mm membrane thickness bridging of 1 mm crack opening at −3 °C at the location of the membrane in the lining has been found possible.
- Testing of shear properties indicate linear shear elasticity up to approximately 1 mm shear deformation.
- Testing of tensile strength show high in situ tensile bond strengths in the range of 1.1–1.5 MPa after 4 years.
- Exposure to cyclic freezing-thawing shows no significant reduction of the tensile bond strength at −3 °C at the membrane location.
- Further work is required to substantiate the performance of an SCL lining structure exposed to high hydrostatic pressures as well as effects of long term mechanical exposure.

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# Paper 6

*Title:*

Effects on ground water pressure in the immediate rock mass around partially drained SCL with bonded waterproof membrane

*Authors:*

Karl Gunnar Holter

Rolf Christiansson

Chhatra Bahadur Basnet

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# EFFECTS ON GROUND WATER PRESSURE IN THE IMMEDIATE ROCK MASS AROUND PARTIALLY DRAINED SCL WITH BONDED WATERPROOF MEMBRANE

Karl Gunnar Holter[1]

Rolf Christiansson[2]

Chhatra Bahadur Basnet[1]

[1] Department of Geology and Mineral Resources Engineering, Norwegian University of Science and Technology, Sem Sælands vei 1, 7491 Trondheim, Norway

[2] SKB Swedish Nuclear Fuel and Waste Management Company, P.O. Box 250, 101 24 Stockholm, Sweden

ABSTRACT

Sprayed concrete tunnel linings (SCL) waterproofed with bonded membranes in principle represent undrained lining structures. When used with a tunnel design with a drained horizontal invert, the undrained walls and crown of the lining may result in a certain water pressure behind the lining and a redistribution of the ground water flow into the tunnel. The tunnel will therefore constitute a partially drained structure in which the ground water pressure in the immediate rock mass surrounding the tunnel will be influenced by the ground water flow towards the invert. This paper presents an investigation of the ground water pressures in the rock mass behind such linings in hard rock environment from four test sites in Scandinavia. The main reason for obtaining this information is to assess any destabilizing effects in the rock mass caused by an undrained lining with the bonded membrane design.

Measurements of the ground water pressures in the rock masses surrounding SCL structures with a drained invert indicate a water pressure reduction of approximately 200-300 kPa from the background hydrostatic pressure along a distance of 3-6 m to 1 m distance from the lining surface. The test sites exhibited background hydrostatic pressures in the range of 680-780 kPa and hydraulic conductivities of the rock mass in the range of $10^{-9}$ to $10^{-8}$ m/s.

The results from a discontinuous numerical model and a continuum numerical model of the water flow in the rock mass indicate an increasing water pressure in the rock mass at decreasing distance to the lining surface, when no consideration for any local features near the lining was taken into account.

A likely explanation for the measured water pressures in the immediate rock mass is the effect of the excavation damaged zone (EDZ). Under the tested conditions this will be a favorable effect for the condition of the lining structure and the stability of the immediate rock mass. Further investigations of the hydraulic properties of the excavation damaged zone are required to substantiate this behavior under a wider range of conditions.

Keywords: Sprayed concrete lining, tunnel waterproofing, sprayed waterproofing membrane, ground water pressure, effects of tunnel lining, excavation damage zone, monitoring

ABBREVIATIONS AND DEFINITIONS

SCL — Sprayed Concrete Lining. Permanent tunnel lining system based on fiber reinforced sprayed concrete as the structural material with alternative possible waterproofing measures which are integrated into the sprayed concrete structure. Such linings may also include rock bolts for rock reinforcement

EDZ — Excavation Damaged Zone. Zone of a certain thickness in the rock mass near the excavated contour of an underground opening which in which hydromechanical and geochemical modifications induce significant changes in flow and transport properties. These changes may, for example, include one or more orders-of-magnitude increase in flow permeability (Bernier et al. 2005)

# 1 INTRODUCTION

SCL waterproofed with a sprayed membrane in principle represents an undrained lining structure. Such linings have been used in several design options including completely tanked linings and partially drained linings in which a significant portion of the tunnel perimeter has no waterproofing.

Several decades of experience is obtained in the Scandinavian countries with sprayed concrete linings applied on the rock surface in a tunnel without any special measures to account for any ground water pressure. The general experience is that the sprayed concrete lining is drained and that ground water does not cause any significant loads on the sprayed concrete. Even during construction of more than 40 subsea tunnels in Norway in hard rock conditions with up to 250 m static head. No detrimental effects of ground water pressure in the immediate rock mass have been observed, other than in severe weakness zones (Nilsen 2014).

Until recently, the normal practice in the hard rock environment in Scandinavian countries has been to construct tunnels with a permanent rock support lining based on rock bolts and sprayed concrete and a separate drainage and thermal insulation lining. The rock support lining is considered a drained structure with this type of design (Fig. 1 left). In this paper, waterproof SCL with a spray-applied membrane in the walls and crown is considered. The category of sprayed membrane investigated in this study are based on ethyl-vinyl-acetate (EVA) co-polymers. The waterproof SCL with a drained invert will allow ground water to enter the tunnel through the invert (Fig. 1, right).

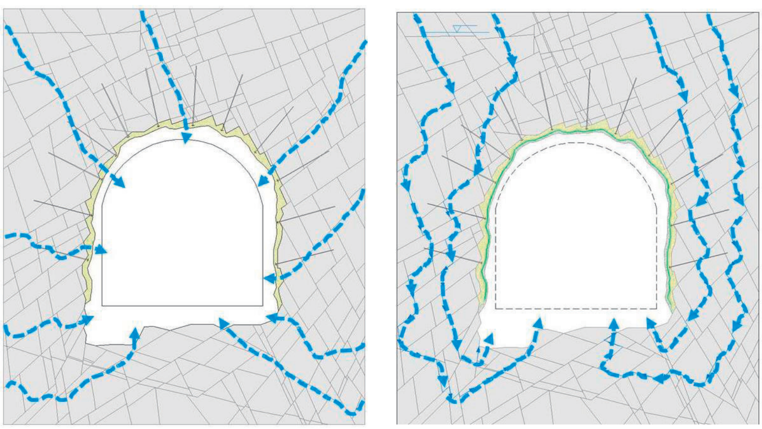

Fig. 1 Drawing showing the hypothesized effect on ground water flow of the waterproof and undrained SCL structure. Left: SCL without any waterproofing. Right: SCL with undrained waterproofing in walls and crown and a drained invert. (Holter 2014).

The partially drained lining option (Fig. 1 right) is the main issue studied in this paper. The motivation for this lining design is to waterproof a sprayed concrete lining with relatively small thickness (totally 150-250 mm) and still expect permanent waterproofing function without long term negative effects. This lining design has so far been used in ground with low hydrostatic pressures up to a maximum of approximately 200 kPa (Nermoen et al. 2011, Holter & Foord 2015). At higher pressures, measures like drainage stripes or drainage channels have been used for water pressure relief at the rock/concrete interface in the lining.

The aim of this paper is to analyze the effects on the water pressure in the surrounding rock mass when constructing the SCL as a waterproof and undrained structure in the walls and crown of the tunnel and drained invert. The important issue to consider is the effect of the undrained waterproofing of the SCL. Such a permanent lining design is only feasible if possible detrimental effects of the ground water pressure to the tunnel lining can be neglected. Furthermore, any possible ground water pressure in the immediate rock mass which eventually could require an increased lining thickness or heavier rock support are important to evaluate. In cases with rock masses with higher hydraulic conductivities, the surrounding rock mass of the lining will also consist of a pre-grouted zone. The detailed effects of this were unable to be experimentally investigated in this study. The study presented in this paper is based on four tunnel sites and numerical simulations of one of the cases, and is illustrated in Fig. 2.

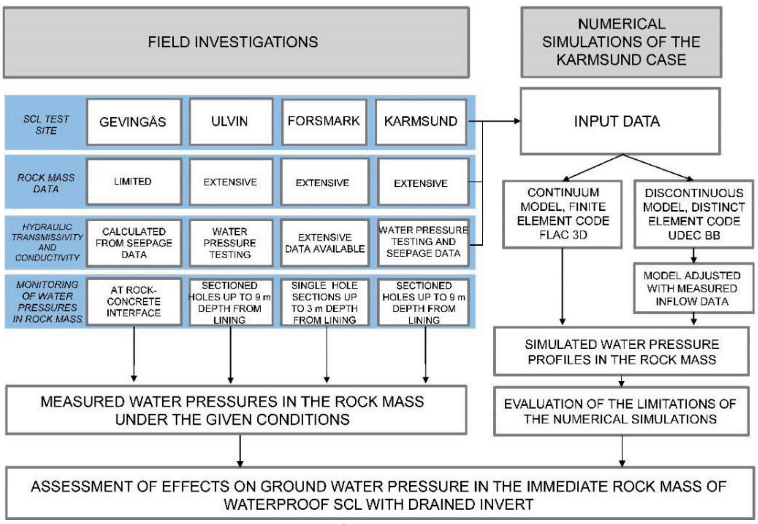

Fig. 2 Diagram with the structure of the analysis in this paper.

## 2 WATER FLOW IN THE ROCK MASS AROUND A TUNNEL

### 2.1 Conceptual model for partially drained lining structure

The bonded property at the interfaces of the constituent materials including the membrane means that there will be no draining function of the lining. The function of the membrane in the SCL structure is to act as barrier for flow of water on cracks in the concrete lining. The lining structure is illustrated in Fig. 3. In a hard rock environment total sprayed concrete thicknesses for modern rail and road tunnels in the range of 150-250 mm are realistic.

The flow of water on paths in the immediate rock mass towards the invert was hypothesized by Holter (2014), and is illustrated in Fig. 1. This model implies the occurrence of a certain water pressure in the immediate rock mass of such a lining. The water pressure in the immediate rock mass will be higher in the case with an undrained SCL structure in the walls compared to a case with a drained lining, since there is a shorter distance to the drained tunnel surface.

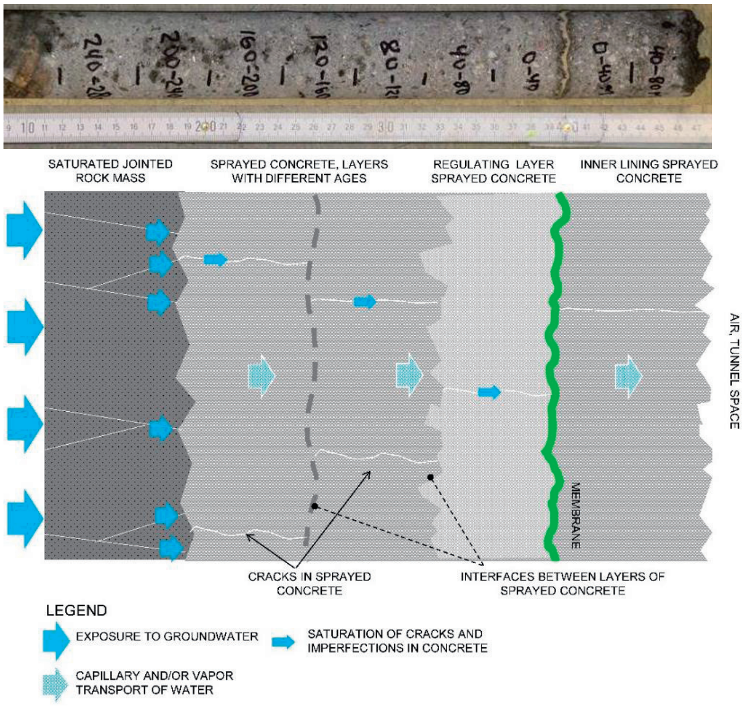


Fig. 3 Top: Photo of a 50 mm diameter drill core through SCL with bonded membrane. Bottom: Conceptual model of the SCL structure with constituent materials, water exposure and water flow processes (Holter 2015).

## 2.2 Models for groundwater flow in crystalline rock masses

In rock masses with hard crystalline rock types the ground water flow is governed by the flow of water on the discontinuities in the rock (NBG 2000, Palmström & Stille 2010, Gustafson 2012). The intact rock material is considered impermeable. At the scale of a tunnel or drill hole sections for pumping tests with lengths in the order of 0.5 to 3 m the water will flow on joints and be governed by the hydraulic transmissivities of the joints. On a larger scale a consideration of the average hydraulic conductivity of the rock mass should be used. However, this depends on the density of the joints (Fig. 4).

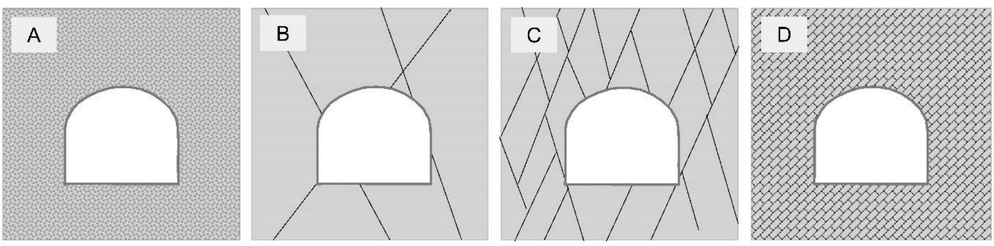


Fig. 4 Drawings showing rock masses with different conductive properties. A: Continuum: rock mass with porous rock and conductive flow through rock material. B: Discontinuous medium: rock mass with impermeable rock material and few conductive joints. C: Discontinuous medium: moderately jointed rock mass with impermeable rock material and conductive joints. D: Continuum: densely jointed rock mass with impermeable rock material and conductive joints (based on STA 2014).

A simplified approach is to consider a rock joint consisting of two parallel planar surfaces with a certain aperture. In reality the surfaces of joints are neither planar nor completely parallel. The joint surfaces can be in contact with each other in several ways. There can be fillings in the fracture which leave only a portion of joint open for fluid flow, such as in limited patches or channels. A conceptual model for this was presented by Butron (2012) and is illustrated in Fig. 5.

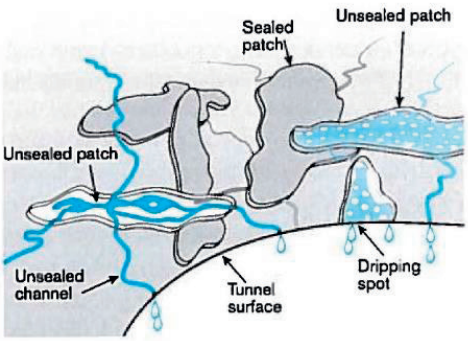


Fig. 5 Conceptual model for a conductive system of rock joints in crystalline rock (Butron 2012).

Studies of the mechanical aperture of joints, their variation and how this influences the hydraulic transmissivity and how this relates to the hydraulic joint aperture have been carried out by Zimmerman and Bodvarsson (1996). For the purpose of flow calculations in this study, the frequently used parameters joint hydraulic transmissivity and the hydraulic joint aperture are important.

In this study water pressure testing was conducted in boreholes section with 500 mm length and hole diameter 67 mm. The equation proposed by Moye (1967), Eq 1, was used to calculate the section transmissivities and the equation proposed by Gustafson (2012), Eq. 2 for the joint transmissivities. The parameters from the water pressure testing are illustrated in Fig. 6 (Gustafson 2012). T is the transmissivity, Q is the water flow during pumping at steady state, $\Delta h$ is the pumping head, L is the length of the pressurized section of the hole, R is an assessed persistence of the joint, normally in the range of 3 to 5 m, and $r_w$ is the diameter of the hole. Gustafson's equation has been found to be valid for cases where the joint transmissivity is very much higher than the hydraulic conductivity k of intact rock.

$$T = \frac{Q}{2\pi\Delta h}\left[1 + ln\left(\frac{L}{2r_w}\right)\right] \qquad \text{Eq. 1}$$

$$T = \frac{Q}{2\pi R\Delta h} \qquad \text{Eq. 2}$$

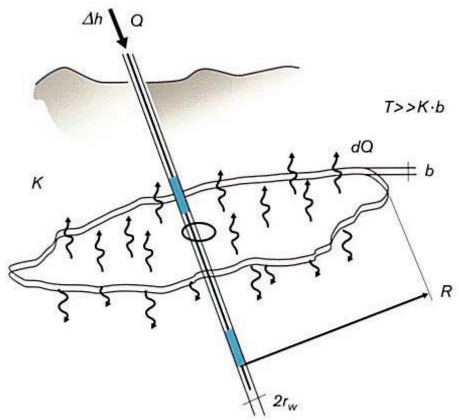


Fig. 6 Hydraulic test in a conductive rock joint (Gustafson 2012).

Using the cubic law (Eq. 3), originally proposed by Louis (1967), the hydraulic aperture $b$ can be estimated from the joint hydraulic transmissivities with the cubic law in the following form (Fransson 2001):

$$b = \sqrt[3]{\frac{T \cdot 12 \cdot \mu_w}{g \cdot \rho_w}} \qquad \text{Eq. 3}$$

Where $T$ is the joint hydraulic transmissivity, $\mu_w$ is the dynamic viscosity of water, $\rho_w$ is the density of water and $g$ is the acceleration of gravity.

## 3 EXECUTED INVESTIGATONS

### 3.1 Overview and goal

An investigation program was undertaken to study the effects on the groundwater pressure in the surrounding rock mass of an undrained SCL structure with bonded waterproofing membrane. The tunnels which were investigated have completely drained inverts. The investigations consist of the following:

- Construction of full scale SCL sections with bonded waterproofing membrane at 3 different tunnel sites
- Water pressure testing for estimation of hydraulic transmissivities of the rock mass at two of the sites
- Monitoring of water pressures in sectioned bore holes up to 9 m length at two of the sites
- Monitoring of water pressures in single section bore holes up to 3 m length at one site
- Monitoring of water pressures at the rock-concrete interface in the lining structure at one site

The locations of the four test sites are shown in Fig. 7. A comparison of the four test sites with the geometrical layout of the tunnel linings and the hydrogeological context is shown in Fig. 8.


Fig. 7 Locations of the four test sites for water pressure monitoring

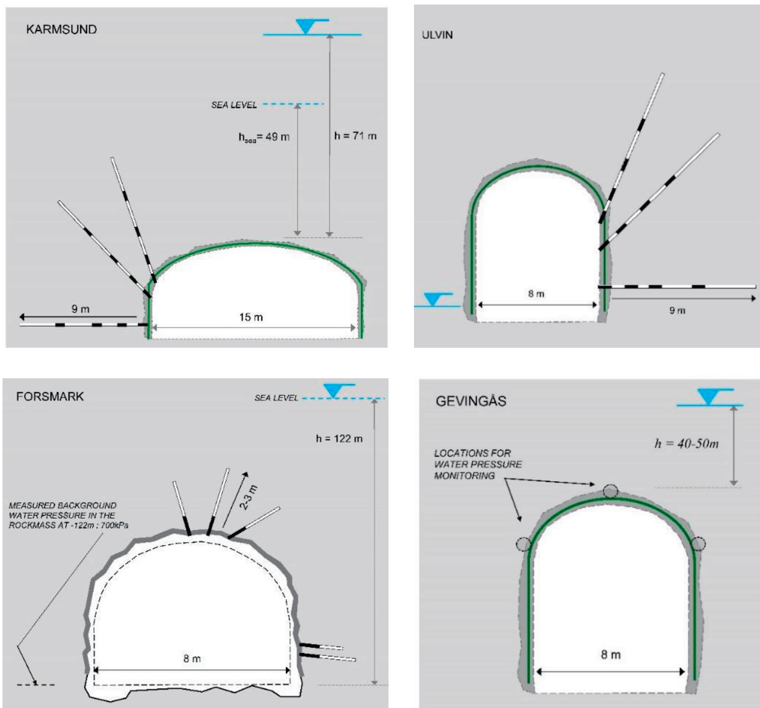


Fig. 8 Conceptual diagrams showing the hydrogeological context for the four test sites with layout of in-situ water pressure measurements

### 3.2 Engineering geological and rock mechanical conditions at the test sites

The four test sites are all located in hard crystalline rock environment. For three of the sites in-situ rock stress data were available from measurements in the vicinity of the test locations. The in-situ rock stress values indicate a horizontal major principal stress in the order of 5 MPa larger than the horizontal stress component induced by gravity only. This is typical for many areas in Scandinavian hard rock environment. The rock mass conditions for the four sites are summarized in Table 1.

Table 1. Main Engineering geological and rock mechanical conditions at the test sites

| Parameter | Karmsund | Forsmark | Ulvin | Gevingås |
|---|---|---|---|---|
| Rock overburden at test location [m] | 130 | 110 | 55 | 40 - 50 |
| Distance below ground water table [m] | 75 (subsea tunnel) | 122 (subsea tunnel) | 0 | 30 - 40 |
| Major principal rock stress, value, dip angle [MPa], [°] | 8.1 (8) | 7.6 (0) | 7.1 (4) | Not measured |
| Rock type | Massive granitic gneiss | Medium to fine grained granite to granodiorite | Banded amphibolitic gneiss | Dark mica schist |
| Uniaxial compressive strength of intact rock [MPa] | 137 - 238 | 139 - 280 | 148 – 285 | Not measured |
| Young's modulus of intact rock [GPa] | 64 – 73 | 66 – 105 | 43 - 49 | Not measured |
| Number of joint sets / joint spacings [m] | 2 + random 0.7 – 1 | 2 + random 0.1 – 0.5 | 3 0.2 - 0 .5 | 2 - 3 0.2 – 0.8 |
| Number of joints per m drillcore | 1 – 3 | 6 - 9 | 1 - 7 | Not measured |
| Rock mass quality Q, range and typical value (in brackets) | 6 – 66 (23) | 10 – 40 (30) | 5 – 12 (8) | 3 -17 (5) |
| Estimated average hydraulic conductivity of rock mass [m/s] | $10^{-8}$ to $10^{-9}$ | $10^{-8}$ to $10^{-9}$ | $10^{-7}$ to $10^{-8}$ | $10^{-7}$ |
| Calculated hydraulic transmissivities of rock joints [$m^2$/s] | $10^{-8}$ to $10^{-10}$ | | | |

3.3 Monitoring method for water pressure

The water pressure monitoring was arranged to directly measure the water pressure at a location in a hole using a hydraulic connection with a hose from the drill hole to the pressure sensor. Two measuring configurations were used:

- Pressure measurement in one section in a hole using a mechanical packer placed 1 m from the collar of the hole. The total hole lengths were in the range of 2 – 3.5 m
- Pressure measurement in four sections with various lengths in each hole with total length 9 m

All pressure sensors were placed in a cabinet and connected to a logging unit. Thin nylon hoses designed for high pressures connected each of the measurement sections to the cabinet with pressure sensors. The pressure sensors recorded absolute pressures. The presented values in this paper are adjusted for the height difference between measurement location and the sensor, as well as the measured air pressure at cabinet location. The measuring station for the Karmsund test site is shown in Fig. 9. Similar stations were used for the Ulvin and Forsmark test sites.

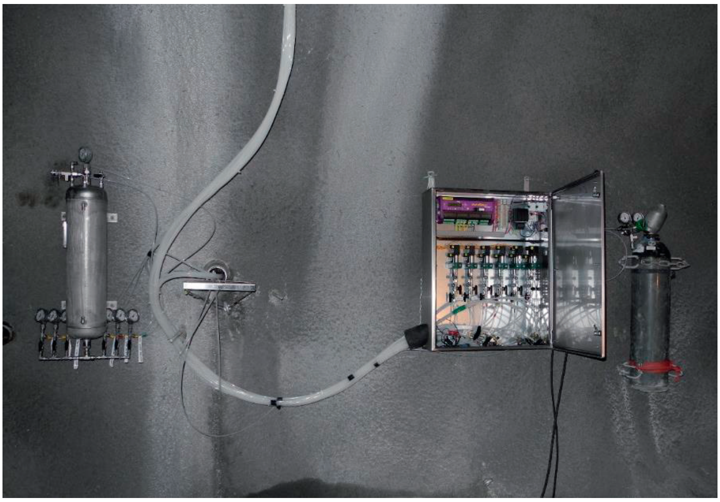

Fig. 9. Arrangement for in-situ monitoring of water pressure with instrument cabinet with pressure sensors and logging unit. High pressure hoses are connected to the drill hole sections for measurements, and to the far left a vessel with pressurized water to keep the packers which section the holes constantly inflated. This measuring principle was used for the Karmsund, Ulvin and Forsmark test sites.

## 3.4 Investigations at the Karmsund site

### 3.4.1 Construction of test site

The Karmsund test site was constructed in a cavern branching off from the highway tunnel in the vicinity of an underground roundabout. The construction of the waterproof lining and the installation of monitoring equipment took place approximately one year after the excavation and construction of rock support of the cavern. The site is located under an island with the invert level at 57 m below sea level. A lake on the island influences the ground water table, which exhibits seasonal fluctuations. A cross section of the test site with the location of the adjacent tunnels is shown in Fig. 10. During excavation of the cavern and tunnels in the immediate vicinity very little seepage was encountered in the probe drilling holes. When grouting the probe drilling holes very little grout takes were experienced when applying a grouting pressure of 80 bars. Some remaining seepage was removed by local post-injections. The measurements of hydraulic transmissivities (Sect. 3.4.4 and 3.6) show no indications of a clear boundary between a grouted and non grouted zone.

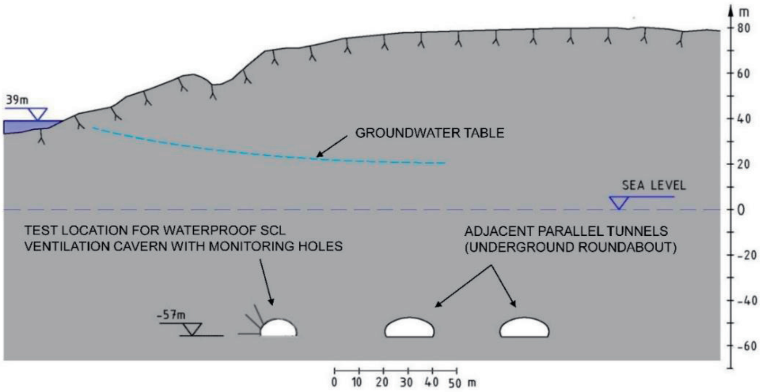


Fig. 10 Vertical section of the Karmsund test site with the location of the test area and hydrogeological context (after Holter 2014).

Fig.11 Interior of the Karmsund test site with the waterproofing membrane recently applied, and LiDar scanning of the surface in progress.

The test site was constructed with waterproof SCL in the walls and crown of the cavern. The interior of the cavern with the surface waterproofing membrane before application of the inner lining sprayed concrete is shown in Fig. 11. In this case the construction of the waterproof lining took place before the testing holes were drilled. Hence all water pressure testing and monitoring was started and executed after the final construction of the waterproof SCL.

3.4.2 Estimation of water ingress and hydraulic conductivity

Images obtained by LiDar scanning of the lining surface before and after application of the waterproofing membrane is shown in Fig.12. Water ingress to the cavern before the application of the membrane occurred as scattered drips and damp spots. Based on the scanned surfaces and observations in the cavern by counting drips and damp spots, the water ingress to the cavern was estimated. After the application of the final inner lining, some damp spots occurred in the lining surface shown in Fig. 12 bottom. Altogether 39 water ingress points could be observed before the application of the membrane. The lower image in Fig.12 shows the occurrence of the remaining damp spots 6 months after construction without any injection works conducted. The construction procedure for such linings normally involves removal of minor seepage points through the final lining by local injection. This was not done in our case. The location of the water pressure testing and monitoring holes is shown in Fig. 12 bottom. The first 250 mm of the holes were equipped with a resin grouted steel casing. No damp points occurred around the measuring locations for the water pressure.

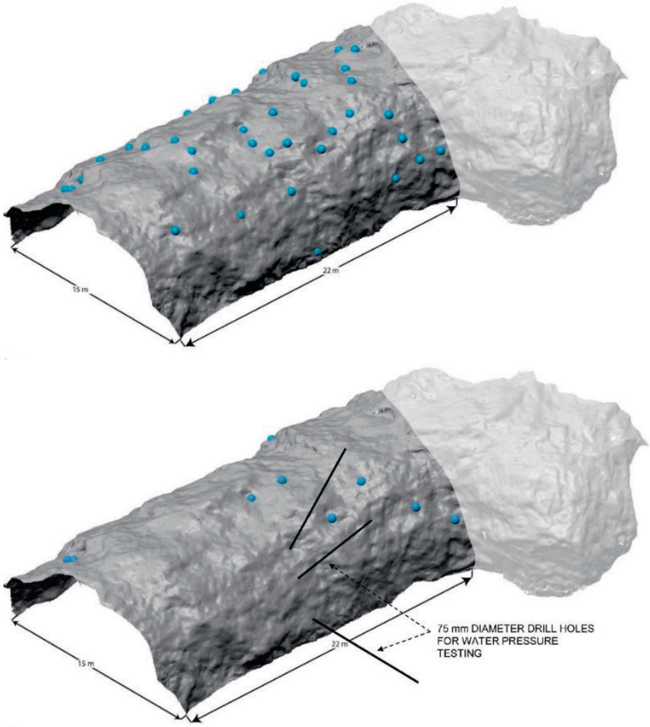


Fig. 12. 3D views obtained by LiDar scanning of the SCL section at the Karmsund subsea tunnel test site. Top: Seepage points in the form of drip spots though the substrate before the application of the waterproofing membrane. Bottom: remaining damp points at approximately 5 months after the application of the final inner lining. No post injection was carried out to remove the remaining seepages.

An estimation of the hydraulic conductivity of the rock mass was done based on the estimated water ingress, the static head and the size of the cavern. The model proposed by El Tani (2003), shown in Fig. 13 and Eq. 4 for a circular shaped underground opening and assuming a homogenous hydraulic conductivity was used for this purpose.

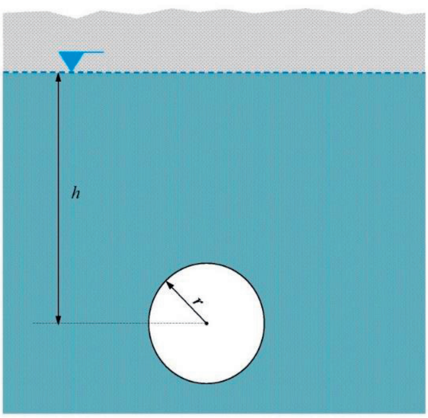


Fig. 13 Conceptual diagram for model with an analytical solution for water ingress $Q$ [$m^3/m{\cdot}s$] in a circular tunnel with radius $r$ [m], located at depth $h$ [m] below the groundwater table in a uniform aquifer with a hydraulic conductivity $k$ [m/s] (El Tani 2003).

$$Q = 2\pi kh \frac{1 - 3\left(\frac{r}{2h}\right)^2}{\left[1 - \left(\frac{r}{h}\right)^2\right] ln\frac{2h}{r} - \left(\frac{r}{2h}\right)^2} \qquad \text{Eq. 4}$$

The water ingress in the invert was unable to be observed. For the purpose of the estimation of the hydraulic conductivity, a quantity of water ingress in the invert in proportion with the perimeter size to the average water ingress in the walls and crown was assumed. An equivalent radius for a circular tunnel with the same perimeter length as the cavern was used in the calculation. The estimated water ingress to the cavern and calculated hydraulic conductivity is shown in Table 2.

Table 2. Parameters for the estimation of hydraulic conductivity in the Karmsund test site

| Parameter | Value | Basis for value |
|---|---|---|
| Estimated total water ingress in entire length of cavern | 0.8 – 1.6 liters per minute | Counting of drips and moist spots |
| Length of cavern section | 24 m | Measured |
| Estimated water ingress per linear m cavern, range | $2.8 \cdot 10^{-7}$ to $1.1 \cdot 10^{-6}$ $m^3/s$ per linear meter | Counting of drips and moist spots |
| Static head | 75 m | Assessed groundwater level based on measurements, Sect 3.3.3 |
| Equivalent radius | 6.8 m | Equivalent perimeter of cavern |
| Hydraulic conductivity | $1.8 \cdot 10^{-9}$ to $7.5 \cdot 10^{-9}$ m/s | Eq. 4 |

### 3.4.3 Water pressure testing

The water pressure testing was executed using a double packer system with 1 m sealing length on either side of the measuring section with 0.5 m length. Hence, sections of 0.5 m length could be tested up to approximately 8 m depth of the 9 m holes. The testing was done by pumping with a pressure in the range of 500-600 kPa higher than the hydrostatic pressure. Pumping was terminated when steady state flow into the section was achieved and the flow rate was recorded. The equipment was set up to measure flow rates with an accuracy of approximately 2 milliliters/minute.

Core material allowed for accurate assignment of joint locations along the measurement holes. Several measurement sections for the pressure testing had no joints at all, and most sections had one or two joints. The hydraulic transmissivites were therefore calculated both as transmissivities for the 0.5 m sections, Eq. 1 (Moye 1967) and transmissivities for single joints, Eq. 2 (Gustafson 2012) assuming one conductive joint per measurement section. From the calculated joint transmissivities, the hydraulic joint apertures were estimated using the cubic law (Eq. 3).

The calculated hydraulic joint transmissivities for all tested sections at the Karmsund site are shown in Fig. 14. The estimated hydraulic joint apertures are shown in Fig. 15. The hydraulic transmissivities for the 0.5 m sections are shown in the compilation with the measured in-situ water pressures in Fig. 16.

The measurements indicate joint hydraulic transmissivities in the range of $1 \cdot 10^{-9}$ to $1 \cdot 10^{-8}$ m/s. A portion of the tested rock joints exhibit hydraulic transmissivities in the order of $5 \cdot 10^{-10}$ m/s. The corresponding estimated joint hydraulic apertures (Fig. 15) are in the range of 0.01 to 0.04 mm with 0.01 to 0.025 mm as the most represented values.

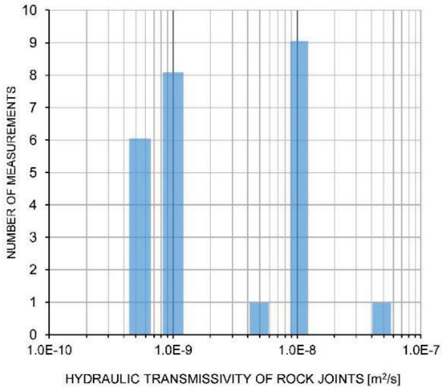


Fig. 14 Calculated hydraulic transmissivities for rock joints from the water pressure testing at the Karmsund test site. Each column shows number of measurements for a value within a certain interval.

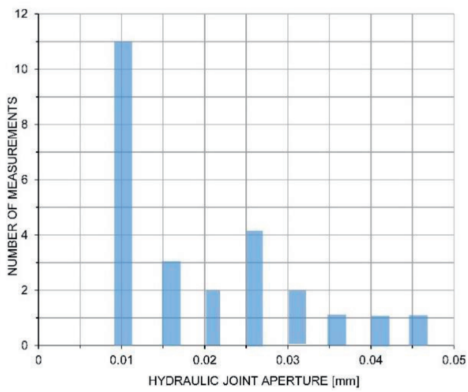


Fig. 15 Calculated hydraulic apertures for rock joints from the water pressure testing at the Karmsund test site.

### 3.4.4 Monitoring of ground water pressures

Monitoring of the in-situ ground water pressure was done in 3 holes with length 9 m. For the water pressure monitoring, the holes were sectioned using inflatable packers (details are explained in Holter 2014). The monitoring could be conducted for a continuous period of 8 months before the monitoring equipment needed to be removed due to the final construction works.

The results from the monitoring of ground water pressure are shown in Fig. 16. The measured pressures are shown in absolute pressures which are compensated for the measured air pressure at the measuring cabinet. Measured section transmissivities and rock jointing respresented as RQD obtained from drill cores are also shown. Further details such as joint transmissivity and location of joints are shown in the compilation in Fig. 20, Sect. 3.5.2. The measured water pressures indicate a slight reduction of water pressure closer to the lining. Two sections indicate very low pressures, one even with pressure below the air pressure. This is likely to be a short term effect of an unsaturated situation and is further discussed in Sect. 4. The development of measured pressures over the 8 months monitoring period is shown for the holes 1 and 3 in Fig. 16 bottom parts. A trend with slight reduction can be observed in the spring months. The changes over time are not consistent in magnitude for all the monitoring sections.

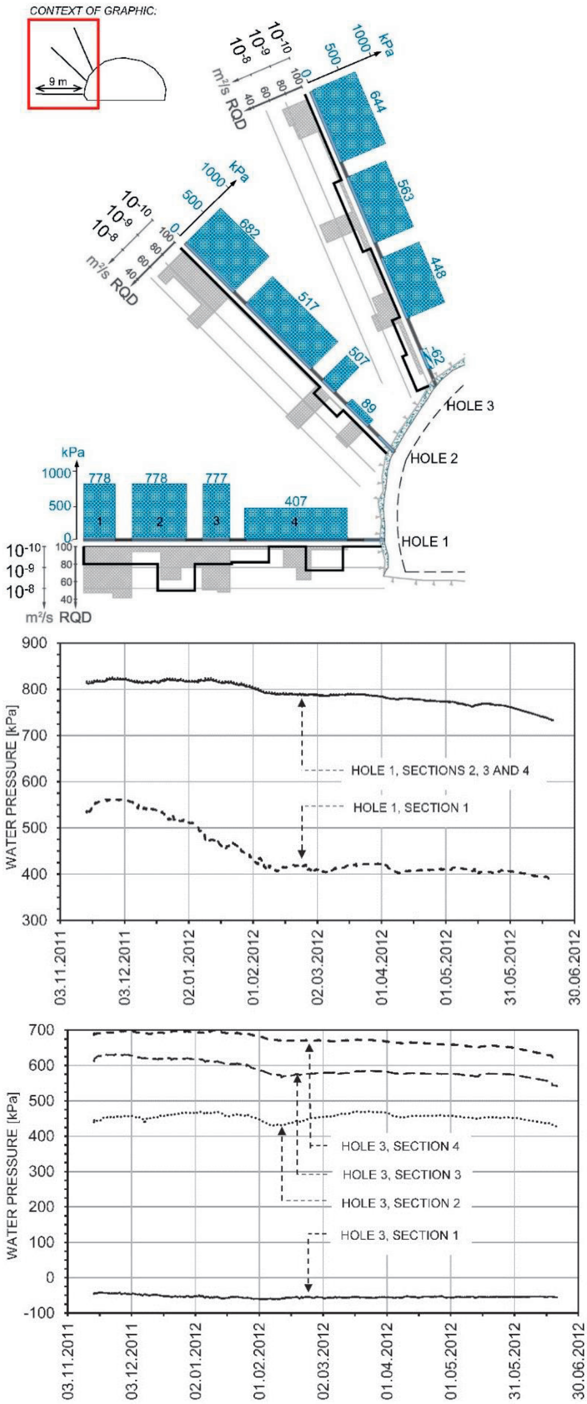


Fig. 16. Measured water pressures in sectioned holes in the immediate rock mass at the Karmsund test site. Top: vertical section with hole locations and measured pressures after 5 months and section transmissivities and rock jointing represented as RQD. Middle and bottom: development of measured water pressures over time.

## 3.5 Numerical simulations of the Karmsund case

Numerical simulations in two different codes; a continuum and a discontinuum code, were carried out in order to estimate the ground water pressure in the rock mass in the immediate vicinity of the lining surface. The numerical simulations were designed to study the effect of ground water pressure, only taking into account the hydraulic properties of the rock mass, without including any local effects, such as EDZ and vapor diffusion near the lining. In each of the codes a model with and without the undrained waterproof lining was established. The purpose of these simulations was to study the effect of the undrained lining considering the average hydraulic conductivity of the rock and hydraulic transmissivity of the rock joints separately. The reason for executing the simulations without accounting for effects of EDZ is to determine which water pressures can be expected in the rock mass without an EDZ with increased hydraulic conductivity near the lining.

For both codes simplified models were constructed to represent the rock mass. Simplified but well established models for ground water flow are applied, such as conductive water flow in a medium consisting of a network of conductive joints according to the cubic law for flow between two parallel surfaces, and conductive water flow in a continuous porous medium according to Darcy's law.

The models were established based on physical mapping of the rock mass, measured in-situ rock stresses in the vicinity and measured values for uniaxial compressive strength and Young's modulus. The models contains the cavern with the waterproof lining in the crown and walls with no waterproofing in the invert. The adjacent tunnel is modelled as a completely drained opening without any lining. No special provisions have been included in the discontinuum model for the EDZ. The effects of the EDZ in the invert are solely produced by the model. For the continuum model an EDZ in the invert of 2.5 m thickness with higher hydraulic continuity was included.

### 3.5.1 Discontinuum model, UDEC BB

For the discontinuum analysis the UDEC BB (Universal Distinct Element Code, Itasca 2013b) with the Bandis-Barton joint model, originally presented by Barton et al. 1985, was applied. UDEC BB is a two-dimensional version of the distinct element method which is specifically designed to simulate the predominant features of jointed rock masses, and models a coupling of the hydraulic pressures and mechanical stresses.

The simulation of the behavior of the rock joints is based on the three key joint parameters: Joint Roughness Coefficient JRC, Joint Wall Compressive Strength JCS and the residual friction angle. The rock mass is simulated as an assemblage of blocks which interact through corner and edges. The intact rock is modelled as an isotropic medium with linear elastic behavior. The fluid flow is modelled according the cubic law, applying an initial hydraulic aperture. No special provisions have been included in the UDEC BB model for the EDZ. The effects of EDZ in the invert are solely produced by the model.

### 3.5.2 Continuum model, FLAC$^{3D}$

FLAC$^{3D}$ is a finite difference model in which the mechanical behavior of a continuous three-dimensional medium is studied numerically as the medium reaches equilibrium or steady plastic flow (Itasca 2013a).

In addition to the mechanical modeling, FLAC$^{3D}$ also models the fluid flow through a permeable solid such as soils and rocks. In this paper, the flow modeling in parallel with the mechanical modeling of rock mass has been considered since the groundwater pressure development in rock mass is of primary interest. Coupled flow and mechanical behavior is modeled to assess the effect on groundwater pressure in the immediate rock mass due to partially drained SCL.

The geometry of the model has been developed based on Fig.10. The adjacent road tunnel has been placed at 15 m distance from the cavern, and is considered a drained tunnel for our purpose. Since it is time consuming and tedious work to model whole topography in the model, a smaller representative 3D model of size of 130 m width, 90 m height and 0.2 m length along tunnel length (Y-direction) has been created in FLAC$^{3D}$. The model is divided into several polyhedral zones. Comparatively smaller zones are created in the vicinity of both cavern and road tunnels and larger zones are created in the outer part (Fig. 20).

### 3.5.3 Construction of models, input parameters

Both models are based on the physical conditions shown in Fig. 10. For the simulation of the road tunnel in the vicinity, this tunnel is placed 15 m to the right of the cavern without any waterproof lining at all. The groundwater table was placed at 78 m above the invert level of the cavern. This corresponds to the recorded water pressure at 9 m depth in the horizontal drillhole, Fig. 16, Sect. 3.4.4) which we have interpreted as the hydrostatic pressure. We have assumed an EDZ in the invert based on the assumption that this area is subject to rough blasting and that no measures are taking to produce an undamaged contour in the invert. In the walls and the crown no EDZ is accounted for in design the models. The input parameters are summarized in Table 3.

Table 3. Rock mechanical properties used as input parameters for the two numerical models

| **Material properties** | **Unit** | **Value** | **Remarks** |
|---|---|---|---|
| Unit weight of rock mass | $MN/m^3$ | 0.0265 | Measured |
| Uniaxial compressive of strength intact rock | MPa | 140 | Measured |
| Young's modulus of intact rock, E | GPa | 70 | Measured |
| Poisson's ratio, ν | - | 0.15 | Measured |
| Geological Strength Index, GSI [1] | - | 75 | Estimated |
| Hydraulic conductivity of rockmass [1] | m/s | $7.5 \cdot 10^{-9}$ | Estimated |
| Hydraulic conductivity of EDZ below invert [1] | m/s | $1 \cdot 10^{-5}$ | Estimated |
| Disturbance factor, D [1] | - | 0 | Assessed |
| Joint compressive strength JCS [2] | MPa | 140 | Measured |
| Joint roughness coefficient JRC [2] | - | 5 / 5 / 4 | Measured values for the three joints sets |
| In-situ rock stress, initial condition | MPa | $\sigma_z$ = 3.3 MPa (vertical)<br>$\sigma_x$ = 2.0 MPa (horizontal)<br>$\sigma_y$ = 6.0 MPa (horizontal) | Based on measurements in the immediate vicinity |

[1] FLAC$^{3D}$ only
[2] UDEC BB only

3.5.4 Output from the numerical models

For the case with an undrained lining in the cavern, the simulated water pressures are shown in vertical section in Fig. 17 for the discontinuous model and in Fig. 18 for the continuous model. For the discontinuous model the water pressures are given as pressures occurring at each joint. For the continuum model the water pressures are given as domain pore pressures. A representation of the simulated water pressures in the rock mass in the immediate vicinity of the cavern is shown in Fig. 20. A simulation of a case with a completely drained lining was also carried out. The simulated water pressures in the immediate vicinity of the cavern with a drained lining, analogue to the case with the undrained lining, are shown in Fig 21.

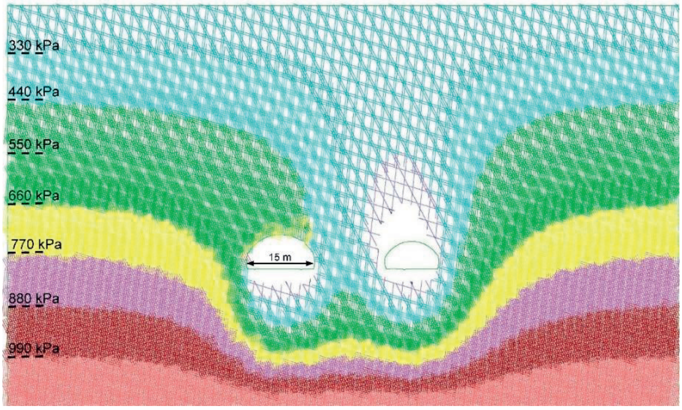


Fig. 17. Image with simulated water pressures with UDEC BB with a waterproof SCL in the wall and crown of the test cavern (to the left) and a complete drained lining in the adjacent road tunnel (to the right). Detailed profiles of water pressures are shown in Figs. 20 and 21.

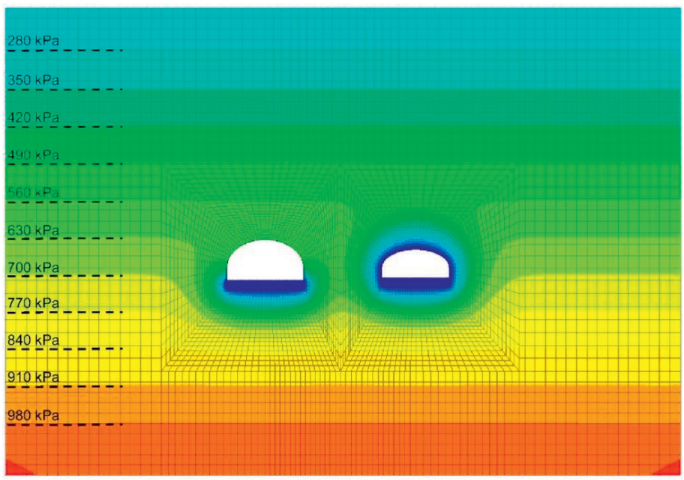


Fig. 18. Image of simulated water pressures in FLAC$^{3D}$ around the cavern with undrained waterproof SCL to the left and the adjacent drained tunnel to the right. Detailed profiles of water pressures are shown in Figs. 20 and 21.

A parametric study in FLAC$^{3D}$ was carried out by only varying the hydraulic conductivity of the rock mass. The model was run with hydraulic conductivities $10^{-7}$ and $10^{-6}$ m/s in addition to the estimated $7.5 \cdot 10^{-9}$ m/s. A larger amount of water will migrate through the rock mass at higher hydraulic conductivities, resulting in a lower water pressure. This is shown in Fig. 19. However, the model still predicts a water pressure of approximately 350 kPa at the lining surface with a hydraulic conductivity of $10^{-6}$ m/s when only taking this parameter into account.

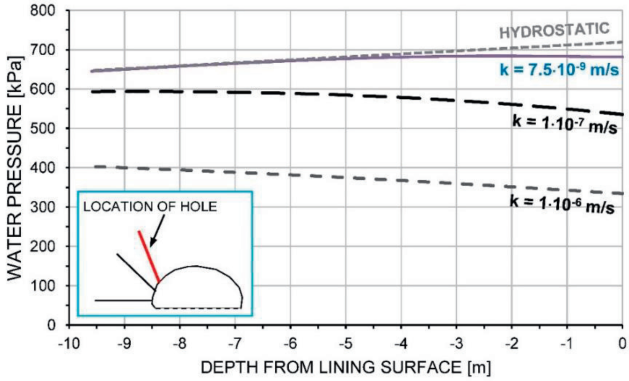


Fig. 19 Simulated water pressures along a drillhole at the Karmsund test site with FLAC$^{3D}$ showing the effect of different rock mass hydraulic conductivities with undrained invert and waterproof lining in the walls and the crown.

## 3.6 Comparison of simulated and measured water pressures at the Karmsund case with SCL in the walls and the crown

A comparison of the measured water pressures, the theoretical hydrostatic pressure and the simulated water pressures is shown in Fig. 20 for the three monitoring holes. Estimated joint hydraulic transmissivities, RQD and observed location of joints along the holes. An imaginary hole 4, oriented vertically in the middle of the crown is added in order to obtain simulated water pressure data at the most distant location from the drained invert.

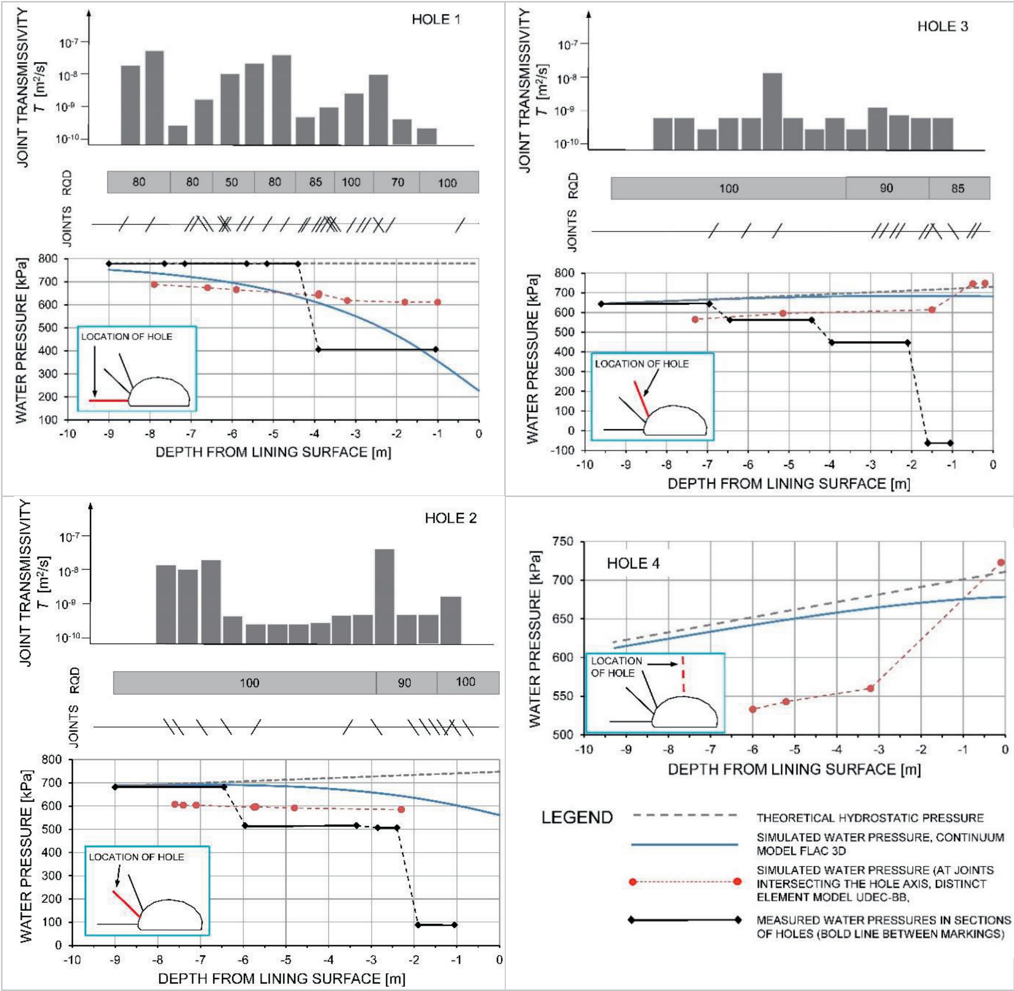


Fig. 20. Results from the Karmsund test site. Compilation of joint transmissivities, rock jointing and simulated and measured water pressures for the holes 1 to 3 for the case with a waterproof bonded tunnel lining in the walls and the crown. Simulated water pressures along a vertical line in the center of the crown are indicated as an imaginary hole 4.

Both numerical simulations predict a trend of water pressures towards the lining surface which is different from the measured. For the horizontal hole 1 and hole 2 with 45° inclination both simulations predict a trend of decreasing water pressure, which is in partial agreement with the measured pressures. For hole 3 the simulations predict an increase in water pressure whereas the measurements show decrease. Along a vertical line in the center of the crown both numerical simulations predict water pressures which are close to a theoretical hydrostatic pressure with an increase in water pressure towards the lining surface. The high values for water pressure predicted by the discontinuous model close to the lining for hole 3 and 4, can be interpreted by as joints which are poorly connected to other joints.

The simulations in both numerical codes for the case without water proof tunnel lining predict a consistent reduction of water pressure towards the lining surface, as shown in Fig 21. These simulations were unable to be compared to any in-situ measurements with these boundary conditions.

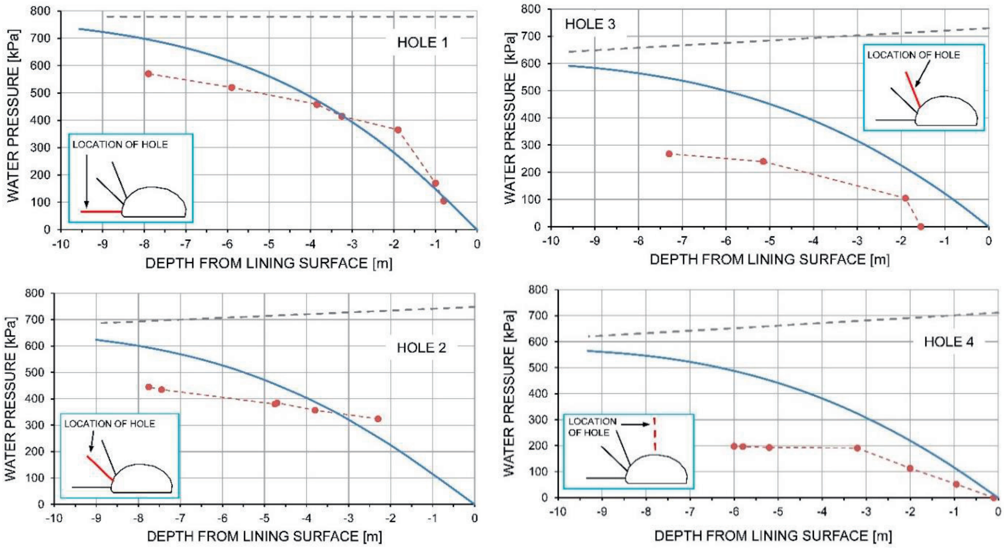


Fig. 21 Compilation of simulated water pressures along lines corresponding to holes 1 to 4 for the Karmsund case without a waterproof bonded tunnel lining. The legend is shown in Fig. 20.

3.7 Field investigations at the Ulvin site

3.7.1 Construction of test site

The Ulvin test site was established in the southern construction access tunnel for the main Ulvin rail tunnel concurrently with the excavation and rock support works approximately 50 - 70 m behind the tunnel face. At the test location the SCL was constructed successfully with a completely dry result.

3.7.2 Water pressure testing and monitoring

The methodology and layout of testing, as well as the monitoring installation was identical with that of the Karmsund site. Shortly after installation, the measured water pressures and observations in two ground water wells in the vicinity indicated that the groundwater level to be lowered to slightly above invert level of the tunnel. Hence, measurements of water pressures under the ground water level and the effects of the waterproof SCL structure could not be obtained as planned. The measured water pressures approximately 3 months after installation are shown in Fig. 22 together with estimated hydraulic transmissivities for borehole sections with 0.5 m length. The measured water pressures are slightly above the measured air pressure. Some sections in the two inclined holes also exhibit water pressures slightly below the atmospheric pressure. This indicates an unsaturated situation in the immediate rock mass. The estimated hydraulic transmissivities are mostly in the range of $10^{-7}$ to $10^{-8}$ m/s.

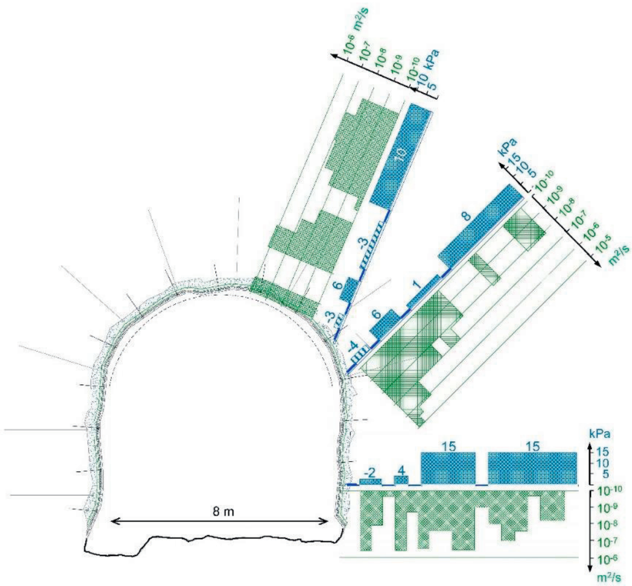


Fig. 22. Measured in-situ water pressures (blue) in sectioned holes with length 9 m and hydraulic transmissivities estimated for hole section lengths of 50 cm.

## 3.8 Investigations at the Forsmark site

The Forsmark site is an underground nuclear waste repository in Sweden, located at the eastern coast. An overview of the site is shown in Fig. 23. The main goal of the SCL test section at Forsmark was to investigate the suitability of this lining method for drip sealing for parts of the facility, particularly the storage chambers for low level nuclear waste. Water pressure monitoring was undertaken in order to investigate effects of the undrained lining and subsequent investigations of the lining materials are planned in order to detect any detrimental effects of the groundwater exposure.

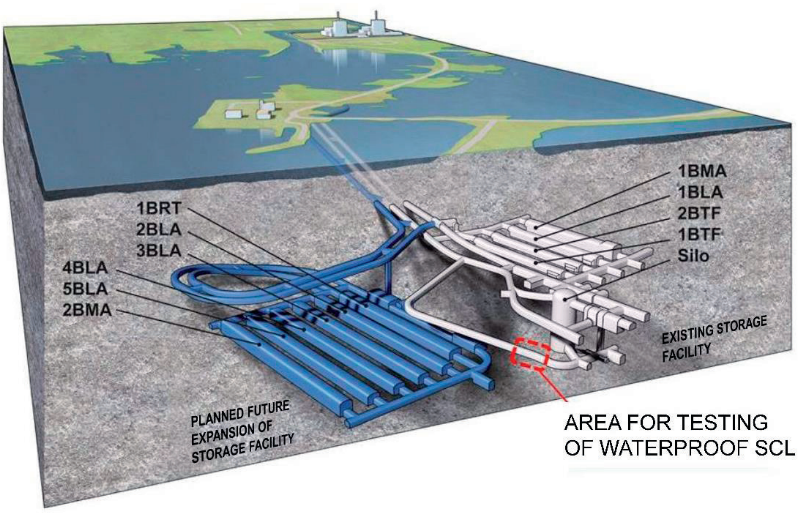


Fig. 23. The Forsmark underground nuclear waste repository and location of test site for SCL.

### 3.8.1 Construction of test section

The test section was established in a service tunnel which was constructed 30 years ago. A part of the access tunnel was selected which had visible drip and moist spot seepages. The area had no pre-grouting during the construction of the tunnel. The location of the 20 m long test section (invert) was approximately 122 m below sea level. The SCL lining type covered the entire contour with crown and walls down to the invert. The invert was left in its given drained condition. The construction of the test section consisted of the following main steps:

- Smoothening layer of 50-60 mm of fiber reinforced sprayed concrete on top of existing rock support lining
- Temporary drainage of a few dripping points
- Spray-application of the membrane
- Injection of the temporarily drained seepage points
- Application of final inner lining with 60 mm fiber reinforced sprayed concrete

No additional rock bolting was a carried out when constructing the test section. A photo of the test section is shown in Fig. 24.

Fig. 24 The test section with SCL at Forsmark after completion.

### 3.8.2 Water pressure monitoring

Monitoring of ground water pressure has been carried out in nearby boreholes since construction of this facility in mid 1980s. A gently dipping borehole in the left wall of the test section is chosen as a reference borehole. The pressure has been stable over time since construction works were completed. Short boreholes were drilled for monitoring behind the seals. The drilling of the boreholes were stopped when water were noticed from the holes, resulting in holes with lengths in the range of 1.6 to 3 m with one measurement section in each hole. The borehole seals (expandable packers) were placed at 1 m depth in each hole. Hence, the water pressure monitoring took place in sections from 1 m depth to the end of each hole. The measurement principle was identical to that of Karmsund and Ulvin apart from the multiple monitoring points in each hole. The holes were placed deliberately in areas with visible seepage, and they were drilled until ground water could be observed seeping into the hole. A series of holes in the left wall and one series of holes in the crown were established. Fig. 25 shows 3D image of the test section with locations of the monitoring holes for water pressure.

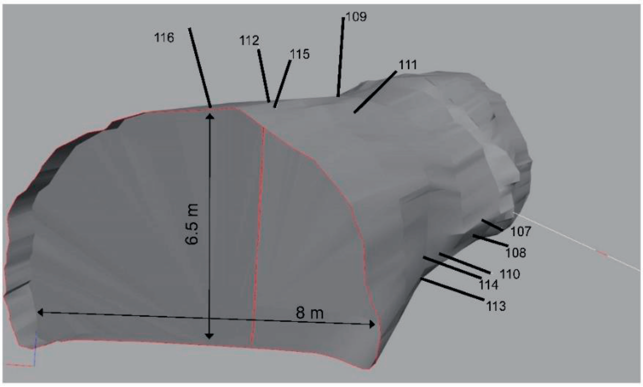


Fig. 25 3D image indicating locations and numbering of drill holes for water pressure monitoring at the Forsmark test site.

### 3.8.3 Results from monitoring of ground water pressure

The background water pressure in the rock mass at the invert elevation half way to another tunnel was measured to 680 kPa. A cross section of the SCL test section with measured water pressures in 5 selected holes is shown in Fig. 26. There are two main observations from these measurements:

- When comparing holes in the same area, longer holes tend to give higher water pressure than what is the case for shorter holes.
- There is significant variation in water pressure when comparing holes with similar lengths

The latter point is particularly notable for several of the holes in the lower wall, where e.g. holes 107 and 108 with lengths 1.3 and 1.4 m located only 1 – 1.5 m from the drained invert exhibit water pressures in the order of 380 kPa. This is an indication that some boreholes are better connected to conductive fractures further away from the tunnel.

The monitoring of water pressure has been continuously maintained, hence, the development of the ground water pressure over time can be evaluated. The development of water pressure from the time of installation, before the construction of the SCL test section took place, and the following 16 months is shown in Fig. 27. The following observations are made from these measurements:

- The measured water pressures reach a constant level with a few exceptions
- Seasonal fluctuations in measured water pressure are unobserved
- A few holes show a slight and constant increase in water pressure of approximately 40-50 kPa over a period of 14 months
- The application of the undrained waterproofing membrane has no measurable impact on the water pressure in the rock mass
- The application of the smoothening layer of fiber reinforced sprayed concrete results in a slight increase of water pressure for a few of the holes

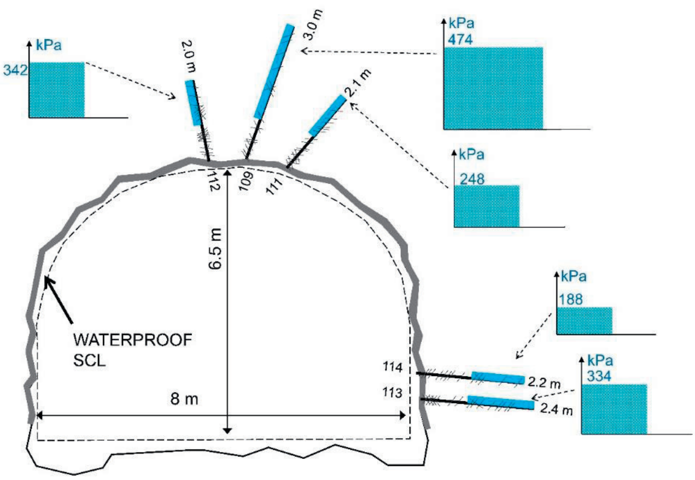


Fig. 26 Measured water pressures in the immediate rock mass, showing results from 5 holes: The hole number designation is shown in Fig. 28.

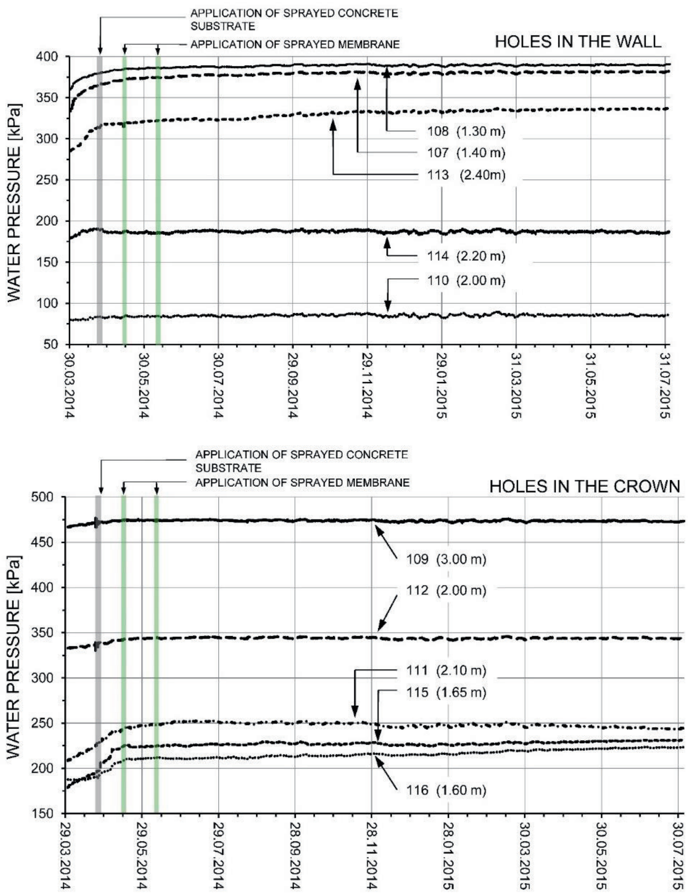


Fig. 27 Measured water pressures in the immediate rock mass of the SCL test section at Forsmark. The time for the application of the sprayed concrete and the membrane is indicated. The different hole lengths are indicated in brackets.

3.9 Investigations of water pressure in the Gevingås rail tunnel

A 1.85 km portion of the 4 km long Gevingås tunnel was constructed with waterproof SCL covering crown and walls down to invert. A section with three monitoring points was established during the construction of the tunnel in 2011. The location for the measurements was selected by the owner based on the observation of the water ingress. The location had higher water seepage than average in the tunnel. The groundwater table is assumed to be approximately 40 m above the crown of the tunnel, due the location of a swamp. The reason for the monitoring of the pressures at this location was to evaluate any development with increasing pressures over time. The monitoring was maintained for approximately 7 months. No pre-grouting had been carried out at this location. The results were made available for this study.

3.9.1 Construction of test site

The measurements were conducted with vibrating wire piezometers which were grouted in short boreholes with length approximately 200-250 mm from the lining surface. The holes were drilled through the waterproof lining into the sprayed concrete on the “rock side” of the membrane. The outer part of the hole was grouted and sealed with resin and mortar. The test location remained completely dry after the installation of the piezometers. The layout of the test site is shown in Fig. 28.

3.9.3 Water pressure testing and monitoring

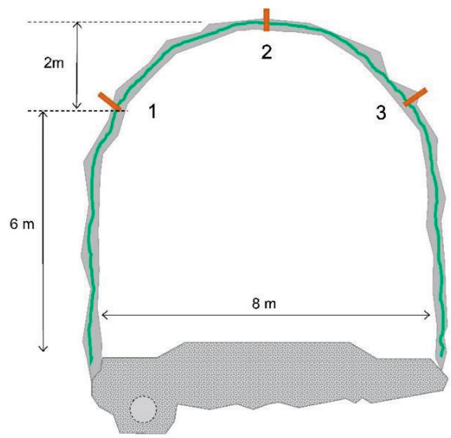


Fig. 28 Vertical section of the Gevingås test site. Locations for monitoring of water pressure with vibrating wire piezometers in the rock mass immediately behind the waterproof SCL.

The measured pressures are shown in Table 4. The measured pressures showed very small fluctuations and were stable for the 9 month monitoring period. The hydraulic conductivity was estimated following the same principle as for the Karmsund site. The observed water ingress amount gives a hydraulic conductivity in the magnitude of $10^{-7}$ m/s.

Table 4. Measured water pressures with piezometers installed in short drillholes behind the waterproof lining at the Gevingås test site.

| Monitoring location | Measured water pressure after 9 months [kPa] |
|---|---|
| 1 | 60 |
| 2 | 135 |
| 3 | 50 |

## 4 DISCUSSION OF RESULTS

### 4.1 Measured water pressures at the Karmsund and Forsmark test sites

These two test sites exhibit similar rock mass conditions and hydraulic saturation of the rock mass given the location below sea level. In the Karmsund case a gradient with a slightly decreasing water pressure in the rock mass towards the undrained lining surface could be observed. In the Forsmark case a trend with slightly longer holes adjacent to shorter holes gave higher recorded water pressures in the longest holes. A trend with lower pressures closer to the lining surface can therefore be observed from the measurements at Forsmark. In both cases the measured pressures at approximately 2 m distance from the lining are in the size order of 200-300 kPa lower than the hydrostatic pressure or background water pressure.

The measured water pressure at Karmsund in hole 2 and 3 at the locations closest to the lining (Fig. 17, Sect 3.4.4) are exceptionally low. The simulations predict significantly higher water pressures at these locations. No obvious explanation can be given for this. The measuring locations were all dry in the immediate vicinity of the holes. A possible explanation can be an unsaturated situation in the rock mass near the lining surface with air entrapped in joints or the measuring section. In the case of a locally unsaturated part of the rock mass, this is unlikely to be a long term situation, since air and vapor will tend to migrate slowly by diffusion through the lining. Effects of fissures in the EDZ with apertures in the capillary range could possibly cause such effects.

Neither of these two cases exhibits a typical grouted zone around the tunnel with significantly lower hydraulic conductivity in the surrounding rock mass in comparison the host rock mass. In the Karmsund case the probe drilling holes were grouted, and selective pre-grouting was carried when water was encountered in the probe drilling holes. Some local post injection was carried out at single water ingress points. The measurements of hydraulic transmissivities (Sect. 3.4.4. Fig. 16, Sect 3.6 Fig. 20) indicate no clear increase in hydraulic transmissivity at increasing depth, which would have indicated the boundary of grouted zone. This will however be the case for many tunnels constructed under the groundwater table. In such cases the water ingress to the tunnel will be mainly governed by the hydraulic conductivity of the grouted zone.

The estimation of the hydraulic conductivity according the model proposed El Tani (2003), given in Sect. 3.4.2 is based on the assumption of a homogenous hydraulic conductivity of the rock mass. In a jointed rock mass inhomogeneity will occur, particularly around a tunnel with a grouted rock mass. No consideration has been given to the effects of seepage into the adjacent tunnel in these calculations. Although no clear effects of the grouting works around the cavern could be observed in the form of an increase in recorded hydraulic transmissivity at increasing distance, a lower hydraulic conductivity around the cavern is likely due to the executed injection works. Based on our measurements this is impossible to quantify. The observed seepage situations prior to the construction of the SCL with membrane in the Forsmark and Karmsund test sites were comparable.

### 4.2 Measured water pressures at the Ulvin site

The measurements suggest that the ground water table is 1-2 m above the invert level of the tunnel, and there is an unsaturated condition in the rock mass at the location of the two upper measuring holes (Fig. 25, Sect. 3.7.2)

### 4.3 Measured water pressures at the Gevingås site

The measuring points were placed in the part of the rock mass which exhibited the largest single dripping zone in the entire SCL portion of the tunnel. No pre-grouting was carried out in this area of the tunnel. The higher seepage in this area indicates a better drainage effect in the rock joints.

The measured pressures indicate a water pressure at the lining surface in the size order of 300 kPa lower than the theoretical hydrostatic pressure.

### 4.4 Simulated ground water pressures

The simulated ground water pressures only take into account of the average hydraulic transmissivities for the joint sets and an average hydraulic conductivity for the rock mass. Although a more sophisticated model with different hydraulic conductivities for different parts of the rock mass surrounding the tunnel, the aim of this part of the study was to investigate the effects on the ground water pressure around the partially drained lining with constant conductivity properties.

For the horizontal hole 1 and hole 2 with $45^o$ inclination both simulations predict a trend of decreasing water pressure, which is in partial agreement with the measured pressures. For hole 3 the simulations predict an increase in water pressure whereas the measurements show decrease. Along a vertical line in the center of the crown both numerical simulations predict water pressures which are close to a theoretical hydrostatic pressure with an increase in water pressure towards the lining surface. The measured water pressure is found to be almost constant within distinct intervals of each borehole. This phenomenon is simulated by the discontinuum code. The differences in the numerical values can be related to local differences in joint geometry. It is also noteworthy that even for the vertical imaginary borehole, the discontinuum code takes into account the natural connection between joints allowing for water flow occurring along joints, resulting in a significantly lower pressure compared to the hydrostatic pressures. Apart from the 1-2 m closest to the cavern, the discontinuum code predicts water pressures which are in good agreement with the measured pressures. It can be seen that the water pressures simulated by the continuum code tend to increasingly deviate from the measured at increasing distance from the drained invert.

The simulations in both numerical codes for the case without water proof tunnel lining predict a consistent reduction of water pressure towards the lining surface, as shown in Fig 21. These simulations could not be compared to any in-situ measurements with these boundary conditions. The discontinuum code generally predicts lower pressure gradients at depths larger than approximately 2 m compared to the continuum code for this case.

The trends predicted by the discontinuum model are more in agreement with the measured trends than the continuum model.

### 4.5 Comparison of measured and simulated pressures.

Both simulations consistently predict an increase of the ground water pressure around the cavern at closing distance with the interface between rock and the undrained SCL, whereas the measured values at the two sites Karmsund and Forsmark show the opposite trend. This indicates that one or several features influence the water flow close to the lining which is not covered by the numerical models. Two possible causes, an EDZ with higher hydraulic conductivity or vapor transport through the lining are considered in our study. These discussed in the following sections 4.6 and 4.7.

### 4.6 Possible effects of increased hydraulic conductivity in the EDZ

The contour of caverns in rock is normally subject to careful blasting in order to produce as little overbreak as possible. However, occurrence of new fractures and fissures or opening of existing fractures is to some extent a likely consequence of the drill-and-blast excavation out to a certain distance from the excavated contour. The in-situ stress situation may cause both dilation and compression of discontinuities, depending of the location on the tunnel perimeter and the fracture geometry. The rock mass immediately below the invert area will normally be subject to larger effects of the blasting than the wall and crown contour due to the higher charging of the holes. This is illustrated in Fig. 29. The EDZ can be exposed to significant hydromechanical and geochemical modification with increased occurrence of discontinuities. These modifications may lead to significant changes in flow paths and may change the hydraulic transmissivities of fractures in this area with a large order of magnitude.  An increased hydraulic transmissivity of the EDZ will have favorable effect in the form of higher tangential and radial water flow and consequently lower water pressures.

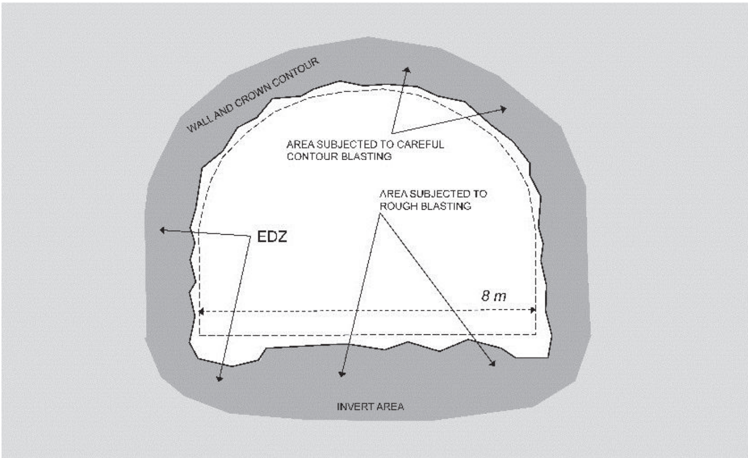


Fig. 29 Schematic illustration of the EDZ.

In a recent study carried out in Sweden in the Äspö hard rock laboratory by the Swedish Nuclear Fuel and Waste Management Company (Ericsson et al. 2015) measurements of the hydraulic conductivity in the invert area of a carefully blasted contour of a tunnel was carried out. The measurements were carried out as double packer tests with 100 mm measuring sections. A special arrangement was made in order to measure the hydraulic transmissivity for the first 100 mm starting at the actual excavated contour. Hence, detailed joint transmissivity data could be obtained in short increments of the immediate rock mass.  The measurements from study at Äspö cover the part of the rock mass in the immediate vicinity of the excavation contour which was impossible to include in our measurements at the Karmsund and Forsmark test sites.

Some results from the study at Äspö are shown in Fig. 30. The measurements indicate an increase in hydraulic transmissivity from $10^{-9}$ $m^2/s$ at approximately 1.3 m depth to a magnitude approximately $10^2$ higher at 500 mm depth and $10^4$ higher at 200 mm depth from the excavated surface.

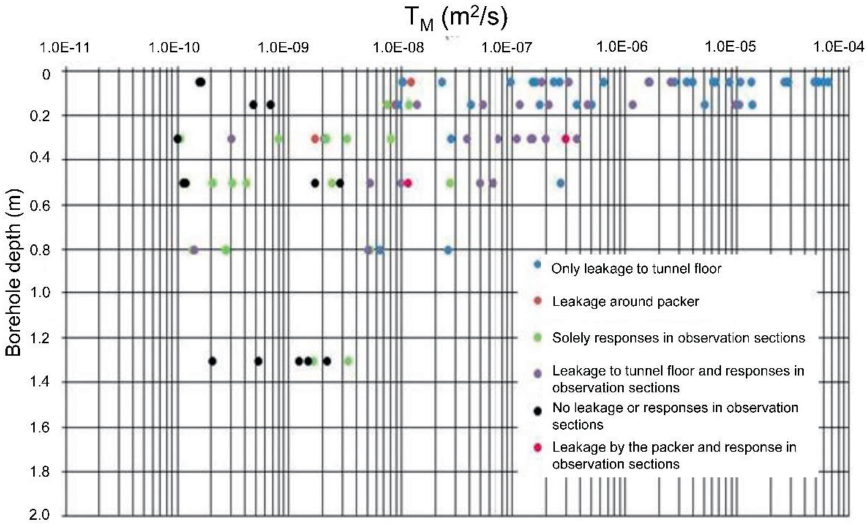


Fig. 30 Estimated hydraulic transmissivities $T_M$ for hole sections with 100 mm lengths from the rock surface to 1.3 m depth (Ericsson et al. 2015)

## 4.7 Effects of water vapor diffusion

Moisture transport through an SCL structure with bonded membrane was studied by Holter and Geving (2015) with boundary conditions corresponding to the Gevingås and Ulvin test sites. A main finding of this study is that a small amount of water migrates through the lining in the form of vapor under certain conditions. This amount of water can be estimated by using numerical simulation and realistic material parameters. The lining sections considered by Holter and Geving (2015) have thicknesses in the range of 200 mm (Gevingås) to 350 mm (Ulvin) whereas the Karmsund and Forsmark sites have total lining thicknesses in the order of 150 mm.

In low permeable ground the vapor permeable behavior of the lining may have an influence on the groundwater flow in the immediate vicinity of the lining/ground interface. An idealized calculation has been made considering the rock mass a homogenous continuum with a fully tanked tunnel with 9 m diameter located at a certain depth and using the flow model proposed by El Tani (2003), explained in Sect 3.4.2. This calculation predicts that at a certain hydraulic conductivity of the rock mass there is a balance between the capillary and vapor transport through the lining and the unrestricted water flow into the tunnel through the ground. Such a balance is illustrated in Fig. 31. This estimate suggests that in rock masses with hydraulic conductivities in the order of $10^{-11}$ m/s the effect of vapor diffusion through the lining will become significant. In our considered cases with hydraulic conductivities in the order of $10^{-9}$ to $10^{-8}$ m/s at the lowest, the effect of vapor diffusion through the lining can only have a minor influence

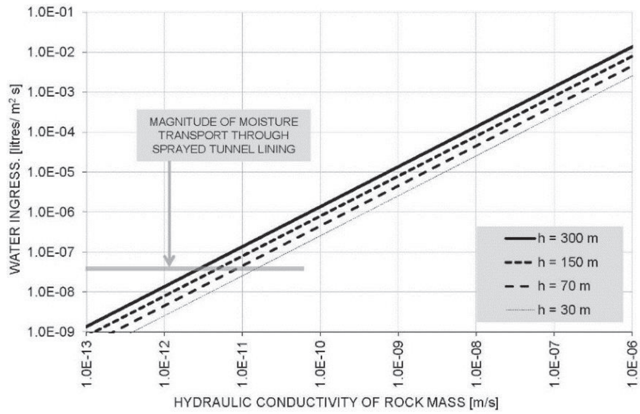


Fig. 31 Water ingress to circular tunnel for a 9 m diameter tunnel given as liters per m$^2$ per second at different ground water levels given as static heads h versus hydraulic conductivity of a rock mass, calculated according to El Tani (2003). The magnitude of possible moisture transport through a sprayed tunnel lining with 250 mm thickness as modelled in WUFI is indicated.

## 4.8 Effect of geometry. Limitation of a 2D model

The UDEC BB model is a two-dimensional representation of a three-dimensional reality. The discontinuous model emphasizes the behavior of the joints. The joints in UDEC BB are represented in vertical cross section as lines with the apparent dip in this cross section. This means that the simulation is done for joint sets which have a strike parallel to the cavern axis. Hence, the blocks in a two-dimensional representation will be oriented parallel to the cavern. This is illustrated in Fig. 32. This creates several challenges in the modelling of groundwater flow. The physical fracture void properties are not possible to represent realistically in a 2D model. The persistence of each joint for the modelling of the water flow is unable to be realized in the model. The simplification which is done, for the modelling of the water flow in a two dimensional model is to give each joint a much longer persistence than in reality.

The effect of the angle between two conductive joint sets is important to consider. This was studied by Panda et al. (1999). It was shown that an angle of 90° between two joint sets gives the highest average block hydraulic conductivity. In the Karmsund case the angle between the two subvertical joint sets is around 80°. This means that the modelling in two dimensions in for the Karmsund case will predict a too low hydraulic conductivity of the rock mass from a geometry perspective.

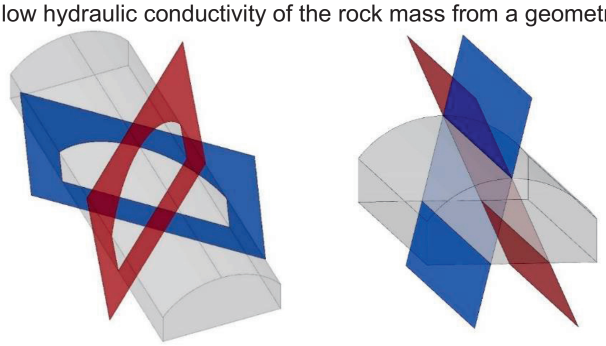

Fig. 32 Representation of a three-dimensional of rock joint pattern in a two-dimensional model. In a two-dimensional model, the three-dimensional configuration of the two joints sets in the left image will be represented with lines with the apparent dip angles in the vertical cross section. Hence, in a two-dimensional model the joints with orientation shown in the left image will be simulated as shown in the right image.

## 4.9 Loads in the rock mass caused by joint water pressure

Water pressures in the immediate rock mass of the lining can lead to destabilizing effects due to reduced effective stresses on joints. Even with a completely drained lining system, which can be expected to give a certain water pressure reduction in the immediate rock mass compared to the hydrostatic pressure, effects of reduced effective stresses have been found to be significant. This is of particular importance in weakness zones with soil character (Anagnostou 2006, Kolymbas 2007). The rock masses considered in this study have Young's moduli of intact rock in the range of 60-100 GPa and uniaxial compressive strengths in the range of 140 to 230 MPa. The in-situ rock stresses for our considered cases are typical for the Scandinavian crystalline bedrock region, with horizontal to sub horizontal major principal stresses in the magnitude of 5 MPa higher than a theoretical gravity induced horizontal stress component. Tangential rock stresses in the immediate rock mass will therefore in many cases be high, and give high normal stresses on rock joints, which in turn will have a strongly stabilizing effect. A reduction of the effective stresses on the rock joints at 1-2 m distance from the lining surface in the magnitude of 0.2 to 0.3 MPa for the Karmsund and Forsmark cases can be assumed based on the in-situ water pressure measurements. In proportion to the high tangential rock stresses which can be expected, this reduction of effective stresses is unlikely to have a significant effect on the friction properties of the rock joints.

Experiences from decades of tunnel projects in hard rock under high ground water pressure, show that collapses caused by ground water pressure are very rare in such conditions (Palmström & Stille 2010). During the construction of more than 40 subsea tunnels hard rock in Norway the last 30 years, with hydrostatic pressures up to 2.5 MPa, there are no reports or records of such collapses having occurred in competent rock (Nilsen 2014).

## 4.10 Observed pressure response during the construction of the lining

The measured water pressures at Forsmark indicate that the application of membrane has no significant influence the ground water pressure. The application of the sprayed concrete shows a slight effect on the measured water pressures (Fig. 27, Sect 3.8.3). The visual effect of the application of the sprayed concrete substrate is a reduced number of wet spots.

# 5 CONCLUSIONS

The tunnel lining system with sprayed concrete and sprayed waterproofing membrane has been investigated with respect to possible effects on ground water pressure in the immediate rock mass. The lining system is an undrained structure, and has been applied on the walls and crown but with a completely drained invert. The cases which have been investigated in this study are all in hard crystalline bedrock with high Young's moduli and high uniaxial compressive strengths of intact rock. The main findings of this study are:

- Recording of water pressures under hydraulically saturated conditions in the immediate rock mass around the tunnel lining do show a gradient with a significant *reduction* of water pressure at closing distance down to approximately 1 m from the lining surface
- Models which account for the joint transmissivity or average rock mass hydraulic conductivities only, not taking into account any damage effects causing increased hydraulic conductivity near the lining, conversely predict an *increase* in water pressure at decreasing distance from the lining surface
- At 2-3 m distance from the lining surface the discontinuum code predicts water pressures which are in good agreement with the measured
- Compared to the continuum code, the discontinuum code predicts hydro-mechanical behavior in closer agreement with the measurements

- The two cases which both had hydraulic conductivities of the rock mass in the magnitude of $10^{-8}$ m/s and a hydrostatic pressure in the range of 680 to 780 kPa both showed a reduction of water pressure near the lining surface
- The investigated case which had ground water level slightly above the invert level of the tunnel, showed constantly measured water pressures in the rock mass up to 9 m depth in the magnitude of the atmospheric pressure
- The investigated case with a water bearing zone with increased rock jointing and a hydrostatic pressure in order of 400 kPa, showed a water pressure in the crown of the tunnel of 135 kPa measured at 150 mm distance behind the undrained lining
- The effect of the undrained membrane on the measured ground water pressure was found to be insignificant in the case where the effect of the primary sprayed concrete could be substantiated
- The effect of the increased hydraulic conductivity of the excavation damaged zone in the first approximately 500 mm of the immediate rock mass is a likely explanation for the measured water pressures
- An EDZ with significantly higher hydraulic conductivity is likely to pose a critical condition for the feasibility of the undrained SCL lining in rock masses with low hydraulic conductivity under high groundwater pressure
- Further detailed investigations of the hydraulic conductivity in the immediate rock mass under a wider range of conditions is required to substantiate this effect on a general basis
- The effect of a pre-grouted zone around the tunnel with presumed reduced hydraulic conductivity needs to be investigated by recordings in longer bore holes, and by modelling with more detailed data

## ACKNOWLEDGEMENTS

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