DOPAS Work Package 3 - Deliverable 3.30

WP3 Final Summary Report

Summary of, and Lessons Learned from, Design and Construction of the DOPAS Experiments

Grant Agreement No:	323273
Authors:	Matt White and Slimane Doudou (Galson Sciences Limited), Jean-Michel Bosgiraud and Régis Foin (Andra), Oliver Czaikowski (GRS), Pär Grahm (SKB), Erika Holt (VTT), Michael Jobmann (DBE TEC), Petri Koho (Posiva), and Jiri Svoboda (CTU)
Date of preparation:	16 November 2016
Version status:	Version B

Start date of the project: September 2012 Duration: 48 months

Project co-funded by the European Commission under the Euratom Research and Training Programme on Nuclear Energy within the Seventh Framework Programme		
Dissemination Level		
PU	Public	X
PP	Restricted to other programme participants (including the Commission Services)	
RE	Restricted to a group specified by the partners of the DOPAS Project	
CO	Confidential, only for partners of the DOPAS Project	

Scope	Deliverable n°D3.30 (WP3)	Version:	A
Type/No.	Report	Total pages	161
Title	WP3 Final Summary Report - Summary of, and Lessons Learned from, Design and Construction of the DOPAS Experiments	Articles:	9

History Chart			
Type of revision	Document name	Partner	Date
Storyboard for discussion at the WP3/WP4 Meeting in Braunschweig, Germany, 01.10.15	WP3 Final Summary Report v1d1	GSL	29.09.15
Rough draft of the report for partner input	WP3 Final Summary Report v1d2	GSL	23.10.15
Full draft for partner comment and additions	WP3 Final Summary Report v1d3	GSL	11.12.15
Second full draft ready for review using the Expert Elicitation process	WP3 Final Summary Report v1d4	GSL	29.02.16
Document in final version for review and approval, following the Expert Elicitation Review	WP3 Final Summary Report A	Andra	31.10.2016
Final version following review statement	WP3 Final Summary Report B	Andra	16.11.2016

REVIEW/OTHER COMMENTS:

The report was reviewed by the DOPAS Consortium members (and later approved by them using review statements). The WP3 leader (Andra) has edited the report for submission and approval by the DOPAS Coordinator (POSIVA).

APPROVED FOR SUBMISSION:

Johanna Hansen ***16.11.2016***

Executive Summary

Report Background

The Full-Scale Demonstration of Plugs and Seals (DOPAS) Project is a European Commission (EC) programme of work jointly funded by the Euratom Seventh Framework Programme and European nuclear waste management organisations (WMOs). The DOPAS Project is running in the period September 2012- August 2016. Fourteen European WMOs and research and consultancy institutions from eight European countries are participating in the DOPAS Project. The Project is coordinated by Posiva (Finland). A set of full-scale experiments, laboratory tests, and performance assessment studies of plugs and seals for geological repositories are being carried out in the course of the project.

The DOPAS Project aims to improve the industrial feasibility of full-scale plugs and seals, the measurement of their characteristics, the control of their behaviour in repository conditions, and their performance with respect to safety objectives. This work does not start from a clean slate, since previous works (at a lower scale) were anteriorly implemented by some of the participants. For example, the work carried out in the FP6 ESDRED project is one of the reference works on which the DOPAS work is partly building on.

The Project is being carried out in seven Work Packages (WPs). WP1 includes project management and coordination and is led by Posiva, Finland. WP2, WP3, WP4 and WP5 address, respectively, the design basis, construction, compliance testing, and performance assessment modelling of five full-scale experiments and laboratory tests. WP2, WP3, WP4 and WP5 are led by SKB (Sweden), Andra (France), RWM (United Kingdom), and GRS (Germany), respectively. WP6 and WP7 address cross-cutting activities common to the whole project through review and integration of results, and their dissemination to other interested organisations in Europe and beyond. WP6 and WP7 are led by Posiva.

The DOPAS Project focuses on tunnel, drift, vault and shaft plugs and seals for clay, crystalline and salt rocks:

- *Clay rocks*: the Full-scale Seal (FSS) experiment, being undertaken by Andra in a surface facility at St Dizier, is an experiment of the construction of a drift and intermediate level waste (ILW) disposal vault seal.
- *Crystalline rocks*: experiments related to plugs in horizontal tunnels, including the Experimental Pressure and Sealing Plug (EPSP) experiment being undertaken by SÚRAO and the Czech Technical University (CTU) at the Josef underground research centre (URC) and underground laboratory in the Czech Republic, the Dome Plug (DOMPLU) experiment being undertaken by SKB and Posiva at the Äspö Hard Rock Laboratory (Äspö HRL) in Sweden, and the Posiva Plug (POPLU) experiment being undertaken by Posiva, SKB, VTT and BTECH at the ONKALO Underground Rock Characterisation Facility (URCF) in Finland, which is also the site of the future Finnish repository.
- *Salt rocks*: tests related to seals in vertical shafts under the banner of the Entwicklung von Schachtverschlusskonzepten (development of shaft closure concepts – ELSA) experiment, being undertaken by DBE TEC together with the Technical University of Freiburg and associated partners, complemented by laboratory testing performed by GRS and co-funded by the German Federal Ministry for Economic Affairs and Energy (BMWi).

Each experiment represents a different stage of development. The Swedish experiment was started prior to the start of the DOPAS Project and was pressurised during the early stages of the Project. The Finnish, Czech and French experiments were designed and constructed during the Project. Initial pressurisation of the Finnish and Czech experiments occurred within the last year of the Project. The French experiment was not pressurised, but dismantling of the experiment was undertaken during the Project. The German tests focused on the early stages of design basis development and on demonstration of the suitability of designs through performance assessment studies and laboratory testing, and will feed into a full-scale experiment of prototype shaft seal components to be carried out after DOPAS.

This report is Deliverable D3.30 of the DOPAS Project, the final summary report of WP3. The objective of this report is to provide an integrated summary of the work undertaken and the lessons learned in the DOPAS Project related to the detailed design and construction of the full-scale experiments. A parallel report, DOPAS Deliverable D4.4 discusses the performance of the experiments and the feedback to the design basis.

This report aims to summarise the work undertaken and identify the lessons learned from the following aspects of the experimental work:

- The objectives of the experiments undertaken in the DOPAS Project.
- The laboratory investigations and *in situ* materials testing that helped in the determination and confirmation of the properties of the materials used in the experiments.
- Analytical and numerical calculations used to underpin the designs and any modifications made to the design basis.
- The approach used to site the full-scale experiments, and the reasons for selecting the location chosen.
- The excavation or construction of the experiment location.
- The installation of the experiment components.

Lessons learned are considered from the perspective of an individual experiment and by cross-comparing the outcomes from the design and construction work undertaken.

Design and Construction of the Experiments

During the course of the DOPAS Project, the designs for the four full-scale experiments have been finalised and the experiments have been successfully constructed.

FSS Experiment

The FSS experiment was a test of the technical feasibility of constructing a drift and Intermediate-level waste (ILW) vault seal at full scale. The test box has an internal diameter of 7.6 m and is 35.5-m long. FSS includes a swelling clay core supported by two low-pH concrete containment plugs. Andra tested two types of low-pH for the containment plugs: low-pH self-compacting concrete (SCC) and low-pH shotcrete.

Construction of the FSS experiment test box commenced in November 2012. Materials research was undertaken in the period August 2012-April 2014, and the main components of the experiment were installed between July 2013 and September 2014.

A range of concrete mixes were tested in the laboratory, and in mock-up tests at the metre and several-metre scales. Design and selection of the SCC mix was undertaken in a three-step process in which the range of options was progressively narrowed. Final selection of the materials considered a global analysis using both technical parameters (compressive strength, shrinkage, organic matter concentration, pH, porosity and permeability, workability) and non-technical parameters (distance of the manufacturer to the test facility and cost). The preferred solution was a binary mix with 50% cement and 50% silica fume. Design and selection of the shotcrete mix followed a similar multi-step process, and similar parameters were used in the global analysis (the analysis also included the odour of the mixture as a result of sulphur presence in the slag materials). The preferred solution was again a binary mix, with the selection particularly affected by the pH and compressive strength of the mix.

Andra has adopted a pellet-based system for installation of the swelling clay core, as the use of pellets is considered by Andra to be an efficient industrial method for implementation of significant quantities of materials. Testing of candidate materials in the laboratory identified a preferred admixture of 32-mm diameter pellets combined with powder made of crushed pellets. The bentonite used was WH2 bentonite from Wyoming (a material very similar to MX-80 a brand more commonly known). Emplacement of the admixture used a dual auger system, preliminary metric-scale testing of which identified the need to arrange the augers one-above-the-other. Although the original target for the admixture density was 1.62 kg/m^3, evaluation of the dry density and swelling pressure for WH2 undertaken in parallel with material testing showed that the required swelling pressure could be achieved with a dry density of 1.50 kg/m^3.

The installation of the FSS components was undertaken to plan. Several lessons were learned regarding the method of installation, for example the need to match the SCC retardant dose to the ambient temperature and the need to manage dust and control pellet breakages during the emplacement of the bentonite admixture. Although the casting of the low-pH SCC was successful as the concrete rose progressively inside the box, with smooth and regular emplacement, some problems were encountered with the shotcrete, notably the management of rebound and formation of gaps.

EPSP Experiment

EPSP is a test of materials and technology, extending laboratory experience to the underground environment and to full-scale tests, and building the practical expertise of the SÚRAO personnel. The experiment consists of two glass-fibre-reinforced low-pH shotcrete plugs separated by a zone containing bentonite pellets and a filter. An injection chamber is located between the inner concrete plug and the back wall of the niche. The tunnel diameter is approximately 3.6 m (5.4 m where slots were constructed for the concrete plugs) and the experiment length is approximately 7.2 m.

Selection of the location of the EPSP experiment within the Josef URC and underground laboratory was undertaken in late 2012. The location was characterised and ground conditions improved in the period January 2013-September 2014. Installation of the EPSP components was undertaken between November 2014 and July 2015.

The shotcreting technology and know-how used in EPSP was developed from previous Czech experience in the installation of concrete plugs using fibre-reinforced concrete. For EPSP, low-pH concretes were required. Low-pH concretes previously developed in the Czech national programme had low compressive strength and were unsuitable for EPSP. Therefore, new mixes were developed. The selected shotcrete was a mix of cement, sand and gravel, microsilica and glass fibres. The ratio of microsilica to cement was approximately 1:1.

One of the main aims of EPSP is to demonstrate the suitability of Czech materials and available technologies for construction of tunnel plugs, including the use of Czech bentonite. Following careful consideration of plug construction requirements, factory-produced B75 bentonite (milled, non-activated Ca-Mg bentonite from the Czech Republic) was selected as the principal material for the bentonite part of the plug. Development of the EPSP structural design was underpinned by the results obtained from two laboratory physical models, which were used to investigate physical processes and to provide parameter values for numerical modelling.

The Josef URC and underground laboratory is a former gold mine, located in a hillside with an overlying rock thickness of 90-180 m. Prior to the commencement of the construction of the EPSP experiment, it was necessary to reshape the experimental gallery niche and improve the ground conditions. Rock quality was improved through the injection of polyurethane resin. Niche excavation was initially undertaken using a hydraulic wedge splitting technique, but this approach was problematic and an alternative pressure disintegration technique using non-detonating gas cartridges was successfully applied during later stages of niche reshaping.

Installation of EPSP was undertaken largely to schedule. Although the inner concrete plug was initially emplaced without concrete grouting, preliminary testing of the plug demonstrated that grouting was necessary. Owing to the small size of the Josef URC and underground laboratory tunnels, the concrete for the shotcreting had to be supplied using small trucks, and there was the need to transfer the mix from a big truck that delivered the concrete from the production plant into the small trucks at the entrance of the facility. However, this did not have a significant impact on installation. Installation of the bentonite pellets was undertaken manually using a range of hand-held compaction machines.

DOMPLU Experiment

The DOMPLU experiment was a full-scale test of the reference deposition tunnel plug in SKB’s repository design. The DOMPLU experiment design consisted of an unreinforced low-pH concrete dome with a watertight seal, a filter layer, and a backfill transition zone located upstream of the concrete component. The DOMPLU experiment was constructed inside a horseshoe-shaped tunnel, with a width of 4.2 m and a height of 4.8 m. The diameter of the plug is 9 m at the centre of the slot excavated for the concrete dome, and the experiment length is approximately 6.5 m, with the concrete dome approximately 3.2-m long.

Excavation of the DOMPLU niche was undertaken in early 2012 and the concrete dome slot was excavated in two phases in April-May 2012 and August-October 2012. The experiment components were installed between January and June 2013.

A concrete mix for a low-pH SCC, denoted B200, had previously been developed specifically for use in the deposition tunnel plugs of SKB’s repository. Testing of this concrete was required to ensure that it provided the necessary strength, shrinkage, creep and binding properties to the host rock, given the ambition to demonstrate that the concrete could be emplaced without reinforcement. It was also necessary to demonstrate that properties measured in the concrete factory were representative of the properties the concrete would achieve when poured at the experiment site underground at Äspö. It was judged possible to use the concrete mix B200 without any further development, based on the results of the concrete testing. However, experiences from the B200 concrete test series also showed that further clarification is needed regarding quality control requirements and acceptance criteria of the young concrete properties.

Experimental tests were also carried out to investigate the properties of the bentonite sealing materials, including compaction properties of the bentonite, strength of the blocks, compressibility, swelling pressure, hydraulic conductivity, and self-sealing of fractures in the rock and slots between the bentonite blocks. The tests confirmed that compressed MX-80 bentonite blocks with a dry density of approximately 1,700 kg/m^3 and a water content of 17%, surrounded by a 10-20 cm thick layer of MX-80 pellets, would be a functional configuration for the seal.

A filter layer is included in DOMPLU to ensure that high pressures are not exerted on the concrete dome until it has cured and developed sufficient strength. Candidate filter materials included different combinations of sand and gravel, and geotextiles. The materials were subjected to compaction, compressibility, hydraulic conductivity and clogging tests. Gravel, with a grain size range between 2-4 mm, was selected as the preferred material for the filter. It was decided to test the geotextile at full scale in the DOMPLU experiment and use it as a delimiter between the gravel filter and the bentonite seal. The purpose was to facilitate distribution of water to the seal from the filter. In addition, light-weight expanded clay/concrete aggregate (LECA®) beams were used both as a delimiter and as part of the filter. LECA® was selected because it has a high hydraulic conductivity and also because it maintains it hydraulic performance when exposed to a water flow with high bentonite content (it does not clog).

Development of the DOMPLU structural design was supported by the testing of the overall design in a scaled laboratory model and by analytical and numerical modelling to predict the hydromechanical and thermal behaviour of the plug.

DOMPLU was sited at the deepest part of the Äspö HRL in order to most appropriately represent the pressure conditions expected in the repository. Siting was supported by rock characterisation activities, including hydraulic testing in a pilot borehole to identify the location of the concrete dome slot. Similar procedures will be applied in the repository and the work in the DOMPLU experiment was a good test of the procedures.

The concrete dome slot was excavated using a new wire sawing technique. This technique might provide a more rapid excavation technique and might reduce the development of an EDZ compared to traditional drill and blast methods, and it may also provide a smooth surface that would allow release of the concrete dome prior to contact grouting. The excavation of the slot by wire sawing was more problematic than expected, mainly as a result of stresses in the rock. However, the method was shown to be feasible and resulted in smooth surfaces.

One of the main outcomes from the DOMPLU experiment was the demonstration that it is possible to build the dome plug system. This includes practical aspects of logistics and arranging of parallel construction activities in a tunnel system. The dome plug was successfully constructed without reinforcement. However, it was found that installation of the concrete beams, filter and seal-pellets near the tunnel ceiling was quite difficult. A cooling system applied to the concrete dome to limit cracking and to pre-stress the concrete prior to contact grouting was successfully applied. The performance of the concrete in response to the cooling system is discussed in Deliverable D4.4.

POPLU Experiment

The POPLU experiment design is based on a wedge-shaped low-pH stainless steel-reinforced concrete structure that is cast in place into a slot that has been notched into the EDZ. The concrete wedge is cast directly adjacent to a filter layer in front of a concrete tunnel back wall. The concrete wedge contains grouting tubes and bentonite circular strips at the rock-concrete interface to ensure water tightness. The steel reinforcement counteracts shrinkage stresses and helps to limit the release of the concrete from the rock. The POPLU experiment was constructed in a horseshoe-shaped tunnel. The tunnel diameter is approximately 4.35 m (6.35 m where slots were constructed for the concrete plugs) and the experiment length is approximately 11 m, with the concrete wedge approximately 6-m long.

Characterisation and excavation of the POPLU niche, including the drilling of pilot boreholes, was undertaken between November 2012 and February 2015. The experiment components were installed between March and December 2015.

The design work undertaken for the POPLU experiment included use of analytical and numerical calculations to underpin the design and identify the required reinforcement on the wedge plug, to plan four mock-up tests, and to select and position monitoring sensors. The work included static and dynamic stress modelling and hydraulic modelling. The work recognised that the addition of bentonite behind the concrete wedge would increase the water tightness of the POPLU experiment plug design. However, building the experiment without a bentonite sealing layer would allow evaluation of the reliability of the concrete structure specifically and would allow much more rapid pressurisation of the concrete plug. Therefore, no bentonite seal was included in the POPLU experiment.

Posiva's reference low-pH concrete is similar to B200, but further development of the concrete mix was required to ensure workability in relation to the significant quantities of steel bar reinforcement used in the POPLU experiment concrete wedge. The preferred concrete was a ternary mix with 38% cement, 32% silica and 30% fly ash, which was judged to be the most durable and best at meeting performance requirements. Three mock-up tests (referred to as method tests by Posiva) were undertaken to finalise the mix and a fourth test was undertaken to finalise the contact grouting mix. Modifications were introduced in response to these tests, including reduction in the aggregate grain size in the upper and lower parts of the concrete wedge.

Six different bentonite strip (or tape) products were tested in the laboratory. The bentonite tape used was chosen as it had a high level of swelling in the first day and had the greatest mass of bentonite within the tape compared to the other products (and thus the lowest amount of foreign materials).

Siting of the POPLU experiment applied Posiva's Rock Suitability Classification (RSC) methodology to the siting of a deposition tunnel end plug for the first time. Concrete wedge slot production used a wedging and grinding method. This method was selected, instead of the originally-planned wire sawing method, owing to operational safety concerns with the best technology that could be demonstrated compared to the costs. It was also deemed beneficial to apply different slot production methods in the POPLU experiment compared to two of the other DOPAS Project experiments.

During installation of the POPLU experiment and related components, it was demonstrated that the construction sequence was well-planned and the schedule could be maintained. The formwork was well-designed and could be implemented and served its purpose. The reinforcement steel bars could be assembled efficiently on-site. The low-pH concrete materials developed in the laboratory within the DOPAS Project proved to be excellent when applied at full scale in the POPLU experiment. The SCC was fluid for placement without vibration, had a good open workability time and could be pumped into place. The low-pH grout material was able to penetrate the contact area between the concrete plug and rock.

ELSA Experiment and Complementary Laboratory Testing

The work in ELSA during the DOPAS Project is largely focused on studies that will support the development of a shaft seal design. The aims of the ELSA project are to develop generic design concepts for shaft seals in salt and clay host rocks that comply with the requirements for a repository for high-level waste (HLW) and to carry out the necessary preparatory work in the shaft seal design project.

Work in the ELSA Project has included a development of a "hard shell – soft core" concept that has been tested in a borehole in the Sondershausen mine in Germany, and development of MgO concrete tested as a plug in a large-diameter borehole also at Sondershausen. In addition, compaction of admixtures of crushed salt and clay intended to be used as a long-term sealing element has been tested *in situ*.

In addition to the above investigations, the laboratory programme of GRS (which is undertaken within the auspices of the LASA, LAVA and THM-Ton Projects) address sealing materials planned to be utilised in the shaft seals. This laboratory programme provides supporting information to the ELSA Project. For example, the programme aims at providing experimental data needed for the theoretical analysis of the long-term behaviour of MgO-concrete and cement-based salt concrete in interaction with the host rock and fluids.

From the *in situ* and laboratory experiments performed so far in the ELSA Project and the complementary laboratory work, several methods and materials have been identified that can significantly improve the sealing capabilities of the long-term sealing element that forms one part of the current reference shaft sealing concept for repositories in salt dome structures. When using pure crushed salt for constructing the sealing element, as in the current reference design, a long time is required to achieve the full sealing abilities through porosity and permeability reduction in response to compaction driven by rock convergence. Using an admixture that includes fine clay, the initial porosity directly after installation can be significantly reduced. In addition, owing to inflowing brine, the clay admixture starts to swell leading to clogging of pores, and an additional reduction in porosity and permeability. Generally, it can be stated that the time which is necessary to develop the full sealing abilities is reduced significantly.

Conclusions

Within WP3 of the DOPAS Project, four full-scale experiments of plugs and seals have been designed and constructed. These include the FSS experiment in France, the EPSP experiment in the Czech Republic, the DOMPLU experiment in Sweden, and the POPLU experiment in Finland. In addition, a series of *in situ* tests and complementary laboratory investigations have been completed in Germany.

Design and construction of the experiments contributes to the readiness level of plug and seal installation in repositories in the near future. The completion of the experiment design and construction represents a successful collaboration between WMOs, research institutes and consultants.

The design and construction of the DOPAS experiments has demonstrated that plugs and seals are more challenging and complex sub-systems of the repository than previously recognised. The challenges and complexities of plug/seal design and construction can be met through available technology, methods and procedures currently available.

Common approaches to design and construction of plugs and seals have been developed and implemented by the WMOs responsible for different disposal programmes. These include, in crystalline rocks, the excavation of a slot through which the plug/seal can be keyed into the rock. In clay and salt host rocks, benefit is drawn from the creep properties of the rock to provide an effective seal when operating in conjunction with engineered features.

Common approaches also include the use of low-pH concrete and/or bentonite systems as the primary components of plugs and seals. Significant work on low-pH concrete and bentonite pellet, tape and block systems has been undertaken within WP3 of the DOPAS Project. Contact grouting is a common feature to all four of the full-scale tests.

The host rock can significantly impact the installation of plugs and seals. Weak rock, the presence of water-bearing fractures and formation of break-outs can be challenging, but approaches have been developed and demonstrated in WP3 of the DOPAS Project to overcome these challenges.

Logistics is a significant issue for plugs and seals. There may be multiple components requiring installation and appropriate time must be allowed for these materials to be installed and evolve prior to installation of the next component. There may be issues associated with manpower and machinery availability (and performance). Therefore, contingency planning, such as the provision of back-ups and spares may be necessary. Contingencies also need to be built into project plans and schedules. Finally workers' safety (e.g. during rock excavation) and workers' health (e.g. bentonite generated dust) must also be considered in the construction process.

Application of the lessons learned from the DOPAS experiments and feedback to reference designs will be considered in WP4 of the DOPAS Project and reported in Deliverable D4.4 (WP4 integrated Report). This will also include an analysis of further work required to develop designs so that they are ready for implementation in repositories in the near future.

List of DOPAS Project Partners

The 14 partners from 8 different countries in the DOPAS Project are listed below. In the remainder of this report each partner is referred to as indicated:

Posiva	Posiva Oy	Finland
Andra	Agence nationale pour la gestion des déchets radioactifs	France
DBETEC	DBE TECHNOLOGY GmbH	Germany
GRS GmbH	Gesellschaft für Anlagen- und Reaktorsicherheit	Germany
Nagra	Die Nationale Genossenschaft für die Lagerung Radioaktiver Abfälle	Switzerland
RWM	Radioactive Waste Management Limited	UK
SÚRAO	Správa Úložišť Radioaktivních Odpadu (Radioactive Waste Repository Authority – RAWRA)	Czech Republic
SKB	Svensk Kärnbränslehantering AB	Sweden
CTU	Czech Technical University	Czech Republic
NRG	Nuclear Research and Consultancy Group	Netherlands
GSL	Galson Sciences Limited	UK
BTECH	B+ Tech Oy	Finland
VTT	Teknologian Tutkimuskeskus VTT Oy (VTT Technical Research Centre of Finland Ltd)	Finland
UJV	Ustav Jaderneho Vyzkumu (Nuclear Research Institute)	Czech Republic

List of Acronyms - Abbreviations

AECL:	Atomic Energy of Canada Limited
ASN:	Autorité de Sûreté Nucléaire (Nuclear Safety Authority in France)
BMU:	Bundesministerium für Umwelt, Naturschutz und Reaktorsicherheit (Federal Ministry for the Environment, Nature Conservation, Building and Nuclear Safety in Germany)
BMWi:	Bundesministerium für Wirtschaft und Energie (Federal Ministry for Economic Affairs and Energy in Germany)
Cigéo:	Centre Industriel de Stockage Géologique (Industrial Repository in France)
DOMPLU:	Dome Plug
DOPAS:	Full-scale Demonstration of Plugs and Seals
EBS:	Engineered barrier system
EC:	European Commission
EDZ:	Excavation damaged zone
EE:	Expert Elicitation
ELSA:	Entwicklung von Schachtverschlusskonzepten (Development of shaft closure concepts)
EPSP:	Experimental Pressure and Sealing Plug
FSS:	Full-scale Seal (test)
GBT:	Green Break Technology
GPR:	Ground penetrating radar
HRL:	Hard Rock Laboratory
ILW:	Intermediate-level waste
LASA:	**La**ngzeitsicherer **S**chachtverschluß im S**a**linar (long-term safe shaft closure in salt)
LECA®:	Light-weight expanded clay/concrete aggregate
LVDT:	Linear variable differential transformer
OPC:	Ordinary Portland Concrete
PHM:	Physical Hydraulic Model
PIM:	Physical Interaction Model
POPLU:	Posiva Plug
PVDF:	Polyvinylidene fluoride
R&D:	Research and development
RSC:	Rock Suitability Classification
SCC:	Self-compacting concrete
STUK:	The Finnish Nuclear Regulatory Authority
TC-Tests:	Triaxial compressions tests
TDR:	Time domain reflectometer

THM-Ton: Untersuchung der THM-Prozesse im Nahfeld von Endlagern in Tonformationen (investigation of THM processes in the near field of a repository in clay)

URC: Underground Research Centre

URCF: Underground Rock Characterisation Facility

URL: Underground research laboratory

WMO: Waste management organisation

WP: Work package

Table of Contents

1. Introduction

1.1 Background

The Full-Scale Demonstration of Plugs and Seals (DOPAS) Project is a European Commission (EC) programme of work jointly funded by the Euratom Seventh Framework Programme and European nuclear waste management organisations (WMOs). The DOPAS Project is running in the period September 2012 – August 2016. Fourteen European WMOs and research and consultancy institutions from eight European countries are participating in the DOPAS Project. The Project is coordinated by Posiva (Finland). A set of full-scale experiments, laboratory tests, and performance assessment studies of plugs and seals for geological repositories are being carried out in the course of the project.

The DOPAS Project aims to improve the industrial feasibility of full-scale plugs and seals, the measurement of their characteristics, the control of their behaviour in repository conditions, and their performance with respect to safety objectives. This work does not start from a clean slate, since previous works (at a lower scale) were anteriorly implemented by some of the participants. For example, the work carried out in the FP6 ESDRED project is one of the reference works on which the DOPAS work is partly building on.

The Project is being carried out in seven Work Packages (WPs). WP1 includes project management and coordination and is led by Posiva, Finland. WP2, WP3, WP4 and WP5 address, respectively, the design basis, construction, compliance testing, and performance assessment modelling of five full-scale experiments and laboratory tests. WP2, WP3, WP4 and WP5 are led by SKB (Sweden), Andra (France), RWM (United Kingdom), and GRS (Germany), respectively. WP6 and WP7 address cross-cutting activities common to the whole project through review and integration of results, and their dissemination to other interested organisations in Europe and beyond. WP6 and WP7 are led by Posiva.

The DOPAS Project focuses on tunnel, drift, vault and shaft plugs and seals for clay, crystalline and salt rocks:

- *Clay rocks*: the Full-scale Seal (FSS) experiment, being undertaken by Andra in a surface facility at St Dizier, is an experiment of the construction of a drift and Intermediate-Level Waste (ILW) disposal vault seal.
- *Crystalline rocks*: experiments related to plugs in horizontal tunnels, including the Experimental Pressure and Sealing Plug (EPSP) experiment being undertaken by SÚRAO and the Czech Technical University (CTU) at the Josef underground research centre (URC) and underground laboratory in the Czech Republic, the Dome Plug (DOMPLU) experiment being undertaken by SKB and Posiva at the Äspö Hard Rock Laboratory (Äspö HRL) in Sweden, and the Posiva Plug (POPLU) experiment being undertaken by Posiva, SKB, VTT and BTECH at the ONKALO Underground Rock Characterisation Facility (URCF) in Finland, which is also the site of the future Finnish repository.
- *Salt rocks*: tests related to seals in vertical shafts under the banner of the Entwicklung von Schachtverschlusskonzepten (development of shaft closure concepts – ELSA) experiment, being undertaken by DBE TEC together with the Technical University of Freiburg and associated partners, complemented by laboratory testing performed by GRS and co-funded by the German Federal Ministry for Economic Affairs and Energy (BMWi).

Each experiment represents a different stage of development. The Swedish experiment was started prior to the start of the DOPAS Project. The Finnish, Czech and French experiments were designed and constructed during the Project. The German tests focused on the early stages of design basis development and on demonstration of the suitability of designs through performance assessment studies and laboratory testing, and will feed into a full-scale experiment of prototype shaft seal components to be carried out after DOPAS.

This report is Deliverable D3.30 of DOPAS, and is part of WP3. This work package addresses the detailed design and construction of the full-scale tests in DOPAS. Deliverable D3.30 is the final summary report of WP3.

1.2 Objective

The objective of this report is to provide an integrated summary of the work undertaken and the lessons learned in the DOPAS Project WP3 related to the detailed design and construction of the full-scale experiments.

This report aims to summarise the work undertaken and identify the lessons learned from the following aspects of the experimental work:

- The objectives of the experiments undertaken in the DOPAS Project.
- The laboratory investigations and *in situ* materials testing that helped in the determination and confirmation of the properties of the materials used in the experiments.
- Analytical and numerical calculations used to underpin the designs and any modifications made to the design basis.
- The approach used to site the full-scale experiments, and the reasons for selecting the location chosen.
- The excavation or construction of the experiment location.
- The installation of the experiment components.

Lessons learned are considered from the perspective of an individual experiment and by cross-comparing the outcomes from the design and construction work undertaken. Common issues, when possible, are also emphasized.

As emphasized by the Experts who carried out the Expert Elicitation process on D3.30, one area of influence that is not included and not mentioned in the design of the experimental designs is the presence of radiological hazard in the actual repository context in the future. It is absent from the experiments in DOPAS Project scope of work, but it may have an impact in the future on the use of the methods now selected as at least some of the plugs are in the vicinity of the waste package disposal operations. Another area (evidenced by the Experts) that has not been addressed in connection with the experiments is the need of resources (personnel, costs) and competences to implement the experiments and the plugs and seals in the future repository.

1.3 Scope and Link to other DOPAS Deliverables

The intended audiences of this report are technical staff and technical management of WMOs, and the report has been written primarily for this audience. The report is also expected to be of potential interest to regulators and TSO (Technical support organizations).

This report (D3.30) is part of a series of WP-level summary reports describing the integrated outcomes of the technical work in DOPAS:

- D2.4, the WP2 Final Report (DOPAS, 2016a), describes the design basis for the plugs and seals considered in DOPAS, conceptual and basic designs, and the strategy adopted in programmes for demonstrating compliance with the design basis. The design basis is presented for both the repository reference design and the full-scale experiment design.
- D3.30, the WP3 Final Summary Report (this document), summarises the work undertaken and the lessons learned from the detailed design and construction of the experiments. These include the full-scale demonstrators, laboratory work and its upscaling, and the learning provided by the practical experience in constructing the experiments.
- D4.4, the WP4 Integrated Report (DOPAS, 2016b), summarises what has been learnt with respect to the repository reference designs for plugs and seals. The report also considers alternatives to the repository reference designs (e.g. the wedge-type plug investigated by Posiva). It considers what can be concluded from the full-scale experiments conducted in DOPAS with respect to the technical feasibility of installing the reference designs, the performance of the reference designs with respect to the safety functions listed in the design basis, and identifies and summarises achievements of DOPAS WP2, WP3 and WP4 at the time of writing. D4.4 also considers the feedback from the work to the design basis.
- D5.10, the WP5 Final Integrated Report (DOPAS, 2016c), describes the conceptualisation of plugs and seals in post-closure safety assessments and the expected long-term evolution of plugs and seals. This includes a description of the evidence that the materials used in plugs and seals will maintain their required performance for the period specified in the design basis.

D3.30 is based on information available by the date of a data freeze of 31 December 2015. At this time, all design and installation work had been completed, although some assessment of the work was on-going. Progress in the experiments by this date was as follows:

- DOMPLU: The DOMPLU concrete dome was cast in March 2013 and contact grouting was undertaken in June 2013.
- POPLU: The POPLU concrete wedge was cast in May and September 2015. Contact grouting was undertaken in December 2015.
- EPSP: The EPSP inner plug was cast in November 2014, the bentonite core was emplaced in June 2015 and the outer plug was cast in June 2015.
- FSS: For FSS, the upstream containment wall was cast in June 2013, the clay core was emplaced in August 2014 and the downstream shotcrete plug was emplaced in September 2014. Investigations of FSS were undertaken in the period November 2014 to July 2015, followed by dismantling between August 2015 and December 2015 (information related to this phase is dealt with in D4.4).
- ELSA: With regard to the long-term sealing element consisting of crushed rock salt and potentially a clay admixture, laboratory and *in situ* compaction tests had been completed by December 2015. In parallel, a small-scale *in situ* test applying MgO-concrete has been performed and a mock-up test of a bentonite shaft seal has started. Small-scale *in situ* tests on the use of bitumen as sealing material were undertaken

during the beginning of 2015. In addition, a laboratory programme undertaken within the auspices of the LASA, LAVA and THM-Ton Projects addresses sealing materials planned to be utilised in the shaft seals. This laboratory programme provides supporting information to the ELSA Project.

Further details of the achievements in DOPAS, based on additional assessment of the design and construction work undertaken after 31 December 2015, is included in experiment summary reports (Andra, 2016; Svoboda *et al*., 2016; Grahm *et al*., 2015; Holt and Koho, 2016; Jantschik and Moog, 2016; Czaikowski and Wieczorek, 2016; and Zhang, 2016). and Kudla *et al*. (2016)) and in the DOPAS Final Project Summary Report (D6.4) (DOPAS 2016d).

As mentioned above, this report focuses on the experiences from the design and construction of the experiments and draws lessons on the conduct of full-scale experiments. As the subject of DOPAS is "plugs and seals", these lessons are principally focused on "plugs and seals", but many of the lessons are also applicable to the conduct of full-scale experiments in general. As noted above, the parallel report D4.4 summarises the monitoring and performance of the plugs and seals, and the learning related to the reference designs. The monitoring and performance discussed in D4.4 includes the monitoring of the materials in response to installation (e.g. the monitoring of the curing temperature and shrinkage of concrete following emplacement) and the monitoring of the plug performance subsequent to pressurisation of the relevant experiment.

1.4 The DOPAS Design Basis Workflow

Work on the design basis in the DOPAS Project has allowed consideration of current practice with regard to both the process used to develop and describe the design basis and the content of the design basis of plugs and seals. The design basis is developed in an iterative fashion with inputs from regulations, technology transfer, tests and full-scale demonstrations, and performance and safety assessments. The learning provided by WP2 has been used to describe a generic process for development of the design basis for plugs and seals called the "DOPAS Design Basis Workflow" (Figure 1.1). This workflow is structured to be consistent with a hierarchy of increasingly detailed designs (IAEA, 2001), as follows:

- Conceptual Design: Conceptual designs describe the general layout of a repository structure, including the different repository components and how they are arranged, and the type of material used for each component (e.g., concrete, bentonite, gravel). In a conceptual design, the environmental conditions (including rock characteristics) are presented in generic terms, for example by describing the nature of the processes occurring rather than quantifying the processes. The performance of the components and the overall structure are described qualitatively.

- Basic Design: In a basic design, the components in the conceptual design are described in more detail with an approximate quantitative specification of geometry and material parameters. The properties of the environmental conditions are presented in detail, which requires characterisation of the site or elaboration of the assumptions underpinning the design. Performance is described quantitatively.

- Detailed Design: In a detailed design, the concept is presented in such detail that it can be constructed, i.e., it provides precise information on all aspects of the structure's components.

At the conceptual design stage, the design basis for a plug/seal includes the stakeholder requirements that define the overall objectives of geological disposal (e.g., the safety criteria that must be met), safety functions for each of the components of the disposal system (e.g., for plugs and seals, this may include limiting groundwater flux through the repository), and the sub-system requirements on each of the components of a plug/seal (e.g., the role of a concrete dome or watertight seal and the plug lifetime). The safety functions are dependent on decisions made on the safety concept, and sub-system requirements are dependent on conceptual design options. Consideration of the site environmental conditions and loads acting on the structures allows conduct of a performance assessment, the results of which feed into a compliance assessment used to ascertain whether the system and sub-system requirements have been met by different conceptual design options. The outcome is selection of a conceptual design of a plug/seal, and elaboration of preliminary design requirements to be tested during development of the basic design.

At the basic design stage, preliminary design requirements are used as the basis for developing preliminary basic designs. During the DOPAS Project, basic designs have been tested through full-scale tests. This has required the development of experiment design specifications, which are used to design the experiment and as the basis for performance evaluation. These design specifications represent working assumptions for reference design specifications, which might be adopted as reference design specifications following evaluation of experiment performance. The full-scale demonstration experiments undertaken in the DOPAS Project have addressed specific objectives, for example, technological feasibility (FSS), performance (DOMPLU), alternative design options (POPLU), and materials research in support of preliminary basic design (EPSP and ELSA).

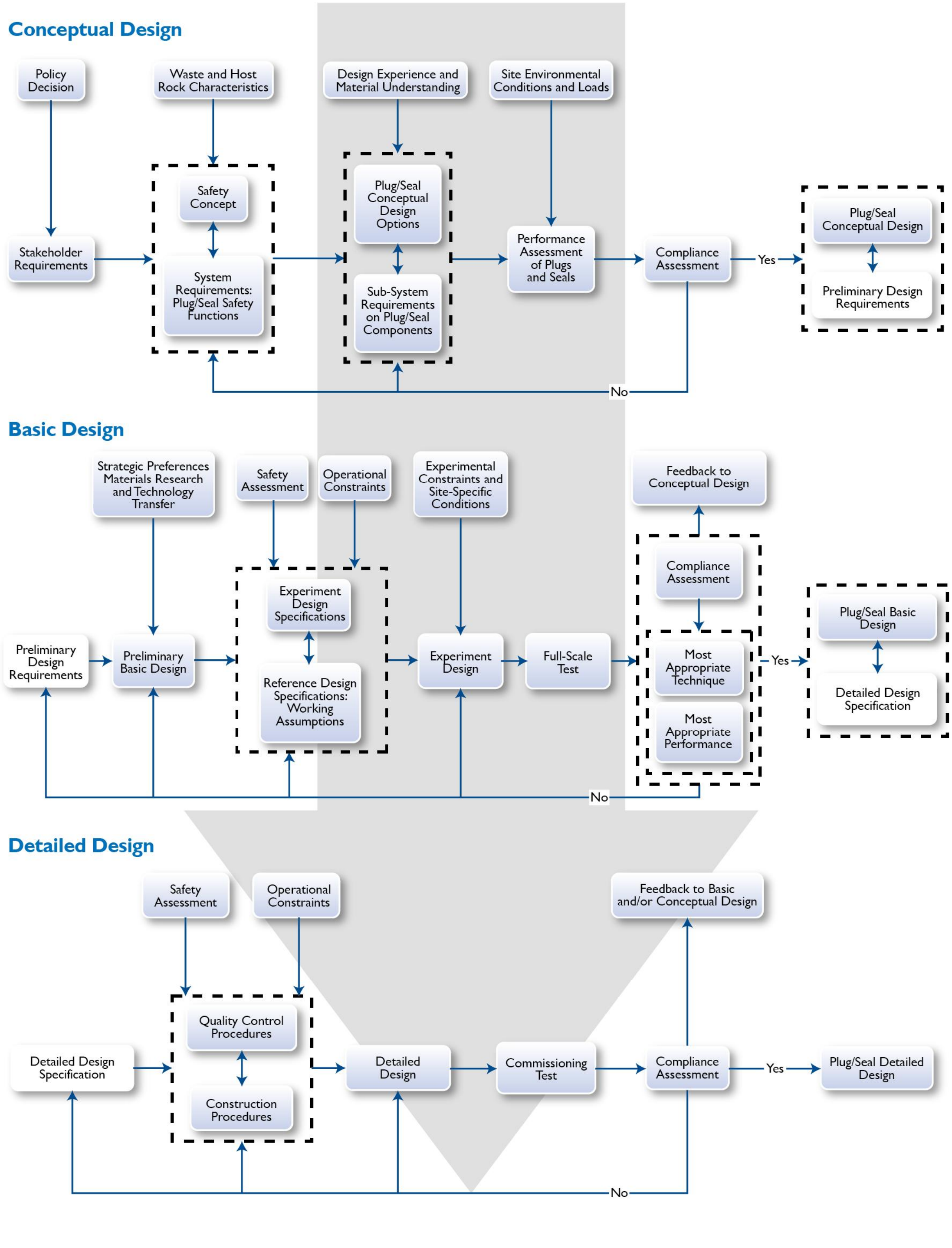


Figure 1.1: The DOPAS Design Basis Workflow, which illustrates the iterative development of the design basis, undertaken in parallel with the development of conceptual, basic and detailed designs. Dashed boxes are used to show activities undertaken in parallel. Terminology used in the Workflow is defined in DOPAS (2016a).

The results of full-scale tests provide further support to design decisions, especially optimisation issues such as the identification of design solutions that represent the most appropriate technique and the most appropriate performance. Compliance assessment at the basic design stage considers the extent to which the experiment results meet the experiment design specifications. Design requirements may be revised based on learning from the experiments, and the result of the compliance assessment can be used to revise the reference design requirements. In parallel, detailed design specifications are prepared based on working assumptions and experiment design specifications used as the basis for the full-scale test. The outcome of a satisfactory compliance assessment is selection of a basic design, and elaboration of detailed design specifications to be tested during development of the detailed design.

At the detailed design stage, the detailed design specification, safety assessment and operational constraints are considered in order to establish quality control procedures and construction procedures. These allow development of a detailed design which may be subject to a commissioning test. In contrast to demonstration testing, the commissioning test is a trial of the plug/seal as it is expected to be implemented in the repository. Consideration may be given to monitoring of these tests over long periods, for example Andra are planning an Industrial Pilot during the early stages of repository operation, which will run for as long as feasible, potentially decades. Compliance assessment of the commissioning test could lead to a revision of the design specifications, for example to write them in a manner that is amenable to checking using quality control or construction procedures. Compliance testing may also identify the need for revisions to the detailed design, which may, therefore, also lead to a need for further testing. Once the compliance assessment is acceptable, the plug/seal detailed design can be finalised and the detailed design specification accepted as the final design specification (subject to further revision based on learning during repository operation).

The DOPAS Design Basis Workflow is based on the design basis work undertaken for plugs and seals within the DOPAS Project. However, the Workflow is generic in nature, and could be applied to other repository design activities. The general applicability of the DOPAS Design Basis Workflow is considered as part of the wider DOPAS Project dissemination activities (WP7).

1.5 Terminology and abbreviations

Throughout this report consistent terminology has been applied. This has required, in places, changing the terminology used in a specific programme or within a specific country. The key terms that have been changed for consistency are:

- In this report, the term used to describe the combination of materials in a specific concrete is *mix*. In specific cases, this term replaces the use of *formulation* and *recipe*.
- In this report, the term used to describe a test of plug/seal components at a reduced scale is *mock-up*. The term *method test* is more frequently applied by Posiva and the tests described in Chapter 5 related to the POPLU experiment would usually be described as method tests by Posiva. The use of the term method test by Posiva, communicates that the test is considering *how* a component is installed as well as demonstrating that it can be installed to meet requirements.

In general, in the DOPAS Project it has been agreed that reference to IAEA glossary (2013) is made in the project to use this glossary for the terms, which are not specifically described in the report's list of abbreviations.

1.6 Report Structure

This report is presented in the following sections:

- Chapters 2-6 provide summaries of the experiences and learning from each experiment (respectively FSS, EPSP, DOMPLU, POPLU and ELSA). For each experiment, the following issues are described:
    - The experiment background and objectives.
    - The development and testing of materials used in the experiment.
    - The structural design, i.e., the work undertaken to select the components in the experiment and their geometrical properties.
    - Siting of the experiment.
    - Excavation of the experiment location or construction of the experiment facility.
    - Installation of the experiment.
    - Lessons learned regarding the design and construction of the specific plug or seal experiment.
- Chapter 7 provides an integrated discussion of the lessons learned and future challenges from the design and construction of the DOPAS experiments.
- Chapter 8 provides conclusions from the report.

2. FSS Experiment

This chapter provides a summary of the learning from design and construction of the FSS experiment:

- In Section 2.1, the background to the experiment and its objectives are summarised.
- In Section 2.2, the testing and selection of materials prior to FSS implementation are described.
- In Section 2.3, the siting of the FSS experiment is explained.
- In Section 2.4, the construction of the FSS test box is described.
- In Section 2.5, the installation of the FSS components is summarised, with particular focus on the novel aspects of the experiment.
- Discussion of the lessons learned regarding the design and installation of the FSS experiment is provided in Section 2.6.

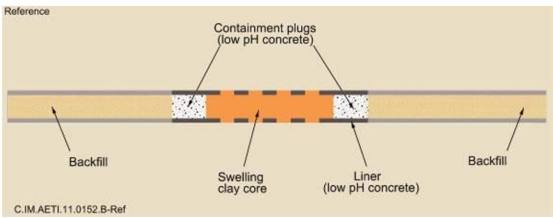
Further details of the design and construction of FSS are available in the FSS experiment Summary Report (D4.8 - Andra, 2016).

2.1 FSS Experiment Background and Objective

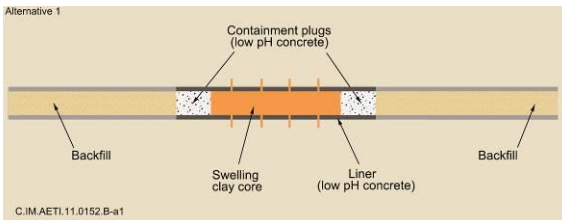
There are approximately 130 seals envisaged at this stage of design in the French reference disposal concept for HLW and ILW (aka Cigéo). Three types of seals are recognised: (vertical) shaft seals, (inclined) ramp seals, and (horizontal) drift and ILW disposal vault seals. Each seal consists of a swelling clay core positioned between two concrete containment walls. The swelling clay core provides the required long-term hydraulic conductivity performance of the seal, whereas the containment walls are included to mechanically contain the clay core when swelling.

Andra has proposed two different conceptual designs for seals (Figure 2.1):

- A **Reference Solution** in which the concrete drift lining in the swelling clay core section is either totally or partially removed prior to emplacement of the core. Removal of the lining allows direct contact of the swelling clay core with the host rock, and the sealing of any potential flow paths within the lining and along its interfaces with the bentonite and host rock.
- An **Alternative Solution** in which radial hydraulic cut-offs (also known as "grooves") are sawn from the host rock allowing the excavation damaged zone (EDZ) to be intercepted by the bentonite material in the seal.



Reference Solution



Alternative Solution

Figure 2.1: Conceptual design for seals in Andra's Cigéo disposal concept.

During the review of Andra's Dossier 2009[1], the nuclear authority (ASN) asked Andra to prove the industrial feasibility of seals by constructing technological demonstrators.

The FSS experiment was built in response to ASN's request, to prove the technical feasibility of constructing a seal at full scale, and especially to:

- Demonstrate the industrial feasibility of emplacement of large volumes of bentonite and emplacement of low-pH concrete and shotcrete at the scale of a Cigéo seal (approximately ten metres in diameter and several tens of metres in length).
- Define the operational requirements useful to obtain the specified properties, for example the tolerances on the density distribution in the core after emplacement or on the ratio of acceptable voids.
- Define and deploy the control means to check the compliance, during construction, with the emplacement requirements.
- Define and deploy the control means to check the compliance, after construction, with the emplacement requirements.

The main difference between the reference and FSS designs for the Andra drift and ILW vault seal is the length of the seal. The real seal underground will be longer than the seal considered in FSS. The FSS experiment investigates two types of low-pH containment wall, one using self-compacting concrete (SCC) and the other using shotcrete, to allow the preferred method to be selected and incorporated into the reference concept.

Technical feasibility includes demonstrating the ability of the approach used to emplace the clay core to be suitable for filling recesses in the clay host rock, i.e., any potential breakouts generated during the removal of the concrete support lining. Therefore, the concrete test box includes recesses that mimic breakouts.

As the experiment is focused on the construction and installation of the seal, the materials were not saturated or otherwise pressurised to check the swelling pressure and hydraulic conductivity. Complementary experiments were undertaken in parallel with FSS. These include the REM experiment, which is part of WP5 of the DOPAS Project and consists of an "as close as possible to *in situ* conditions" resaturation test undertaken in a surface laboratory with the same bentonite pellets/powder mixture as used in FSS.

The conceptual design of the FSS experiment is illustrated in Figure 2.2 and the schedule of activities is summarised in Table 2.1. Further information on the FSS experiment conceptual design and design basis is presented in DOPAS (2016a).

[1] The Dossier 2009 presented Andra's operational safety case and developments to the design of the Cigéo repository since Dossier 2005 (Andra, 2005).

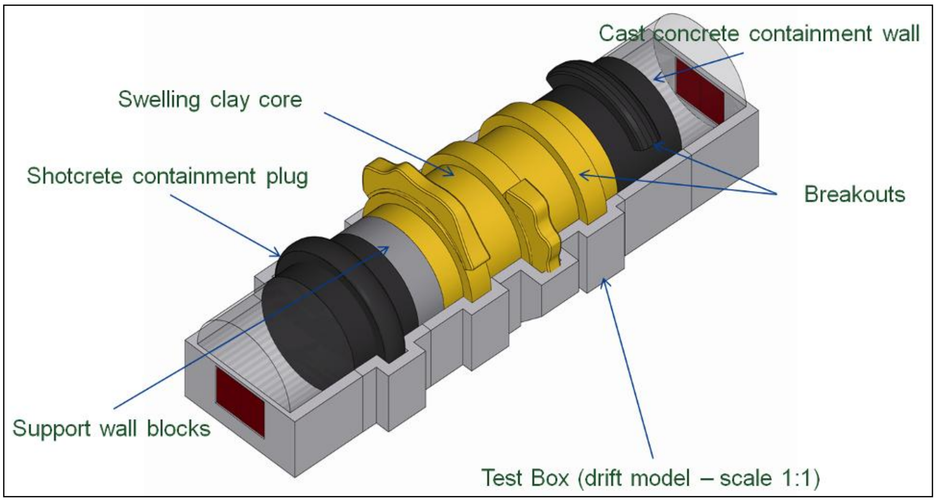


Figure 2.2: Schematic illustration of the FSS experiment design (Bosgiraud and Foin, 2013a).

Table 2.1: The schedule of activities undertaken in the FSS experiment.

Period	Activity
August 2012	Beginning of studies
November 2012 – June 2013	Mock-up (box) construction
August 2012 – July 2013	Low-pH SCC concrete and shotcrete development
August 2012 – April 2014	Development of bentonite materials and methodology to emplace the core
July 2013	Low-pH SCC containment wall construction
August 2014	Bentonite core construction
September 2014	Low-pH shotcrete containment wall construction
October 2014 – July 2015	Scientific investigations
August 2015 – December 2015	Dismantling

2.2 FSS Material Testing and Development

2.2.1 Development and Testing of the SCC Mix

Studies were carried out to develop and test the mix for the low-pH SCC, first in a laboratory, then in tests at the metre and several-metre scales. Further details of the development and testing of the low-pH SCC mix are provided in Bosgiraud and Foin (2014).

The design and selection of the concrete mix was undertaken in the following three steps:

1. ***Step 1: Identification of Binder Options:*** The first step in development and testing of the SCC mix was testing of different binder compositions in a laboratory. Eight compositions were studied (Table 2.2) and their impact on pH, compressive strength and curing temperature were measured (Table 2.3). After this investigation, four of the eight binders were selected for further testing based on their potential suitability for application in FSS. The compositions selected for further testing were:
    - B50 CEM I 52.5 Le Teil.
    - T3 (L) Le Teil.
    - B50 CEM III/A 42.5 Héming.
    - B50 CEM III/A 52.5 Rombas.
2. ***Step 2: Laboratory Testing of the SCC Mix Options:*** The four selected binders were tested in concrete mixtures in a laboratory to determine their properties. Slump flow measurements were used to test "flowability". Visual examination gave valuable information on the concrete quality, based on the distribution of aggregates, aggregate/paste separation and bleed water. Laboratory testing also included measurement of the temperature increase due to curing, the compressive strength, the porosity, and the pH of the pore solution. After this testing, it was decided to omit the T3 (L) Le Teil composition from further testing.
3. ***Step 3: Metric-scale Testing of the SCC Mix Options:*** The three remaining mixtures were then tested at the metric scale in field conditions using a ready-mix plant selected for the FSS project (Figure 2.3). For this step, the concrete mixtures were produced using industrial methods and used to fill test boxes with a volume of 1 m^3 (Figure 2.4). After 28 days, cores were extracted for analysis.

Following Step 3, a multi-criteria analysis was carried out in two additional phases to select the most suitable mix. The first phase was a technical analysis based on the measured compressive strength, pH, porosity and permeability of the samples (Table 2.4). This phase did not allow clear identification of a preferred composition, and, therefore, a second phase which included non-technical considerations was undertaken (referred to as a global multi-criteria analysis). As illustrated in Figure 2.5, five criteria were included in this second phase: pH, shrinkage (referred to as "retrait" in Figure 2.5), distance of the manufacturer from the test facility, presence of organic matter and cost (referred to as "coût" in Figure 2.5).

Based on the global analysis, B50 CEM III/A 52.5 Rombas was identified as the preferred composition for the low-pH SCC. Further testing of this mix in a test box with a volume of 12 m^3 verified the industrial feasibility of using this mix, based on a consideration of its curing and hardening.

Table 2.2: Binder compositions considered during Step 1.

Component	Binary CEM I		Ternary CEM I		Binary CEM III			
	B40	**B50**	**T1(CV)**	**T3(L)**	**B20**	**B30**	**B40**	**B50**
Cement	60%	50%	37.5%	20%	80%	70%	60%	50%
Silica Fume	40%	50%	32.5%	32.5%	20%	30%	40%	50%
Fly Ashes	-	-	30%	-	-	-	-	-
Slag	-	-	-	47.5%	-	-	-	-
Binder Silica Content	~53%	~61%	~56%	~54%	~42%	~50%	~57%	~64%

Table 2.3: Measured pH, compressive strength and curing temperature for the eight binder compositions considered in Step 1, and their rank order for each parameter - pH and compressive strength were measured after 28 days.

Binder Composition		pH		Compressive Strength		Maximum temperature increase	
		Value (slurry)	**Rank**	**Value (MPa)**	**Rank**	**(°C)**	**Rank**
Binary CEM I	B50 Le Teil	11.6	4	50.1	2	14.4	5
	B40 Le Havre	11.5	3	41.9	6	10.0	3
Ternary CEM I	T3(L) Le Teil	11.8	5	42.0	5	9.5	1
	T3(L) Le Havre	-	-	42.3	4	-	-
Binary CEM III	B40 III/A Rombas	11.9	6	51.0	1	-	-
	B40 III/A Héming	11.9	6	-	-	-	-
	B50 III/A Rombas	11.3	2	44.3	3	11.5	4
	B50 III/A Héming	11.1	1	39.0	7	9.7	2

(a) Concrete mixer-truck used to carry concrete © Paul Calin

(b) Sample for testing

Figure 2.3: Delivery of concrete for metric-scale testing of concrete mixtures (Bosgiraud and Foin, 2014).

Figure 2.4: Test box used for metric-scale testing. In this photograph, the fluidity of the concrete is shown at the outlet of the pump and evidence of self-compaction is provided by the smooth surface produced (no vibration or shocks were applied during the test) (Bosgiraud and Foin, 2014).

Table 2.4: Measured compressive strength, pH, porosity and permeability for the three mixtures tested at the metric-scale.

Metric Test Blocks Composition	Compressive Strength (MPa)			pH		Porosity	Permeability m^2
	Cylinder		Cores				
	28 d	90 d	28 d	28 d	90 d	28 d	28 d
B50 CEM III/A 52.5 Rombas	37.8	50.7	34.9	11.8	10.1	19.7%	17.10^{-18}
B50 CEM III/A 42.5 Héming	36.9	49.6	31.3	11,8	10.2	19.,9%	19.10^{-18}
B50 CEM I 52.5 Le Teil	46.7	61.9	46.2	12,2	10.3	17.7%	-

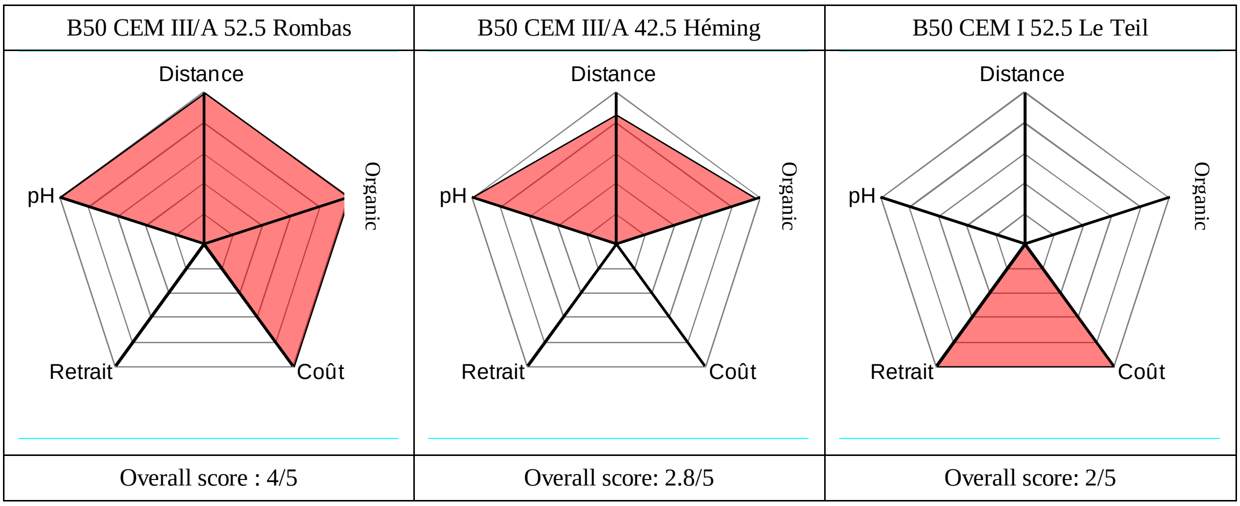


Figure 2.5: Global multi-criteria analysis to identify the preferred option for the low-pH SCC to be used in FSS. The French terms are defined in the text (Bosgiraud and Foin, 2014).

2.2.2 Development and Testing of the Shotcrete Mix

The research to identify the preferred low-pH shotcrete mix for construction of the downstream containment wall used a similar approach to the research used to identify the preferred option for the low-pH SCC. Three candidate mixes were selected for testing:

- B50 CEM III/A 52.5 Rombas.
- B50 CEM I 52.5 Le Teil.
- B50 CEM III/A 52.5 Héming.

After test spraying of metric-scale test panels (Figure 2.6), cores were extracted to measure the same properties as measured for SCC, i.e., compressive strength, pH, porosity and shrinkage (Table 2.5). In addition, the quantity of material experiencing rebound during emplacement was estimated by collection and measurement of material in front of the test panels (Table 2.5).

B50 CEM III/A 52.5 Rombas (100 % silica fume)

B50 CEM I 52.5 Le Teil (50% + silica fume + 50% nanosilica)

B50 CEM III/A 52.5 Héming (100 % silica fume)

Figure 2.6: Photographs of the outcomes from the test spraying of metric test panels.

Table 2.5: Technical results for the three shotcrete mixtures tested at the metric-scale.

Parameter		B50 CEM III/A 52.5 Rombas	B50 CEM I 52.5 Le Teil	B50 CEM III/A 52.5 Héming
Slump (mm)	t_0	200	200	200
	t_0+1h	220	200	210
	t_0+2h	180	90	180
Percentage of rebound		2.2 %	8.0 %	9.1 %
Compressive strength at 28 days (MPa)		11±3	24±3	19±3
pH after 28 days		11.3	11.4	12.3
Water porosity after 28 days		21.1%	22.3%	19.8%
Shrinkage after 28 days (µm/m)	Endogenous	19	12	19
	Total	420	280	500

As for the selection of the low-pH SCC, final selection of the low-pH shotcrete was based on a global analysis (Figure 2.7). In addition to the parameters considered in the low-pH SCC analysis, the global analysis for selection of the preferred low-pH shotcrete mix also considered the odour of the mixture resulting from sulphur present in the slag materials (“odeur” in Figure 2.7) and the compressive strength at 28 days (“résistance” in Figure 2.7).

The preferred mixture based on this global analysis was identified as being B50 CEM I 52.5 Le Teil, and this mixture was subsequently taken forward for shotcreting of the downstream containment wall.

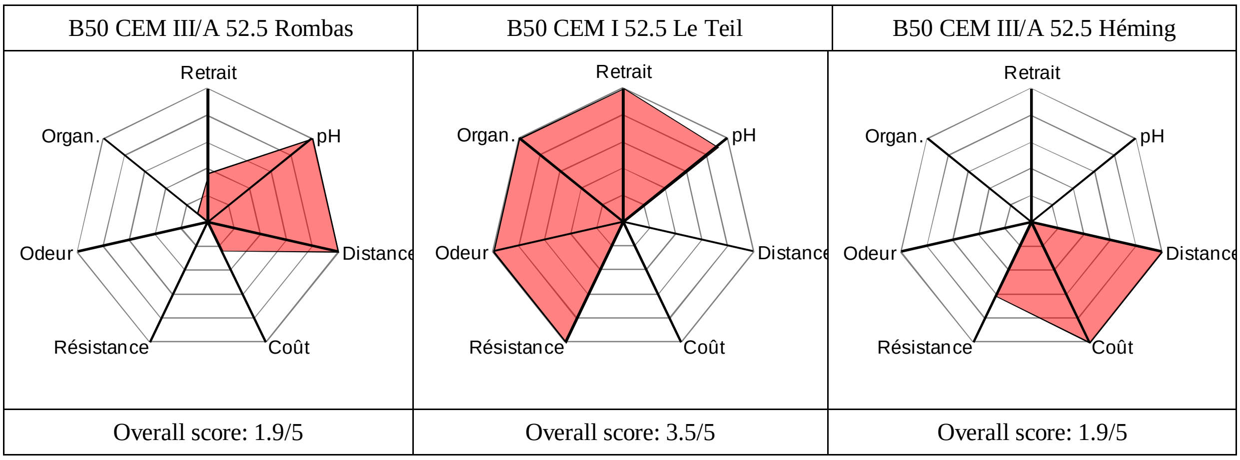


Figure 2.7: Global multi-criteria analysis to identify the preferred option for the low-pH shotcrete to be used in FSS. The French terms are defined in the text.

Further details of the development and testing of the low-pH shotcrete mix are provided in Andra (2016) and references therein.

2.2.3 Selection of Bentonite Materials

The selection of the bentonite materials for the swelling clay core was undertaken in the following steps:

- First, a decision was made to adopt a pellet-based admixture rather than use of pre-compacted bentonite blocks. This is motivated by Andra's belief that the use of pre-compacted blocks (bricks) is not a choice technically commensurate with the need for emplacing very large volumes of swelling clay in Cigéo in an industrial way.
- Second, laboratory testing of bentonite pellet and powder mixtures was undertaken in parallel with manufacturing tests to identify the appropriate pellet and powder mixture, and initial water content.
- Third, mock-up testing and desk-based design work was used to test and develop the design of the bentonite emplacement method.

Adoption of a Pellet-based System

Andra selected a pellet-based system instead of pre-compacted bentonite blocks because this solution is considered by Andra to be a more efficient industrial method of implementation for significant quantities of material. The method is similar to that proposed by Nagra for emplacement of bentonite buffer materials (Kohler *et al.*, 2015). In the case of FSS (and the large seals envisaged by Andra for the Cigéo repository), the pellets can be filled by conveyor systems while the blocks are to be positioned by human action or robots at a much lower emplacement speed. Furthermore, the erection of a wall of blocks raises the issue of its stability, since the blocks are not assembled with a mortar.

Laboratory Testing and Manufacturing Trials of Pellet and Powder System

Andra specified the use of a pure sodium bentonite, without any other constraints, for the swelling clay core. The contractor chose WH2 bentonite from Wyoming. The raw WH2 material is equivalent to the well-known MX-80, but is provided by a different supplier.

Several studies were carried out to determine the required properties of the bentonite clay core material to be used in FSS. The initial requirements on the swelling clay materials focused on the swelling pressure and hydraulic conductivity; these are 7 MPa and $1x10^{-11}$ m/s, respectively. During material testing, the dry density value of 1620 kg/m^3, corresponding to a swelling pressure of 7 MPa after hydration, was specified to ensure that the required swelling pressure could be achieved.

Laboratory studies were carried out to determine the preferred pellet size and choice of powder. The objective was to obtain the best possible dry density for the emplaced pellet-powder admixture. The tests showed that the best value for the dry density was obtained using an admixture with the following properties:

- The pellets had a cylindrical-spherical form with dimensions of: 32 mm (height) and 32 mm (diameter), and weight of around 42 g. This size was selected as a compromise between the optimum void between the pellets (to allow for a good emplacement of the powder) and the manufacturing capacity of the pellet producing machine (diameter vs compacting pressure).
- The 32 mm pellets were manufactured with a material composed of 90% WH2 powder and 10% WH2 fine particles with a grain size <160 μm. The dry density of the manufactured pellets was ≥2040 kg/m^3. The water content of bentonite was adjusted to around 4.5% (between 4% and 5%). This water content was the best compromise to obtain a good density of pellets.
- The bulk dry density of the pellets measured in the laboratory is approximately 1.10 kg/m^3, while the bulk dry density of standard WH2 powder is 1060 kg/m^3. Based on geometrical considerations, an optimum density value was observed for a 70% pellet and 30% powder mixture ratio. An addition of 30% of powder to the pellets can provide only an additional 320 kg/m^3 of dry density to the admixture. Therefore, the use of 32-mm pellets and WH2 powder does not provide the dry density of 1620 kg/m^3 required. To achieve the required density, the pellets were mixed with crushed pellets rather than with powder. The ratio of pellets to crushed pellets was 70%-30%. Laboratory measurement of this admixture confirmed that its bulk density was 1620 kg/m^3, i.e., the bulk density of the admixture in the laboratory matched the initial specification.
- The powder made with crushed pellets was controlled by sieving of the material to ensure a maximum grain size of 4 mm. This maximum was imposed to ensure that the powder could fill the voids between the pellets; powder with a grain size >4 mm could stay on the top of the heap (creating a bridging effect) preventing the powder from accessing the inter-pellets voids.

Mock-up Testing and Emplacement Machine Design

In parallel with the laboratory studies, a mock-up metric-scale setup was developed for testing the selected bentonite admixtures and the backfilling device. This device is based on two augers for emplacement of pellets and powder (Figure 2.8).

The metric emplacement tests demonstrated that the auger devices are the best adapted means capable of backfilling the drifts in an industrial way. However, the obtained bentonite density was less than the specified value of 1620 kg/m^3 which was obtained in laboratory. The best values of the obtained density in this mock-up test were 1510 kg/m^3 with the powder auger above the pellets auger and 1470 kg/m^3 with the two augers side by side. The main reason for these lower values was deemed to be the breakage of some pellets during the handling process resulting in closure of inter-pellet spaces and preventing the powder from accessing some voids. As a result, mechanical resistance tests were introduced to measure the “hardness” of the pellets (resistance to erosion or breakage due to a compacting effort as envisaged inside the screw conveyor pipe) at the production workshop and at the FSS site before emplacement.

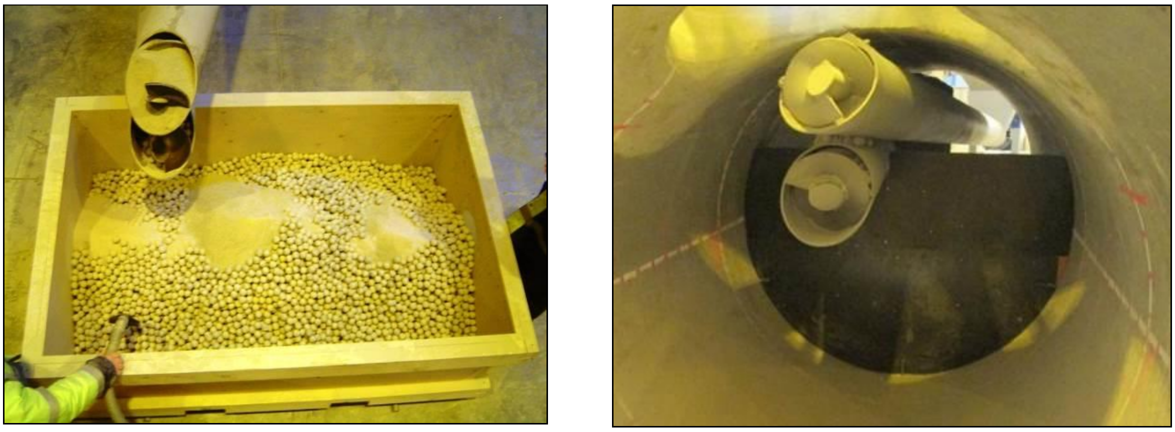


Figure 2.8: Mock-up tests at the metric scale of the bentonite pellet-powder admixture emplacement using augers.

Evaluation of the relationship between dry density and swelling pressure for WH2 undertaken in parallel with the design work described above showed that with a dry density of 1.5 kg/m^3 the swelling pressure would be around 5 MPa. This pressure was finally considered by Andra to be sufficient, and the FSS specifications were modified so that the required average dry density in the clay core would be 1500 kg/m^3 instead of 1620 kg/m^3 specified originally.

Further details of the selection of bentonite materials are provided in Andra (2016) and references therein.

2.3 Siting of FSS

The FSS experiment was carried out in a warehouse at a surface facility in Saint-Dizier, which is close to the French underground research laboratory (URL) at Bure. The choice of a surface facility was made for the following reasons (Bosgiraud and Foin, 2013a):

- Carrying out the experiment underground requires a large excavation, with a significant duration, and a considerable amount of equipment and materials to be mobilised and emplaced. The Bure URL is essentially a qualification facility, in which the logistical flexibility to undertake large experiments such as FSS is limited. Logistical limitations include, for example, the means to transport significant quantities of materials, the number of people permitted underground at any one time, and the geometry of the underground tunnels, which limit the ability to use large equipment. A surface facility does not have these logistical limitations, and, in addition, allows investigations of the experiment from the external surfaces of the “test box”.
- Several experiments were already planned to be carried out at Bure at the same time as the period in which FSS was to be conducted; this also limited the ability to use Bure for FSS.
- The experimental costs, schedule and requirements on dismantling also favoured the use of a surface facility.

The Saint-Dizier site was proposed by the Contractor in charge of the FSS experiment (GME), and accepted by Andra, for three reasons: the vicinity of Bure (30 km) where many Andra staff are located, the height of the warehouse (more than 10 m of free gap under the roof frame), and the proposed building was suitable from the perspective of controlling the air humidity and temperature, as it had a double-insulated roof and atmosphere control equipment available.

2.4 Construction of the FSS Test Box

The FSS "test box" (or drift model) design concept was specified by Andra to contain the FSS experiment. Bosgiraud and Foin (2013b) provides a detailed description of the test box construction. The test box is made of Ordinary Portland Concrete (OPC) and is shown in Figure 2.9. One of the main requirements on the test box is for it to remain stable during the various construction and filling operations. Displacements and deformations must not exceed 5 mm and must be monitored throughout the whole duration of the FSS test (Bosgiraud and Foin, 2013a). The results of preliminary geotechnical investigations on the soil of the warehouse concluded that there was a need for replacing the alluvium layer present underneath the facility floor with a substrate (a limestone aggregate) between -2 m and -4 m to reinforce the soil beneath the test box. The construction procedure of the test box is summarised below and is described in more detail in Bosgiraud and Foin (2013b).

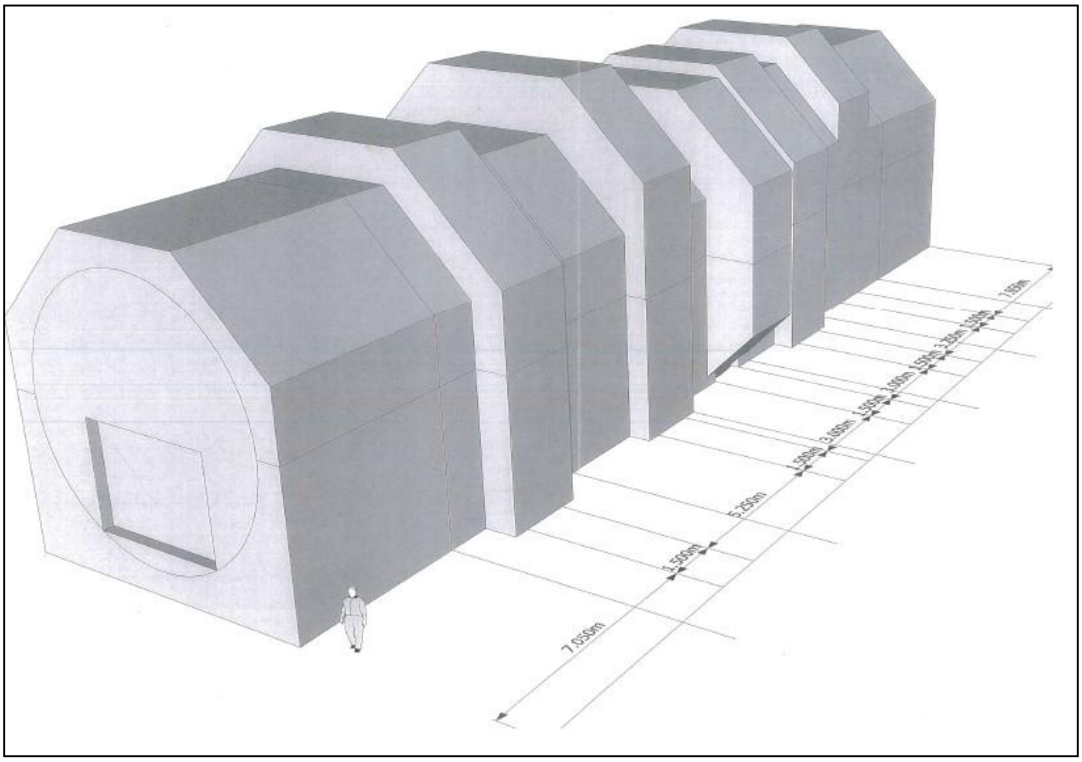


Figure 2.9: 3D schematic of the FSS test box (Bosgiraud and Foin, 2013a).

The first step in construction of the test box was to pour a concrete foundation slab on the newly created limestone aggregate platform (Figure 2.10a). Construction of the lower part of the concrete box framework commenced in December 2012 and was completed in nine weeks. The box structure was then built with seven lower blocks and seven upper blocks (each 5-m long). A wooden formwork was used to make the circular inner form, while for the outside a classic steel formwork was used (Figure 2.10b).

The concrete lining recesses were shaped by adding wood rings onto the principal inner formwork, and a special folio was pasted onto the rings to simulate the texture of the argillite walls in the breakout zones (Figure 2.10c).

After completion of the lower part of the text box, a layer of sand was laid on the ground inside the box in order to enable working on a flat floor and three steel sheets were used to slide the scaffolding equipment from a given block casting position to the next one (Figure 2.10d).

a) View of the concrete foundation

b) The inner wooden form and steel form

c) the folio pasted on the wood circular inner form to simulate the argillite walls

d) The completed lower part of the test box with a sand layer at the bottom and scaffolding also shown

e) Removal and translation of the wooden formwork between the two phases of the upper block casting

f) The completed test box

Figure 2.10: Different stages in the construction of the FSS test box (Bosgiraud and Foin, 2013b).

After that, the inner wooden framework was turned upside down and deposited on a shoring system made of a support beam and brackets so that the construction of the upper part of the box could be performed. Two phases of casting were carried out to complete the upper part of the box. Between the two phases of concrete casting, the external framework was totally removed, while the internal framework was slipped on rollers (Figure 2.10e).

It took thirteen weeks to construct the upper part of the box (Figure 2.10f), with construction completed in May 2013. A set of stairs was used to access the top of the test box. Twelve observation windows were installed to enable observation and checking of the bentonite filling operations. A local "mine-like" exhaust ventilation system was also installed, with a closing door in the front of the box, in order to control the ambient temperature (18°C < q < 30°C) and humidity (50% < HR < 75%) inside the box, as required by the experiment specification.

The test box construction was successfully commissioned as "test ready" by mid-June 2013, paving the way for the first step in the seal construction, i.e., installation of the upstream low-pH SCC containment wall.

2.5 Installation of FSS Components

Installation of FSS started with the construction of the first concrete containment wall (Section 2.5.1) followed by bentonite clay core emplacement in parallel with a support wall (Section 2.5.2). The last installed component of FSS was the second containment wall made of shotcrete (Section 2.5.3). The installation of the measurement system was undertaken in parallel with installation of the other components and is described in Section 2.5.4. The installation of the components is described in detail in Andra (2016).

2.5.1 First Containment Wall

The casting operations for the first concrete containment wall took place in July 2013. The low-pH SCC casting operations were carried out with two main objectives (Bosgiraud *et al.*, 2014):

- To realise a monolith type containment wall (with a volume of ~250 m^3) to minimise (as much as possible) the concrete shrinkage and cracking extent.

To achieve these objectives, it was decided to pour concrete in batches of 7 m^3 at a time, i.e., the maximum capacity of the truck. In order to simulate the expected logistical constraints in a repository, for the majority of the SCC containment wall a period of two hours elapsed between starting one casting operation and starting the next. This period is consistent with the expected time for transport of concrete from the surface to underground emplacement sites in the Cigéo project. However, for the upper part of the wall, one batch per hour was poured to avoid rinsing the concrete pump pipes between two mixer-trucks and to maintain a sufficiently fluid concrete mass in the upper part of the wall to improve bonding.

The fabrication of the low-pH concrete batches took place in the concrete mixer plant in Saint-Dizier (some 5 km from the FSS site); the same plant was used for the preliminary test phases. The concrete was transported by mixer trucks. Each mixer truck batch represented a layer about 15-20 cm thick in the containment wall construction progress. Each pour was completed in approximately twenty minutes followed by a pause for approximately one hour and forty minutes before commencement of the next pour. All of the mixer truck 7 m^3 batches passed two qualifying slump flow tests, one just after mixing (i.e. at plant site) and another immediately before emplacement (i.e. at FSS construction site). The slump flow target was 550-750 mm.

The temperature of the concrete significantly affected the rheology of the fresh SCC. Therefore, during emplacement of the low-pH SCC containment wall, it was decided to adjust the retarding agent with respect to the ambient temperature; the higher the ambient temperature, the higher the retarding agent dosage. The superplasticiser dosage was kept unchanged. Moreover, it was required that the maximal temperature reached in the concrete should be less than 50°C. Thus, the heat emitted during curing was measured during trials and later used to estimate the maximum temperature at which the concrete could be poured to meet the requirement. It was found that the maximum temperature increase during curing and hardening was 24°C. In practice, it was then decided not to pour concrete with an ambient temperature greater than 26°C. Those ambient conditions were effectively respected at the time of casting operations.

The casting operations were judged to be successful as the low-pH concrete rose progressively inside the box, with smooth and regular emplacement (Figure 2.11). To finish the operation, it was necessary to totally close the formwork before the concrete was injected in the upper part of the containment wall (Figure 2.11).

Figure 2.11: Pouring and even emplacement of SCC inside the test box (left) and emplacement of SCC close to the recess (right) (Bosgiraud and Foin, 2014).

After casting, the SCC containment wall was left for preliminary curing for about one week, and then the formwork was stripped from the concrete with no particular difficulties. Some 28 days of hardening later, the injection of low-pH slurry (grout) commenced in order to bond the containment wall to the test box concrete liner. The grout was emplaced through small pipes installed at the higher part of the cast concrete (two for grouting and two others to serve as air evacuation). The mix for the grout is provided in Table 2.6 and the grouting machine is shown in Figure 2.12. The quantity of injected slurry turned out to be very small (a few tens litres). It was thus inferred that, in FSS, there was only a small gap generated by concrete shrinkage and/or that the bonding had already taken place. This issue was investigated further during the dismantling of FSS, which is reported in D4.4 (DOPAS 2016b).

Table 2.6: The mix for the low-pH grout used in FSS.

Component	Quantity (kg/m^3)
CEM III/A 52.5 Rombas	311.7
Silica fume	311.7
Very fine sand ("Sablon")	781.7
Water	438.0
Glenium ®Sky 537	1.9% of (C + SF) : 11.4 L
Rhéomac ®SRA 872	2.0% of (C + SF) : 12.5 L

Figure 2.12: Grouting machine used in the FSS experiment.

2.5.2 Bentonite Clay Core and Support Wall

The swelling clay core backfilling activities were commenced in October 2013 (after completion of the SCC containment wall).

Before the start of bentonite emplacement, all material was delivered and stored at the warehouse, including 847 tonnes of 32-mm diameter pellets (in 770 octabins) and 368.5 tonnes of crushed pellets powder (in 335 big bags). A specially developed "backfilling" machine for the purpose of bentonite emplacement was constructed (Figure 2.13). The machine comprises a carriage that moves in the *y*-direction into the test box on two rails overlaid by a movable turret in the *x*-direction. This turret incorporates all components used for conveying of materials: a hopper for pellets, a hopper for crushed pellets, and a crane used to move the augers in the *z*-direction as bentonite emplacement progresses.

Figure 2.13: The bentonite emplacement machine.

The filling procedure commenced with emptying of the octabins (containing bentonite pellets) and the big bags (containing bentonite powder) into separate movable hoppers (Figure 2.14). The movable hoppers were then lifted by forklift trucks and emptied into fixed hoppers positioned on the filling machine (Figure 2.15). Filling of the lower two thirds of the clay core volume was completed with the two augers (one for the pellets and one for the powder) one above the other to reproduce the laboratory experiments in which the best results were obtained (Figure 2.16). Filling of the upper third was completed with the augers side-by-side to enable the filling of the recesses (reproducing the metric tests).

The process of transferring bentonite powder into hoppers and then to the forklift trucks followed by placement in the concrete test box generated high-levels of dust. Examination of the ventilation system after the first day of use identified non-ideal performance of the mine-type filter, which was then cleaned, resulting in lower levels of dust in the warehouse facility.

Figure 2.14: Emptying of an octabin containing bentonite pellets (left) and a big bag with bentonite powder (right) into hoppers.

Figure 2.15: Transferring the bentonite into the filling machine using forklift trucks (left) and then transfer of pellets through a conveyor belt into the auger (right).

Figure 2.16: Bentonite materials transferred through the augers (left) into the core of the seal (right).

Emplacement of the bentonite materials was facilitated by construction of supporting walls. At first, a supporting wall was partially constructed (at mid-height) to contain the first part of the bentonite core. The supporting wall was then progressively constructed concurrently with the bentonite filling operations. This wall was built with half-cubic-meter low-pH concrete blocks made with the same SCC as that used for the first containment wall. The swelling clay core emplacement was finished in August 2014.

During the filling operations, two problems were encountered and mitigated:

- Too many pellets were found to be broken during the emplacement process, which would have a significant impact on the emplaced density. The augers were inspected and excessive wear of the transport screw was discovered. Subsequently, the screw was replaced, resulting in far less breakages.

- The gear motor of the bentonite powder auger also required replacement.

2.5.3 Second Containment Wall

The second low-pH shotcrete containment wall was constructed after emplacement of a mortar to close the joints in the supporting wall that would separate the swelling clay core from the shotcrete wall. The mortar was emplaced between blocks and at the contact with the test box wall. This containment wall was built by spraying shotcrete in wet conditions in order to test an alternative to the use of low-pH SCC. The low-pH shotcrete containment wall (Figure 2.17) was finished in September 2014 after solving initial challenges with fabrication of the shotcrete:

- The rheology of shotcrete was initially found to be highly variable; some batches of shotcrete were found to be too stiff and some too soft. This difficulty was solved by increasing the time of mixing from 30 seconds to two minutes so as to have a better incorporation of silicate fume. After making three batches (with a volume of 3 m^3), a check was then carried out to adjust the amount of additives for the last four batches to get the expected final consistency.

- The rebound was around 10 to 12% and the difficulty was to thoroughly remove it after each mixer-truck emplacement to prevent its incorporation into the wall.

Figure 2.17: Installing the second containment wall made of shotcrete in FSS.

2.5.4 System for Measuring FSS Components Parameters

The main environmental condition parameters measured in the FSS experiment to ensure that they are similar to those expected in Cigéo included, ambient temperature (measured continuously), humidity (measured continuously), and dust in the air (measured randomly as done in a mine). For the concrete and shotcrete containment walls, continuous measurements of the curing temperature and shrinkage were implemented. For the bentonite core, the quality of the material used at each step of fabrication and the quality of the filling and mass balance of the admixture were checked. 3D scans were performed for evaluation of the bentonite backfilled volumes at pre-defined phases. The emplaced bentonite dry density was also evaluated. More information on the measurement system is available in Andra (Report D4.8, Bosgiraud et al. 2016). It is important to mention that the monitoring of FSS was dedicated only for the test run-time of the experiment until its dismantling.

2.6 Lessons Learned from the Design and Installation of FSS

Prior to the commissioning of the FSS experiment, some of the lessons learned concern the need to allow enough time to design the experiment, especially with no previous experience of full-scale tests of plugs and seals. All the experiment specifications need to be individually checked and cross checked to ensure consistency (for example, the temperature and pH values for concrete, type of materials for the bentonite and their density, etc.).

An important lesson for the material specifications is the selection of realistic and achievable targets. This was the case with the bentonite material dry density specification. Initially a high value of the dry density was specified even though a more realistic and lower target was found to be sufficient and more easily achievable. Where feasible, a range of values for material specifications should be provided rather than absolute values as this will facilitate quality control during installation. It furthermore provides “robustness” to the concept.

Besides, for the WH2 admixture, the original target swelling pressure of 7 MPa turned out to be somehow a real “overkill” and was reduced to 5 MPa since this value is compatible with the seal permeability target and the argillites EDZ self-sealing.

For instance, the requirement for the concrete temperature (50°C) was not a requirement given in D2.4, but a decision made by Andra for the input for design and work specifications given to the contractor. The explanation about this is that Andra thought it right to have a safety margin (i.e. 10°C less than the usual 60°C temperature requirement commonly found for curing of concretes). On the other side, for the low-pH SCC, it was found that the temperature at the experiment site impacts on concrete, therefore, this will need to be appropriately managed underground. For the shotcrete, it was concluded that new metric or plurimetric tests would need to be implemented in order to have a better know how of the shotcrete emplacement methods.

Selection of both the SCC and shotcrete mixes used both “technical” (e.g., pH, shrinkage, compressive strength and presence of organic matter) and “non-technical” considerations (e.g. distance from manufacturer and cost). Overall, the shotcrete mixes tested had a lower strength than the SCC mixes.

Similarly, the concrete pH value requirements were changed from the original pH 10.5-11 @28 days to pH 10.5-11 @90 days. The justification is that the original requirement was some type of “overkill”, since pH value keeps diminishing with time for these types of concrete.

Development of the bentonite admixture recognised the need to combine pellets with crushed pellets rather than the standard WH2 powder (the crushed pellets are referred to as "powder") to achieve higher densities, and the need to arrange augers vertically rather than horizontally to achieve the best density. However, emplacing the bentonite near the tunnel ceiling requires the augers to be arranged horizontally.

During experiment installation, one of the main challenges was dealing with the vast amount of dust generated by handling bentonite in the warehouse. The use of a scraper conveyor to transfer material into the filling machine instead of forklift trucks would reduce the dust generated during unloading of bentonitic components (powder in particular) into the forklift truck hoppers and also reduce the transfer time of material. A special hooded vacuum tube with a filter to remove dust could be incorporated into the design of the bentonite emplacement machine.

During construction of the experiment, it was emphasised that the safety and health of staff is paramount, especially with material which can potentially be hazardous (e.g., silica fume, bentonite dust, etc.). Supervisors need to be mobilised all the time during the full-scale test in order to verify that the procedures and regulations are always adhered to.

3. EPSP Experiment

This chapter provides a summary of the learning from design and construction of the EPSP experiment.

- In Section 3.1, the background to the experiment and its objectives are summarised.
- In Section 3.2, the testing and selection of materials prior to EPSP implementation are described.
- In Sections 3.3, the structural design work undertaken as part of the DOPAS project is summarised.
- In Section 3.4, the siting of the EPSP experiment is explained.
- In Section 3.5, the construction work undertaken in the EPSP niche is described.
- In Section 3.6, the installation of the EPSP components is summarised, with particular focus on the novel aspects of the experiment.
- Discussion of the lessons learned regarding the design and installation of the EPSP experiment is provided in Section 3.7.

Further details of the design and construction of EPSP are available in the EPSP experiment Summary Report (SÚRAO and CTU, 2016).

3.1 EPSP Experiment Background and Objective

The EPSP experiment is the first time that SÚRAO has carried out any detailed work on repository plugs and seals. The key objectives of the experiment are to test materials and technology, extending laboratory experience to the underground environment and to full-scale tests, and to build the practical expertise of the SÚRAO personnel and other partners. Implementation of the reference design itself is not being tested. At this early stage in the Czech geological disposal programme, about 50 years prior to the scheduled commencement of operation, it is considered more important to build knowledge and experience rather than to refine implementation designs for an, as yet, unidentified site with unknown mechanical, hydrogeological and chemical characteristics (SÚRAO, 2012). However, EPSP will also provide an important test-bed in developing a final plug design and procedure for implementation, will contribute towards the development of a reference design for tunnel plugs, will give indications on crystalline host rock requirements and may support the site selection programme (Svoboda *et al.*, 2015).

The conceptual design for EPSP includes the following components (see Figure 3.1):

- Pressure Chamber: The pressure chamber (or injection chamber) is an open area that can be used to pressurise the inner concrete plug. The chamber contains an inlet valve and a drain valve that can be used to fill the chamber with air (gas), water or bentonite slurry. The chamber was built to be as small as possible to allow the pressure to be readily controlled. The pressure chamber was sealed with a membrane.
- Concrete Walls: Concrete separation walls (or blocks) were used to facilitate construction of EPSP. Three concrete walls were built: one between the pressure chamber and the inner concrete plug, one between the bentonite and the filter, and one between the filter and the outer concrete plug.

- Inner Concrete Plug: The inner concrete plug is one of the sealing components in EPSP and was constructed using glass-fibre-reinforced low-pH shotcrete. The mix and pH values were determined during the laboratory testing stage.
- Bentonite Pellets: The bentonite pellet zone is comprised of B75 bentonite (a locally extracted material), a natural and high-smectite-content Ca-Mg bentonite with notably high iron content in the octahedral layer of the smectite. The purpose of the bentonite is to seal and absorb/adsorb any water that leaks across the inner concrete plug. The bentonite zone is 2-m long.
- Filter: The filter collects any water that is not absorbed by the bentonite. This is most likely to occur if the leakage rate across the inner concrete plug is sufficient for piping and erosion of the bentonite to occur. The filter may also be used to reverse the direction of pressurisation of EPSP.
- Outer Concrete Plug: The outer concrete plug is similar to the inner plug (i.e., made using glass-fibre-reinforced low-pH shotcrete) and was designed to hold the other components of EPSP in place. However, should the direction of pressurisation of EPSP be reversed, the outer concrete plug will have to perform as well as the inner concrete plug, and, therefore, the requirements on the outer concrete plug are the same as the requirements on the inner concrete plug.

The schedule for the EPSP experiment is summarised in Table 3.1, and further information on the EPSP experiment conceptual design and design basis is presented in DOPAS (2016a).

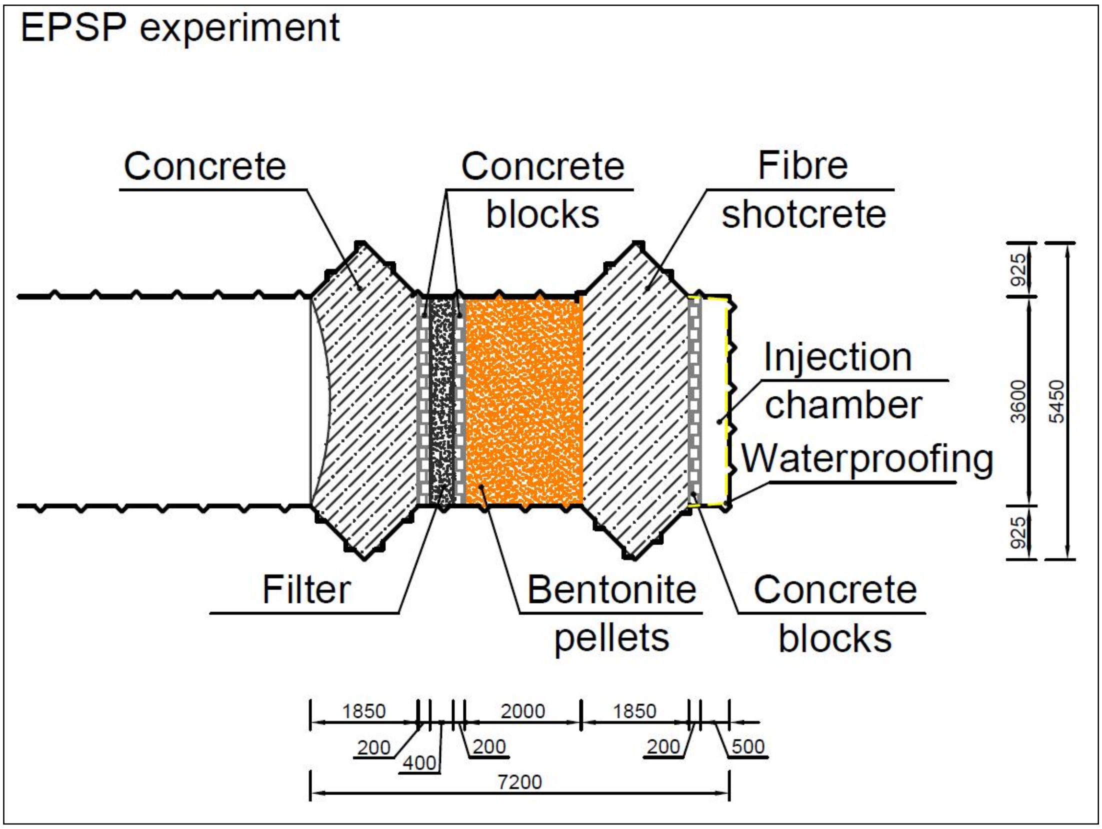


Figure 3.1: Schematic illustration of the EPSP experiment design (Figure provided by CTU). Dimensions are in mm.

Table 3.1: The schedule of activities undertaken in the EPSP experiment.

	Period	Activity
September 2012 – December 2012		Niche selection
January 2013 – April 2013		Site preparation
October 2012 and February 2013		Geological mapping
May 2013 – September 2013		Tendering for Phase 1 work
Phase 1 work	November 2013 – July 2014	Drift shape adjustments
	October 2013 and December 2013 – September 2014	Rock improvement (grouting)
	November 2013 – September 2014	Connecting boreholes drilling, casting, grouting
	June 2014 – August 2014	Rock bolting
	January 2015 – May 2015	Contact grouting – inner plug
July 2015 – August 2015		Contact grouting – outer plug
January 2014 – October 2014		Tendering for Phase 2 work
Phase 2 work	October 2014	Pressurisation chamber adjustment
	November 2014	Separation wall installation
	November 2014	Inner plug erection
	December 2014 – May 2015	Inner plug tests
	June 2015	Bentonite emplacement
	June 2015	Outer plug erection
	February 2015 – July 2015	Technology installation and tests
January 2013 – July 2015		Monitoring preparation and installation

3.2 EPSP Material Testing and Development

In 2013-2015, a series of laboratory tests focused on the sealing properties of the selected bentonite, and development activities and tests on the proposed concrete mix and bentonite were undertaken. The sections below describe the work and results obtained from laboratory testing. More details on results of laboratory testing can be found in Vašíček *et al*. (2014).

3.2.1 Selection and Testing of the Concrete Mix

At the outset of the EPSP experiment, it was decided to use glass-fibre-reinforced low-pH shotcrete for the Inner and Outer Concrete Plugs. The decision was based on previous experience with iron-fibre shotcrete from the Háje gas storage pressure plugs (Hilar and Pruška, 2011). For EPSP, glass fibres were selected as reinforcement instead of iron-based fibres to avoid the potential for corrosion of the iron-based fibres to affect the post-closure performance of plugs in the Czech repository, and also to avoid the introduction of additional iron into the system. These fibres also significantly help to reduce (micro) cracking from shrinkage. Moreover, low-pH concrete is required to limit possible impacts on bentonite.

ÚJV's previous experience with the preparation of low-pH concrete mixes and the experience of a commercial producer of concrete mixtures for building purposes were used in developing initial proposals for the shotcrete mix. The project required that the concrete mixture should be "low-pH concrete", which, in the Czech Republic, is generally interpreted to mean that the pH value of pore water (leachate) in equilibrium with the concrete is ~11.5 or less.

Low-pH concrete mixtures with a pH of <11 had been developed by ÚJV prior to the DOPAS Project. Two types of cements were used in these concrete mixtures:

- CEM III/B 32.5 N-SV – Heidelberg.
- CEM II A-S 42.5R – Lafarge.

The concretes were produced by mixing the cement with silica fume (Addiment Silicoll P), fine milled limestone (D8 – Lhoist) and plasticizer (Addiment FM 935). Although the mixtures developed by ÚJV exhibit low-pH leachate, they also have a low compressive strength and therefore are not directly suitable for practical use in construction projects.

A reduction in the pH of the leachate of the concrete mixture can be obtained by the partial replacement of the cement by another type of binder; it can also be achieved by increasing the ratio of fine SiO_2 (micro silica/silica fume) to the cement content or by the partial replacement of cement by metakaolin. The addition of metakaolin decreases leachate pH values from pH~13 to pH~12 after 5 weeks of hardening. However, the addition of metakaolin in the cement mixture causes a decrease in strength. For SiO_2, the decrease of pH values is greater, i.e., a pH of approximately 12 is achieved after one week of curing and a pH~11.5 is achieved at later times (Vašíček *et al*., 2014).

Following the initial studies based on existing concrete mixes described above, ÚJV worked with the supplier to develop a concrete mix suitable for testing in EPSP. Two mixes were tested and their suitability assessed based on a consideration of pH, compressive strength and rheology, amongst other parameters (Table 3.2). The two mixes were also subject to mock-up tests in a testing niche in the Josef URC and underground laboratory.

Selection of the preferred concrete mix was ultimately determined by the chemical performance, as one of the concrete mixtures exceeded the pH target (pH = 12.0-12.2), whereas the other met the target (pH = 11.3-11.5) and all other requirements.

Table 3.2: Measured strength and pH for the two mixes considered for the EPSP shotcrete plugs.

Parameter	Mix 1	Mix 2
Compressive Strength (mixture)	59.2 MPa	51.4 MPa
Compressive Strength (core drills)	44.4 MPa	46.5 MPa
Flexural Strengths	5.8 MPa	6.7 MPa
pH - filtrate	11.3	12.1

The materials that were used in the EPSP concrete mix were:

- Cement: CEM II / B – M (S-LL) 42.5 N.
- Sand and Gravel: 0-4 & 4-8 Dobřín.
- Plasticiser: SIKA 1035CZ.
- Retardant: SIKA VZ1.
- Accelerator: SIKA Sigunit L93 AF.
- Microsilica: SIKA FUME.
- Glass fibres: crack HP (Sklocement Beneš).

The ratio of microsilica to cement was approximately 1:1.

3.2.2 Selection and Testing of Bentonite Materials

One of the main aims of EPSP is to demonstrate the suitability of Czech materials and available technologies for construction of tunnel plugs. In order to form a basis for identification and selection of candidate bentonite materials, plug construction requirements (e.g., hydraulic conductivity) were considered alongside a more general set of requirements:

- The bentonite material had to be sourced from the Czech Republic.
- The bentonite material had to be non-activated. This follows on from the first requirement that the bentonite material had to be sourced from the Czech Republic. There are no sodium bentonite deposits in the Czech Republic and artificial activation increases costs with no long-term guarantee that the activated materials will not revert to their non-activated state. Therefore, the focus is on non-activated materials.
- The bentonite material had to be capable of fulfilling sealing requirements (see White *et al.*, 2014).
- The bentonite material used in the experiment had to be homogenous from a chemical and mineralogical point of view.
- The bentonite material had to be available in sufficient quantity.
- The bentonite material had to be available for use during the timeframe of the EPSP experiment.

Following careful consideration of plug construction requirements, factory-produced bentonite (milled, non-activated Ca-Mg bentonite) was selected as the principal material for the bentonite part of the plug. The commercial product “Bentonit 75” (B75) was the only material available at that moment fulfilling all the requirements. The bentonite B75 is produced by Keramost Plc from the Černý vrch deposit.

B75 bentonite is produced in powder form which is not ideal for sealing plug purposes owing to the low density of the raw material. Therefore, the first stage involved the selection of the best compaction technology commercially available in the Czech Republic. Eventually, three technological processes were selected for further consideration.

The first method originated from a factory which produces compacted kaolin clay pellets (cylinders with a diameter of 12mm) by means of a roller compaction machine. A number of tests were conducted concerning the manufacture of the bentonite pellets, the main aim of which was to determine the conditions for the industrial compaction and production of the bentonite pellets with the most suitable dry density value. The final dry density of the compacted pellets depends on the water content of the material; B75_2013 bentonite had to be moistened prior to compaction. Subsequently, material with a water content of around 16% by mass was selected for further use. The pellets selected from this producer, code-named B75 PEL_12, have a diameter of 12 mm, a length of up to 4cm and a dry density value of 1.80-1.85g/cm^{-3}. Pellets produced by this technology were used for the major part of the sealing section.

The second compaction method was based on a small roller compaction machine which produced bentonite pellets with a diameter of 8mm. This method, however, was not selected and further developed due to the low level of bentonite compaction and the amount of time required for production.

The third method considered (employing a roller mill) was the result of consultation with a Czech bentonite production company. The pellets (fragments of highly-compacted bentonite plate) produced employing this procedure are not available commercially but the machinery involved is in common use. Laboratory testing revealed a good level of compaction (dry density 1.70-1.98 g/cm^{-3}) with a relatively low water content value. The advantage of this technology is the production of pellet fragments with various sizes. It allows mixing in various proportions in order to achieve the best dry density value following emplacement. The resulting material was codenamed B75_REC. This material was used for shot clay technology.

Selection of B75 was supported by experience from previous research (Trpkošová *et al.*, 2013), where B75 was found to fully comply with the required hydraulic conductivity ($\leq 1 \times 10^{-12}$ m/s) and swelling pressure (≥ 2 MPa) at a dry density of 1.4 kg/m^3. This research was based on material delivered by the producer in 2010. As bentonite deposits are heterogeneous, the B75 used for EPSP, which was delivered in 2013 (and is referenced as B75_2013), was subject to laboratory testing to determine its mineralogical and chemical composition, and to confirm its properties against the requirements set out in White *et al.* (2014).

The mineralogical and chemical composition of B75_2013 is presented in Figure 3.2 and Table 3.3 respectively. Various laboratory tests were performed on B75_2013 material to verify its properties by CTU and ÚJV.

CTU undertook laboratory tests to determine the specific density and Atterberg limits of the bentonite powder. This was followed by the determination of the relationship between the dry density of compacted samples, and hydraulic conductivity and swelling pressure. B75 is produced in powder form which is not ideal for sealing plug purposes due to the low level of compaction. Therefore, the testing of the most appropriate technology for the manufacture of the pellets, in cooperation with potential Czech producers, was also carried out by CTU. The main conclusion from this work was that B75_2013 bentonite demonstrated sufficient dry density levels and, therefore, can be used to achieve the required geotechnical behaviour of the bentonite seal in the EPSP experiment (Vašíček *et al*., 2013; Vašíček *et al*., 2014).

Laboratory testing of B75_2013 bentonite by ÚJV focused on the chemical composition, the measurement of pH in suspensions of bentonite and distilled water at different ratios, and the analysis of leachates (cation concentrations). It was concluded that the main characteristics of B75_2013 bentonite remained constant and fulfilled all the expectations, limits and requirements for the construction of the experimental plug (Vašíček *et al*., 2014).

3.2.3 Selection of Filter Materials

The filter functions as a permeable layer for collection of any water passing through the sealing part of the experiment. It has no other function. Inert gravel was used in EPSP, with the exact type and grain size determined by the supplier.

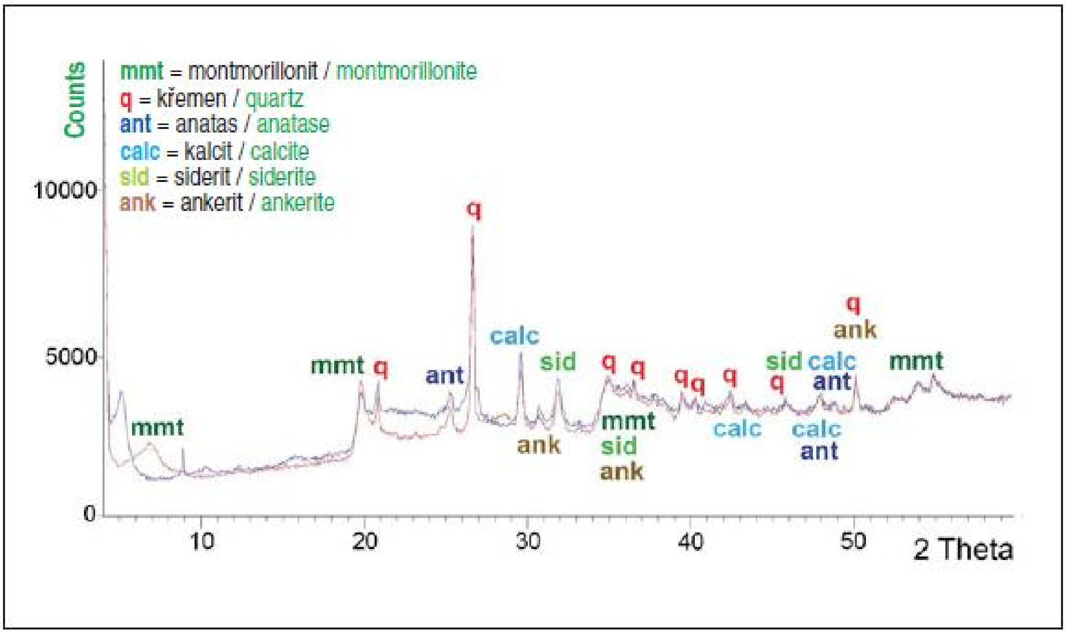


Figure 3.2: X-ray diffraction pattern for B75_2013. Red spectrum shows the pattern for untreated (i.e., no glycolation or other treatment was applied) B75_2013 and the blue spectrum shows the pattern after glycolation. Both samples (red spectrum and blue spectrum) were air-dried at laboratory temperature 20°C.

Table 3.3: Chemical composition of B75_2013.

Oxide	Weight%
SiO_2	49.83
Al_2O_3	15.35
TiO_2	2.82
Fe_2O_3	10.90
FeO	3.74
MnO	0.09
MgO	2.88
CaO	2.01
Na_2O	0.67
K_2O	1.05
P_2O_5	0.63
CO_2	3.66

3.2.4 Selection of Rock Grout

The niche selected for the location of the EPSP experiment is traversed by quartz and quartz-carbonate veins with a maximum thickness of 14 cm. The ground conditions had the potential to impact the performance of the EPSP experiment in several ways:

- The low rock strength meant that pressurisation of the plug components could lead to reactivation of the rock fractures and failure of the surrounding rock mass.
- The fracture network could lead to excessive water leakage from experiment.
- The location of the Josef URC and underground laboratory is close to a water reservoir. Any grouting material used to improve the ground conditions required a certificate to confirm that its use would have only an insignificant impact on groundwater quality.

Therefore, before the installation of EPSP, the surrounding rock had to be grouted to improve the rock strength and to reduce the permeability of the rock mass. As part of the development and selection of grouting materials, tests were undertaken to ensure that any interactions between the low-pH leachate released from the concrete plugs and the rock mass would not significantly impair the performance of the grouted rock mass.

Laboratory tests were carried to test commonly used grouting materials (based on polyurethane). These materials were selected based on their common usage and suitability for rock grouting in the geological conditions of the Josef URC and underground laboratory.

The chemical composition and stability, possible interaction, physical properties and applicability of the grouting material were verified. The extent of the interaction of the grouting with the cement and the bentonite leachates serves to confirm the stability or otherwise of the grouting. Initial tests indicated that common polyurethane based grouting materials were not influenced by low-pH solutions and should not be affected by cement leachates in the grouting of the experimental plug; no organic components were found to have leached into the alkaline solutions. In a further laboratory test, it was proved that the polyurethane-based grouting material has a sufficiently low hydraulic conductivity up to a water pressure of 2.5 MPa to be suitable for use in EPSP.

3.3 Development of the EPSP Structural Design

The design work undertaken for the EPSP experiment included the use of physical model tests in the laboratory (Section 3.3.1), and analytical and numerical calculations to underpin the design (Section 3.3.2). More details on physical model tests can be found in Vašíček *et al*. (2014) and more information on the analytical and numerical calculations can be found in Svoboda *et al.* (2015).

3.3.1 Physical Model Tests

Since the simulation of unsaturated swelling materials is complex and the EPSP underground laboratory experiment will not be dismantled during the course of the DOPAS Project, two series of physical model tests were conducted by ÚJV at the laboratory scale on the candidate EPSP materials to support the design of EPSP.

The objectives of the Physical Hydraulic Model (PHM) tests were to investigate the hydraulic and mechanical processes during saturation of bentonite and to derive data for the subsequent calibration of numerical models of the bentonite material saturation. Two PHM tests were conducted; one with bentonite powder and the other with bentonite pellets (Figure 3.3), in which the samples were gradually saturated with synthetic granitic water under pressure. The data were used to develop water retention for the bentonite. The two water retention curves obtained through the two tests were comparable and were applied in numerical modelling of the plug performance.

The objectives of the Physical Interaction Model (PIM) were to study the interactions between the bentonite and grouted granite, and between the concrete and grouted granite interfaces. PIM incorporated all of the materials expected to be used in EPSP, including bentonite, concrete and polyurethane materials. In the PIM apparatus (Figure 3.4), two blocks of low-pH concrete surround a block of compacted bentonite. One side of the PIM apparatus is connected to a source of synthetic granitic water under 2 MPa pressure to simulate the conditions expected in EPSP. Saturation of the middle bentonite part is monitored by a relative humidity sensor and water passed through PIM is collected for chemical analysis. The dismantling of PIM was planned for the end of 2015 and the results of this dismantling were therefore not available for inclusion in this report. The dismantling will include investigation of changes in the chemical, physical and mineralogical properties of the bentonite and the low-pH concrete.

3.3.2 Numerical Modelling and Site-specific Design Issues

Numerical modelling was undertaken to evaluate the structural performance of the proposed plug design. The aim of the analyses was to verify the design of the concrete plugs, their stability, deformation and the changes of stress inside the surrounding rock. A basic model was studied where both plugs including the surrounding rock and the pressurisation chamber were included. A detailed model of one concrete plug in contact with the rock was also developed.

The numerical models simulated the construction of the experiment and the subsequent loads experienced by each component. The models did not incorporate stress changes inside the rock owing to the excavation of the niche, as the niche was excavated between 1981 and 1991, and it was assumed that any deformation arising from this excavation had already taken place. Groundwater ingress was not considered in the models because the surrounding rock was sealed using grouting within 5 m of original excavation and because the water load was simulated by including the overpressure in the injection chamber.

The calculation was done according to Czech standard ČSN EN 1997-1 (Eurocode 7) – Design Approach 2, using a finite element package dedicated to the deformation and stability analysis of underground works and geotechnical structures (CESAR-LCPC). The models evaluated the structural response to loads generated by self-weight (compaction), shrinkage, pressure in the chamber and swelling pressure, and the combination of these forces, and accounted for the impact of fractures on the spatial variability of rock strength.

Once the models and their various load states had been calculated, the maximum stress state in the concrete plugs was compared with the strength of the glass-fibre-reinforced low-pH shotcrete as determined by the laboratory tests described earlier (Section 4.2). The results of the structural analysis showed that the selected design of the plug and its materials should withstand all experimental loads without difficulty (see further discussion in Svoboda *et al.*, 2015).

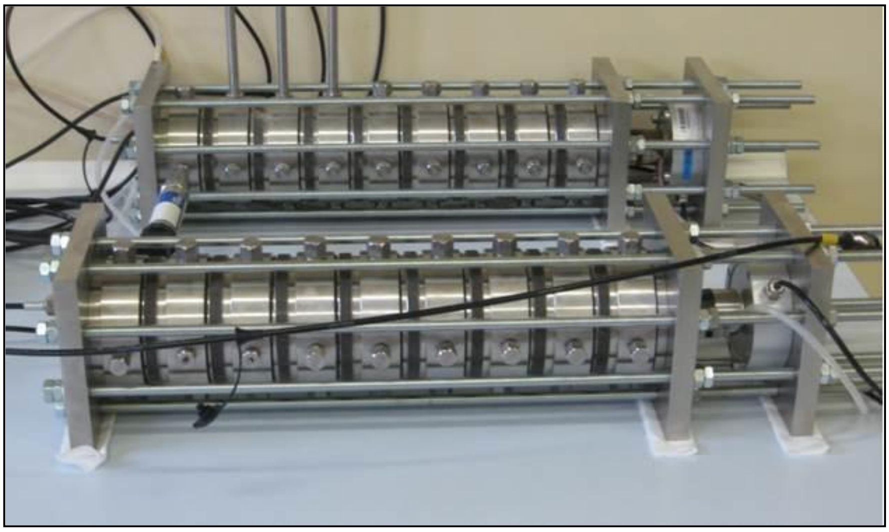


Figure 3.3: The two PHM tests used to investigate the hydraulic and mechanical processes during saturation of bentonite.

Figure 3.4: Photograph of the PIM test.

3.4 Siting of EPSP

The Josef URC and underground laboratory is located near the Slapy dam close to the villages of Čelina and Mokrsko in the Příbram district of Central Bohemia, Czech Republic. The total length of the tunnels in the facility is approximately 8 km and the length of the main drift is 1,835 m, with a cross-section of 14–16 m^2. The overlying rock thickness is 90-180 m. Two parallel tunnels lead from the entrance portals, each having a length of 80 m and a cross-section of 40 m^2.

There are two main geological formations present in the Josef URC, each with different physical and material properties which change in character towards the contact zone and which include many local fracture zones and several intrusions. This provides a high level of flexibility with regard to choosing the appropriate place for conducting experiments depending on the conditions required, for example, fracture systems, rock stability, rock strength and mineralogy (see Svoboda *et al*. (2015) for more details on the siting of EPSP).

The EPSP experiment is located in a short gallery situated in the granitic area of the Josef URC (the M-SCH-Z/SP-59 experimental gallery niche, Figure 3.5 and Figure 3.6). The necessary technology for the experiment is located in the parallel niche M-SCH-Z/SP-55 (Figure 3.5). The niches are interconnected by cased boreholes equipped with tubing for pressurisation media circulation and for monitoring.

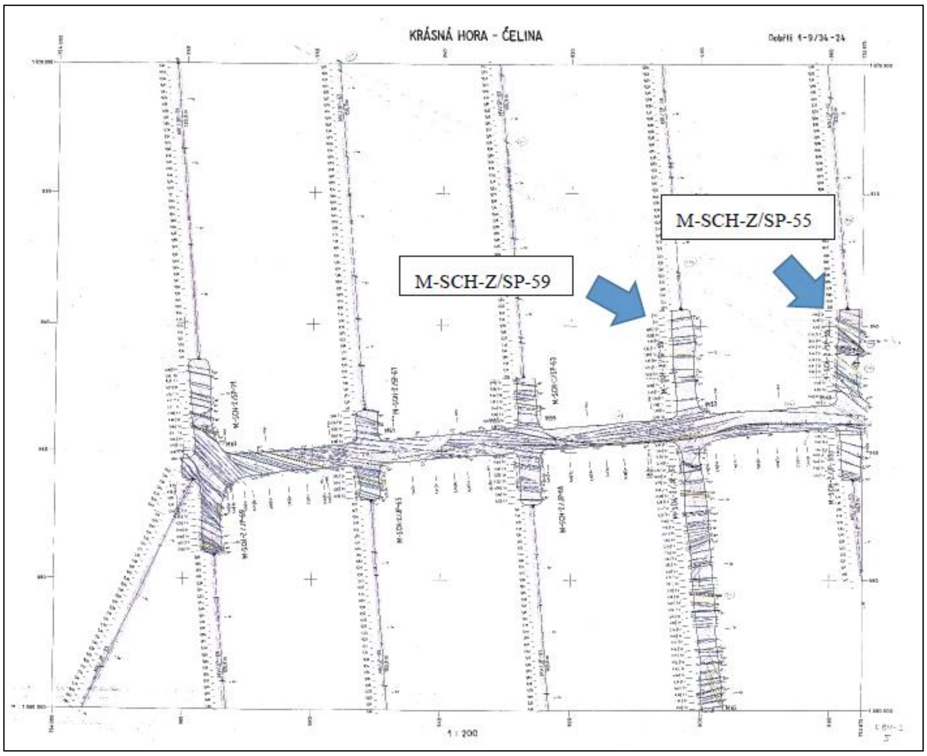


Figure 3.5: Location of the EPSP experiment in the Josef URC and underground laboratory.

Figure 3.6: Photograph of the EPSP experiment niche.

These niches were selected based on the following considerations:

- The ground conditions in the niches was considered appropriate for construction of EPSP, in particular, the granitic rock does not contain any major fracture zones.
- The operation of the EPSP required two free adjacent niches to be present, and this available space was provided by the M-SCH-Z/SP-59 and M-SCH-Z/SP-55 niches.
- The size of the M-SCH-Z/SP-59 experimental niche profile and length) was sufficient for construction of EPSP without significant additional excavations (other than shaping of the rock mass).
- The location of M-SCH-Z/SP-59 meant that there would be no significant impacts on other ongoing experiments in the Josef URC and underground laboratory.

3.5 Excavation of the EPSP Tunnel: Reshaping and Ground Improvement

As noted above, EPSP was conducted in an existing niche that was excavated between 1981 and 1991 (Figure 3.6). Prior to the commencement of the construction of the EPSP experiment, it was necessary to reshape the experimental gallery niche and improve the ground conditions.

The reshaping and ground improvement activities started in October 2013 with 3D scanning of the existing niche profile. Based on the results of the scanning, the precise location of the EPSP experiment was determined. The location selected was the one that would minimise the need for additional ground works.

Once the position of EPSP was fixed, the excavation works started with excavation of the slots in which the shotcrete plugs would be emplaced. The excavation was carried out gradually, in two stages. First, rough excavation of rock was undertaken in the upper half of both plugs, working from a platform built in the existing experimental niche. Subsequently, the platform was disassembled and the excavation of the lower half of both plugs was undertaken. Following the rough works, the rock surfaces were smoothed through diamond sawing and chiselling.

Selection of the rough excavation method was constrained by a requirement that excavation was undertaken without blasting. This requirement was introduced to minimise the potential for EDZ development. Initially, a hydraulic wedge splitting technique was applied, but this technique was found to be particularly challenging for excavation of the EPSP shotcrete plug slots owing to the following:

- Application of the technique did not result in a smooth excavation profile.
- The splitting of the rock required high pressures; the unconfined compression strength reached a maximum of 120 MPa. This made use of the technique physically demanding for workers and slow to undertake.
- Use of the hydraulic splitting technique left unbroken ends of 45mm-diameter boreholes with variable depths.

In response to the identification of these challenges and in order to test the use of an alternative, a second technique was used for construction of part of the outer plug. The second technique was a pressure disintegration technique using Green Break Technology (GBT) cartridges (non-detonating gas expansion cartridges). The GBT technology significantly accelerated the work on the excavation for the plugs. Nearly no remains of boreholes were left when the GBT technology was used. Seismic activity was monitored during the GBT activation and demonstrated that the surrounding rock massif experienced only minimal shaking. During the application of the technology, the niche ventilation system was intensified by adding an efficient mobile duct to evacuate the gases generated by activation of the GBT cartridges. The excavated opening contour was more precise and smoother, compared to the hydraulic splitter technique.

During the excavation operations, the dimensions of the plugs were checked against the requirements using a triangular measurement tool. When the shape of the space excavated for the plug was not compliant with the requirements, fine enlargement work was carried out using a diamond saw; and the incised rock was removed by hand using chisels.

Following construction of the slots, the rock mass was injected with polyurethane resin at high pressure so as to improve the quality of the host rock. The required hydraulic conductivity value of the massif following injection was a maximum of 1 x 10^{-8} m/s. The requirement was to improve the quality of the massif surrounding the experiment up to a radius of 5 m. The injection mixture, consisting of WEBAC 1401 polyurethane resin, was injected into a total of 72 injection boreholes which were fitted with mechanical packers. The resin was injected into the boreholes by means of a WEBAC IP 2 high-pressure grouting set. Injection was terminated once a pressure level of approximately 35 MPa had been attained. A total of 760.45 kg of WEBAC 1660, WEBAC 1410, WEBAC 4170T, WEBAC 150 and WEBAC 1403 PU resins were used so as to achieve the required hydraulic parameters within the rock mass in the required area.

Borehole hydraulic tests were conducted and confirmed that the modified hydraulic conductivity of the rock mass in the space for the plug met requirements. Following completion of the tests, the boreholes were filled by injection of 32 litres of WEBAC 1660 resin.

Thirteen 23 m-long connecting boreholes were drilled between the SP-59 experimental niche and the SP-55 technological niche for the purpose of pressurising the experiment and for instrumentation requirements (some of the boreholes are illustrated in Figure 3.7). Eight of the boreholes were used for pressurisation and five for cabling associated with the experiment monitoring system.

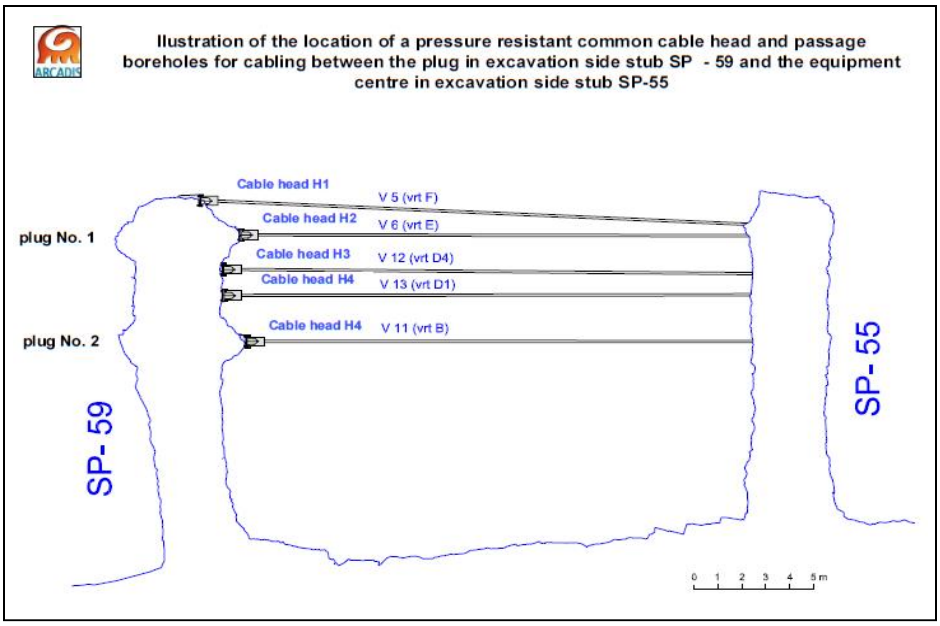


Figure 3.7: Lead-through boreholes (for wiring).

Further details on the reshaping and ground improvement of the EPSP experimental gallery niche can be found in SÚRAO and CTU (2016).

3.6 Installation of EPSP

The EPSP experiment was installed in following steps:

1. Preparation of the pressurisation chamber <2 m^3 (including installation of pressurisation tubing).
2. Treatment of the pressurisation chamber with waterproofing.
3. Installation of the first concrete separation wall between the pressurisation chamber and the inner plug.
4. Installation of the inner plug using glass fibre low-pH shotcrete.
5. Installation of bentonite sealing material, second concrete separation wall, filter and third concrete separation wall, which was undertaken using a concurrent process.
6. Installation of the outer plug using the same material and design used for the inner plug.

Monitoring instrumentation was installed as construction progressed.

The different stages in the installation of EPSP are described below and illustrated in a series of photographs in Figure 3.8. Further details on the installation of the EPSP experiment can be found in SÚRAO and CTU (2016).

The installation of the plug, from installation of the first concrete separation wall to installation of the outer concrete plug, took about three months. This does not include the time required for grouting, and monitoring technology. The total time from chamber adjustment to fully operation experiment was 10 months. A major delay was caused by contact grouting of inner plug where several campaigns were done unfortunately with long waiting time for curing in between. The shotcreting action (plug erection) by itself was very fast. It took less than 24h to erect each plug followed by one month curing.

3.6.1 Installation Activities Associated with the Pressure Chamber

The walls and floor of the pressurisation chamber were prepared using shotcrete and SikaTop-122SP (www.sika.com). The thickness of the profiling was such that there was a gap of 100 mm between the remodelled chamber surface and the next structure (the first concrete separation wall). The surface of the remodelled chamber was treated with a 3 mm thin waterproofing finish SikaTop Seal 107 (www.sika.com). The pressurisation chamber was closed by installation of the first concrete separation wall.

The shotcreting of the pressure chamber also served as a test of the technology to be used for shotcreting of the inner plug.

3.6.2 Inner Concrete Plug

The inner plug was built using glass-fibre-reinforced low-pH shotcrete. The wet mix shotcreting procedure was used for construction of this plug. The thickness of this inner plug is 1850 mm. The shotcreting was performed in approximately 100 mm thick layers and in a non-stop run in 23 hours. Measurements and observations during the experiment demonstrated that contact grouting between the plug and the rock was necessary to ensure water tightness.

The major influence on the speed of the shotcreting process was logistics. The concrete mix was produced at a concrete plant in Prague and transported by road to the Josef URC and underground laboratory. At the entrance to the facility, the mixture was reloaded into small trucks (each capable of transferring 1 m^3 of concrete) because the small profile of the Josef tunnels limited the size of the trucks that could access the experiment location.

Within the Josef facility, there is only one location where the trucks supplying the concrete could pass, and, therefore, the turnaround time for each truck (40 minutes) was the rate-limiting step in the shotcreting process. That meant that every 20 minutes a new batch of concrete was available for shotcreting. The shotcreting process itself was much quicker. The time between the arrival of each batch of concrete was used to clean rebound from the area around the shotcrete plug and to install monitoring equipment.

Once the plug had cured, a pressure test was undertaken using water and air. Measurements and observations of water flow across the plug during the test demonstrated that contact grouting between the plug and the rock was necessary to ensure water-tightness. The contact grouting, which used CarboPurWF and CarboPurWFA resin, was undertaken in several campaigns with a pressure test conducted after each campaign. During each campaign, a series of boreholes were drilled around the circumference of the plug on the contact plane between the shotcrete and the rock. These holes were backfilled with WEBAC 1660 resin following the installation of each grout campaign.

3.6.3 Bentonite Seal

The bentonite was emplaced between the inner shotcrete plug and the second separation wall. Ninety-five percent of the bentonite seal was composed of bentonite pellets that were vibration-compacted. The crown space of the seal (the remaining 5%, which corresponds to approximately 1.5 m^3) could not be accessed using the vibrator and was therefore emplaced using spraying technology. The filter separation wall(s) served as support for the emplaced bentonite, and were constructed in parallel with bentonite emplacement.

Based on a pilot test (see Vašíček *et al.*, 2014) which demonstrated good compaction of the pellets, two vibration-desk machines (the NTC compaction plate and the Masalta vibration plate) were selected for bulk works. The bentonite pellets were emplaced in horizontal layers, each with a maximum height 3 cm, and were vibration compacted. Electric hammer drills (HILTI TE 3000-AVR and HILTI TE 1500-AVR) with a plate were used for the compaction of the bentonite pellets around measurement sensors, around the drift wall and in the upper part of the drift where the space available for utilising the vibration-desk machines was limited.

Originally, consideration had been given to using a mixture of bentonite and ice for the spraying, which could potentially provide high densities for the emplaced bentonite. However, the ice spraying technology has a limited throughput, and owing to time constraints for installation of the bentonite seal, this technology was not used. Nonetheless, after fine tuning of other machinery, it has been possible to achieve the same density of deposited material with sprayed bentonite pellets only (at a much higher deposition speed).

3.6.4 Filter

The gap between the second and third separation walls was used for the gravel filter. The filter was manually emplaced in steps. At first, the lower part of the walls (approximately one-third to half of the overall height) was erected and the gravel filter was emplaced in the resulting gap. Following this, the bentonite emplacement commenced. Once the bentonite level reached the level of the walls (and the filter) then new layer(s) of concrete blocks were constructed and the filter emplaced. The final layer of the separation walls and the gravel was emplaced immediately after shot-claying was completed.

3.6.5 Outer Concrete Plug

The outer concrete plug was constructed in exactly the same manner as the inner plug. The only difference between the inner and outer plug was the installation of grouting tubes around the circumference of the outer plug prior to shotcreting.

Once the plug had cured, grouting was undertaken using the preinstalled tubes. Initial pressure testing of EPSP demonstrated that this grouting was not sufficient and additional grouting was introduced in a similar way as for the inner plug.

3.6.6 Monitoring System

The primary aim of monitoring of EPSP is to investigate the various processes underway inside each plug component, to verify component behaviour and to assist in assessing their performance in order to build a knowledge base for the construction of a future repository plug.

The key processes and locations inside EPSP have been identified and sensors have been specially selected in order to capture them. Monitoring of EPSP focuses on water movement inside the experiment and the experiment's response to pressurisation.

Water movement inside the experiment is monitored in terms of water inflow, water content distribution within the bentonite seal and water (pore) pressure distribution.

The mechanical response of the plug is monitored by means of strain gauges installed at key locations in the concrete plugs and instrumented rock bolts positioned within the rock. Moreover, contact stress measurement is deployed between the rock and the plug.

Temperature distribution is monitored since it is important not only to understand the hydration heat generated through curing, but it is also used as a reference base for sensor compensation during the loading of the experiment.

Several measures were taken in order to ensure the provision of reliable data such as cross validation (sensors working on different principles are used to measure similar phenomena) and redundancy. Only pretested/calibrated/verified sensors were used in the experiment.

An integral element of the monitoring process consisted of the presentation of the measured data for further analysis; therefore the data were instantly available online to end-users via a simple web interface. This is discussed further in D4.4 (DOPAS 2016b).

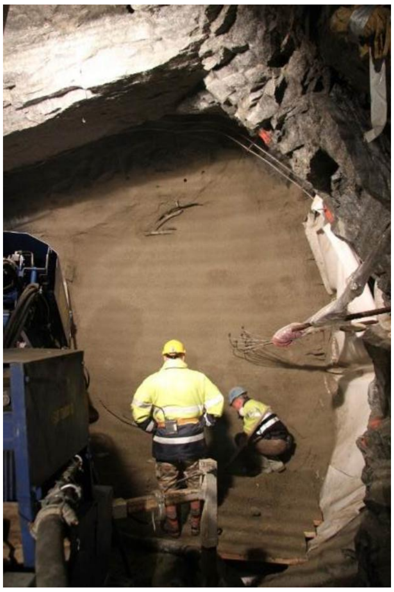
Experimental niche prior to reshaping and ground improvemen (2.11.2012)

Excavation of slots for plugs

Grouting works

Excavated slots for the plugs

Shotcreting of chamber (27.10.2014)



Finished shotrcreting (27.10.2014)

Figure 3.8: Photographs of the installation of EPSP.

Separation wall erection (4.11.2014)

Finished separation wall (5.11.2014)

Inner plug erection and sensor installation (12.-13.12.2014)

Figure 3.8: Photographs of the installation of EPSP (continued).

Sensor assembly on inner plug before bentonite emplacement (5.6.2015)

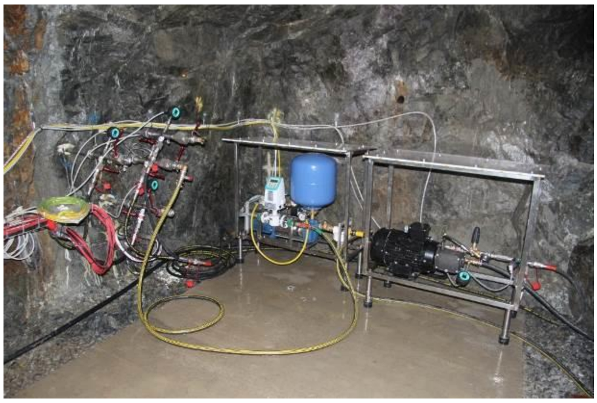
First part of filter erected (photo taken at the start of bentonite installation, 5.6.2015)

Pellets emplacement (compaction)

Upper part of the drift – space for backfilling by the spraying

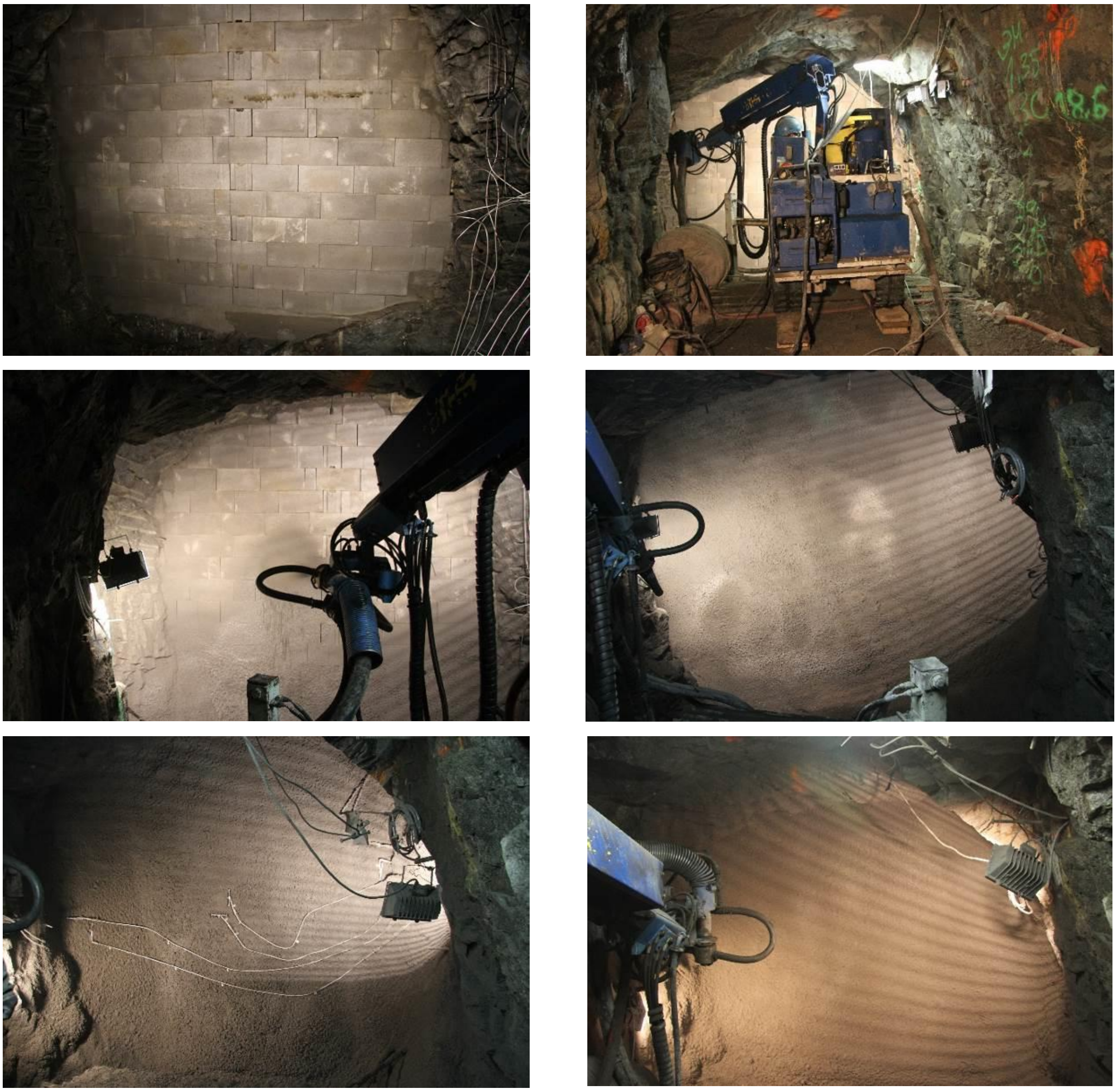
Shot claying

Figure 3.8: Photographs of the installation of EPSP (continued).



Technology installation in the niche (24.2.2015)

Technology installation in the niche (24.2.2015)



Outer plug erection (19.-20.6.2015)

Figure 3.8: Photographs of the installation of EPSP (continued).

Finished outer plug

Start of the experimental programme (21.7.2015)

Start of the experimental programme (21.7.2015)

Figure 3.8: Photographs of the installation of EPSP (continued).

3.7 Lessons Learned from the Design and Installation of EPSP

EPSP has demonstrated that there can be significant programmatic benefits from undertaking full-scale experiments in pre-existing near-surface facilities, especially for programmes with a long period before submission of a licence application. The conduct of the test has contributed significantly to the practical experience and expertise of individuals working on the Czech radioactive waste disposal programme. This includes greater experience and expertise in the operator skill required to effectively emplace shotcrete, in the management of rebound and dust during the shotcreting (and shot-claying) activities, and in the documentation and quality assurance of practical activities.

Conducting experiments in pre-existing near-surface facilities will always have its limitations. In the case of EPSP in particular, and the Josef URC and underground laboratory in general, two limitations were the ground conditions and the impact of the size of the underground tunnels on the delivery of the shotcrete to the experiment site. The limitations posed by the ground conditions were addressed through extensive rock grouting operations, which slowed the progress of the experiment, but should not be necessary to the same extent in an actual repository site (if used close to the waste, the acceptability of grouts incorporating organic material will require demonstration). The size of the access tunnels limited the delivery of the mixed shotcrete to 1 m^3 batches and slowed the installation of the shotcrete plugs.

EPSP has demonstrated that in a pre-existing, near-surface facility, the selection of the experiment site is particularly important, as the impact of the ground conditions (as discussed above), tunnel dimensions and electricity supply can all be limiting factors on the manner in which the experiment can be conducted.

One of the concerns for implementation of EPSP was workers' safety, in particular mitigation of the potential impacts from dust and engine exhaust gases hazards. These hazards were mitigated by:

- Limiting the use of diesel engines; the only diesel engines that were used were the shotcrete delivery trucks. Limiting the use of diesel engines led to additional stress on the electrical power network of the facility.
- Providing additional ventilation of the experimental niche. The air from the experimental niche was extracted using a fan and piped into a nearby niche. The niche was equipped with several geotextile walls that filtered the extracted air.

EPSP has been undertaken as an integrated project, with significant materials development and testing, which has led to an enhanced knowledge of concrete and bentonitic materials in the Czech programme. The additional understanding of local bentonite materials (e.g. B75), will be of use in the design of plugs and seals, and also in development of designs for buffers and backfills.

EPSP has also benefitted from transfer of knowledge from other industries, most notably in the use of fibre-reinforced shotcrete, as used in plugs and seals installed in gas storage facilities in the Czech Republic several decades below. This illustrates how industrial analogues can be successfully incorporated within specific waste disposal programmes.

Initial testing of the Inner Concrete Plug of EPSP demonstrated that contact grouting was necessary to achieve appropriate performance of concrete seals. This learning was applied in the Outer Concrete Plug, where grouting tubes were installed prior to emplacement of the shotcrete.

EPSP has also applied several processes that would not be considered optimised in other programmes. These include the excavation of the slots for the Inner and Outer Concrete Plugs using highly-manual methods (including chiselling), and the use of compaction plates for compression of the bentonite pellets. These processes may require further "industrialisation" before application in a repository, but there is sufficient time available in the Czech programme to allow this. Such processes may also ultimately be feasible for installation of the engineered barrier system in the Czech repository.

4. DOMPLU Experiment

This chapter provides a summary of the learning from design and construction of the DOMPLU experiment:

- In Section 4.1, the background to the experiment and its objectives are summarised.
- In Section 4.2, the testing and selection of materials prior to DOMPLU implementation are described.
- In Section 4.3, the structural design work undertaken as part of the DOPAS project is summarised.
- In Section 4.4, the siting of the DOMPLU experiment is explained.
- In Section 4.5, the construction of the DOMPLU niche and the adjacent monitoring niche is described.
- In Section 4.6, the installation of the DOMPLU experiment components is summarised, with particular focus on the novel aspects of the experiment.
- Discussion of the lessons learned regarding the design and installation of the DOMPLU experiment is provided in Section 4.7.

More details and illustrations regarding the DOMPLU experiment can be found in the DOMPLU summary report (Grahm *et al*., 2015).

4.1 DOMPLU Experiment Background and Objectives

The DOMPLU experiment was a full-scale test of the reference deposition tunnel plug in SKB's repository design. The DOMPLU experiment design consisted of an unreinforced low-pH concrete dome with a watertight seal, a filter layer, and a backfill transition zone located upstream of the concrete component (Figure 4.1). The function of the concrete dome is to keep the watertight seal, filter and backfill in place. The watertight seal is made of bentonite blocks and pellets. Its function is to seal groundwater leakage paths and to ensure an even pressure is applied on the concrete dome. The filter is made of gravel[2]. Its function is to collect water draining from the deposition tunnel so that no water pressure is applied on the concrete dome before it has cured and gained full strength. The plug also contains concrete beams[3] to aid construction as well as pipes for drainage, cooling and grouting. The backfill transition zone consists of bentonite blocks and pellets. Further information on the DOMPLU experiment conceptual design and design basis is presented in DOPAS (2016a). The experiment schedule is summarised in Table 4.1.

---

[2] The term "Geological Filter" is used to describe the part of the filter made of gravel.

[3] As described in the text, in the DOMPLU experiment, one of the concrete beams also forms part of the filter; the term "Stiff Filter" is used to describe the part of the filter made of concrete beams.

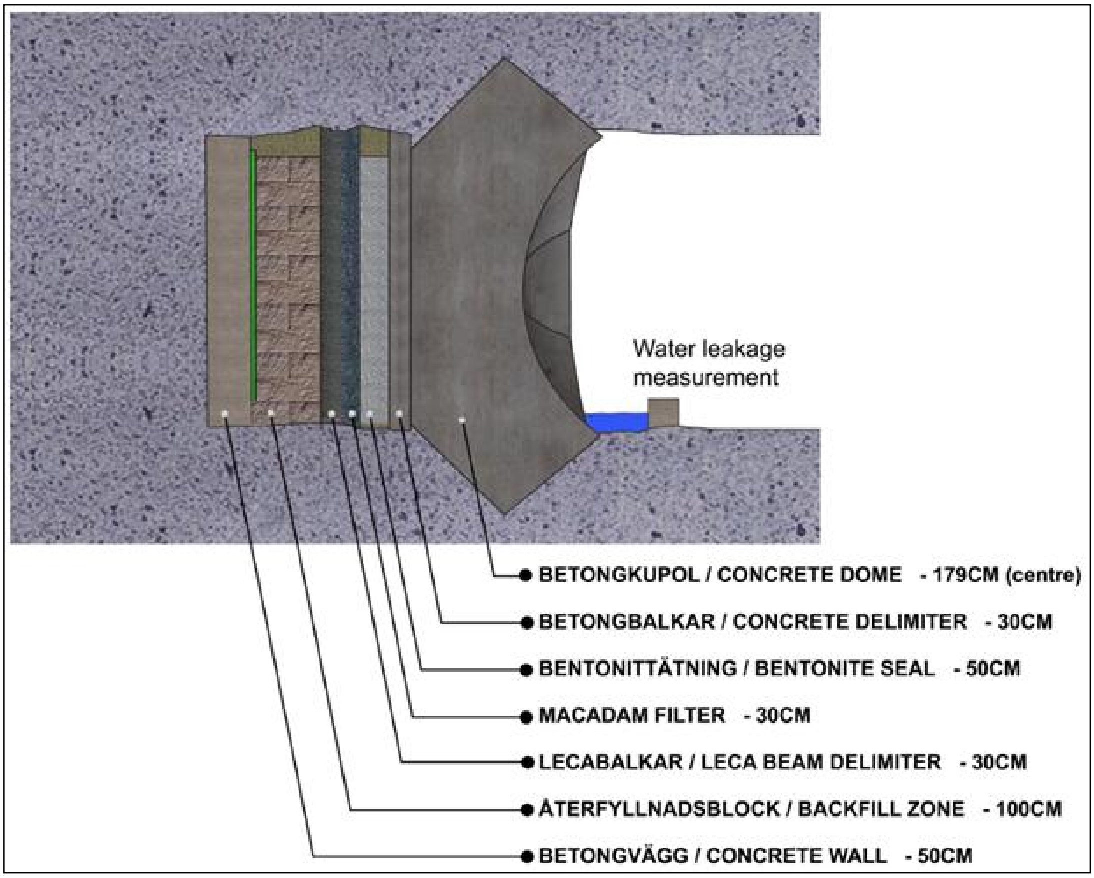


Figure 4.1: Schematic illustration of the DOMPLU experiment design (Grahm *et al.*, 2015).

The DOMPLU experiment is part of an on-going SKB testing and demonstration programme. The overall objective of the test is to reduce uncertainties in the long-term performance of deposition tunnel plugs and in the description of the initial state of the deposition tunnel plugs. The DOMPLU experiment design represents a detailed iteration of the reference design rather than a fundamental change. Specific objectives of the DOMPLU experiment were:

- To finalise the details of the reference design.
- To demonstrate the feasibility of plug installation.
- To validate requirements on construction methods.
- To demonstrate that the plug works as intended under realistic conditions, up to the reference design total pressure of 7 MPa. The load case is a combination of the hydrostatic pressure from the groundwater (up to 5 MPa) and the swelling pressure from the backfill transition zone (approximately 2 MPa), acting together on the plug system.
- To develop a method for measurement of leakage across the plug, use the results to determine a requirement on the leakage rate across deposition tunnel plugs, and evaluate whether a sufficiently low hydraulic conductivity can be achieved; e.g., a leakage rate of <0.1 litres/min as discussed by Grahm *et al.* (2015).
- To improve testing and quality control during repository construction.

Table 4.1: The schedule of activities undertaken in the DOMPLU experiment.

Period	Activity
February-March 2012	Excavation of DOMPLU niche
April-May 2012	Excavation of concrete dome slot: Phase 1 – assembly of safety scaffolding; drilling 16 holes and wire sawing by blind and pulling cuts; problems occurred with jamming of wire; process interrupted
June 2012	Installation of concrete back wall
August - October 2012	Excavation of concrete dome slot: Phase 2 – modified wire sawing procedure by blind cuts; blasting of the stone segments cut free by the wire; and cleaning of slot and tunnel
November – December 2012	Preparation of lead-through pipes, sensors and pressurisation system. Laser scanning of slot surfaces.
January 2013	Parallel installation of the backfill transition zone and LECA® beams
January 2013	Parallel installation of the filter, bentonite seal and outer delimiter
February 2013	Test of filter drainage function during construction
February 2013	Installation of grouting tubes, cooling pipes and formwork assembly
March 13, 2013	Casting of the concrete dome
May 2013	Reference measurements (laser scanning) of the concrete dome
June 2013	Contact grouting of the concrete/rock interface

Note: this DOMPLU construction sequence took place out of the DOPAS Project Schedule.

In order to meet the experiment objectives, the DOMPLU experiment design included specific design modifications compared to the reference design. These modifications were introduced to test if they could be adopted in the reference design in the future:

- The use of unreinforced low-pH concrete instead of reinforced low-pH concrete for the concrete dome. The use of an unreinforced structure was proposed in Malm (2012). Malm (2012) concluded that the dome plug is strong enough without reinforcement, that reinforcement has some undesirable properties (e.g. potential for cracking due to autogenous shrinkage and the corrosion of the reinforcement), and cost and time implications during construction of the dome.

- In the DOMPLU experiment, the backfill end zone was redefined as a backfill transition zone where the swelling pressure from backfill is reduced to a level that is similar to the resulting swelling pressure of the bentonite seal (about 2 MPa). The purpose of introducing a transition zone is to reduce the displacement of the plug system components and pressure on the concrete dome.
- In the DOMPLU experiment, the innermost (towards the backfill) delimiter was considered to be part of the filter. Instead of concrete beams, lightweight expanded clay/concrete aggregate (LECA®) beams and gravel with a high hydraulic conductivity were used. The filter thickness was 600 mm, made up of 300 mm of gravel (with a particle size of 2-4 mm) and 300 mm of LECA® beams, compared to a thickness of 700 mm, which is specified in the reference design for the filter.
- The middle delimiter between the filter and the watertight seal was composed of a geotextile instead of concrete beams. The geotextile simplifies installation and facilitates distribution of water from the filter into the bentonite seal.
- The outer delimiter was composed of low-pH concrete beams as for the reference design. There was also a double geotextile layer between this delimiter and the concrete dome to prevent adhesion of the delimiter to the concrete dome, and therefore avoid potential cracking of the concrete dome during shrinkage.
- Cooling pipes were specified to be made of copper. This is a material easy to work with, likely to be approved for future use and common for similar applications.
- Grouting tubes were made of cross-cut 50 mm plastic drainage tubes. This is a new promising design of injection tubes, but has never been tested by SKB before.
- The thickness of the bentonite seal was 500 mm in the DOMPLU experiment, which was practical and considered sufficient for the timescale of the experiment. It is suggested that the final seal thickness should correspond to the thickness of reference backfill blocks, thus the same production tools can be used. The same clay material for the seal as for the buffer system is recommended and therefore MX-80 material was used in the DOMPLU experiment.
- The filter installed dry density was 1400 kg/m^3 in the DOMPLU experiment while a value of 1900 kg/m^3 is considered in the reference design. In the reference design, compaction of the filter was presumed, but this was evaluated as impractical and not useful for the grading chosen of the filter material. The result is, of course, a larger compression of the filter by the swelling pressure, which has to be taken into account in the design of the backfill transition zone.

Since the development of groundwater pressure inside the plugged volume, as well as the swelling pressure development in the seal, was predicted to take many years, the plan was to pressurise the DOMPLU experiment by injecting water. Hence, a small gravel pocket was placed at the floor level directly in front of the concrete wall at the tunnel face. This feature is not included in the reference design; it is only part of the full-scale test to inject water behind the backfill and was used for drainage tests, wetting of the bentonite clay and pressurising of the DOMPLU experiment.

4.2 DOMPLU Material Testing and Development

For the DOMPLU experiment, material testing and development focused on:

- Testing of the preferred concrete mix to demonstrate suitability for use in the concrete dome.
- Testing of materials and design of the bentonite seal and the transition zone based on the requirement to reduce swelling pressures acting on the concrete dome.
- Testing of different filter material options to select a preferred material suitable for effective drainage.
- Testing of different options for the delimiters and selection of the preferred materials.

Further details of the testing and development are provided in the DOMPLU experiment report (Grahm *et al*., 2015) and the modelling and tests report (Börgesson *et al*., 2015).

4.2.1 Selection of the Concrete Mix

A concrete mix for a low-pH SCC, denoted B200, has previously been developed specifically for use in the deposition tunnel plugs of SKB's repository (Vogt *et al*., 2009) (Table 4.2). However, testing of this concrete was required to ensure that it provided the necessary strength, shrinkage, creep and binding properties to the host rock, given the ambition to demonstrate that the concrete could be emplaced without reinforcement. Details of the concrete testing can be found in Mathern and Magnusson (2014) and Flansbjer and Magnusson (2013a and 2013b). It was also necessary to demonstrate that properties measured in the concrete factory were representative of the properties the concrete would achieve when poured at the experiment site underground at Äspö, as it takes two hours to transfer mixed concrete from the factory to the experiment site at -420 m in the Äspö HRL.

Table 4.2: Mix composition of the B200 low-pH self-compacting concrete.

Constituent	Quantity (kg/m^3)
CEM I 42.5 MH/SR/LA	120
Silica Fume (Densified)	80
Water	165
Limestone Filler L25	369
Sand 0-8 mm (65% of aggregates)	1037
Gravel 8-16 mm (35% of aggregates)	558
Superplasticiser Glenium 51	6.38

Tests of the hardened concrete properties (compressive and tensile splitting strengths) were performed as part of concrete casts made at the concrete factory, concrete casting made on site at the Äspö HRL, and in conjunction with other laboratory tests. Overall, the measured compressive strength obtained at the concrete plant and on-site at the Äspö HRL were close to each other (see Figure 4.2), indicating that the compressive strength of the concrete was not affected significantly by the two-hour transportation time from the concrete plant to the site. However, results from tests on cubes from the back wall casting, and to a lesser extent also from the specimen casting, have shown a lower compressive strength on-site than at the concrete plant.

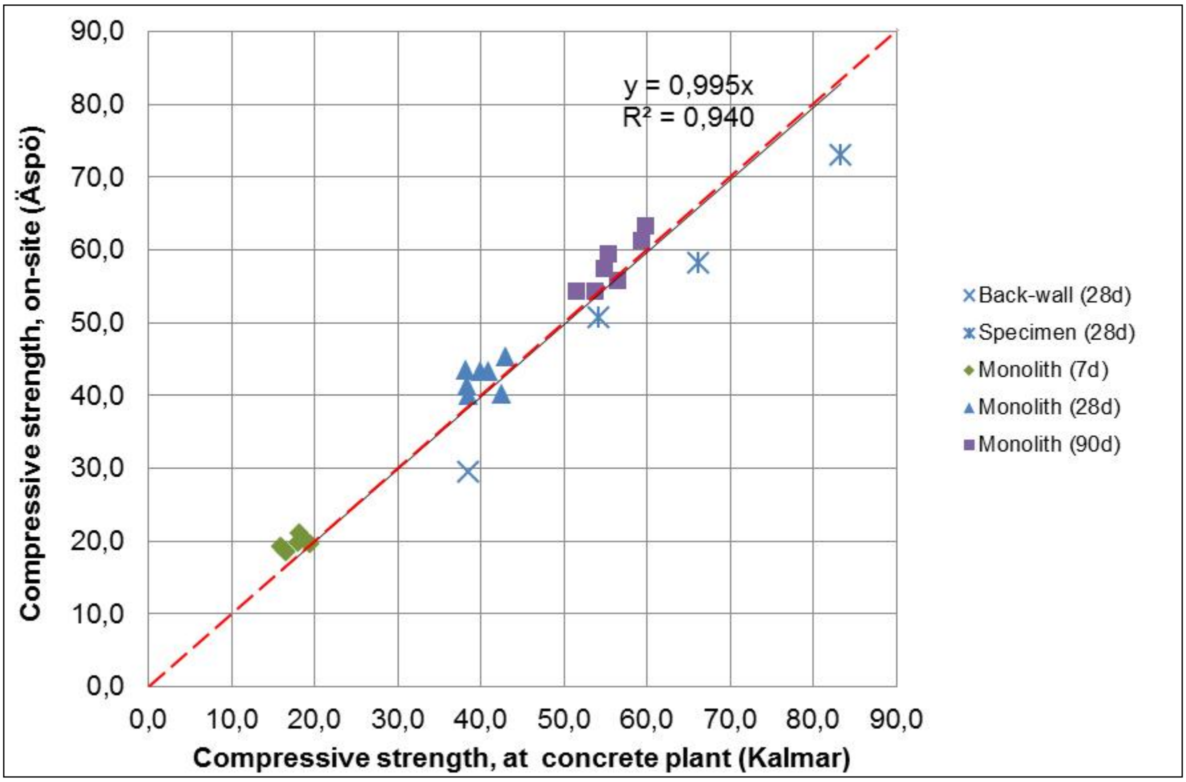


Figure 4.2: Comparison of compressive strength for cubes cast at the concrete plant and on-site at the Äspö Hard Rock Laboratory (average values for each concrete load when more than one cube tested per load) and trend line considering all the test series (Mathern and Magnusson, 2014).

The shrinkage of the low-pH concrete was tested in the laboratory to obtain information on the expected change in volume of the concrete with time. Shrinkage measurements were conducted on prisms of dimensions 100x100x400 mm during approximately 3 years. Three different curing conditions were considered: specimens kept in 50 % relative humidity at 20°C; sealed specimens and specimens stored in water at 20°C.

It can be noted that the results from the previous shrinkage measurements conducted by Vogt *et al.* (2009) for sealed specimens and specimens stored at 50 % relative humidity are very similar to the results measured in the current project (see Figure 4.3).

According to Malm (2012), the estimated gap due to autogenous shrinkage at the time of grouting was estimated to be about 3.7 mm in the top of the concrete dome and 1.8 mm on its sides (assuming that the plug has fully released from the rock). The cooling of the structure performed prior to contact grouting increases this gap further and Grahm *et al.* (2015) predicted that the further increase would be 0.8 mm at the top. The total gap at the top of the dome would, according to this simplified estimation, be approximately 4.5 mm. The design of the concrete dome includes contact grouting of the shrinkage gap three months subsequent to casting of the dome.

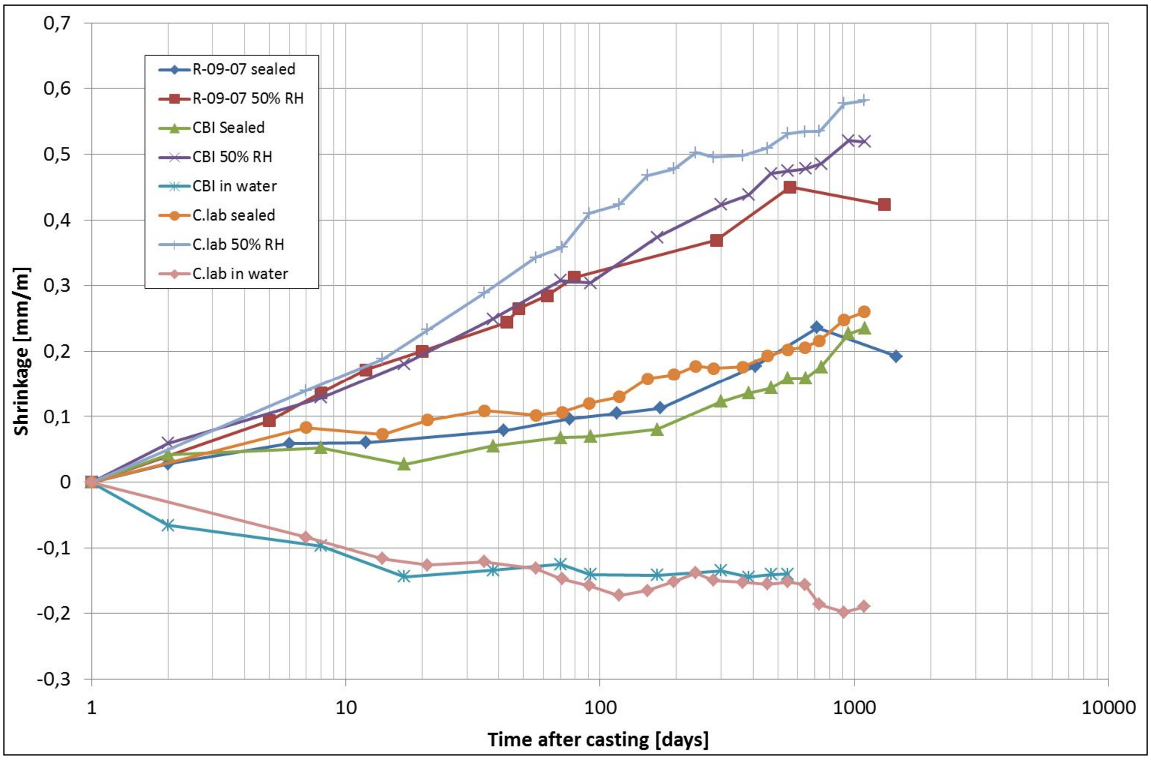


Figure 4.3: Average shrinkage results for the different curing conditions for specimens tested at the concrete laboratory, at the Swedish Cement and Concrete Research Institute and previous results from Vogt *et al.* (2009) (Mathern and Magnusson, 2014).

Concrete creep of the B200 mix was also tested in the laboratory. The creep tests were performed for different stress levels: 40%, 50% and 75% of the concrete compressive strength. The results indicated that impacts of creep were not significant, even for high stress levels. According to Mathern and Magnusson (2014), the results show no indication that the stress levels applied as sustained loading will lead to failure in the concrete. It was, however, recommended to continue the creep measurements over a longer period.

Bond tests on the mechanical properties of the interface between the rock and the concrete were conducted in order to estimate the release of the concrete from the rock during curing, and to derive parameters for numerical models. The mechanical tests were performed on specimens core-drilled from small-scale rock-concrete blocks manufactured in the laboratory. The behaviour in shear and tension of the interface between the concrete plug and the wire-sawn rock surface was investigated at different times after casting.

According to Mathern and Magnusson (2014), the results from direct tensile tests showed that the tensile bond strength between the concrete and the rock proved to be in the same order of magnitude as the tensile strength of the concrete. However, the failure mode at the rock/concrete interface was much more brittle than that of the concrete. One of the design requirements for the dome plug is that it should be considered as stress free 90 days after casting. This means that the shrinkage during the first 90 days should be considered as free shrinkage, i.e., without restraint. Therefore, the concrete plug has to undergo sufficient shrinkage during this time so that release from the rock surface is achieved.

A general conclusion from the concrete testing was that it is possible to use the concrete mix B200 without any further development. However, experiences from the B200 concrete test series showed that further clarification is needed regarding quality control requirements and acceptance criteria of the young concrete properties. In addition, the design decision to force the dome to release from the rock must be evaluated during the full scale test. Thereby, if the measured shrinkage in full scale is less than used in the design calculations it may no longer be a future requirement for the concrete dome to release from the rock.

4.2.2 Selection of Sealing Materials

The design of the DOMPLU experiment includes a bentonite seal of MX-80 positioned behind the concrete dome (Figure 4.1). Bentonite sealing blocks are placed in a section excavated by blasting. The blasting technique results in uneven rock surfaces, which means that bentonite pellets were needed to fill the floor, ceiling and contacts with the wall to even out the rock surface. Various experimental tests were carried out to investigate the properties of the bentonite sealing materials, including compaction properties of the bentonite, strength of the blocks, compressibility, swelling pressure, hydraulic conductivity, and self-sealing of fractures in the rock and slots between the bentonite blocks (Börgesson *et al.*, 2015).

The main conclusions from the test programme are presented below. For further information regarding the test methods used, and a more extensive summary of the results, see Börgesson *et al.* (2015).

Initial calculations demonstrated that blocks with a relatively low density (1400-1500 kg/m^3) and a high water content (~28%), would be needed to achieve a swelling pressure of ~2 MPa in the bentonite seal. However, during the project design phase, it was realised that the density requirements of the sealing blocks must be increased to 1700 kg/m^3 if the effect on the overall density of the pellets used to fill the gaps against the rock around the blocks was taken into account. Consequently, the sealing blocks must have a higher density in order to reach an average density of 1400-1500 kg/m^3 in the seal section.

To identify the required water content of the raw bentonite material, a series of strength tests were undertaken. Bentonite blocks with the highest dry density for a compaction pressure of 25 MPa were achieved when powder with a water content of 17% was compacted. These blocks also had the highest strength. The strength of blocks compacted with a water content of around 14, 22 or 24% was high, while the samples compacted with 10% water content were very fragile with low strength, mainly due to the low density. The specimen with a water content of 28% had a very low dry density, 1525 to 1550 kg/m^3, but the high water content resulted in beams with good strength. The results are provided by Figure 4.4.

The performed beam tests show that in order to produce blocks with the intended density (originally 1400-1500 kg/m^3) and with high strength, bentonite with a low water content (≤10%) should be avoided. The highest strength was achieved for the specimens having a water content of 17% and high dry density. With an adjustment of the compaction pressure this water content may be the optimal from a block strength point of view but an alternative could be to manufacture the blocks with water content of 28%. Based on the results of the tests, it was recommended to use in the DOMPLU experiment water content for the raw material of 17% to achieve blocks with density 1700 kg/m^3 and with high strength.

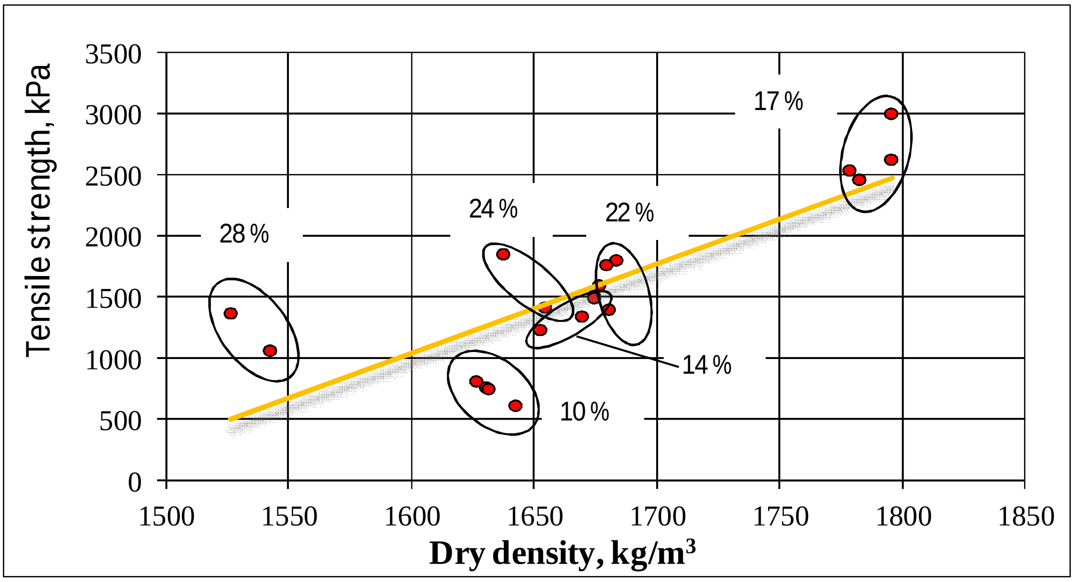


Figure 4.4: Summary of the strength test results showing the maximum tensile strength at failure for a series of beam tests, plotted as function of the dry density of the specimens prepared using 25 MPa load, from Börgesson *et al*. (2015).

Establishing the compressibility properties of the sealing blocks is of considerable importance, if modelling of the movements of the backfill behind the dome plug is to be accurate. The results from the Oedometer tests agreed well with what was expected for MX-80 bentonite. These results were used for comparisons with results from numerical models, especially stress paths in the void ratio versus net stress plane (both axial and radial).

The aim of testing the swelling pressure and the hydraulic conductivity of the bentonite blocks was to investigate whether the blocks with a high degree of initial saturation showed the same properties as blocks with a low degree of initial saturation. The tests were performed by using a swelling pressure Oedometer (Fig 2-12 in Börgesson *et al*. 2015).

The performed testing showed that there is no significant influence of the water content of the bentonite specimen on its swelling pressure and hydraulic conductivity. The swelling pressure obtained by the current study is essentially identical to the reference values and the hydraulic conductivities measured are just slightly lower, which is interpreted as being due to the fact that the higher initial water content gives somewhat more homogenised specimens and hence a slightly lower hydraulic conductivity value.

Based on previous studies and a substantial database, the influence of a 1% salt concentration in the pore-water was not expected to have a substantial effect on the swelling pressure or hydraulic conductivity (Börgesson *et al*., 2015), and the new data shown in Figure 4.5 is consistent with these expectations.

The time for the bentonite blocks to swell and seal the initial gaps between them is of high interest for the function of the plug. In order to study this behaviour, a series of specially designed tests were performed. The results from the tests performed by Börgesson *et al*. (2015) showed, as expected, that the width of the gap between the bentonite blocks is of great importance in their saturation evolution. In tests performed with a 2 mm gap, the radial pressure build up was much faster compared to tests with 4 mm gap. Another observation was that the initial water content of the bentonite block also influenced the results. The time for swelling and filling up the gap with bentonite and the start of pressure build-up at the contact between the bentonite and the experimental cell walls took longer for the samples with high initial water contents (and a high degree of saturation).

In summary, the tests confirmed that compressed MX-80 bentonite blocks with dry density of approximately 1700 kg/m^3 and a water content of 17%, surrounded by a 10-20 cm thick layer of MX-80 pellets, would be a functional configuration for the seal.

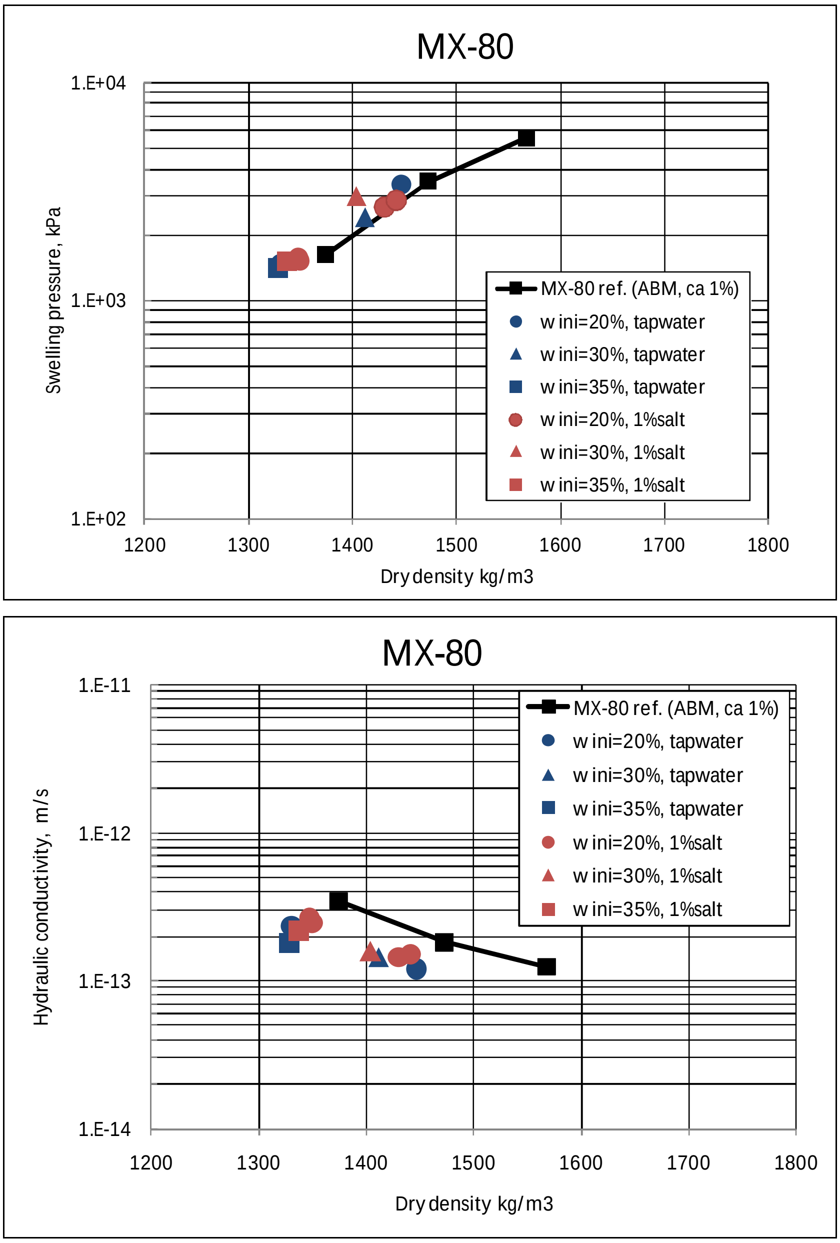


Figure 4.5: Plots showing the test results of MX-80 clay. For comparison some results from measurements within SKB's ABM test are also shown in the figures. Upper: swelling pressure as function of dry density. Lower: hydraulic conductivity as function of the dry density, from Börgesson *et al.* (2015).

4.2.3 Selection of Geological Filter Materials

Different filter materials for the plug system were tested to select the most appropriate composition. Three materials with different attributes were investigated (Börgesson *et al.*, 2015):

- Natural sand/gravel with a grain size range of 0-4 mm.
- A combination of gravel of <5 mm grain size and stone dust (MakPak®).
- Gravel with a grain size range of 2-4 mm.

Geotextiles with two surface densities, 1000 and 1200 g/m^2, were also tested. These materials can be used for both material separation and draining purposes.

For the different kinds of materials, several different tests were performed (compaction tests, compressibility, hydraulic conductivity and clogging tests). Each test had to be adapted to the tested specimen types and this is described by Börgesson *et al.* (2015). The main results are briefly summarised below.

The testing identified gravel, with a grain size range between 2-4 mm, as the preferred material for the filter. Gravel 2-4 mm had the highest hydraulic conductivity and its hydraulic conductivity was also only slightly affected by an increased dry density. Gravel 2-4 mm also maintains its draining ability when exposed to water flow with high bentonite content.

Both geotextiles behaved in a similar manner with regards to compressibility and hydraulic conductivity. The hydraulic conductivity was clearly reduced when the stress on the specimen increased. The clogging test indicated that the draining ability of the geotextile will be reduced with time and that it should not be used for draining water with eroded bentonite present in it. However, it could still function to distribute water if artificial wetting of the system is required. Thus, it was decided to test the geotextile at full scale in the DOMPLU experiment and use it as a delimiter between the gravel filter and the bentonite seal. The purpose was to facilitate distribution of water to the seal from the filter.

4.2.4 Selection of Stiff Filter Materials

In the plug reference design, the concrete beams between the backfill transition zone and the filter were judged to be a poor solution since the inflow of groundwater to the filter would be restricted by the concrete. Therefore, two stiff filter materials were evaluated to consider their suitability to be used as stiff filter materials in the DOMPLU experiment:

- LECA®.
- Light concrete.

According to Börgesson *et al.* (2015), LECA® was found to be the superior option, in terms of its hydraulic conductivity and also because it maintains it hydraulic performance when exposed to a water flow with high bentonite content (it does not clog). The main concern during LECA® beam testing was its compressive strength which did not correspond to the product specification. There is, however, no requirement for this parameter in the tunnel plug design basis and the material was considered suitable for use in the DOMPLU experiment. Consequently the concrete delimiter was replaced by 30-cm-thick LECA® beams between backfill and gravel filter. The LECA® beams were considered to be part of the filter and thus the thickness of the gravel layer could be reduced to 30 cm.

4.3 Development of the DOMPLU Structural Design

The structural design work undertaken for the DOMPLU experiment included use of a scale model test in the laboratory, and analytical and numerical calculations, which are described in full in Börgesson *et al*. (2015). These two different approaches were mainly used to decide the geometrical properties of the DOMPLU experiment, to simulate the behaviour of the plug system, and to assist with the design of the pressurisation sequence and the water pressure elevation in full-scale testing (Börgesson *et al*., 2015).

4.3.1 Scale Model Test

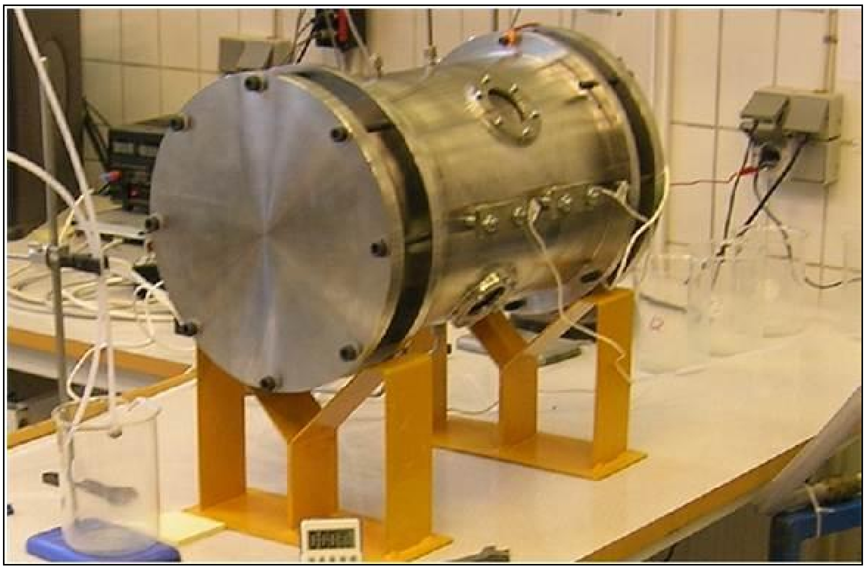
A laboratory physical model at a radial scale of 1:20 and axial scale of 1:10 was designed in order to test the functionality of the plug and its components. A total of six laboratory model tests were implemented with the following main objectives:

- Demonstrate the function of the draining components (LECA® and gravel filter).
- Demonstrate that the bentonite seal can withstand a water pressure of 5 MPa without leakage.
- Investigate whether a bentonite seal component is needed when using grouted concrete beams.
- Investigate the bentonite seal function against grouted and non-grouted concrete beams.
- Investigate how the sealing function is influenced by the void space between the concrete beams.
- Study how fast and how soon a water pressure can be applied to the sealing component after water filling of the filter.

The assembled model equipment and a schematic of the model cross section are shown in Figure 4.6.

Based on the results from the six scale model tests, the following recommendations were made for consideration in the DOMPLU experiment:

- The filter draining function should be tested with a water flow rate that represents the expected total flow in a deposition tunnel.
- The erosion of bentonite past the plug during the drainage period should be determined and evaluated to confirm that it is within the acceptable range.
- The void space between the concrete beams and between the concrete beams and the tunnel wall should be minimised for maximum sealing function. The void space between the concrete beams and the tunnel wall should be filled with paste.
- The filter should be filled with water early on to give the bentonite seal access to water as early as possible.
- The water pressure elevation should be carefully implemented. If the water pressure needs to be reduced for some reason, the re-pressurisation also needs to be carefully performed.



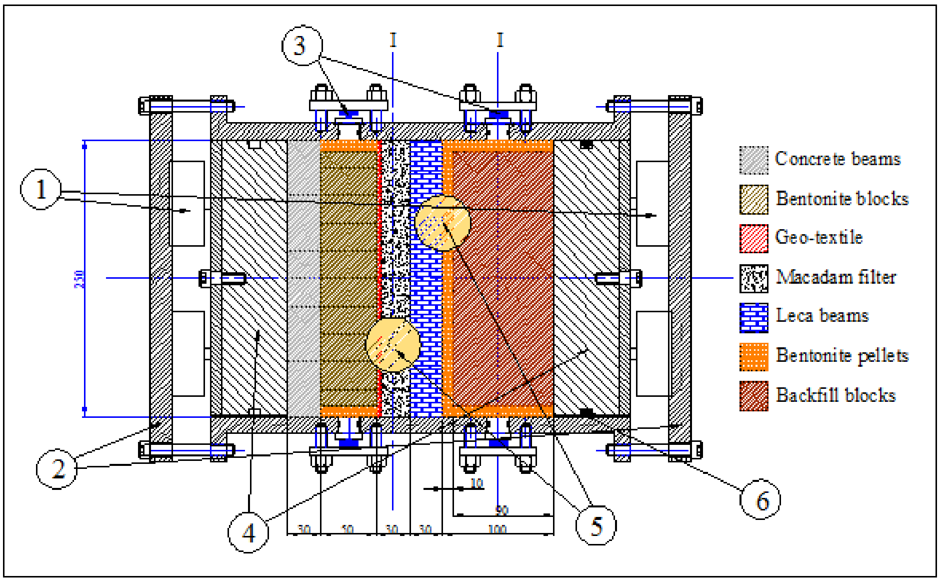


Figure 4.6: The DOMPLU experiment scale model test equipment (top), and a cross section through the scale model (bottom).

4.3.2 Analytical and Numerical Modelling

Analytical and numerical modelling work was undertaken to support the dimensioning of the experiment and to predict the hydro-mechanical and thermal behaviour of the plug components. The modelling work was extensive and performed outside of the scope of the DOPAS project.

The analyses and evaluation of the response of the bentonite seal and the filter are summarised in Börgesson *et al.* (2015). Calculations feeding into the design choices for the filter and bentonite seal as well as different configurations of the plug system were carried out. Modelling of the hydro-mechanical behaviour of the whole plug was also carried out in order to predict the real behaviour of the plug system before deciding the design of the full-scale test. The aim of these calculations was to identify an approach to minimise the swelling pressure from the bentonite backfill in order to reduce displacements in the plug system. The aim was to reduce the swelling pressure from 6 MPa (the pressure expected from the backfill) to 2 MPa. The main results from the numerical models concerned the stress on the plug, the stress distribution along the tunnel axis; the displacements of the different plug components; the extent of homogenisation; and the timescales of hydration. All numerical analyses of the plug components, except the concrete dome, were performed using the Code_Bright modelling tool based on the finite element method.

Grahm *et al.* (2015) presents the numerical analyses performed to predict the thermal and structural response of the concrete dome. A number of numerical models were developed and analysed during both the design of the concrete dome and in preparation of the actual full-scale test. The aim was to obtain a better understanding of the response of the concrete dome, both in the early stages of the project including casting, curing and other preparations before the full-scale test programme and during the test programme.

The extensive and detailed modelling results, i.e., complete predictions of the full-scale test behaviour available in Börgesson *et al.* (2015) and Grahm *et al.* (2015) respectively, will be used to compare the results from monitoring of the performance of the DOMPLU experiment.

4.4 Siting of the DOMPLU Experiment

During initial planning of the project, a set of requirements was developed in order to find a feasible test site for the DOMPLU experiment. It was decided that a purpose-built experiment tunnel should be exclusively reserved for the experiment since sensitive monitoring and high water pressures would be used.

In practical terms, a requirement was established to install the DOMPLU experiment in the crystalline rock at -450 m below the ground surface, where a groundwater pressure of about 3.2 MPa could be expected. At this depth, conditions would be as close to the final repository as is feasible within the Äspö HRL and would facilitate pressurisation more readily than shallower depths.

A preferred location for the DOMPLU experiment niche at the lowest level of the Äspö HRL was identified, but to be accepted as the location for the experiment, confirmation that the site met the rock requirements was needed. These requirements included:

- No continuous fractures longer than the tunnel diameter in the experiment niche.
- No fractures, longer than 1 m, or water-bearing features at the plug location.
- No fractures with an angle of incidence less than 30º against the tunnel axis in the location of the concrete dome.

These requirements ensure that the desired boundary conditions for the DOMPLU experiment are met. The shortest possible test tunnel was estimated to be 13.5 m. The length of the first pilot borehole was 30 m.

The feasibility of the preferred experiment tunnel location was tested against these requirements by drilling a core using a single pilot borehole followed by high-pressure stepwise injection tests in the borehole. The results of the core logging and hydraulic testing indicated that the rock had no fractures of the type specified in the rock requirements.

Examination of the borehole indicated that the preferred location was acceptable, and that the most suitable geological section for the DOMPLU experiment construction was between approximately 7 and 11 m depth from the tunnel entrance.

4.5 Excavation of the DOMPLU Experiment Niche

Drill and blast techniques were used to excavate the experiment tunnel. It was decided to attempt to reduce the magnitude of the EDZ through careful blasting in a two-step excavation method. In the two-step method, first, holes in the middle of the tunnel are drilled and blasted before perimeter holes around the tunnel are blasted. By using this approach, the risk for any continuous excavation-induced EDZ-fractures is minimised. As a consequence, the test tunnel was considered to provide good conditions for the experiment following construction. A picture of the fully excavated tunnel is shown in Figure 4.7. The tunnel cross-section geometry conforms to the reference KBS-3V deposition tunnel (SKB, 2010b).

Figure 4.7: The fully excavated DOMPLU experiment tunnel.

Based on the hydraulic testing in the pilot borehole, a decision was made to place the slot for the concrete dome between 7.2 m (downstream) and 10.4 m (upstream) from the borehole casing. Since the total length of the DOMPLU experiment, including the bentonite seal, filter section and the backfill transition zone, was designed as a 6.5-m-long installation, the total tunnel length to be excavated was determined to be 13.7 m.

The octagonal slot for the concrete dome plug was excavated by use of a wire sawing technique. Wire sawing was selected for the following reasons:

- Wire sawing might allow more rapid excavation than drill and blast or grinding techniques.
- The use of wire sawing might allow the slot to be excavated without the development of an EDZ.

- Wire sawing will result in a smooth surface that should facilitate release of the concrete dome from the rock during early shrinkage and during pre-stressing of the concrete prior to contact grouting.

The wire sawing technique was tested and demonstrated in a separate test prior to the DOMPLU experiment (Grahm and Karlzén, 2015). The purpose with this initial study test was to demonstrate feasibility and evaluate different ways of drilling holes and arranging the diamond wire for cutting. For instance both blind cuts and pulling cuts were performed in this pilot field study.

To protect workers during the full-scale excavation of the concrete dome slot, safety scaffolding was used and dimensioned so that it would be able to support big rock segments cut free by wire sawing. After the rock was removed and the octagonal slot had been cleaned, laser scanning was performed to measure deviations on the surfaces compared to the theoretical geometry. In general, the measurements showed that the wire had been centred with quite good precision in the boreholes in order to stay within this range when sawing began.

Two deviations were noted near the tunnel wall in which the cut surfaces were 7 cm and 6 cm deeper than the theoretical surface respectively. These deviations are thought to result from initial problems with a clamped wire.

Other deviations were measured at the end of some cuts, near the intersection of the slot. According to Grahm and Karlzén (2015), a main finding from the measurement results was that the excavated surfaces were deeper than the theoretical plane near the intersection point for eight of the sixteen performed cuts. This is believed to be a consequence of stresses in the rock. It can be noted that when the first cut on the downstream side for each segment had been made, the subsequent cut from upstream side was performed closer to the target cut line. This is likely to be an effect of released stresses in the rock as a result of the nearby cuts.

An evaluation of the surface deviations showed that there was no risk for reduced stability of the concrete dome. Calculations confirmed that the octagonal shape of the slot is tolerant of small deviations.

A picture of the excavated slot abutment for the concrete dome is shown in Figure 4.8. As can be noted by the dark fields in the excavated slot, two of the boreholes for the diamond wire hit a water bearing structure. Consequently the requirement for no water bearing fractures at the plug location could not be fulfilled. This result needs to be considered when evaluating the final results from DOMPLU (see further discussion in DOPAS 2016b).

In conjunction with excavation of the experiment tunnel, an existing niche in the main tunnel was expanded to accommodate the pressurisation system and a water tank for supply of water. The niche also included a data cabinet to collect the signals from sensors in the backfill, filter and bentonite seal. Three 21-m-long holes were drilled for the purpose of leading cables and pipes between the experiment niche and the monitoring niche. The lead-through pipes and sensor cables were sealed to withstand high water pressures.

Further information on the excavation of the DOMPLU experiment niche is provided in Grahm *et al*. (2015).

Figure 4.8: The excavated slot abutment for the concrete dome.

4.6 Installation of the DOMPLU Experiment

All installed components of the plug system in the DOMPLU experiment are illustrated in Figure 4.1. Some components of the experiment were installed separately while others were installed in parallel with assembly of the components progressing vertically. The stages in installation were as follows:

- The concrete back wall.
- The bentonite backfill (transition zone) and LECA® beams inner delimiter.
- The gravel filter and geotextile delimiter together with the bentonite seal and concrete beams outer delimiter.
- The concrete dome.
- The contact grouting (not shown in Figure 4.1).
- The plastic sheet (not shown in Figure 4.1).
- The monitoring system was installed in parallel with installation of the components but is discussed separately.

The different stages in DOMPLU experiment installation are described below and shown in Figure 4.9. Further information on the installation of the DOMPLU experiment is provided in Grahm *et al*. (2015). Three components, the concrete back wall supporting the backfill zone, the LECA® delimiter, and the front delimiter supporting the concrete dome, were given fixed coordinates prior to installation. Other components were emplaced consistent with the locations of these fixed elements.

The concrete back wall during formwork removal after casting.

Parallel installation of backfill bentonite blocks, pellets and LECA® beams. Sensors to measure displacements are also shown.

Parallel installation of filter gravel, geotextile delimiter, bentonite sea and outer concrete beam delimiter.

Installed cooling pipes with supporting steel frame inside the dome.

Completed plug after dome casting.

Installed plastic sheet to prevent water evaporation.

Figure 4.9: The different stages in the DOMPLU experiment installation.

The installation of the plug, from installation of the backfill blocks to casting of the concrete dome, and including installation of the monitoring system, took about two months. This period does not include the time required for excavating the tunnel, excavating the slot abutment, casting the concrete back wall, drilling lead-through boreholes and other preparations such as concreting of the remaining boreholes from wire sawing and casting of plinths for the LECA® delimiter and the concrete delimiter.

4.6.1 Concrete Back Wall

The concrete back wall was constructed using the low-pH SCC B200 mix. It was casted without reinforcement, i.e., the same procedure as for the concrete dome. The reasons to cast this wall were to achieve a controlled back surface of the tunnel and to get the correct depth of the tunnel needed for the experiment. The casting of the back wall was also a practice for the casting of the concrete dome. Temperature sensors were installed inside the concrete back wall in order to measure the heat generated due to hydration of concrete. The pressure on the formwork was also recorded.

4.6.2 Backfill Transition Zone and LECA® Beams

The backfill transition zone consists of bentonite blocks and pellets. The backfill blocks used for the DOMPLU experiment were manufactured of bentonite from Ashapura, India. These backfill blocks for the transition zone had been manufactured in brick size (300 x 150 x 75 mm) and could thus be easily stacked by hand. The total length of the backfill section included in the full-scale test was 100 cm. The design of the backfill transition zone for the DOMPLU experiment, aiming for 2 MPa of swelling pressure, also included 15 cm pellets between the backfill blocks and the LECA® filter.

The installation started with emplacement of the first LECA® beam, between the backfill and the filter, followed by emplacement of a 10-cm-thick layer of bentonite pellets on the tunnel floor. Subsequently, bentonite blocks were stacked by hand. A band saw was placed close to the experiment location which was used to cut some of the blocks for optimal fitting in the horse-shoe-shaped tunnel.

The gap between the bentonite blocks and the rock was also filled with bentonite pellets. The installation of bentonite blocks and pellets was performed in parallel to installation of the LECA® beams. The installation was performed in stages where bentonite blocks were stacked up to a certain level, LECA® beams were then stacked to the same level and then the slots were filled with pellets. The LECA® beams were piled on each other and plastered together in order to prevent concentrated leakage of bentonite fines between them. Temporary steel brackets were used to lock each LECA® beam to the rock wall, to assure worker safety during ongoing installation. These brackets were later removed to allow some filter displacement to take place during pressurisation, due to the bentonite swelling.

At the tunnel roof, the pellets were installed using a shotcrete machine to fill the void between backfill blocks and the rock wall. In order to fill up the last volume, a few litres of soaked pellets were used to prevent them from falling out.

4.6.3 Filter, Bentonite Seal and Outer Delimiter

The filter component of the DOMPLU experiment consists of two material layers, 30-cm-thick LECA® beams and a 30-cm layer of gravel with grain sizes of 2-4 mm. The LECA® beams were included in the filter to act as a delimiter for the bentonite backfill blocks and to carry the load from bentonite blocks during installation. The LECA® beams were installed in parallel with the backfill bentonite block installation as described above. The installation of the gravel section of the filter was performed in parallel with installation of the bentonite seal and outermost delimiter of concrete beams. A geotextile delimiter was also installed to separate the gravel from the bentonite seal. In addition, two drainage pipes were installed from the gravel filter. These pipes pass through the bentonite seal and the concrete dome and drain groundwater during construction.

The bentonite seal consists of MX-80 bentonite blocks, compacted by uniaxial compression to a size of 500 x 571 x 300 mm. The 300 mm design height of a seal block was chosen to comply with the 600 mm height of a concrete beam. The total design depth of the bentonite seal for the DOMPLU experiment was 500 mm. The bentonite blocks were compacted with a pressure of 25 MPa and had a water content of 17% with a resulting dry density of about 1700 kg/m^3 (as described in Section 4.2.2).

The slot between the bentonite blocks and the rock was manually filled with bentonite pellets. The outermost concrete beams were placed at the same time as the bentonite seal. Concrete was cast to fill the remaining gap between the rock wall and the concrete beam ends to restrict leakages of bentonite and concrete. Two layers of geotextile were installed at the downstream face of the concrete beams to prevent a bond forming between the concrete dome and the concrete delimiter and, thereby to allow the dome to move freely during shrinkage.

4.6.4 Concrete Dome

Before casting the concrete dome, cooling pipes were placed inside the dome. An advanced cooling scheme was used to control the temperatures in the dome at different stages:

1. To reduce the effect of hydration heat.
2. Cooling of the dome in order to force it to release from the rock.
3. Additional cooling of the dome prior to contact grouting.

A formwork was designed in order to cast the concrete dome. A low rate of casting was used, defined as less than one meter per hour. This resulted in the concrete curing earlier in the bottom and thereby developing some strength by the time the last batch of concrete was cast at the top.

The concrete properties were measured during casting of the full-scale DOMPLU experiment, including measurements of the slump flow, air content and density. The properties of the fresh low-pH self-compacting concrete were controllable and batches which satisfied the requirements were pumped into the formwork. The concrete temperature was consistent with the target properties for fresh concrete. For all but one of the concrete batches, the slump flow was either according to specifications or could be adjusted to satisfy the requirements. For the other batch, the flow of the concrete was judged to be acceptable despite a slump flow of just 660 mm (target value was 700±30 mm). This concrete was used in the base of the concrete dome and would have been compacted by the weight of concrete poured on top of it. Batches that had to be adjusted had lower slump flow than required. This is preferred compared to batches being received at the site with higher slump flow than required as addition of superplasticiser to increase the slump flow is a relatively easy process. Had the slump flow been too high, the only options would have been to delay the pouring of the concrete or to reject the non-compliant batch.

Due to a problem with the used superplasticiser, the air content of the low-pH self-compacted concrete was higher than the air content tested previously in laboratory and factory tests. In agreement with the structural designers, a decision was made to cast the dome plug with the high air content concrete. The high air content will result in the mechanical parameters of the hardened low-pH SCC to differ from the pretested parameters. Sufficient numbers of samples were prepared to determine all relevant affected concrete properties. The alternative would have been to terminate the pour, identify the exact reason for the high air content, and most likely wait for a delivery of new superplasticiser and try again. Due to the risk of swelling bentonite in the backfill which might jeopardise the whole experiment, the former approach was taken.

The intensive test programme of fresh concrete properties ensured that concrete of sufficient quality was pumped into the formwork. Some adjustments were required, but no concrete batches had to be rejected.

4.6.5 Contact Grouting

Grouting pipes were used to perform contact grouting at the interface between the rock and the concrete dome. The purpose of the contact grouting was to seal gaps that may have occurred between the concrete dome plug and the surrounding rock. During the first 90 days, significant autogenous shrinkage of the concrete dome occurred. In addition, the thermal cooling of the concrete dome prior to contact grouting also caused it to contract. Both of these effects forced the concrete dome to release from the rock. Contact grouting was performed in three stages. In the first stage, contact grouting was performed with the inner tube, located on the upstream side of the dome. In the second stage, contact grouting was performed with the outer tube, located on the downstream side of the plug. The third and final grouting stage was performed with the centre tube.

4.6.6 Plastic Sheet

The final step in construction of the plug involved covering the downstream side of the concrete dome with a plastic sheet connected to a weir for accurate leakage collection and measurement. The sheet prevents water from evaporating from the dome due to ventilation. The tunnel ventilation system in the main tunnel was subsequently shut off as the monitoring phase began.

4.6.7 Monitoring System

The sensors were installed in parallel with installation of each plug component. A permanent measurement system was used to monitor the concrete dome; it consisted of sensors cast in the concrete and sensors connected to the edge of the concrete dome. Due to the symmetry of the dome, effects from, for instance, shrinkage, water pressure, etc. are expected to give the same results if they had been placed in any of the four quarters of the dome at the same depth and radius. However, due to gravity, the largest stresses are expected in the upper half of the dome if a bond between concrete and rock is present. Therefore, the instrumentation of the concrete dome was primarily made in one quarter (in the upper half) of the concrete dome. In order to obtain redundancy in the measuring set-up, sensors were placed at a similar radius and depth and sensors from different manufacturers were used (*Grahm et al.*, 2015).

The objectives of the instrumentation of the concrete dome are to:

- Measure the response of the concrete dome from the point of casting the concrete up to the point when it is subjected to high loads due to water pressure.
- Determine if the concrete dome releases from the rock caused by early age shrinkage.
- Measure the heat development and cooling of the concrete dome.
- Support comparison of the performance of the concrete dome with the predicted performance.

In the formwork, five pressure sensors were installed to measure the pressure on the formwork generated by the concrete dome.

For the bentonite seal and backfill, the sensors were installed in nine sections, where four were in the bentonite seal and five in the backfill. In addition, two pore pressure sensors were also installed in the excavated slot for the concrete dome. The objectives of the measurements in the backfill and bentonite seal are to:

- Follow the saturation process of the bentonite.
- Monitor the development and distribution of swelling pressure.
- Evaluate the sealing function.
- Detect displacements between the material zones.

All sensors that were installed in the bentonite backfill and in the bentonite seal were subjected to high water pressures. Therefore, these sensors and cables were designed to withstand a hydrostatic water pressure of at least 10 MPa.

Different approaches were used to route the cables, connected to the installed sensors, to the measuring cabinets. The approach used for each sensor depended on where in the plug system the sensors were installed. The aim was to minimise disturbances that may affect the experiment to as little as possible. All sensors installed on the upstream side of the bentonite seal (sensors that will be subjected to high water pressures), were routed from the experimental tunnel through two lead-through pipes that were installed in the rock.

In contrast, the sensors installed in the bentonite seal and against the rock in the slot for the concrete dome were drawn to the inside of the concrete dome in a steel pipe lead-through crossing the concrete dome. The reason for this was that it was preferred not to intersect the bentonite seal with cables, which could short-circuit the seal and cause a leakage path through it.

The sensors that were embedded in the concrete dome, did not use any specific lead-through, instead the cables were routed out directly through the downstream side of the concrete dome.

4.7 Lessons Learned from the Design and Installation of the DOMPLU Experiment

One of the main outcomes from the full-scale test was showing that it is possible to build underground the dome plug system. This includes practical aspects of logistics and arranging of parallel construction activities in a tunnel system (Grahm *et al*., 2015).

In the full-scale test, all installations were made manually while the installation of bentonite blocks is planned to be performed with an automated system in the repository. The DOMPLU experiment also included items such as ~100 sensors and a water injection system connected to four lead-though pipes. The time required for installation of a plug in the Spent Fuel Repository will therefore most likely be shorter than in the full-scale test.

The installation of the DOMPLU experiment was in general successful with a few important experiences gained from construction, taken from (Grahm *et al*., 2015) and summarised below.

4.7.1 General Design Process

An extensive materials testing and development programme underpinned the DOMPLU experiment. Significant understanding of material behaviour and scaling of the DOMPLU experiment was provided, for example, by the use of a scaled physical model combined with numerical modelling of the plug behaviour.

4.7.2 Siting

The siting of the DOMPLU experiment followed a standard procedure for characterisation of deposition tunnels prior to their excavation. However, for the DOMPLU experiment the slot encountered an unexpected water bearing strata, and this experience will have to be taken into account when developing the construction procedures for excavation of the repository. Water inflow into two of the holes used for wire sawing the plug slot made the work complicated and thus water bearing fractures should always be avoided at the plug locations.

4.7.3 Excavation of the Experimental Niche and Dome Slot

The excavation of the DOMPLU experiment niche used a two-step blasting method. The method was followed successfully in excavation of the niche and no particular problems were encountered. No detrimental impacts owing to the use of this method have been detected to date.

The excavation of the slot by wire sawing was more problematic than expected. The method with blind cuts was however feasible and resulted in smooth surfaces but with unwanted deviations in planarity for a few cuts. Further practical improvements can be introduced for future excavations, for example release of rock stress prior to cutting and developing more productive methods to drill holes for the guide pulleys. There is also a need for site-specific testing of excavation at the Forsmark site, where rock stresses will be even higher than in the Äspö HRL. There is a need to define new requirements on fracture-free rock and flat surfaces, potentially free from continuous EDZ. Safety aspects, to protect workers from the risk of falling rocks, will also need continued focus.

4.7.4 Installation Method

It was found that installation of the concrete beams, filter and seal pellets, near the tunnel ceiling, was quite difficult. For instance, it was not possible to achieve a vertical wall of geotextile and gravel; instead it was decided to fold in the geotextile over the gravel which resulted in an empty space at the uppermost 0.5 m. This space was later filled with bentonite pellets that were blown in over the bentonite seal blocks.

Detailed methods and instructions for installation should be produced during detailed design. The worker's safety aspect of working at heights, for instance, during installation of cooling pipes where a sky-lift could not easily be used, should also be studied further.

4.7.5 Formwork

The formwork was judged by the staff to be both solid and well designed. A modification with a man-hole was suggested by the workers for easier positioning of the larger upper parts. Possibly the upper part of the formwork could also be re-designed as one piece to facilitate fitting. The pressure measurements during casting showed that a future formwork can be built with thinner dimensions. An improved routine should also be developed for positioning of the formwork frame, especially regarding the extension with fitting pieces to the rock.

4.7.6 Casting of the concrete dome

The concrete control station and the organisation at the test site functioned well. The formwork was tight and not more than 0.2 litres of concrete escaped to the front. For the purpose of risk mitigation, a redundant concrete pump was available at the site but it was never needed.

The concrete used was good quality and its behaviour was as expected of the B200 concrete. Measurements of concrete properties (e.g. compressive strength) were shown to be consistent with on-site measurement of test blocks. An exception to the initial expectations was that the concrete was found to have high air content (6-9%) at delivery which was probably related to the mixing of additives and the long transport. Another exception was the lower amount of autogenous shrinkage, which likely was caused by the higher air content (see DOPAS (2016b) for further discussion).

The design work underpinning the DOMPLU experiment, and the subsequent construction of the concrete dome, has built confidence that the concrete dome can be constructed with sufficient strength without the need for reinforcement. Further information on the concrete performance is being collected during the pressurisation of the DOMPLU experiment and will be reported in DOPAS (2016b).

4.7.7 Cooling

Installation of the prefabricated parts of the cooling system was successful but was challenging to accomplish in full compliance with the drawings. There was a need for a bigger tolerance (±10 cm) near the intersection points of the slot since it was hard for the technicians to reach these areas.

The cooling procedure was successful (see DOPAS (2016b) for further discussion) and can be specified in detail for future plugs. The redundant cooling machine was never needed but it is considered important to have a spare cooler on site in case the operating machine fails.

4.7.8 Contact grouting

The installation of grouting tubes was time consuming and took almost two weeks. For the upper parts of the slot, work had to be performed from a sky-lift. However, installation of grouting tubes in the lower part of the slot was even more difficult because the workers tend to slide downward on the steep and smooth rock surface.

The contact grouting procedure, in combination with cooling, worked as intended and the leakage monitoring indicated that the grout made the plug watertight enough for the initial pressure increments.

5. POPLU Experiment

This chapter provides a summary of the learning from design and construction of the POPLU experiment.

- In Section 5.1, the background to the experiment and its objectives are summarised.
- In Section 5.2, the testing and selection of materials prior to POPLU implementation are described.
- In Section 5.3, the design work undertaken as part of the DOPAS project is summarised.
- In Section 5.4, the siting of the POPLU experiment is explained.
- In Section 5.5, the construction of the POPLU niche and the adjacent monitoring niche is described.
- In Section 5.6, the installation of the POPLU experiment components is summarised, with particular focus on the novel aspects of the experiment.
- Discussion of the lessons learned regarding the design and installation of the POPLU experiment is provided in Section 5.7.

More details and illustrations regarding the POPLU experiment can be found in the POPLU Summary Report (Holt and Koho 2016).

5.1 POPLU Experiment Background and Objective

The reference design for the Posiva deposition tunnel plug is the same as that described for the DOMPLU deposition tunnel plug in Chapter 4. However, the POPLU experiment design is based on an alternative design to that of the dome-shaped design. POPLU is a wedge-shaped low-pH stainless steel-reinforced concrete structure that is cast in place into a slot that has been notched into the EDZ (Haaramo and Lehtonen, 2009). The differences to the reference design arise from a desire to demonstrate performance of a potentially simpler plug design. The current conceptual design of the POPLU wedge plug is illustrated in Figure 5.1. The design consists of a wedge-shaped concrete structure cast directly adjacent to a filter layer in front of a concrete tunnel back wall. The concrete wedge contains grouting tubes and bentonite circular strips (or tapes) at the rock-concrete interface to ensure water-tightness. The steel reinforcement counteracts shrinkage stresses and helps to limit the release of the concrete from the rock. The POPLU experiment wedge plug is significantly longer than the dome plug design implemented in the DOMPLU experiment (6 m, compared to ~1.8 m for the DOMPLU experiment concrete dome), and this provides additional flexibility in constructing the steel reinforcement. In the POPLU experiment, the concrete was cast in two sections rather than one, in order to manage the concrete curing temperatures.

Should the POPLU experiment provide evidence that a potentially simpler concrete structure with fewer components (e.g. no sealing layers as in the reference concept) will perform as required, the plugging process could become more straightforward to implement. For example, a design with fewer components might be easier to construct and to model. Should the POPLU experiment be successful, there may be two options for the deposition tunnel plug available during the implementation stage, and, possibly, the wedge design might replace the dome design as the reference design.

The POPLU experiment is implemented to fulfil the YJH-2012 (Posiva, 2012a) plans to:

- Construct a full-scale deposition tunnel plug.
- Develop the detailed structural design for the deposition tunnel plug, including development of the concrete mix.
- Develop the method for excavation of the deposition tunnel plug location.
- Produce a quality manual for the deposition tunnel plug, including development of quality control practices and risk mitigation measures.
- Develop instrumentation and techniques for monitoring the performance of deposition tunnel plugs (e.g., mechanical load transfer, concrete shrinkage and water tightness), including modelling of the performance.
- Observe and solve practical challenges associated with installation of deposition tunnel plugs prior to repository operation, including, for example, challenges related to health and safety, documentation, quality assurance and practical work procedures.

The schedule of activities undertaken in the POPLU experiment are summarised in Table 5.1. Further information on the POPLU experiment conceptual design and design basis is presented in DOPAS (2016a).

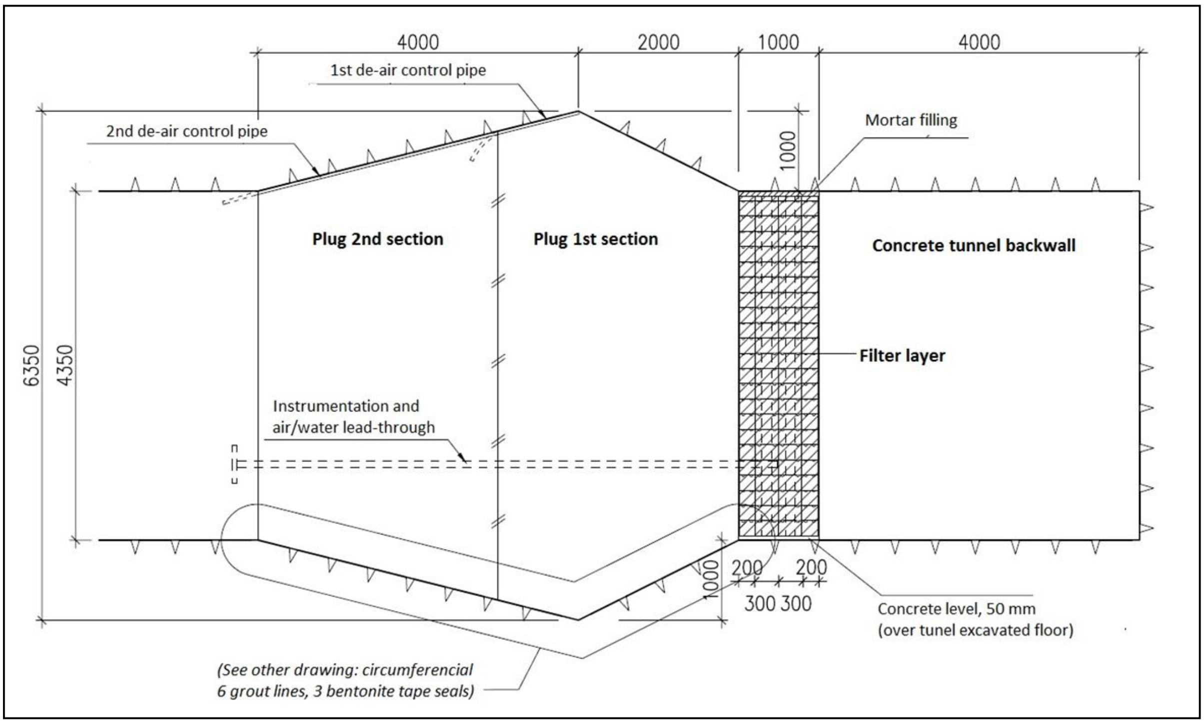


Figure 5.1: Schematic illustration of the POPLU experiment design (Holt, 2014) Dimensions are in mm.

Table 5.1: The schedule of activities undertaken in the POPLU experiment.

Period	Activity
November 2012	Decision to adopt the wedge-shaped plug design rather than a dome-shaped plug
November-February 2012	Pilot boreholes, Rock Suitability Classification (RSC) assessment and excavation planning
December 2012-September 2013	Completing detailed design and instrumentation plans
February-September 2013	Concrete mix performance evaluation
September-December 2013	Tunnel excavation
July 2014-February 2015	Plug Slot excavation
March 2015	Installation of rock lead-throughs
April 2015	Installation of back wall, including formwork, instruments and concrete
May-June 2015	Installation of filter layer and instrumentation, grouting tubes, bentonite tapes, Plug Part One, formwork and reinforcement
15 July 2015	Casting of Concrete Wedge Section One
16 September 2015	Casting of Concrete Wedge Section Two
16-30 December 2015	Grouting of plug/rock interface

5.2 POPLU Material Testing and Development

Extensive testing has been performed within the scope of the POPLU experiment concerning the materials to be used. This includes testing of the low-pH concrete mix, studying the properties of the bentonite sealing material to be used in the bentonite strips around the plug, and materials for the filter layer. All materials used in ONKALO needed to pass a strict "Foreign Material Acceptance" review by Posiva safety experts, so as not to jeopardise the natural condition of the site. This caused some limitations to the type of building and construction materials that could be used in the POPLU experiment.

5.2.1 Selection and Trials of the Concrete Mix

The selection of the low-pH SCC for the POPLU experiment was based on previous experience gained through projects undertaken by Posiva, SKB and Atomic Energy of Canada Limited (AECL). Full results of the concrete mix development and laboratory performance evaluation are provided in Holt (2014).

Posiva's reference low-pH concrete mix is described in the Backfill Production Line reference design (Posiva, 2012b). This reference is similar to the concrete mix (B200) used in the DOMPLU experiment (see Section 4.2.1). The reference mix was the starting point for the concrete mix development in POPLU. In the POPLU experiment, it was necessary to use a different type of superplasticiser to Glenium 51, which is used in the Posiva reference mix. The alternative chosen is a naphthalene based superplasticiser called Pantarhit LK (from Ha-Be Betonchemie GmbH & Co.), which is currently being used for Posiva's low-pH cementitious grouting of rock bolts in ONKALO. The use of a naphthalene-based superplasticiser is preferred over the use of polycarboxylate-based superplasticiser (e.g. Glenium 51 as used in B200) because naphthalene has lower potential for radionuclide sorption than polycarboxylate (Andersson *et al*., 2008). Other modifications to the concrete mix were also required to meet the workability and temperature requirements for the massive concrete wedge.

After initial laboratory testing of the concrete, two mix designs were chosen for large-scale testing, a binary mix with binder composition of 60% cement and 40% silica, and a ternary mix with a binder composition of 38% cement, 32% silica and 30% fly ash.

Compared to the Posiva reference mix, the binary mix was prepared with a lower water/cement ratio, a maximum aggregate size of 32 mm, and a higher superplasticiser dosage. This binary mix had a binder composition equal to the B200 mix; the plasticiser was changed to Pantarhit LK, the limestone filler was replaced with quartz, and the water content was lowered (Holt, 2014).

The ternary mix was a mix design from Aaro Kohonen Oy. Initially, this mix design did not generate a workable concrete without adding more water. Slight mixture changes were made to the aggregate gradation curve to minimise the water content of this mix design so that a workable concrete was obtained. This ternary mix was the one used for the POPLU experiment method tests and plug casting, as it was judged to be the most durable and best at meeting the performance requirements.

Three mock-up tests were carried out with the chosen ternary mix[4]. These mock-up tests were done at the surface and underground at the ONKALO facility site, using the same ready-mix factory supplier that would be used by the POPLU experiment plug construction contractor. The first mock-up test (Figure 5.2) was done in two casts in July 2014.

The main lessons from the first mock-up test were:

- The mass is self-compacting and can fill the upper peak when pumped from below.
- Continuous casting is needed, which means that several trucks had to be available for delivery of the mixed concrete.
- The workability of the concrete needs to be maintained for 30-45 minutes, and may be influenced by the ambient air temperature and dosages of superplasticiser.
- Sampling of each concrete batch for quality control has to be completed before pumping of the previous batch is complete.

Figure 5.2: The POPLU experiment Mock-up Test 1.

The second mock-up test was cast in a cubic box (Figure 5.3) in October 2014. The main lessons from this test were:

- Work techniques still required improvement before use of the concrete mix in the POPLU experiment as the casting of the experiment plug was expected to be demanding, for instance regarding pumping and quality control sampling.
- It was possible to achieve a good workable mix, with sufficient open time prior to stiffening.
- A lubricating mass (i.e. 0.5 m^3 for plug casting) needs to be flushed through the pumping tubes prior to actual plug concreting to ensure good flowability and pipes not getting clogged.

[4] Three mock-up tests focused on the concrete mix and a mock-up test focused on the low-pH injection grout mix were all undertaken outside the scope of the DOPAS Project, but have been discussed here for completeness of the description of the POPLU experiment.

It was also noted that the aggregate, which had a grain size of 32 mm, did not properly penetrate between the spacing of the reinforcement bars. Therefore, a decision was made to use a concrete mix with a smaller maximum aggregate size of 16 mm in the top and bottom of the plug, i.e., the areas where the majority of the reinforcement bars would be situated in the POPLU experiment plug.

The third mock-up was undertaken in February 2015 using the same mould as the first mock-up test, but using the concrete mix with the 16-mm maximum aggregate grain size. Steel reinforcement was used near the top and bottom sections. There were no problems observed during this test, and, therefore, the mix was considered ready for casting in the POPLU experiment tunnel back wall and plug sections.

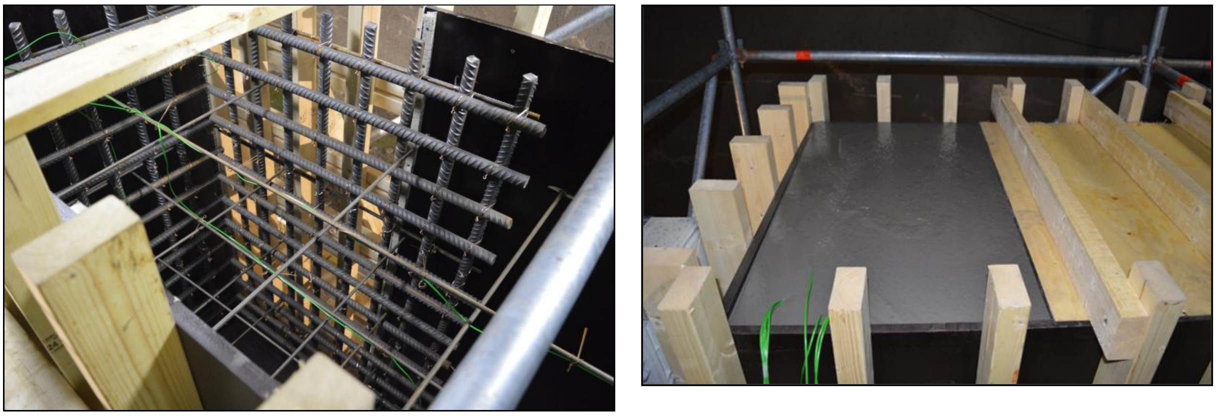


Figure 5.3: The POPLU experiment Mock-up Test 2.

5.2.2 Contact Grouting Mix

Owing to concrete shrinkage, the gap created between the plug and the rock needs to be filled with grout to make the plug as watertight as possible. Grouting of the plug was carried out with cement-based grouting mortar which has a low-pH. The amount of silica fume in the grouting mortar is high and thus a slurry superplasticiser was used to aid workability, instead of powder superplasticiser. Accelerating agents were not included in the grout mix because they restrict the penetrability. The grouting mixes were developed and their performance verified in earlier work, as reported in Raivio and Hansen (2007); and Ranta-Korpi *et al.* (2007). The mix was further adapted in 2015 during the POPLU experiment project, though this work was outside of the scope of the DOPAS project.

A fourth mock-up test was undertaken in November 2015 using the low-pH grout mix prior to contact grouting of the plug. The test arrangement included a reinforced concrete cap cast on the rock tunnel floor in ONKALO. Grouting tubes were led to the interface of the concrete base and rock, which was then contact grouted using the method planned for the concrete wedge. The mock-up showed that the testing arrangement was not properly designed or constructed, because no grout could be injected. Post-test concrete core sampling and analysis showed that the gap was probably too small to allow the grout to infiltrate and that the grouting tubes were clogged with concrete. The tubes were installed as per the contractor's instructions, but would need to be re-designed for future mock-up tests on grout. The work done during this mock-up test was helpful in verifying the grout mix and injection techniques, so the POPLU contact grouting was planned to proceed based on the available work, and no further mock-up tests were undertaken.

5.2.3 Bentonite Tape Materials

The POPLU experiment plug structural design includes bentonite tapes in the gap between tunnel rock walls and the concrete wedge. The bentonite tapes are intended to (i) to support the grouting process by keeping the grout in place during the first low-pressure injections and (ii) to increase the long-term tightness of the plug by preventing seepage through the rock/concrete interface. Six different products/suppliers (Figure 5.4) available in Finland were evaluated to select the most appropriate material for the bentonite tape by assessing the bentonite swelling properties in varying water compositions. The products tested were:

- Meltex Oy: Super Stop.
- Solcon Oy: Bentorub.
- Semtu Oy: PC.
- Betonstrip, Kaitos Oy: Waterstop RX 101.
- Muottikolmio: C
- Muottikolmio: CJTA.

The tapes were tested in VTT laboratories for sealing in three types of water (concrete capillary water, groundwater, and tap water). The swelling was assessed for unconfined samples in these different water types. The swelling was greatest in tap water compared to groundwater or concrete capillary water. The final bentonite tape chosen for use in POPLU was Super Stop (manufactured by RPM/Belgium N.V.), which had a high level of swelling in the first day. This brand was also chosen as it had greatest mass of bentonite within the tape compared to the other products (and thus the lowest amount of foreign materials).

The performance of the bentonite tape sealing layers in contact with the concrete was also assessed in the mock-ups by using microscopy to visually assess samples taken across the bentonite-concrete interface. It was apparent that there was no detrimental reaction between the Super Stop tape and the low-pH concrete in the mock-ups.

Further discussion of the testing and selection of the bentonite tape materials is provided in the POPLU experiment summary report (Holt and Koho 2016).

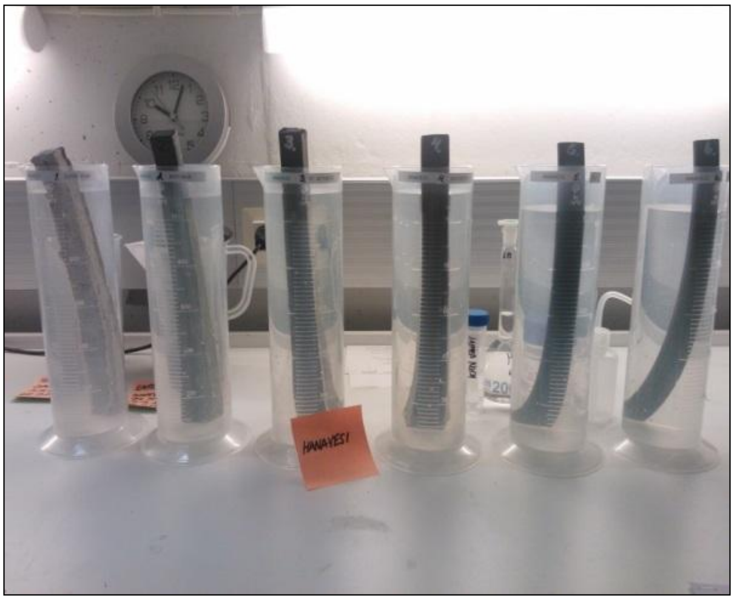


Figure 5.4: Bentonite tape materials tested for use in the POPLU experiment.

5.2.4 Filter Materials

The filter layer was constructed from lightweight concrete blocks, manufactured by Rakennusbetoni- ja Elementti Oy using LECA® with a maximum grain size of 10 mm. The blocks were made using a low-pH paste having binder proportions similar to the plug binary concrete mix design (40% silica fume and 60% cement as the binder, and a maximum binder content of 200 kg/m^3). Preliminary trials were undertaken in the VTT laboratories to determine the mixture proportions and properties such as void content, density, water permeability and compressive strength. Based on the initial laboratory results, manufacturing proceeded to the factory where 1,200 blocks were produced for Posiva in spring 2015. The dimensions of the block (590 mm long x 290 mm wide x 190 mm height) were constrained by the available moulds from the factory.

During installation, the blocks were secured together and to the rock wall with low-pH mortar. The mortar had a maximum paste binder content of 200 kg/m^3 and a ratio of 40% silica fume to 60% cement. The aggregate was natural Finnish sand with a maximum diameter of 1 mm. The first layer of filter blocks was secured on a footing, also made of low-pH mortar approximately 1.2-m long and 2 to 10-mm thick depending on the tunnel floor roughness.

Further discussion of the testing and selection of the filter materials is provided in the POPLU experiment summary report (Holt and Koho 2016).

Figure 5.5: Low-pH LECA® blocks used for the filter layer in the POPLU experiment.

5.3 Development of the POPLU Structural Design

The design work undertaken for the POPLU experiment included use of analytical and numerical calculations to underpin the design and identify the required reinforcement on the wedge plug:

- Modelling of static stress in the concrete wedge resulting from the emplacement of the concrete and its curing (Section 5.3.1).
- Modelling of the dynamic stress in the concrete wedge and surrounding rock resulting from pressurisation of the POPLU experiment (Section 5.3.2).
- Modelling of the hydraulic performance of the POPLU experiment (Section 5.3.3).

The results of the structural and mechanical integrity modelling were used for design of reinforcement, planning of the mock-ups, and selection and placement of monitoring sensors. The hydraulic modelling results also influenced the selection and design of the bentonite tape sealing strips.

The POPLU experiment was constructed without a bentonite sealing layer behind the concrete (unlike the DOMPLU experiment, which contained a bentonite watertight seal). Inclusion of a bentonite sealing layer behind the concrete in the POPLU experiment was considered during the structural design work. However, numerical modelling of the plug hydraulic performance concluded that the massive concrete structure, together with contact grouting of the concrete-rock interface and the introduction of bentonite tapes around the concrete would be sufficiently watertight during pressurisation. The planning work recognised that the addition of bentonite behind the plug would increase the water-tightness of the POPLU experiment plug design, but building the experiment without a bentonite sealing layer would allow evaluation of the reliability of the concrete wedge specifically. Absence of the bentonite seal would also allow much more rapid pressurisation of the concrete wedge. Further structural design information is documented in Holt and Dunder (2014) and modelling in Rautioaho *et al.* (2016), and a summary of the numerical modelling used to design and evaluate the expected plug performance is provided below.

5.3.1 Static Stress

Static calculations were used to determine the stresses in the plug prior to the selection of the concrete grade and the required reinforcement. The concrete grade and required steel reinforcement are selected based on an understanding of the tension and compression forces in the concrete, the aperture of cracks that form in the concrete and the heat of hydration of the concrete.

The most critical load controlling the minimum reinforcement is the hydration load. Steel reinforcement is used in the plug structure to minimize the crack widths that may result due to loading, thermal gradients and shrinkage of the concrete material. The allowed maximum cracking width is 3 mm. The reinforcement is stainless steel ribbed bar type B600KA2. The amount of steel reinforcement is approximately 20,000 kg (Holt and Dunder, 2014).

The initial POPLU experiment structural calculations were undertaken using Autodesk Simulation Mechanical (www.algor.com) using a linear elastic material model. The modelling results were output in graphical form as displacement and stress plots. The magnitude of the horizontal displacement and the resulting compressive stress perpendicular to the surface in contact with the tunnel in the event of an incomplete bond between concrete and rock were also estimated (Holt and Dunder, 2014).

5.3.2 Dynamic Stress

The next set of structural modelling was done by Pöyry Oy to evaluate the mechanical integrity of the concrete plug together with the behaviour of the surrounding rock mass. This was simulated using the programme 3DEC 5.00 (Itasca Consulting Group, (2012)), which uses a finite-element approach for modelling structures represented as a continuum. Three scenarios were evaluated, which addressed varying degrees of heterogeneous rock. These focused on different fracturing associated with excavation of the POPLU experiment niche. The displacements modelled in these scenarios prior to pressurisation were a maximum of a few millimetres in the rock and less than 1 mm in the concrete wedge, as shown in Figure 5.6 (top figure). Following pressurisation, displacements are expected to be a maximum of 3 mm on the side of the concrete wedge facing pressurisation Figure 5.6 (bottom figure). The conclusions from this modelling were that the concrete wedge can be expected to deform more than the rock in response to pressurisation and that the concrete wedge deformations may be asymmetric due to the heterogeneous nature of the rock mass lithology and structure (Rautioaho *et al.*, 2016).

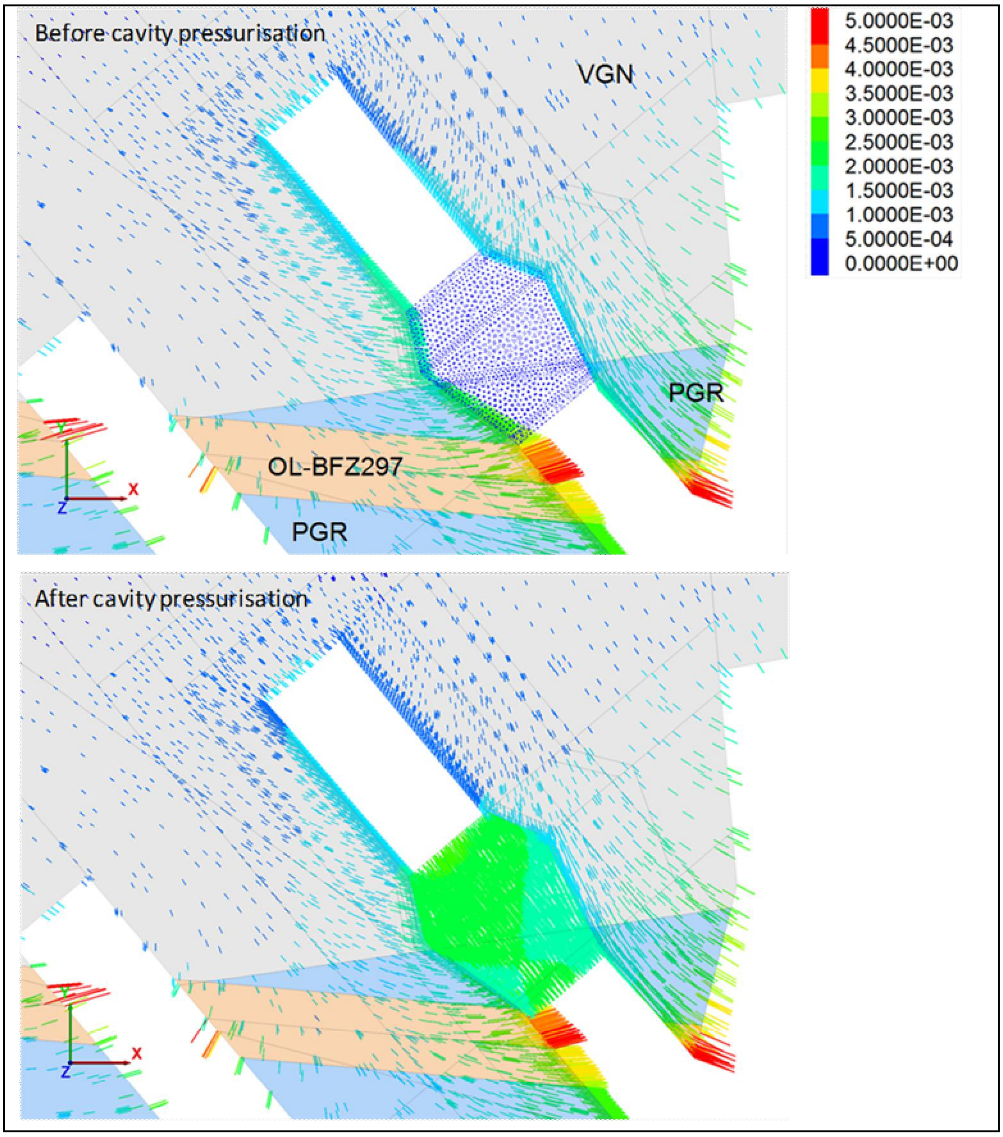


Figure 5.6: An example of the displacement simulations in [m] for scenarios of heterogeneous rock, modelled without (top) and with (bottom) cavity pressurisation to 10 MPa.

5.3.3 Hydraulic Modelling

In the hydraulic modelling, different design options for a bentonite seal were quantitatively evaluated and the flow through a saturated seal at the end of the concrete wedge and the interface between the wedge and the rock were assessed. The objective was to assess the effectiveness and suitability of different design options for the putative seal at the end of the plug, and to support the structural design of the POPLU experiment, specifically when deciding if a layer of bentonite clay blocks would be needed behind the plug to support water tightness (as discussed above).

A three-dimensional hydraulic numerical continuum model was created to simulate the flow through the seal and the gap in contact with the plug, and to predict the total outflow for the different design options and leakage scenarios in fully-saturated, steady-state conditions. The analyses were performed by B+Tech Oy and Clay Technology AB with the finite-element software CODE_BRIGHT (Olivella *et al.*, 1996). The results showed that the outflow from the concrete wedge would be 17 to 1380 times lower when using a bentonite seal than the case without a bentonite seal, depending on the size of defect and type of sealing materials assumed in the calculations (Rautioaho *et al*., 2016). The results of this sensitivity study were used to support a decision that the POPLU experiment would not include a bentonite watertight seal, in order to focus on the performance of the concrete plug.

5.4 Siting of the POPLU Experiment

The plug construction was carried out in the ONKALO demonstration area, at -420 m, where two plug demonstration tunnels were specially constructed for the POPLU experiment (Demonstration Tunnels 3 and 4), northeast of pre-existing tunnels (Demonstration Tunnels 1 and 2) (Figure 5.7). The excavation of the tunnels was completed at the end of 2013. Demonstration Tunnel 4 contains the plug and Demonstration Tunnel 3 contains the monitoring equipment. The tunnel lengths are approximately 21 m and 25 m from the centre line of the central tunnel, respectively. The two tunnels are 4.35 m high and 3.5 m wide, with a cross-sectional area of approximately 14.46 m^2.

To verify the suitability of the plug demonstration tunnels and to select the location for the plug within the tunnels, the RSC methodology developed by Posiva Oy was applied (Kosunen, 2014).

Posiva's RSC methodology comprises a stepwise procedure for identifying bedrock volumes suitable for hosting the various parts of the repository such as, repository panels, deposition tunnels, tunnel plugs, and deposition holes, based on a series of suitability assessments carried out as construction proceeds and increasingly detailed data on the properties of the bedrock is collected through various investigations. The suitability of host rock volumes is evaluated using criteria defined for the different parts of the repository; for example, to be classified as suitable for hosting a deposition tunnel plug, a volume of rock has to fulfil every criterion (concerning host rock properties) set for a deposition tunnel plug. The criteria are based on requirements stemming from aspects of long-term safety, related to the functioning of the bedrock as a natural barrier as well as to ensuring proper conditions for the functioning of the EBS. The criteria pertaining to all underground rooms, deposition tunnels or deposition holes are described in McEwen *et al*. (2012).

The area immediately northeast of the existing demonstration tunnels 1 and 2 (Figure 5.7) was considered a possible location for the POPLU experiment on the basis of investigations and detailed-scale modelling of bedrock structures carried out earlier, during the construction of the two demonstration tunnels. The suitability of the rock in this area was assessed twice using the RSC. The first classification was based on information gained from pilot holes (one drilled for each planned tunnel location) and as a result, the approximate chainage 11-17 m of the planned demonstration tunnel 4 was selected as the primary candidate for the plug location. The second classification was carried out after the excavation of the demonstration tunnel 4 had proceeded past the suggested plug location, and based on the tunnel observations, the suggested location was verified to be suitable for hosting a plug (Kosunen, 2014).

Further information on the siting process for the POPLU experiment, including the application of the RSC is available in Kosunen (2014).

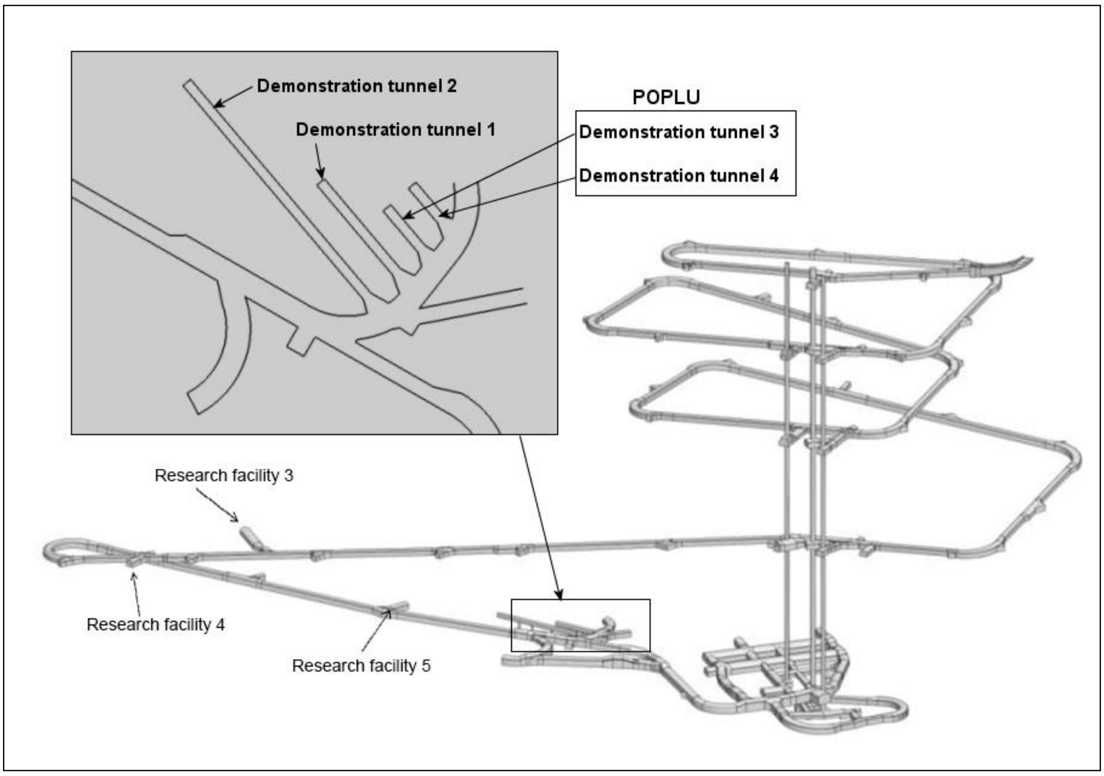


Figure 5.7: The POPLU experiment tunnel locations in ONKALO.

5.5 Excavation of the POPLU Experiment Niche

The selected methods for the concrete wedge slot production were drilling, wedging and grinding. These methods were selected, instead of the originally-planned wire sawing method, owing to operational safety concerns with the best technology that could be demonstrated compared to the costs. It was also deemed beneficial to apply different slot production methods in the POPLU experiment compared to the other DOPAS Project experiments (i.e. hydraulic wedge splitting and pressure disintegration techniques as used in EPSP and wire sawing as used in the DOMPLU experiment) to be able to compare and contrast the methods. Full description of the excavation of the POPLU niche is provided in the POPLU experiment summary report (Holt and Koho 2016).

The POPLU experiment was the first time that control hole drilling, wedging and grinding have been used in ONKALO and, therefore, application of the methods under the site-specific conditions found in ONKALO have not yet been optimised. During the POPLU experiment, improvements in the method were made in relation to:

- The accuracy of drilling angle measurements and drilling depths.
- The practices for ensuring tolerances and excavation quality, which were verified.
- Improvements in estimating the cost of applying the techniques in terms of both time and labour.
- The definition of the requirements for application of the techniques was improved based on feedback from the public procurement of the slot excavation works.

Based on the slot production experience, further work to optimise the method has been identified.

Before excavation, two rock extensometers were installed adjacent to the slot area from Demonstration Tunnel 3 (see Figure 5.7 for the location of Demonstration Tunnel 3). The extensometers were used to evaluate rock movements during excavation. The exact location of the slot was marked by laser scanning and surveying. Excavation was carried out using two construction devices. A Holland excavator with 235 planes was used as the base for the grinding attachment. The excavator boom was modified with a hydraulic extension boom for this purpose. The wedging was done with a Nemek drill boom modified for this work. The modification included adjusting the clamping device and how the drill device was aligned with the drill bar. A Webster TD-140 hydraulic cutting unit was installed at the end of the hydraulic extension boom of the excavator for milling the rock to the final dimensions.

The slot excavation work was undertaken throughout the second half of 2014. First, the sides were excavated, then the floor section and finally the ceiling. The plug area was roughly excavated to within 100 mm of the theoretical dimensions using the drill and wedge method. The control holes were drilled with a milling cutter, with the location of each hole marked on the tunnel walls. The clamping drill was placed on the centre line with an 89 mm drill diameter. The drill holes were spaced 400 mm apart. Initially, on the side walls, the section of rock removed by each drill section was very small and, therefore, the removal of the rock could be done unsupported. Records were kept of the volume of rock removed by drill and wedging. Final dimensions were achieved by grinding of the surface. During excavation, intermediate laser scans were taken to control the dimensions and grinding locations.

After the slot excavation, geological point mapping was done in accordance with Posiva's standard procedure for tunnel documentation. Extra measurements were also done with high-frequency ground penetrating radar (EDZ-GPR) to evaluate the depth, density and connectivity of fractures in the EDZ. These measurements were acquired in three lines along the slot separated by one metre intervals. Figure 5.8 and Figure 5.9 illustrate how the EDZ was mapped around the plug slot area. From these scans, it was possible to assess the hydraulic conductivity risks around the plug area. The information was also incorporated into 3D rock models for application during deposition tunnel plug design work in the future.

Throughout the slot excavation process, safety practices were of highest concern. It was necessary to have tunnel support against ceiling rock falls at all times, so as to not work unprotected in the tunnel. Shotcreting that was in the slot area for tunnel support prior to slot excavation was removed during the process. To ensure minimal fracturing of the tunnel rock, no rock bolts or netting could be used in the excavated slot area to support the rock. The slot excavation was done by using remote controlled equipment so that it was not necessary to access the unreinforced slot area during standard operation. When the slot area needed to be accessed, a crane with a shelter against rock block falls was used. After excavation of the slot, a temporary sheltering system was built from scaffolding for protection during the construction activities.

The plug slot excavation and dimensions were achieved accurately, in accordance with the plan. The surface excavation was done to a high tolerance and was required for acceptance of work completion. The shape of the slot is shown by the laser scanning image in Figure 5.10. It was necessary to have much iteration of laser scanning and additional localised grinding to final dimensions. An example of this iteration is shown by the laser scanning image of Figure 5.10 (with the red dots indicating excavation areas not yet achieving zero tolerance). This frequent reiteration to reach final tolerances took more than one month to complete.

The oversight of the Finnish regulator, STUK, was done during the excavation of the demonstration tunnels. Thus STUK had the authority to review the tunnel excavation plans, the produced tunnels and the generated documentation after the completion of the tunnel excavation. The plug slot excavation, plug construction, and plug testing were classified as R&D work and STUK was kept informed of the planned actions by providing the relevant documents to them beforehand. This did not provide any exceptional work compared to standard ONKALO site practices for Posiva. No issues arose based on the STUK reviews prior to excavation.

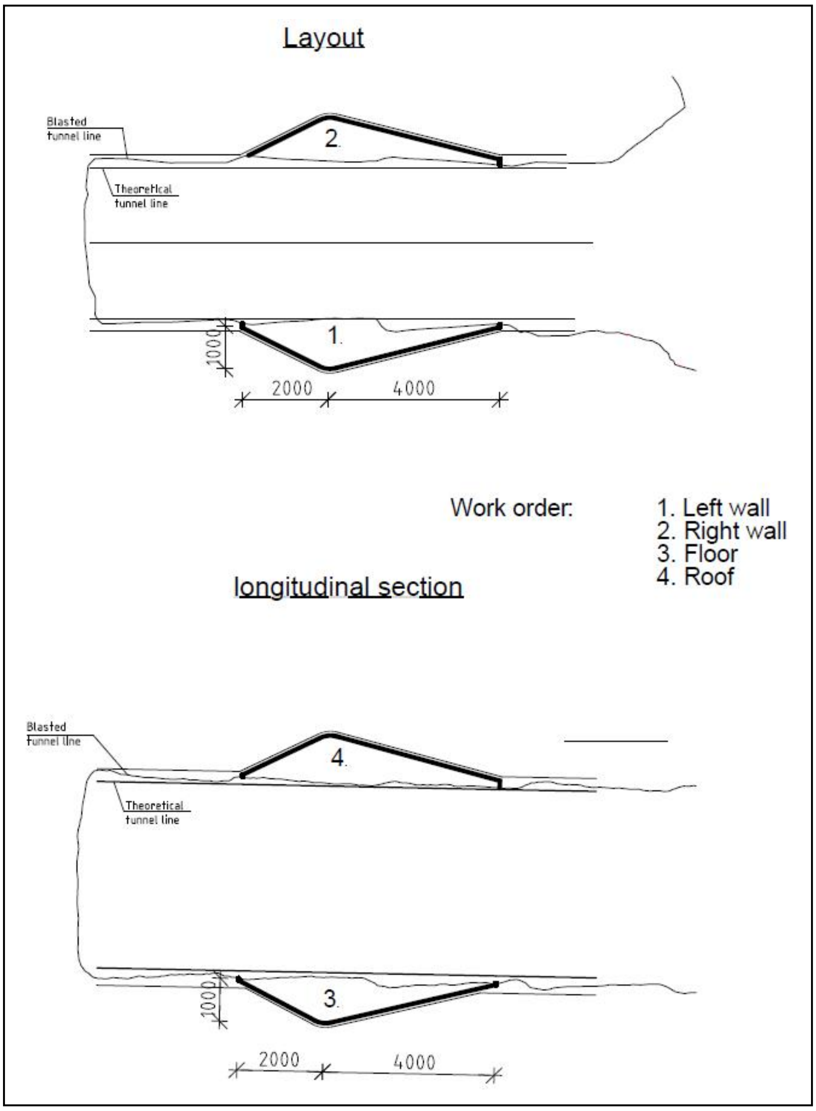


Figure 5.8: Plug slot area and corresponding EDZ removal at edges.

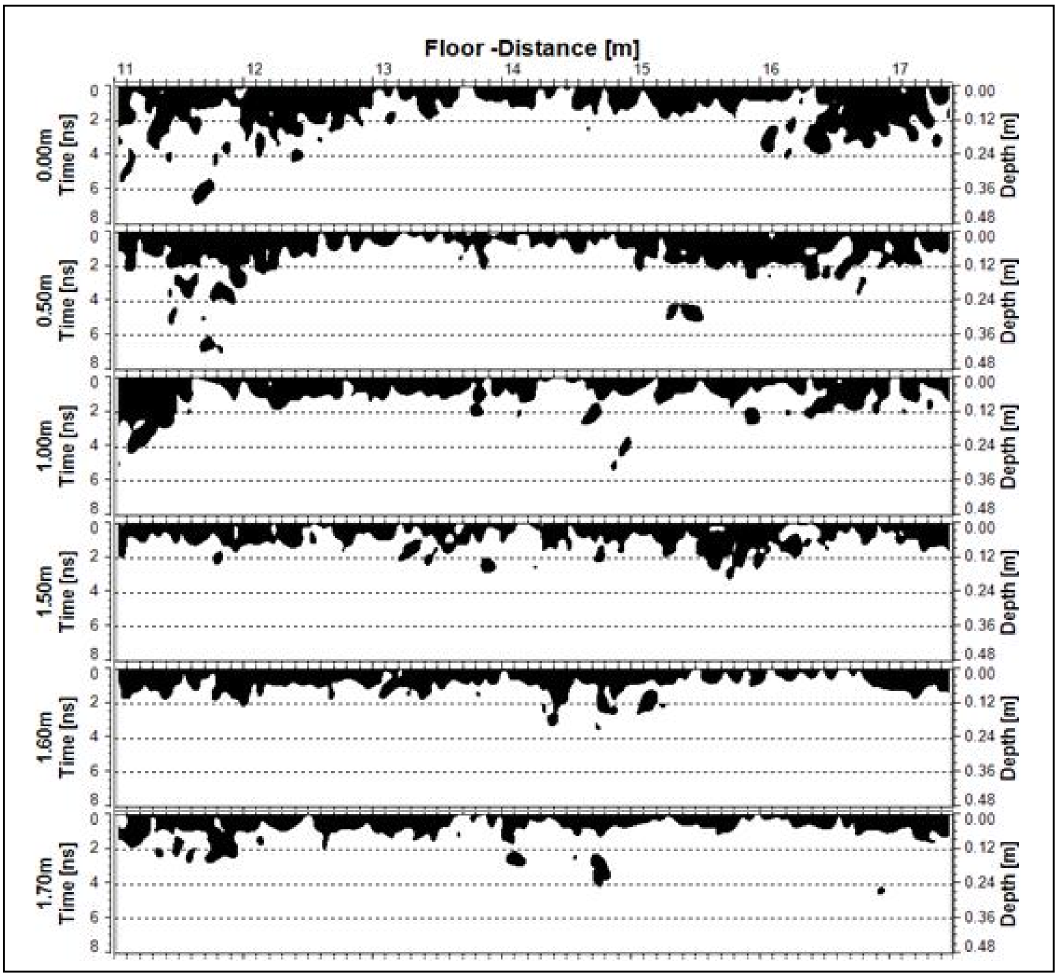


Figure 5.9: Plug slot area and corresponding EDZ removal at edges.

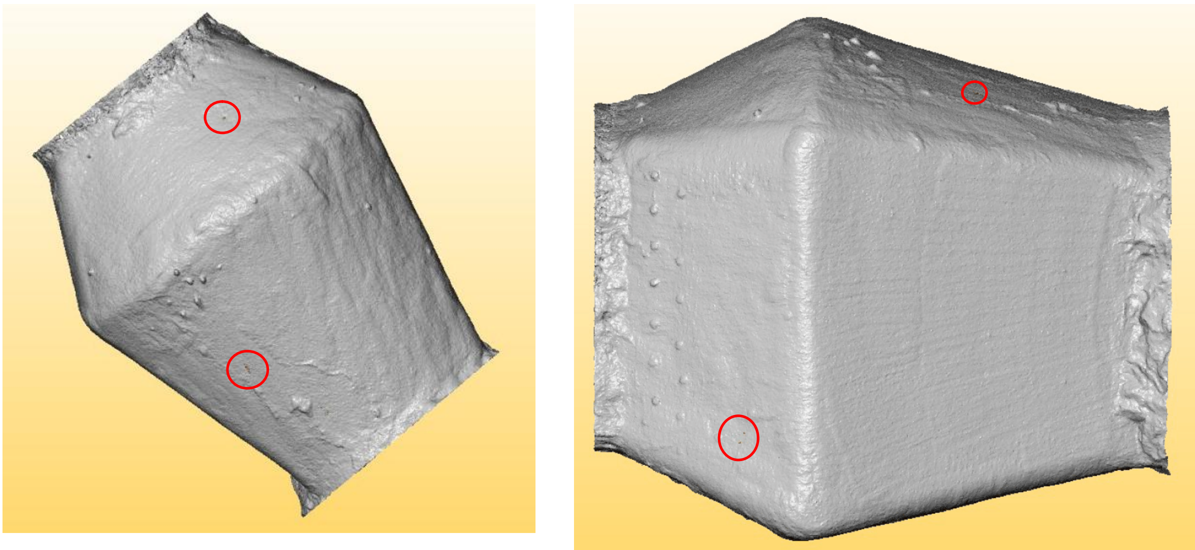


Figure 5.10: Example of laser scanning images showing the shape of the slot. Images of this type were used to assess grinding compared to tolerances during excavation. Red circles indicate areas not yet meeting tolerances.

Some of the key lessons learned during slot excavation included:

- The tolerances defined for the dimensions of the slot excavation may have been stricter than necessary, for instance having an inner tolerance less than the zero line. In the future, the plug design and dimensioning should be completed after the slot excavation, while in this case a preliminary design was made first, which meant that the excavation work had no allowable tolerance because of the shape of the designed support structure.
- The machine working time for the equipment was quite long. The tools should be further improved, for instance for the efficiency of machine grinding and exchange of the booms and/or clamps. There were more equipment breakdowns and resulting delays than expected. The contractor should have enough staff trained in use of the specialty equipment, so that work can progress faster without dependence on only a select few persons. In future operations, it is estimated that the required work time for slot excavation could be reduced to about half (from 6 months to 3 months) based on optimization of the equipment and ease of operations gained with experience, and access to the slot area from both directions of the tunnel.
- The results of the POPLU experiment excavation activities showed that Posiva's reference method of wire sawing may not be the best method from the operational production point of view. Use of wire sawing would need to have development regarding worker safety when conducting excavations of the ceiling and walls, where large rock masses are dropped. Working within the unreinforced tunnel section remains a high concern.

5.6 Installation of the POPLU Experiment

The principal components of the POPLU experiment are illustrated in Figure 5.1. The installation of these components is described below, as follows:

- Section 5.6.1 describes the installation of the rock support.
- Section 5.6.2 describes the installation of the rock lead-throughs.
- Section 5.6.3 describes the installation of the formwork.
- Section 5.6.3 describes the installation of the concrete tunnel back wall.
- Section 5.6.5 describes the installation of the filter layer.
- Section 5.6.6 describes the installation of the concrete wedge, incorporating the reinforcement, the grouting tubes and bentonite tape, the concrete lead-through, and the cast concrete.
- Section 5.6.7 describes the installation of the monitoring system.

Further discussion of the installation of the POPLU experiment is provided in the experiment summary report (Holt and Koho 2016).

5.6.1 Rock Support

The first step in construction of the POPLU experiment was to build a protective shelter and scaffold to protect against falling rock within the slot area of Demonstration Tunnel 4. In February 2015, a temporary sheltering area was built, including a walk-through platform across the plug slot (Figure 5.11). This provided access to the area behind the plug during construction. So as not to create further fractures or impact the EDZ, no rock support, in the form of rock bolts or netting, was allowed in the slot area.

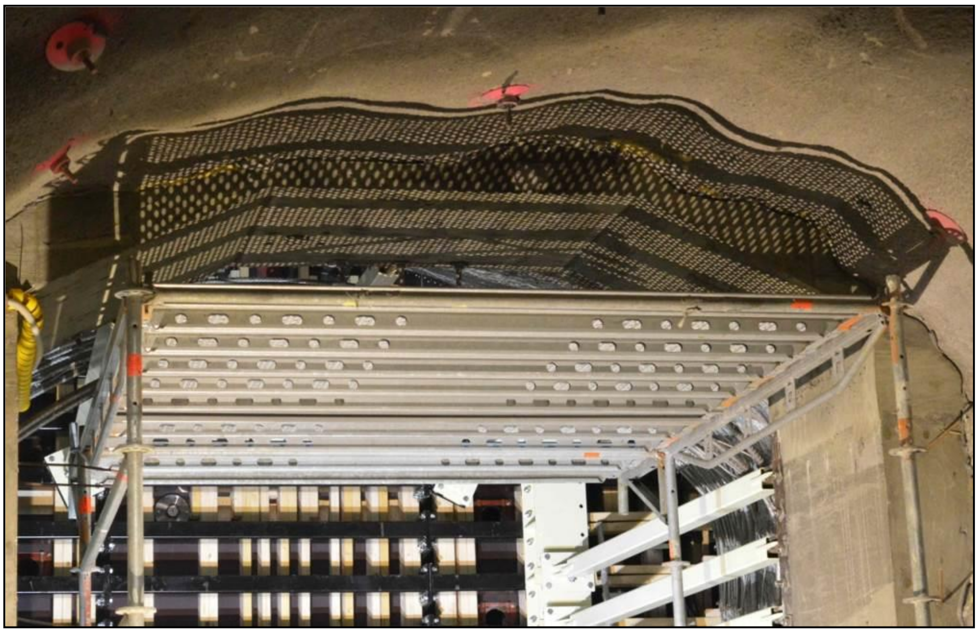


Figure 5.11: Example rock support scaffolding within the slot area.

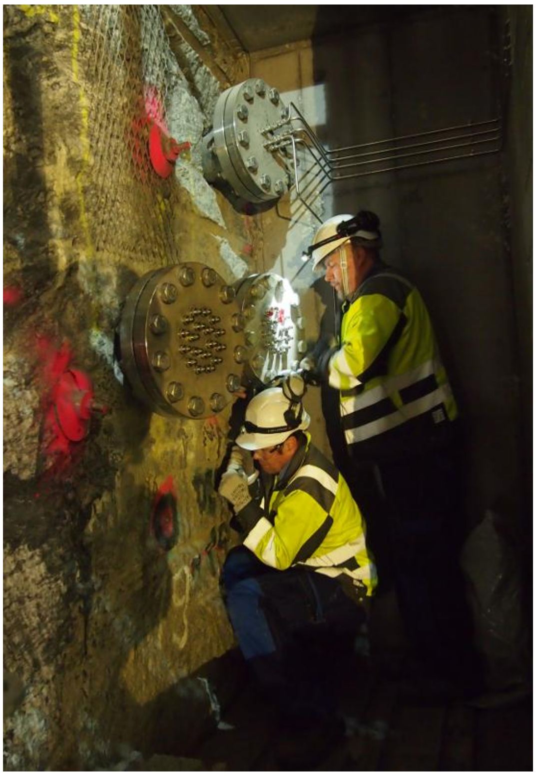
5.6.2 Rock Lead-throughs

The materials for the stainless steel lead-through were ordered by Posiva and took about a month for procurement and delivery of the various parts. TVO quality managers inspected all materials after delivery to ONKALO. All components were then sent to IS Works Oy in Pori, who assembled the various components. This included the flange on each pipe within Demonstration Tunnel 4. All three pipes were designed to be a combination of three sections. Non-destructive testing by x-ray scanning and surface quality assessment was also done at IS Works Oy. The three parts for each lead-through were welded together after delivery back to Olkiluoto. The welding was done underground and took approximately one day per pipe. The welding was again quality control tested upon completion of the welding.

Installation of the rock lead-throughs on-site at ONKALO was undertaken in March 2015. For installation, the first step was to put bentonite blocks around the ends of the pipe, 1 m from the end of each pipe, to be within the rock lead-through and add water tightness. After assembly, each of the three pipes was pushed through the pre-drilled rock holes. Each metal lead-through also had three separate smaller diameter metal tubes (18 mm). Contact grouting was done from both Demonstration Tunnel 3 and Demonstration Tunnel 4 to fill the junction between the lead-through pipe and rock surface. The polyurethane flanges that were originally planned to be used to add water tightness were omitted due to non-acceptance by ONKALO foreign materials review. This aspect could be improved in the future, so that the design is modified earlier to account for acceptable materials. Figure 5.12 illustrates the rock lead-throughs.

The next step was to install the five pressurisation tubes within the lead-through pipes. This work included installing, bending and connecting water pipes and then connecting their nozzles. The average length of each pipe was approximately 35 m (20 m visible in Demonstration Tunnel 3 and 15 m within the structure) from the pressurisation pump system connection to within the filter layer, going via the rock-lead-through pipes.

(a)



(b)

Figure 5.12: Installation of the rock lead-throughs. a) flange with Teflon pipes for shielding, b) closing lead-through flanges.

5.6.3 Formwork

The plug formwork was built by the contractor, starting with building a tunnel circumferential footing or frame along the floor, wall and ceiling. The width was approximately 1 m. Bolts were extruding from the footing, for attachment of jacks and plug formwork steel beams. The formwork was constructed contemporaneously with the rock lead-through installation. Beams were used to hold the actual formwork mould in place, which was attached to a bracing frame (or circumferential footing) in the rock. The bracing pieces were designed to be mostly installed by hand, for instance with three components that could be screwed together. The formwork design was successful and avoided drilling attachments to the rock. The formwork system was heavy and maybe more massive than needed. The design was based on hydrostatic pressure related to the SCC, with the estimated range during casting to be designed for 0.2 MPa at the base to 0.05 MPa at the top. Pressure meters were put on the formwork to observe actual pressure during casting.

5.6.4 Tunnel Back Wall

The tunnel back wall was constructed in April 2015, for shortening the tunnel adjacent to the plug since no backfill clay was used. The back wall had no steel reinforcement, and was cast with the same low-pH concrete as used in the plug. Four L-shaped beams were placed in the back wall for the purpose of attaching instrumentation sensors for performance monitoring. For casting, a small concrete pump was used that fit easily and was manoeuvrable in the tunnel. This same pump was not available for the plug casting, but it performed well and is suitable for use in the future in confined spaces. Approximately 40.5 m^3 of concrete were poured using 11 delivery trucks, over a 6-hour period on 28 April 2015. Quality control included measurements of slump and air content, as well as cube samples made for long-term testing of strength, permeability and pH leachate. The temperature, humidity and displacement of the back wall were monitored for the first few weeks after casting. The formwork was demoulding approximately 10 days after casting. The back wall was then used as an attachment location for water pressurization pipes entering the filter layer.

The primary lessons learned during back wall casting were the practical arrangements to be utilised during the actual plug casting. It was important to practice the formwork erection, monitoring methods, concrete delivery sequence, concrete pumping and quality control methods. The back wall structure would not be used in plugs for an actual repository, but it was beneficial in the POPLU experiment as a learning experience related to installation of the concrete wedge.

5.6.5 Filter Layer

The filter layer construction was started by attaching the pressurisation water pipes to the back wall. The exact locations for the pipes were slightly adjusted and tailored based on the available geometry. The filter layer construction was started by casting 2 to 10-cm thick footing. This footing was made from low-pH mortar, similar to that subsequently used for securing the filter blocks to each other. The mortar was prepared in a mixer on-site underground. After hardening, one layer of lightweight blocks was installed. After installation of the sensors over four days, the filter layer erection continued (Figure 5.13). The filter blocks were cut on-site with a hand saw to match the geometry of the tunnel. The space around the flanges was left free from blocks in an area about 5 cm diameter around the flange (Figure 5.13(b)), which was then filled with LECA® pellets to give flexibility for any movement or displacements, as it was critical that the flanges would not be damaged by displacements during pressurisation.

(a)

(b)

Figure 5.13: Photographs taken during installation of the filter layer, a) against tunnel back wall, b) around rock lead-through flange.

5.6.6 Concrete Wedge Sections

The following sections describe the construction of the concrete wedge, including the reinforcement, grouting tubes and bentonite tape, concrete lead-through and concreting. These are grouped together for both Concrete Wedge Section One (work from mid-May to mid-July 2015) and Concrete Wedge Section Two (work from beginning of August to mid-September 2015), since most of the steps are identical for each section. Differences in the two sections are noted accordingly. Photos of the installation of the concrete wedge are provided in Figure 5.14.

5.6.6.1 Reinforcement Installation, Phase 1

The steel reinforcement of the concrete wedge was designed to serve the secondary function of rock support within the slot area. Therefore, the first step in construction of the concrete wedge was to remove the temporary rock support scaffolding and tunnel floor platform, then proceed with reinforcement installation around the wedge circumference (adjacent to the rock) for both Concrete Wedge Section One and Concrete Wedge Section Two. This reinforcement was installed from the front face (near the central tunnel) working towards the filter layer, so as to protect the workers. About 1 m of empty space, or a gap, remained open around the plug circumference in the middle section of the plug to position the formwork between Concrete Wedge Section One and Concrete Wedge Section Two. This area was covered with a temporary metal mesh for tunnel rock fall protection.

5.6.6.2 Grouting Tubes and Bentonite Tape Installations

The six sets of grouting pipes (three for each section) and three sets of bentonite tape were then attached to the rock surfaces around the slot circumference using stainless steel screws spaced every 200 mm for pipes and 300 mm for bentonite tape.

5.6.6.3 Reinforcement Installation, Phase 2

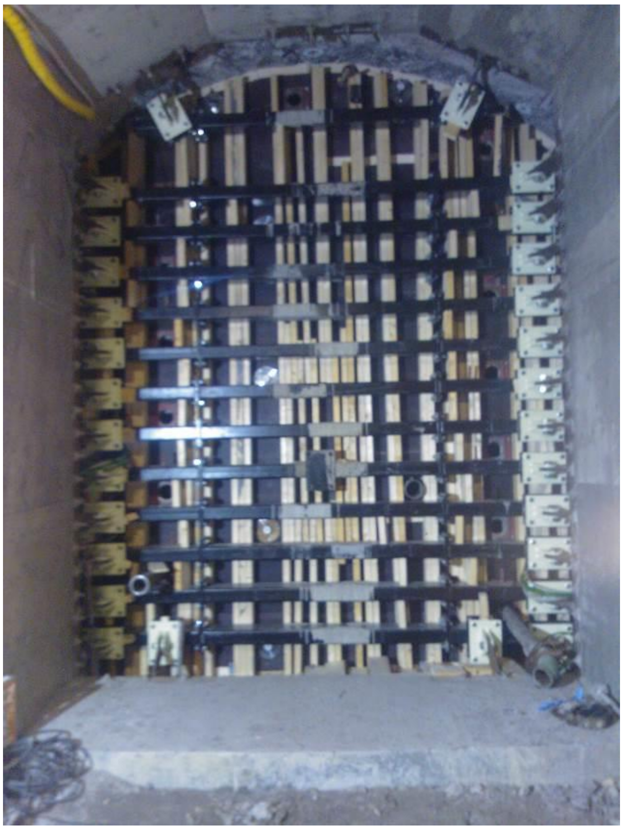
The final sections of reinforcement within Concrete Wedge Section One needed to be placed near the ceiling after the grouting pipe and tape installation. After that point there was no exposure to the tunnel slot rock. Then the next reinforcement was installed for the inner reinforcement and back-section reinforcement in front of the filter layer.

Another subsequent short step was attaching the grouting feed pipes to the circumferential grout tubes. This was done after the reinforcement was ready in both plug sections. The feed pipes were attached for support to the reinforcement bars using stainless steel tie wire. The man-hole in the formwork was then closed in preparation for concreting.

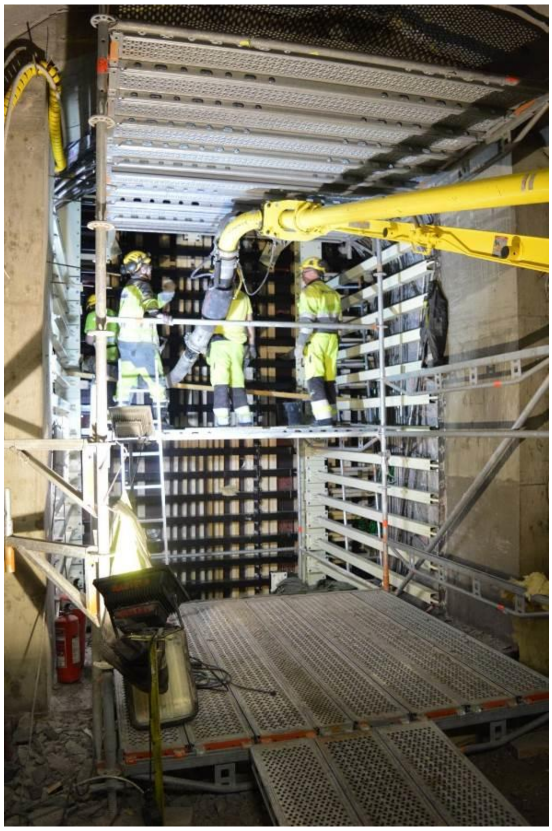
5.6.6.4 Concrete Lead-through Installation

The concrete lead-through pipe was installed during the phase of final reinforcement installation for Concrete Wedge Section One. As the lead-through was embedded in the filter layer and then supported by the formwork, it did not need much support beside a few ties to the internal reinforcement bars. The lead-through pipe had been initially assembled above-ground and quality control checked.

After casting of Concrete Wedge Section One, the second lead-through section or pipe was welded to the first section, and non-destructive testing of the weld quality was undertaken. Bentonite tape was added adjacent to the internal flanges along the tube and against the formwork. After the second casting, the final flange including pipe closure valves was welded to the lead-through opening on the front face of the plug. Welds were again checked by quality control methods.



Formwork ready for casting, Concrete Wedge Section One.



Concrete pumping into the mould, September 2015.

Quality control 1 m^3 cubes for 16 and 32 mm aggregate mixtures, Concrete Wedge Section One.

Fresh concrete measurement of slump flow, underground during quality control before pumping.

Figure 5.14: Plug section and concrete during emplacement period.

5.6.6.5 Reinforcement Installation, Phase 3

The third phase of reinforcement installation was done in the Concrete Wedge Section Two. It proceeded similar to the Phase 2 installation, with the exterior bars being added to supplement the existing reinforcement used as rock protection. The instrumentation then proceeded, followed by ceiling and central reinforcements. For Concrete Wedge Section Two, the reinforcement work took a total of six days.

5.6.6.6 Concrete casting

Casting of each concrete wedge section was completed in a single day in 10-12 hours. Concrete Wedge Section One was cast on 15 July 2015 and Concrete Wedge Section Two on 16 September 2015. The procedures for both sections were nearly identical, and are thus described here together.

A concrete casting readiness review meeting was held a few days prior to casting of the concrete wedge sections. The regulatory authority (STUK) was invited as an observer to these meetings. The pump truck was positioned underground the day before casting and all quality control equipment was ready. The small concrete pump truck used for casting of the back wall was not available for the plug casting and a larger one needed to be used.

Two quality control cubes, 1 m^3 each in size, were poured from each mix (16 mm and 32 mm maximum aggregate sizes, see Figure 5.14). For Concrete Wedge Section One, these large cubes did not include any reinforcement steel. For Concrete Wedge Section Two, the bottom and one side included 32-mm diameter steel bars, so the concrete flow adjacent to the formwork could be observed. Formwork on the quality control 1 m^3 cubes was removed after 2-4 weeks and the concrete cubes have been stored for future sampling, as needed. The quality control cube moulds were re-used for Concrete Wedge Section Two casting.

A preliminary 0.5 m^3 concrete slurry batch, made from 8 mm maximum aggregate size, was used to flush through the pump truck and tubes. This was done to lubricate the equipment approximately 30 minutes before the actual plug concrete delivery. Concrete was delivered underground to ONKALO in intervals of approximately 20-30 minutes. The drive time from the batching plant was approximately 20 minutes.

Quality control tests were done prior to the truck dispatching concrete to the pump truck. These included air content, density, temperature, and slump flow. Cylindrical and cubic samples were made for testing compressive strength, water tightness (permeability) and pH leachate at various ages. Quality control tests were also done by Rudus Oy at the factory at the time of production, before truck delivery.

Pumping was arranged via a rigid steel pipe which was input through the lowest window in the formwork mould. The pipe was slowly extracted during pumping, so that there was not a large distance for the concrete to flow or drop. Visual observations through the lower formwork indicated that the concrete remained level. This could also be seen when casting the 1 m^3 quality control blocks. No internal or external vibration was used.

For Concrete Wedge Section One casting, 24 trucks of concrete were delivered starting from 9.30am. For Concrete Wedge Section Two casting, 20 trucks of concrete were delivered, at 4 m^3 each, starting from 8am. The concrete volume needed for casting the whole plug was estimated to be 161.5 m^3, based on dimensional scanning of the tunnel after slot excavation. In actuality, a total of 172 m^3 was used, comprised of 94 m^3in the first section and 78 m^3 in the second plug section. The extra material was utilised partially for the quality control samples.

A de-airing pipe was used to evaluate when the mould was full. When concrete flowed out of the air tube, it was withdrawn. The uppermost or last hole in the formwork had concrete pumping or casting with an applied pressure of 0.05 MPa maintained for 30 minutes.

During construction, it was possible to monitor the temperature of the plug and pressure on the formwork. The demoulding was undertaken approximately five days after casting, based on the temperature development profiles and a preliminary measurement of the compressive strength. Plastic sheeting covered the front face of the plug for the first three weeks after casting, to prevent drying and shrinkage. The Concrete Wedge Section Two concrete was allowed to harden for 90 days prior to contact grouting of the rock-plug interface.

It was noted by the contractor that the concrete was very good, having high workability and being easy to work with. There was a good workability time, so there was enough time for quality control tests and placement via pumping. During Concrete Wedge Section Two casting, one batch of concrete needed to have slightly more superplasticiser added to obtain the target range of slump spread.

Lessons learned from casting were that, from an emplacement point of view, it would be feasible to consider casting the plug in one section rather than two. The concrete was self-levelling and did not require vibration, thus it could flow for larger distances. There were no problems in using two different concrete mixes, with varying maximum aggregate sizes. However, data on other aspects of the concrete performance are required before a conclusion on the concreting can be made (e.g. curing temperature, which is discussed in DOPAS (2016b)). It was helpful to work with the contractor to make mock-up or metric-scale demonstration tests prior to actual concrete wedge casting. The contractor noted that there could have been more detailed plans for the many phases of construction, to save time and utilise the best available on-site experience and practices.

5.6.7 Grouting of Interface

Contact grouting of the plug-rock interface around the circumference of the plug was performed during a five-day period in December 2015. A total of six grouting loops were grouted according to the work descriptions. The work began with grouting of the first two loops, the outer loops, on consecutive days, and was then halted for a drying period of eight days and for Christmas holidays. Grouting continued on the 28 December 2015 and was completed in three consecutive days.

The grout was mixed on the POPLU experiment site and quality control testing was performed before accepting the grout for use. During contact grouting, the grouting pressure and consumption of the grout for each loop was recorded.

5.6.8 Instrumentation Installation

5.6.8.1 Back Wall

The tunnel back wall casting was used to develop experience on sensor installation in the low-pH concrete. For that purpose, four thermocouples and three relative humidity sensors were installed at different locations inside the concrete back wall. The monitoring of the concrete back wall was not intended to be permanent, but to serve as a temporary test run for the systems for a duration of about two months.

The thermocouples were installed in the centre location of the concrete back wall as well as next to the lateral rock wall, the concrete mould and a lower edge in the direct vicinity of the rock wall and mould. The thermocouples provided important information about the hydration heat development during concrete hardening and the maximum temperature gradients within the massive concrete structure.

All three relative humidity sensors were installed at the same location in the centre of the back wall. There were two different types of humidity sensors, one based on a temperature-compensated resistance meter, and one based on an electronic capacitive hygrometer. The latter is not designed for direct water contact. Therefore, two sensors of this type were placed in the back wall, one without water protection (as provided by the supplier) and one additionally covered with a Gore-Tex cloth to prevent direct water contact, but to allow for the transport of relative humidity.

The uncovered electronic capacitive hygrometer failed after contact with fresh concrete. All other sensors survived and provided accurate data before the wires were cut when the rock lead-throughs had to be closed during construction of the filter layer.

5.6.8.2 Filter Layer

Five pressurising pipes were installed inside the filter layer. Four of the five pipe ends carry nozzles to guarantee an evenly distributed outflow of the pumped water used for pressurising. One pipe end was left open, but protected with a wire mesh against blockage. The pressurising pipes were mounted onto the concrete back wall surface and led to the top rock lead-through flange. After passing the flange, which serves as a tight barrier, the pressurising pipes were continued to the instrumentation tunnel, which hosts the pressurisation system including a water tank.

During three installation campaigns in summer 2015, preparatory works were done and sensors installed inside the filter layer. At first, the rock lead-throughs were prepared (installing Teflon pipes inside the rock lead-throughs, attaching locks and closing of the flanges) and necessary arrangements done inside the measurement container in the instrumentation tunnel. Afterwards, the installation of four pore pressure sensors, four total pressure sensors and three displacement sensors inside the filter layer took place. An essential part of the work was the wiring of the sensors. Due to the high expected hydraulic pressure, all cables and wires had to be sheltered by stainless steel tubes. The tubes were fixed inside and along groves that were cut into the filter layer blocks.

5.6.8.3 Concrete Wedge Section One

The instrumentation for Concrete Wedge Section One was then installed, including work to bend pipes for instrumentation wire shielding. In total, five pore pressure sensors, five total pressure sensors, three relative humidity sensors, two standalone thermocouples and 13 strain gauges were installed inside Concrete Wedge Section One. The wires for the pressure sensors, each of them running inside a stainless steel sheltering tube, were directed through a cable flange towards the rock lead-through flanges in the filter layer. Therefore, all ten pressure sensors had to be installed prior to the final construction of the filter layer. The wires of the remaining sensors were led to a cable flange, which was located between the two concrete wedge sections. The wires of the strain gauges and relative humidity sensors were protected by tubes made from Polyvinylidene fluoride (PVDF), which are more flexible and easier to install between and along the reinforcement bars.

Before closing the manhole in the formwork, laser scanning of the sensor locations was undertaken in order to document all sensor and wire locations as part of the quality control process.

In parallel with the installation work, the data acquisition system was set up and all sensor wires were connected to the loggers located in a cabinet in front of the concrete wedge or at the measuring container in the instrumentation tunnel. Data logging started in the beginning of July 2015 as a test run. Immediately before concrete wedge casting, all sensor data was re-checked and formal data acquisition commenced.

(a) pore pressure sensor within a filter layer block

(b) total pressure sensors in a filter layer block

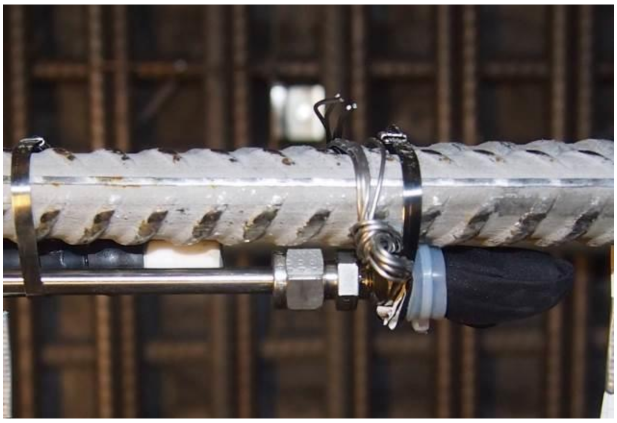
(c) sensor for perdendicular displacement (perpendicular to tunnel axis)

(d) sensor for longitudinal displacmeent (along tunnel axis)

Figure 5.15: Examples of sensors installed in and near filter layer.

5.6.8.4 Concrete Wedge Section Two

More sensors were installed in Concrete Wedge Section Two than elsewhere in the POPLU experiment. In total, 19 strain gauges, six standalone thermocouples, four relative humidity sensors, two pore pressure sensors and two total pressure sensors were installed. The sensor wires were sheltered by PVDF tubes. Before the wires inside the PVDF tubes reached the three cable flanges, which were used to lead the wires outside of the plug, the sheltering tubes were changed from PVDF to stainless steel. This measure guarantees that if pressurised leakage water entered a tube, it would not burst where it is exposed to atmospheric pressure outside of the plug concrete. As already done in Concrete Wedge Section One, all sensor locations were measured and documented using laser scanning.



(a) relative humidity sensors

(b) strain gauges

(c) thermocouple

(d) wiring inside plug

Figure 5.16: Examples of sensors installations in the plug sections.

Data logging was continued for the sensors in the filter layer and in Concrete Wedge Section One, and started for all sensors in Concrete Wedge Section Two. Again, all sensor data was checked immediately before the concrete casting of Concrete Wedge Section Two to guarantee a flawless function of the sensors and data logging systems.

5.6.9 Pressurisation and Leakage Systems

After the concrete wedge was installed, the pressurisation system and leakage measurement system were transported to ONKALO and installed. The pressurisation water pipes were connected to the pump unit and artificial saline deep groundwater was prepared for pressurisation. Two tracers were added to the pressurisation water for tracking water flow in the near field. Base line measurements of the near field waters were taken. The leakage measurement system was placed in front of the plug to collect any potential water from around the plug or along the concrete lead-through. The plug response to pressurisation is reported in DOPAS (2016b) and DOPAS 2016d). At the time of writing of this report, the pressurisation was scheduled to begin in January 2016 and to progress to 4.2 MPa over a 12 week period.

5.7 Lessons Learned from the Design and Installation of the POPLU Experiment

The following sections summarise some of the key lessons learned during the POPLU experiment, which will be considered in Posiva's future repository planning and may also be useful for other programmes.

5.7.1 Repository Safety

Working in a future repository site provided a set of challenges during the POPLU experiment, which others may encounter also in the future. The foreign materials acceptance procedure was successfully implemented for review of safety of the site. This process should be maintained in the future, but requires extra preparation to obtain necessary details from material suppliers (i.e. material data sheets are not always easily available for some equipment or supplies). The construction sequence also needed to be flexible when working in the repository environment, due to other activities also on-going (such as parallel equipment demonstrations for buffer installations in the neighbouring tunnel; blasting for tunnel expansion in other areas underground that can cause vibration for sensors in monitoring of the POPLU experiment).

Also for the underground environment, worker safety was always taken as highest priority. Within the POPLU experiment, this was realised by, for example, taking extra or new precautions to support the tunnel ceiling rock in the slot area (this was because normal means of rock bolting or wire mesh could not be used, including proper support like scaffolding during casting through the upper parts of the formwork 5 m above the tunnel floor level), ventilation and lighting. Underground safety chambers were moved nearby the POPLU experiment area in case of emergency and all workers were well-trained for underground environmental working condition safety. Efficient ventilation must be available at the demonstration site, especially in tasks where there is an identified risk of exposure to dust or exhaust fumes.

Finally, many lessons were learned during the interaction with the radiation protection authorities (STUK), in the sharing of information during the POPLU experimental process. Discussion meetings held approximately every 6 to 9 months provided a forum for discussion and a means of providing clarifications and suggestions. The process was helpful to all parties involved and aids future regulatory understanding and reviews.

5.7.2 Design

During the design phase, it is important to establish clear and quantifiable performance requirements, along with any possible allowable tolerances. For instance, the justification for the slot excavation methodology can be governed by the necessity of rock smoothness, which can be tied to the structural design functionality of the plug. This also supports the benefits for frequent dialogue between the various parties responsible for the plug design. Only with a clear understanding of the requirements and their justification, can a safe structure be designed and implemented during construction. The transparent communication is critical for maintaining the schedule and avoiding conflicts due to lack of knowledge between various parties (structural design, material specifications, monitoring plans, modelling, construction, and quality control).

The instrumentation and monitoring system proved to work effectively. Many lessons were learned about the suitable types of sensors, connections, and attachments and shielding. Water tightness methodologies were developed. There was great experience learned about the monitoring computer programmes for data collection, storage and transfer from underground to the surface.

5.7.3 Excavation and Construction

The tunnel excavation progressed smoothly, and the rock suitability classification methodology was validated for selecting the tunnel and slot locations. The EDZ could be effectively removed with the wedge and grind method for the slot, though the process took more time than originally expected. Clear lessons were learned about how requirements were set and validated. It was learned that the wire sawing method for slot excavation needs to have a stronger implementation plan for worker safety against falling rock. The slot surface quality requirements need to have synergy with the plug structural design expectations in order to have harmony between the design and implementation. In the case of the POPLU experiment, the project resulted in a need to redefine the plug slot surface quality requirements to better represent the chosen slot production method.

During installation of the POPLU experiment and related components, it was demonstrated that the construction sequence was well-planned and the schedule could be maintained. The formwork was well-designed and could be implemented and served its purpose. The reinforcement steel could be assembled efficiently on-site. Extra care was needed to identify suitable materials used for connections and attachments of materials within the plug area, so care should be made in the future when planning auxiliary materials and having their foreign materials acceptance review. The working environment during excavation and construction installations was demanding, with very little space to work. The sequence of persons in the plug area had to be well planned, so as to maximise effective working time. The space needed to be well-maintained for tidiness.

The low-pH concrete materials developed in the laboratory within DOPAS proved to be excellent when applied at full-scale in the POPLU experiment. The self-compacting concrete was fluid for placement without vibration, had a good open workability time and could be pumped into place. The performance test results from quality control testing showed it was a good mix (raw materials and proportions) and met the requirements for strength, permeability and water tightness. Based on the experience from the concrete early age properties (flowability), it is possible that the plug could potentially be cast in one part rather than two sections. This would save material costs, especially for reinforcement steel, and time, but further information on the performance of the concrete, including hardening and maximum curing temperature would need to be assessed before a decision could be made. Discussion of this performance is included in DOPAS (2016b).

The low-pH grout material was able to penetrate the contact area between the concrete plug and rock, even though the Mock-up Test 4 was unsuccessful to the *in situ* experimental test arrangement. The grouting methodology was suitable for the plug, though the design pressures could not be achieved due to failure of the grouting tubes. Caution should be taken to verify with the designers and material suppliers about the safety factors.

The instrumentation and monitoring system was successfully installed and less than 20% of the sensors were lost during the construction and pressurisation starting phase. The data collection system was successful for off-site evaluation and rapid evaluation of plug performance. The near field monitoring plan and implementation was helpful in evaluating the plug response to pressurisation.

After construction, the contractors have noted that they would have preferred to have had more detailed method descriptions and designs, so that they could have more accurately planned and implemented the work. Yet contractors also noted that the documentation should have some level of flexibility and allowed deviations that could be implemented after suggestions, review and approval. In this way they can bring their experience and cleverness to the project, which can result in a safer structure and/or more economic implementation.

The pressurisation programme of the POPLU experiment was updated based on the experiences from the DOMPLU experiment, where the maximum pressure of the experiment was limited to the value of 4.2 MPa corresponding to the groundwater pressure on the demonstration area. This decision was based on the desire to prevent harming the local rock by over pressurising and opening fractures.

5.7.4 Scheduling and Project Management

One of the overall lessons from the project was that keeping the schedule for the long duration of design, construction and implementation was challenging. When the original schedule was delayed with certain tasks, the infrastructure and resources on the site for construction and implementation may be removed, causing possible further delays and cost increases. If unexpected changes occur during the construction activities, the re-sequencing of the activities needs to be carefully considered so that all the activities get done even if the order of the work has changed, thus striving to ensure minimal effects to budget and schedule. The planning of the schedule and budget of full-scale experiments is a demanding task owing to scale of the experiments (both physical size and duration) and the large number of parties involved e.g. sub-contractors. The planning of the sequencing of the sub-contractor's work needs to be carefully done when there are multiple sub-contractors on site. This is even more important when there are several experiments and tests going on at the demonstration site at the same time.

The risks assessment plan and mitigation was helpful for preparation at all phases of work. Open and frequent dialogue between the project team members, both within Posiva staff and contractors, was helpful in maintaining schedules, documentation and brainstorming for risk avoidance. More time should be taken for proper written documentation of design decisions and locking these in good time, also for aspects of planned quality control.

The delays in the schedule of the POPLU experiment compared to the original DOPAS Project plan were mostly due to the plug being first-of-a-kind in the world. Most of the delays were within the preparations for construction, such as the tunnel and slot excavation methods. In an operational repository, these tasks would be sequenced to be completed at least one year before needed. In an operational repository, it is expected that the plug could be constructed a couple months faster than in the POPLU experiment now that the process is better established for the multiple steps. An actual operational plug would also have much less (if any) instrumentation and monitoring, thus the installation time is further shortened.

6. ELSA Experiment and Complementary Laboratory Testing

6.1 ELSA Background

The DOPAS experimental programme in Germany focuses on studies that will support the development of a shaft seal design. No full-scale test as part of the German experimental programme is being carried out within the DOPAS Project. The aims of the experimental programme are to develop generic design concepts for shaft seals in salt and clay host rocks that comply with the requirements for a repository for HLW (Jobmann, 2013; Kudla *et al*., 2013; Herold & Müller-Hoeppe, 2013) and to carry out the necessary preparatory work in the shaft seal design project. Large-scale *in-situ* demonstration test of individual shaft sealing elements will be undertaken after the DOPAS Project.

In order to meet the requirements laid down in the repository regulations and mining law, the primary safety function for shaft and drift seals in a salt formation has been specified as being to provide a sufficiently low hydraulic conductivity to avoid brine paths into the repository and the movement of radionuclides out of it. The period assumed for function of the shaft sealing system in the preliminary safety assessment for the Gorleben site was 50,000 years (Müller-Hoeppe *et al*., 2012a and Müller-Hoeppe *et al*., 2012b). However, integrated process modelling has been performed it showed that the functional time needed can be shorter. Accordingly, the minimum functional period necessary is until the backfill in the repository drifts, access ways and emplacement fields seal the repository in response to compaction driven by host rock creep, which is achieved, depending on the boundary conditions, after some thousands up to 20,000 years.

6.2 Work carried out in WP3 for ELSA

The work in ELSA during the DOPAS Project is largely focused on studies that will support the development of a shaft seal design. The aims of the ELSA project are to develop generic design concepts for shaft seals in salt and clay host rocks that comply with the requirements for a repository for HLW (Jobmann, 2013; Kudla *et al*., 2013) and to carry out the necessary preparatory work in the shaft seal design project.

The detailed objectives of ELSA (Phase 1 and 2) were to:

- Give a summary of the state-of-the-art in shaft sealing systems that would remain stable over the long term.
- Provide documentation on how to achieve compliance of a shaft sealing system design with national and international standards and regulations (design basis analysis).
- Compile boundary conditions for shaft sealing systems in Germany.
- Define requirements for shaft sealing systems in Germany.
- Develop new and modular-based shaft sealing concepts for HLW repositories in Germany sited in both a salt and a clay environment.
- Perform *in situ* tests of specific functional elements (modules) of a reference shaft sealing design or modifications to them (Figure 6.1).
- Develop mathematical models to characterise the material behaviour of specific sealing elements of a reference shaft sealing design or modifications to them.

These objectives have been tackled during the Phases 1 and 2 of the ELSA Project (within the DOPAS Project). One or two large-scale demonstration experiments of particular sealing components and adjustment of the sealing concept will be undertaken in Phase 3 (not part of the DOPAS Project). The main objectives of the experiment(s) in Phase 3 will be to demonstrate technical feasibility and long-term compliance with requirements.

In addition to the above investigations, the laboratory programme of GRS (which is undertaken within the auspices of the LASA, LAVA and THM-Ton Projects) addresses sealing materials planned to be utilised in the shaft seals. This laboratory programme provides supporting information to the ELSA Project. For example, the programme aims at providing experimental data needed for the theoretical analysis of the long-term behaviour of MgO-concrete and cement-based salt concrete in interaction with the host rock and fluids. These kinds of materials have been considered as sealing elements and abutments in the first reference shaft sealing concept (Figure 6.1) for the Gorleben site developed during the preliminary safety analysis for the Gorleben site (Müller-Hoeppe *et al*. 2012).

The data acquired in the LASA, LAVA and THM-Ton projects will be used to assess the long-term performance of the seals with respect to the required hydraulic conductivity. The experiments comprise the following mechanical and geochemical investigations:

- Uniaxial multistep creep tests on samples of concrete for the determination of creep parameters.
- Triaxial compression tests on samples of concrete with axial flow of gas for determination of time-dependent compaction and damage evolution.
- Long-term re-compaction tests on pre-damaged samples of concrete under increasing isostatic load (evolution of gas permeability as the self-sealing indicator).
- Long-term experiments of rock salt/concrete systems using large hollow salt cylinders filled with concrete under varying isostatic load and constant brine pressure.
- Batch experiments with crushed concrete used to determine mineral alteration reactions and the geochemical evolution of the concrete pore waters up to the point where equilibrium between the material and brine is established.
- In-diffusion experiments with concrete and brine in order to determine the porous matrix alteration rate.
- Experiments with the concrete and brine at the contact with the EDZ in order to determine the sealing material alteration rate in response to advective flow at the boundary with the rock formation.

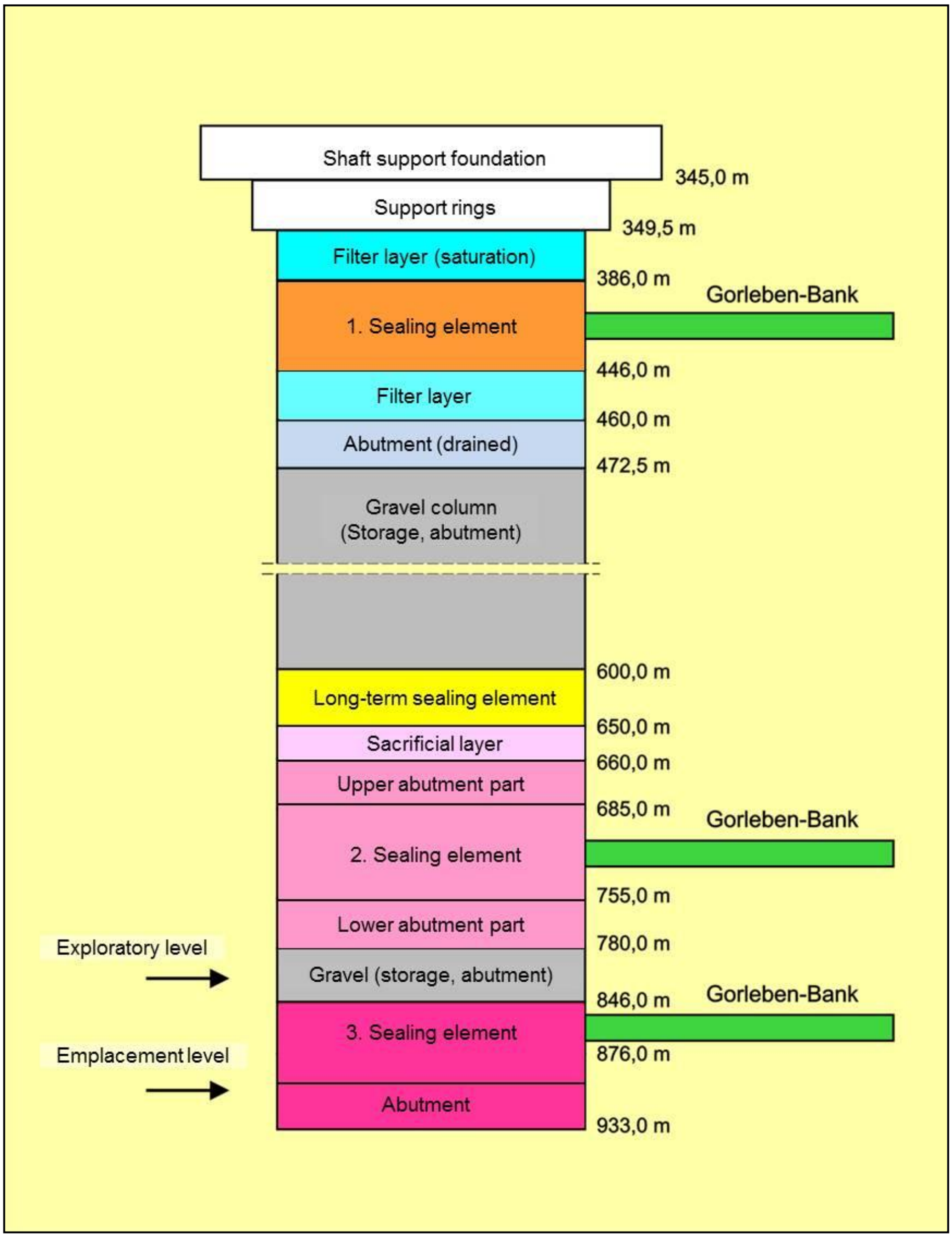


Figure 6.1: Reference conceptual design for the German shaft seal. The Gorleben-Bank is a folded anhydrite layer in the rock salt (Müller-Hoeppe *et al*. 2012a).

This chapter provides a summary of the studies undertaken in DOPAS that are part of the ELSA, LASA, LAVA and THM-Ton programmes:

- In Section 6.3, the ELSA Phase 2 *in situ* tests are summarised.
- In Section 6.4, selected experimental investigations relevant for the hydro-mechanical long-term material behaviour according to the LASA programme are summarised.
- In Section 6.5, selected experimental investigations relevant for the hydro-chemical long-term material behaviour according to the LAVA programme are summarised.
- In Section 6.6, selected experimental investigations relevant for the hydro-mechanical long-term material behaviour according to the THM-Ton programme are summarised.
- Discussion of the lessons learned regarding the design of shaft seals in Germany is provided in Section 6.7.

Further details of the ELSA Phase 2 experiments are described in the final synthesis report of the ELSA Phase 2 report (Kudla *et al*. 2016).

6.3 Results of ELSA Phase 2

In the framework of the ELSA R&D project, shaft sealing concepts are currently under development for both salt and clay host rocks. These concepts are to be modular in design so that they can be tailored to site-specific conditions once potential repository sites will have been identified.

With regard to the safety requirements, sealing systems must be designed based on redundancy and diversity. In practice, this means combining several sealing components made from different materials or from materials with different properties. Proof of functionality must be provided independently for each sealing component.

A multi-layer 'hard shell – soft core' seal concept is currently being investigated (Figure 6.2). This concept combines bitumen sealing element with low viscosity, good wettability and good penetration capacity into the surrounding rock, and confining layers with higher viscosity, no outflow and good confinement of the sealing bitumen).

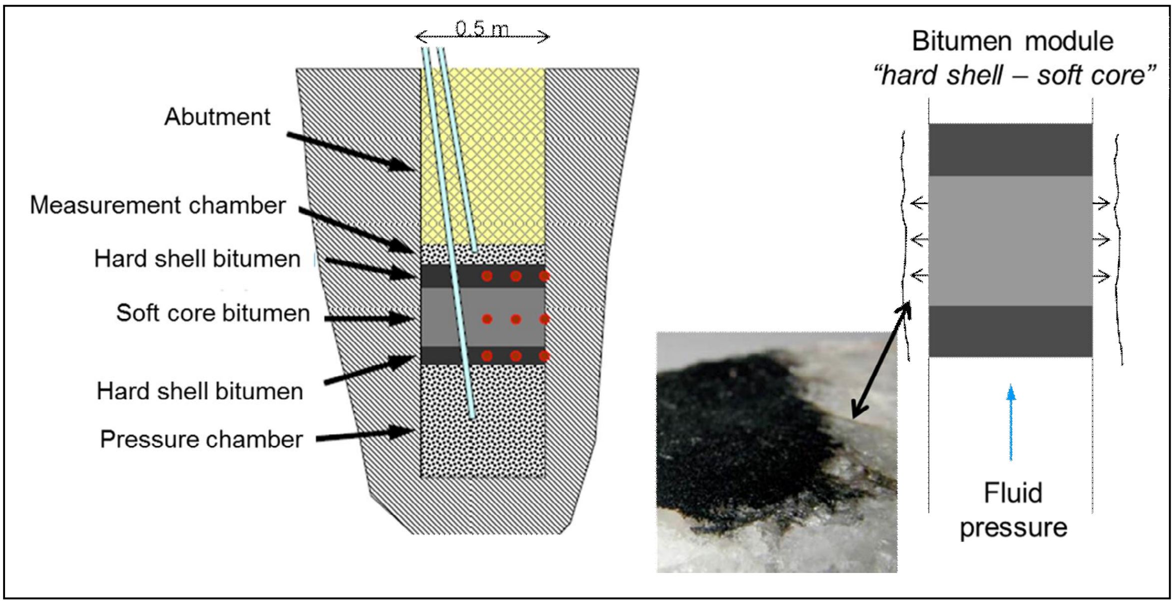


Figure 6.2: Design of the *in situ* experiments considering bitumen as sealing material.

A seal based on the hard shell – soft core concept has recently been tested in an *in situ* large-diameter borehole at the Sondershausen mine in Germany. The measured permeabilities were in the range of 1 x 10^{-20} m^2, which is equivalent to the permeability of the salt host rock. This is taken as demonstration of the tightness of the seal.

Magnesium oxide materials are always employed in the conventional salt mining industry when cement-based materials cannot be used because of the risk of an inflow of magnesium-containing solutions. This generally affects all building and construction work in the potash industry and also any structures in the rock salt industry that are located in mine workings with a magnesium-salt environment. In the latter case, a corresponding magnesium level will establish itself in the solution even with the influx of a mainly pure rock salt solution, as the solubility of the magnesium salts is greater than that of the rock salt.

Wide-ranging investigations carried out in conjunction with the Institute of Inorganic Chemistry at the Freiberg University of Mining and Technology have resulted in the development of a magnesium oxide concrete that exhibits long-term stability. This newly developed formula was the focus of the *in situ* borehole test in the Sondershausen mine (Figure 6.3).

The tests included measuring the temperature development in the MgO concrete during the setting reaction (Figure 6.3). As expected, the temperature at the rock profile was low (<50°C) because of the high thermal conductivity of the rock salt. The maximum temperature measured at the core of the plug was 70°C.

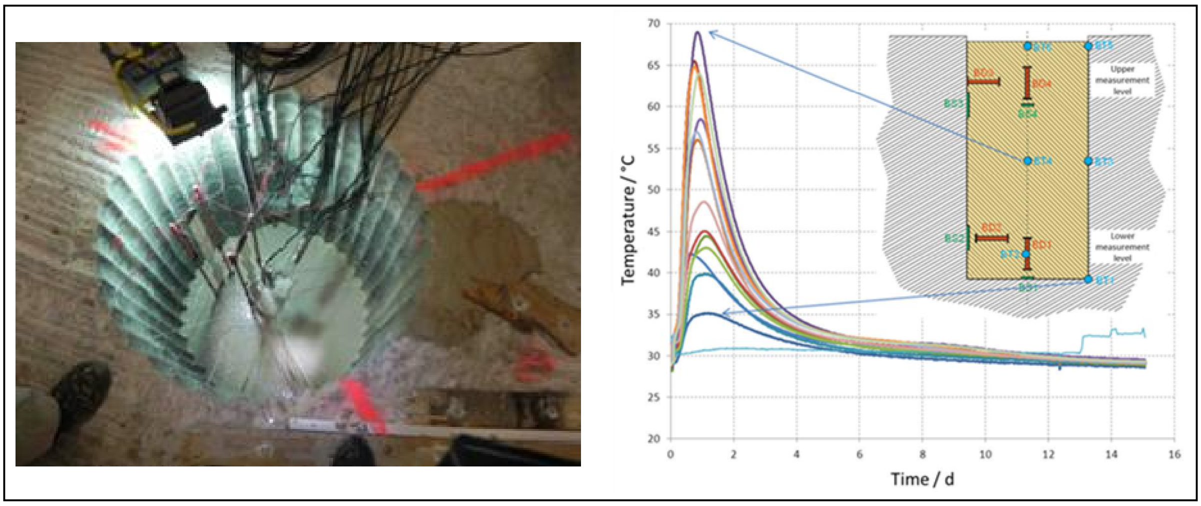


Figure 6.3: Photograph of the large-borehole test in the Sondershausen mine used to investigate MgO concrete (left) and temperature profiles (right) measured in the MgO plug after installation (Kudla *et al*., 2015).

The MgO concrete plug can be made watertight by employing appropriate injection measures to consolidate the contact zone between the MgO concrete and the surrounding strata, and by preventing the formation of cracks in the concrete. This makes it possible to design a range of sealing elements based on a favourable combination of MgO concrete and bitumen/asphalt elements.

A combination of clay and bitumen/asphalt has over the years proved extremely effective for constructing seals in mining industry shafts (Kudla *et al.* 2009).

Other diversified systems involve the combined use of bentonite and compacted mixtures of crushed salt and clay. The use of filling columns consisting of crushed salt has been considered for many years as potential supporting elements for closing shafts in rock salt. In the recently developed German reference concept for repository shaft sealing in rock salt compacted crushed rock salt is used as a long-term sealing element. The problem is that it is rather difficult to achieve an initial porosity of the sealing element of less than 10% during the installation. Compaction the material can be improved and a lower initial porosity be achieved when a mixture of crushed salt and clay is used instead. During laboratory tests, initial porosities of less than 10% could easily be achieved. Problems arise however when attempts are made to achieve this target porosity using *in situ* compaction means. Conventional industrial compaction equipment such as vibrating plates can be applied easily and with little technical effort. However the compaction force that can be achieved is relatively low. During *in situ* tests (Figure 6.4) in the Sondershausen salt mine, initial porosities of about 15% could be achieved by using different vibrating plates.

A more effective solution is to use impulse compaction with a high impact force of several tonnes. *In situ* tests have been performed in a stone quarry. For this purpose a “little” shaft was constructed in the bedrock and installed with a steel liner supported by concrete (Figure 6.5).

Figure 6.4: In situ compaction tests of crushed salt and clay mixtures. © Martin Uhlig, TU Bergakademie Freiberg.

Figure 6.5: *In situ* compaction test using pre-compaction by vibrating plates and pulse compaction with high impact forces for of crushed salt and clay mixtures - © Uwe Glaubach, TU Bergakademie Freiberg.

The pulse compaction system used during the *in situ* tests has been developed by the TERRA-MIX company. Three different crushed salt and clay mixtures have been prepared and implemented in three levels in the little shaft. Each level has been pre-compacted using a vibrating plate prior to the impulse compaction. As a result, with the exception of the first metre, the target initial porosity of the mixture of less than 10% could be achieved in the lower part of the little shaft. However to deploy this technology inside a repository shaft (i.e. underground) technical modifications and adaptations to the equipment will be necessary. Nonetheless, the technical feasibility has been demonstrated in principle. It means that this kind of mixtures can be regarded as additional materials for use in shaft sealing elements.

Details of the experiments are described in the final synthesis report of the ELSA Phase 2 report (Kudla *et al*. 2016).

6.4 Results of the LASA Experimental Programme

The LASA programme focuses on the hydro-mechanical properties of candidate seal materials in a salt rock such as MgO and cement-based salt concrete. The LASA programme (in conjunction with the LAVA programme, Section 6.5) aims to provide experimental data required for the theoretical analysis of the long-term behaviour of cement-based salt concrete (second sealing element) and MgO concrete (third sealing element) including the interaction with the host rock and formation water (see Figure 6.1). The data gained will underpin the understanding of the long-term evolution of the hydraulic conductivity of the seals.

Samples for experimental investigations were drilled from an *in situ* construction in a former salt mine. The drift sealing element was constructed at the 945 m level of the mine in January 1992. Its dimensions were 8.0 m in length, 5.5 m in width and 3.4 m in height. The samples were taken from two boreholes (B4 and B5, see Figure 6.6). The salt concrete, which is referred to as Mül10, is a mass concrete consisting of a cement matrix with crushed salt filler. The composition is given in Table 6.1. At the time of sampling the seal had been subjected to convergence of the surrounding rock salt for approximately ten years. Figure 6.6 shows some of the samples before testing.

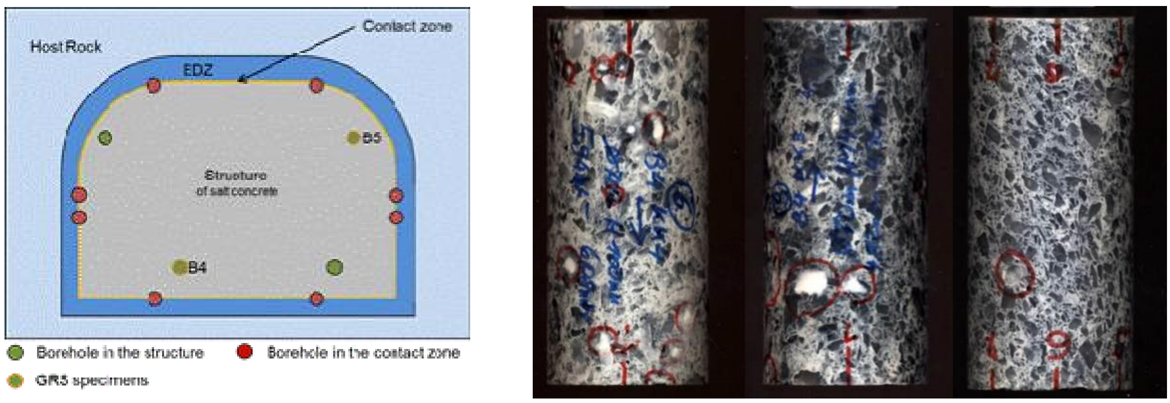


Figure 6.6: Definition of the three parts of a sealing system and identification of the sample boreholes (left); Salt concrete samples before testing (right).

Table 6.1: Composition of the Mül10 salt concrete.

Component of salt concrete	Proportion kg/m³	Proportion mass %
Blast furnace cement	380	18.3
Crushed salt	1496	72.1
NaCl-brine	198	9.5
Total	2074	100.0

The following laboratory tests were performed:

- Uniaxial tests to determine salt concrete long-term deformation behaviour.
- Triaxial tests on salt concrete for investigating mechanical stability.
- Hydro-mechanical investigations of the combined system of salt concrete/rock salt to determine the re-compaction behaviour.

6.4.1 Long-term deformation behaviour

The tests aim at determining the deformation of the samples in terms of strains and strain rates in order to describe the time-dependent uniaxial creep behaviour of salt concrete at different stress states. Uniaxial creep tests were performed in a rig in an air-controlled room. The rig allows five samples of 80 mm diameter and 160 mm length being simultaneously tested at the same load up to 500 kN at ambient temperature.

Figure 6.7 shows the deformation rig. Axial load was applied equally to the five samples by means of an oil balance with accuracy higher than ±0.5%. Axial deformation of each sample was originally measured by linear variable differential transformer (LVDT) displacement transducers with an accuracy of ±0.1 mm. Radial strain was measured by strain gauges with a resolution of 1 x 10^{-6}, which were directly glued on to the samples. In the following analysis, only the results for axial deformation measured by LVDT are considered. The reason for this approach is that the LVDT measure the change of the whole length of the specimen while the strain gauges provide only a point measurement. Thus, the measurement with the LVDTs is more representative for the overall sample behaviour. The individual specimens were labelled SC (1048) up to SC (1052) for identification purposes.

The tests were executed at three different stress states. First, the axial stress was set to 5 MPa, and then it was increased to 10 MPa and 20 MPa, respectively. During the tests, the temperature was held at approximately 25°C. The axial stress causes a reduction of the length and an increase in diameter of the specimens. All of the tests lasted approximately 300 days with each step duration of 76 to 106 days. Figure 6.7 shows the axial strain and the strain rate as a function of time for a representative sample. Strain rate is averaged over seven days. The figure shows that at the first two stress levels, there is only a minor increase in strain while strain rates decrease rapidly (i.e. there is no stationary creep). Strain increases at the third stress level, and strain rate seems to stabilize in the range of ~1 x 10^{-10} s^{-1}, which could imply a steady state creep at this stress level.

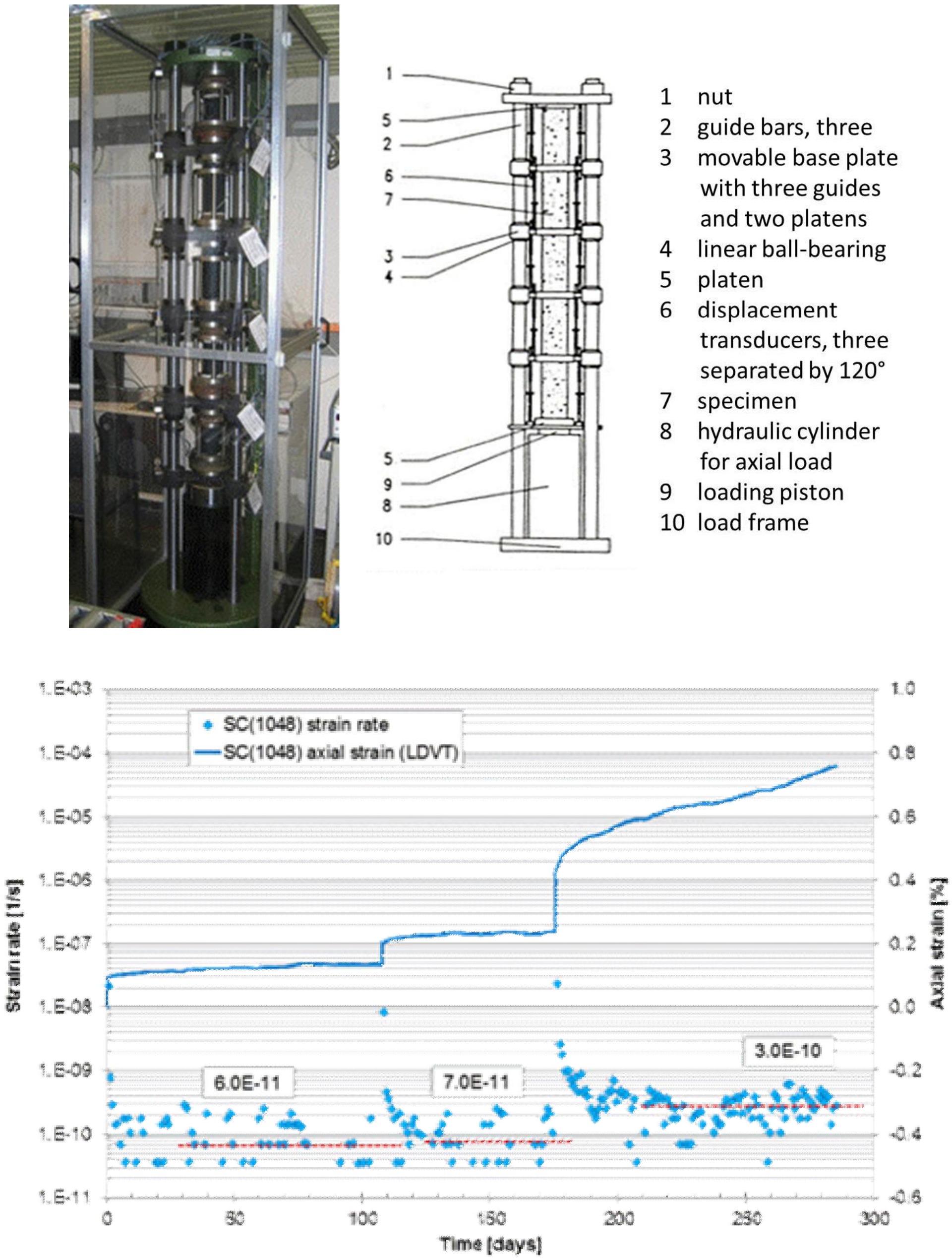


Figure 6.7: Rig for uniaxial creep tests on a stack of five samples (above); Long-term uniaxial creep behaviour of salt concrete sample no.1048 under multi-step loads – axial strain and derived creep rates (below).

The results of the uniaxial creep test show that material behaviour is different at lower stress levels of 5 MPa and 10 MPa and at a stress level of 20 MPa. While strains are small at lower stresses, a distinct creep deformation occurs at a stress level of 20 MPa. The reason for the different deformation behaviour at various stress levels might be that the cement structure of the salt concrete bears at uniaxial stresses up to 10 MPa. Cement is expected to have an elastic material behaviour without viscoplastic deformations after the water curing process has finished. Therefore, no stable creep rates could be derived at lower stress. When the stress level was increased up to 20 MPa, the cement structure of the salt concrete was damaged. Consequently, the salt grit structure of the salt concrete was subjected to the load, and because of the viscoplastic material behaviour of salt grit, the specimens exhibited explicit creep behaviour.

6.4.2 Material stability

Triaxial compressions tests (TC-Tests) were performed in order to investigate the mechanical stability of salt concrete. Onset of dilatancy, start of gas flux and failure of the specimens were determined under different radial stresses. The objective of the TC-Tests was to develop a greater understanding of the deformation behaviour of salt concrete than developed through the uniaxial creep testing described previously (Section 6.4.1). Damage tests were carried out on three salt concrete samples in a triaxial apparatus, and deformation and gas permeabilities were measured under various stress conditions. The samples are shown in Figure 6.6. Open voids stemming from the hydration process marked in red on the samples were filled with resin during the sample preparation procedure. The average porosity of the samples is about 6 % with a grain density of 2.17 - 2.2 kg/m^3. The average water content is approximately 2 wt%.

The samples were subjected to two phases with different loading regimes. The purpose of the initial isostatic pre-compaction phase was to re-establish the intact state of the samples with respect to permeability. No effective healing was expected during the pre-compaction phase. This phase was performed by simultaneously increasing axial and radial stress to 5 MPa at a loading rate of 1 MPa/min, then keeping stress constant for 22 hours. Then, the axial and radial stresses were increased twice up to a level of 20 MPa. After 24 hours, the pre-compaction phase ended. The idea of this pre-compaction phase is to reach the state of compaction that the specimens had before extraction from the drift sealing element.

In the second step of the test, the specimens were deformed under deviatoric stress. For that purpose, three specimens were subjected to three confining stresses of 1, 2 and 3 MPa. The axial stress was increased until the failure load level of the sample was reached. A low strain rate of 1 x 10^{-7} s^{-1} was applied. During the load tests, axial strain was measured by a displacement-transducer installed outside of the cell, while volumetric strain was determined both directly from the volume change of the confining oil in the cell by using a pressure/volume controller and indirectly by strain gauges attached on the sample surface at the middle of the length. Permeability changes induced by the mechanical loading were measured along the sample axis by injecting dry nitrogen gas to the bottom at constant pressure and recording the outflow at the opposite side. The gas outflow was continuously recorded by using a burette. During steady state gas flow, the permeability is determined according to Darcy's law for compressive media.

The results of the TC-Tests with a confining stress of 3 MPa are presented in Figure 6.8. On the left side, the deviatoric stress and the volumetric strain are shown versus the axial strain. The deviatoric stress increases up to a stress level of 40 MPa. The failure stress is reached at an axial strain of 2 %. After failure, gas permeability measurements are performed. Axial deformation is kept constant during the measurement. Hence, stresses show relaxation behaviour during the gas permeability measurements. Afterwards, axial stress is increased further. The volumetric strain decreases in the beginning of the test, due to the compaction of the specimen. Volumetric strain starts to increase when axial strain reaches values of about 0.9%. This point is defined as the onset of dilatancy, which is marked by formation of microcracks leading to volume increase. At this point, the deviatoric stress is about 36 MPa. By ongoing increase of deviatoric stress the microcracking and the specimen volume increases further. At a certain point of the test the microcracks are connected and a gas flow can be detected, Figure 6.8 (right hand side).

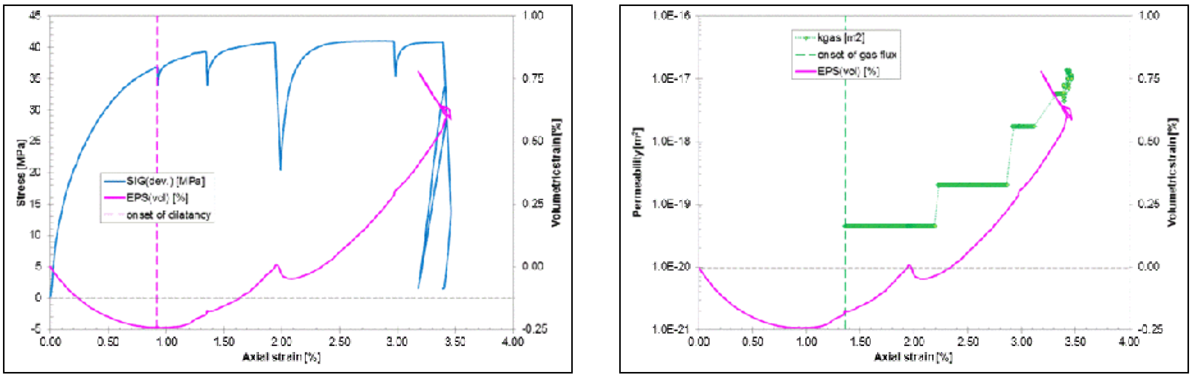


Figure 6.8: Stress-strain behaviour (left) and strain-permeability behaviour (right) of a salt concrete sample deformed by deviatoric loading at confining stress of 3 MPa.

The results of the triaxial compression tests show that salt concrete exhibits reversible (elastic) and irreversible (plastic) material behaviour during the compaction phase. The compaction test phase was identical for all three specimens. During the deviatoric stress phase, the results of the tests are different due to different confining stresses. It is possible to identify the onset of dilatancy by the evolution of volumetric strains. In all three tests, the onset of gas flux is measured at higher deviatoric stress levels than the onset of dilatancy. The boundary for the failure of the specimen increases with higher confining stress, analogue to the onset of dilatancy. The onset of gas flux and the failure of the specimens occurred nearly at the same deviatoric stress level. Consequently, the test results show that the investigated salt concrete samples were gas-tight until the load limit was reached. Generally, no damage is expected in the salt concrete specimens below deviatoric stresses of 30 MPa.

6.4.3 Evolution of combined concrete/rock salt system

The test procedure is first to perform gas permeability tests of the dry sample while stepwise increasing the compressive load. After unloading the sample saturated brine is injected and the stepwise loading is repeated, while the permeability of the seal system to brine is recorded. The evolution of the permeability of the composite sample to gas or to brine is the essential variable characterizing the recovery of the EDZ and the closing of the contact seam between salt concrete core and surrounding rock salt.

The samples are coated with rubber jackets and placed in isostatic cells equipped with hydraulic lines to allow for axial flow-through of gas or liquid and determination of the system permeability. A photo of a coated sample and the cell arrangement are shown in Figure 6.9.

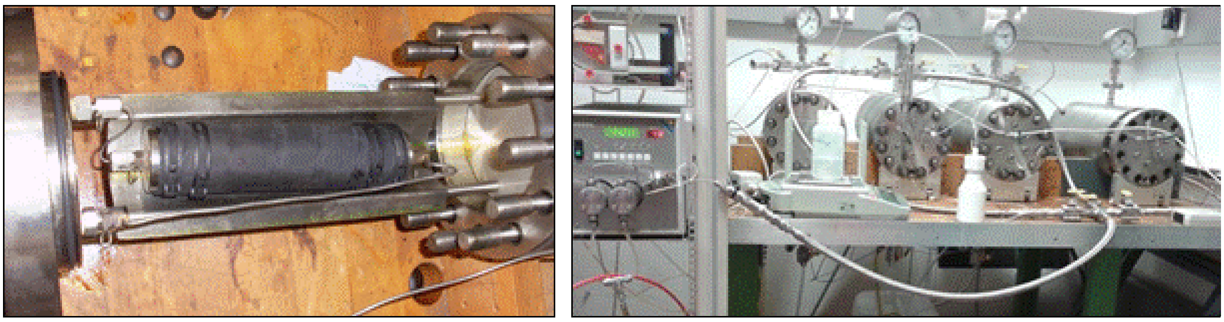
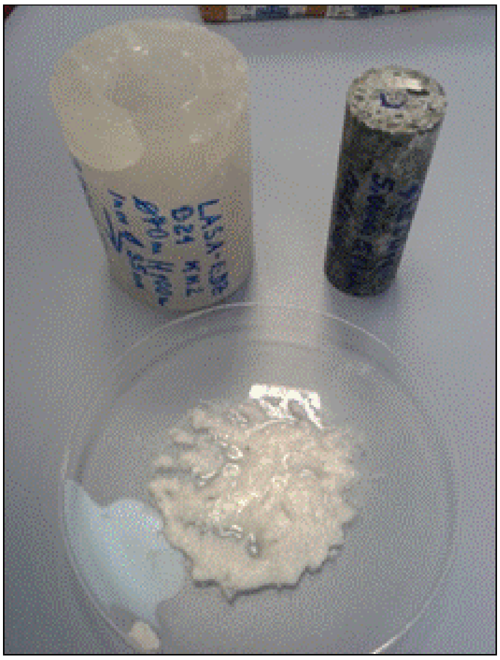


Figure 6.9: Coated sample before installation in the isostatic cell (left), isostatic cell arrangement (right).

For performing experiments on the system consisting of a salt concrete seal element and the surrounding dilated rock salt at the laboratory scale, the idea is to use hollow salt cylinders furnished with a central core of salt concrete which are placed in isostatic cells to impose confining stress. Thus, a combined sample represents the system of seal element, contact seam and surrounding rock at a small scale. The salt concrete core (diameter 35 mm) is placed in the hollow cylinder (70 mm outer diameter by 100 mm height) with a snug fit. In order to fill up potentially remaining voids, the core is coated with salt slurry and then pushed into the hollow salt cylinder. Excess slurry is removed and the sample is dried in an oven to avoid undefined saturation states. A photo of a salt concrete core, a hollow cylinder and the completed sample is shown in Figure 6.10.

A pilot test was performed in order to check whether the sample composition and the measurement technique were suitable. After the test, the sample was dismantled and inspected. The pilot test showed that the test method is adequate to investigate evolution of the overall permeability of dry and brine-containing composite samples as a function of compressive load. It also showed that load changes have to be applied cautiously in order to maintain integrity of the testing arrangement.

For the start of detailed testing, two new samples were prepared. In order to increase the range of rock properties tested, one sample featured an intact salt concrete core as taken from the *in situ* seal, while for the other sample a larger salt concrete core was loaded triaxially to the failure point and a smaller core was machined from the damaged core. The rationale for using a damaged salt concrete core as seal element is the fact that shrinkage fractures of a seal during construction cannot be excluded. The gas permeability of both the salt concrete cores and the rock salt cylinders was measured before preparing the combined samples. Afterwards, gas and liquid testing in the isostatic cells was started similarly to the pilot test.



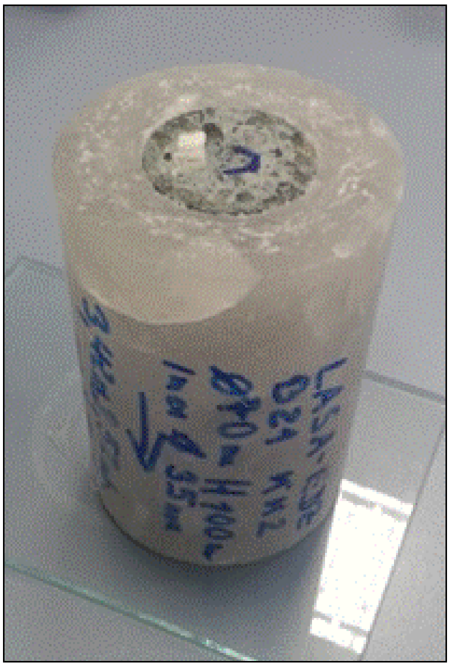


Figure 6.10: Hollow salt cylinder, salt concrete core and salt slurry (left); complete combined sample (right).

For characterization of the salt concrete cores, these were placed in isostatic cells and the load dependent gas permeability was determined under stepwise loading from 1 to 5 MPa. The permeability of the intact salt concrete core, calculated using Darcy's law, amounted to 2.3×10^{-20} m^2 at 1 MPa isostatic load and <1 x 10^{-22} m^2 at 2 MPa. This also demonstrates that any flow along the jacket interface is insignificant. For the damaged core, a moderate permeability decrease from 4.1×10^{-18} m^2 at 1 MPa load to 1.6×10^{-18} m^2 at 5 MPa was observed. The salt cylinders were subjected to the same tests before drilling the central holes for reception of the salt concrete cores. They also showed a permeability decrease with load: from 2.6×10^{-19} m^2 to 2.5×10^{-21} m^2 for the cylinder used with the intact salt concrete core and from 5.2×10^{-20} m^2 to <1 x 10^{-22} m^2 for the cylinder used with the damaged salt concrete core. The results of the load-dependent permeability tests of the individual material samples are summarized in Figure 6.11.

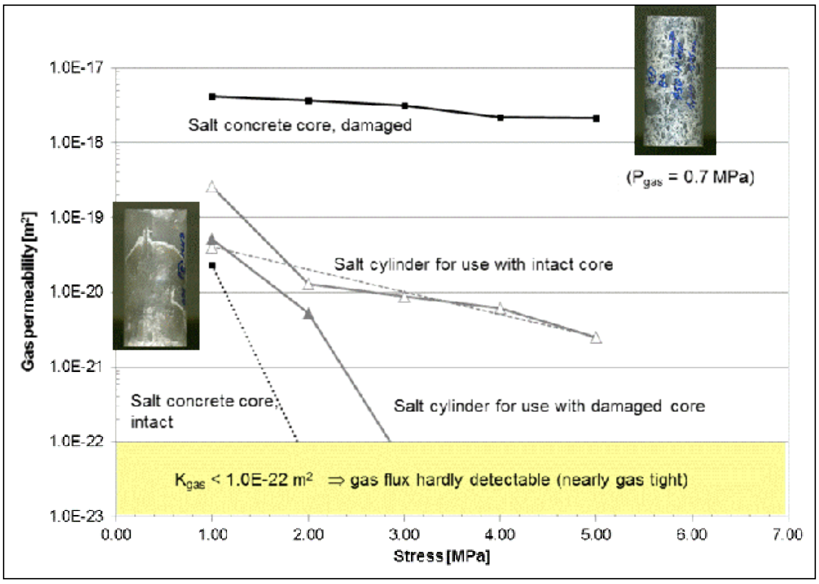


Figure 6.11: Gas permeability of salt concrete cores and salt cylinders before assembly of combined samples.

After characterization of the salt concrete and salt cores, the central holes were drilled into the salt cylinders and the combined samples were prepared as described earlier (see Figure 6.10). The potential effect of the sub-coring on the salt core gas permeability could not be quantified.

In the first stage of testing the combined samples, the confining load was increased by 1 MPa steps to 5 MPa over two weeks, and gas flowed axially through the samples. For the sample with the intact seal element of salt concrete, the gas permeability decreased slightly from 2.5×10^{-14} m^2 to 1.3×10^{-14} m^2. When the sample was unloaded, the gas permeability remained at this value. The permeability measured is close to the upper limit of the testing arrangement. In fact, for the sample with damaged seal element gas permeability remained above the upper measurement limit of 5×10^{-14} m^2. The gas tests on dry samples show the high impact of the contact seam – the overall permeability is much higher than the gas permeabilities of the individual materials. A second result of gas testing is that under dry conditions and moderate confining stress up to 5 MPa, a reconsolidation of the EDZ or closing of the contact seam is negligible in the short term. After gas testing, the samples were unloaded and brine was injected. Then, the samples were again loaded in steps by 1 MPa up to 5 MPa, and the permeability to brine was measured. The measurement results are shown in Figure 6.12. The time to maximum load was three weeks for the two samples. Both samples showed a significant decrease of permeability to liquid with increased loading.

For the sample with the intact seal element, permeability decreased from an initial value of 4.5×10^{-15} m^2 to below the detection limit of 10^{-20} m^2 at a load of 5 MPa. Obviously, the salt surrounding the seal element became soft enough to enable very effective sealing of the contact seal and the EDZ in short time. The sample behaves significantly differently from the dry case, where only a slight gas permeability reduction was observed. The sample was afterwards stepwise unloaded again. Permeability remained below the detection limit, showing that irreversible compaction had occurred. It is envisaged to dismantle the sample and confirm the reconsolidation and associated reduction of pathways by microscopic inspection.

As expected, the sample with the damaged seal element started at a higher permeability of 3.6×10^{-13} m^2 (note that the measurement limits for permeability measurement with liquid are different from those for gas tests). With increasing confining stress the permeability decreased by two orders of magnitude, but it remained much higher than for the sample with the intact seal element. While a compressive load of 5 MPa was sufficient to effectively seal the contact zone and EDZ in the presence of brine in the experiment with the intact seal element, the same cannot be postulated for the experiment with the damaged seal element. Obviously, reconsolidation of the damaged seal element was not achieved. The permeability value measured at 5 MPa confining stress, however, seems too high to attribute it to the flow through the seal element alone, as the gas permeability of the damaged salt concrete ranged between 10^{-18} m^2 and 10^{-17} m^2 (see Figure 6.11).

The sample with the damaged seal element was not unloaded. The confining stress was increased in steps up to 10 MPa and the measurement of permeability to brine continues. Obviously, the permeability value decreases not only with increasing confining pressure (light grey phase). The measurements show a decrease of more than two orders of magnitude from 10^{-15} m^2 down to 10^{-18} m^2 within the next 100 days (2400 hours). It is expected that the permeability value will tend towards the detection limit without further increase of the confining pressure.

Extensive calculation activities are underway for interpretation and further prognosis of the experimental data.

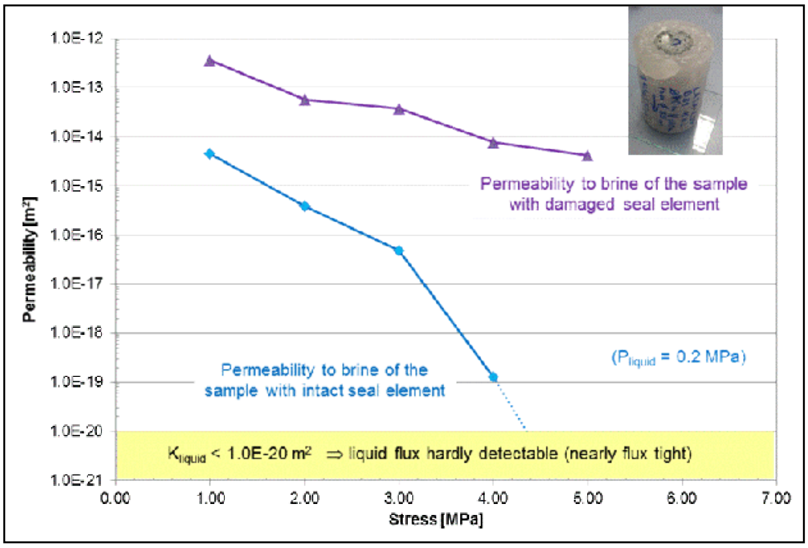


Figure 6.12: Brine permeability of combined samples as a function of confining stress.

6.5 Results of the LAVA Experimental Programme

The LAVA programme focuses on the hydro-chemical properties of candidate seal materials in a salt rock. The chemical stability of cement-based sealing materials is of vital importance for the longevity of sealing elements. In scenarios where there is a permanent contact between an aqueous solution and the sealing element, dissolution and precipitation processes can occur which eventually might result in changes of porosity and subsequently mechanical stability. Two evolutions are conceivable: firstly, porosity and consequently permeability increase. This may lead to a loss of mechanical stability. Secondly, porosity could decrease due to a net increase of solid phase volume. This would result in a decrease of permeability (Meyer *et al*., 2003).

The composition of the aqueous solution depends on the ambient host rock formation. For the present considerations, the most important mineral phases in salt rock are halite (NaCl), anhydrite ($CaSO_4$), gypsum ($CaSO_4*2H_2O$), sylvite (KCl), kieserite ($MgSO_4*H_2O$), polyhalite ($K_2Ca_2Mg[SO_4]_4*2H_2O$), carnallite ($KMgCl_3*6H_2O$) and kainite ($K_4Mg_4[Cl_4SO_4]_4*11H_2O$). Naturally occurring brines in equilibrium with rock salt are always saturated with respect to halite. Equilibration with other mineral phases leads to quinary (without Ca) or hexary solutions (with Ca), with the composition specific to the particular set of mineral phases with which the solutions have equilibrated. For example, equilibration with potash salts results in brines that are dominated by Mg^{2+} and Cl^- (Herbert, 2000).

The corrosion mechanism of sealing elements depends significantly on the construction material and the solution composition. NaCl-based salt concrete is stable against NaCl-saturated brine (hereafter referred to as "NaCl-solution") but corrodes in the presence of high $MgCl_2$-concentrations. A specific $MgCl_2$-rich solution in equilibrium with halite, sylvite, carnallite, kainite, and polyhalite is of particular interest and hereafter is referred to as "Mg-rich-solution". $MgCl_2$-based sorel cement shows an opposite behaviour to salt concrete: it is stable in Mg-rich-solution and corrodes in NaCl-solutions containing small amounts of $MgCl_2$ only (Krauke and Fliß, 2008).

The objective of laboratory tests executed by GRS was to investigate the reaction path and diffusive and advective transport mechanisms in salt and sorel concrete in contact with Mg-rich- and NaCl-solution. The experiments described below were performed in the GRS laboratory and are on-going.

6.5.1 Batch experiments

"Cascade experiments" are executed to investigate the reaction path between sealing material and solution. The cascade experiment is a sequence of batch experiments and has to be performed in air-tight vessels for the elimination of carbon dioxide. In any batch experiment, powdered concrete is exposed to solution in a defined solid-solution-ratio. Vessels are shaken in an over-head-shaker during the whole reaction time. If equilibrium is reached between concrete and solution, the solution is separated from solid and is exposed to new powdered concrete for the next cascade in the same solid-solution-ratio as before. This process is repeated until no free aqueous solution can be regained anymore after an equilibration step. Solution and solid phases are analyzed at the end of each cascade. Singular batch experiments were executed before starting cascade experiments. This aims to determine the reaction time between solution and concrete until the equilibrium is reached. Therefore, four preparations (in PE-bottles) in each system (Sorel concrete / Mg-rich-solution, Sorel concrete / NaCl, Salt concrete / Mg-rich-solution, Salt concrete / NaCl) were estimated. The bottles have been shaken by hand every day. Samples were taken after 2, 4, 7, 9, 11 and 18 days. The test procedure is schematically depicted in Figure 6.13 together with the development of solution composition in batch experiment with salt concrete / Mg-rich-solution. The increase of Ca-concentrations results from the dissolution of salt concrete typical CSH-phases in consequence of Mg-attack. This experiment is still in progress because the equilibrium has not yet been attained.

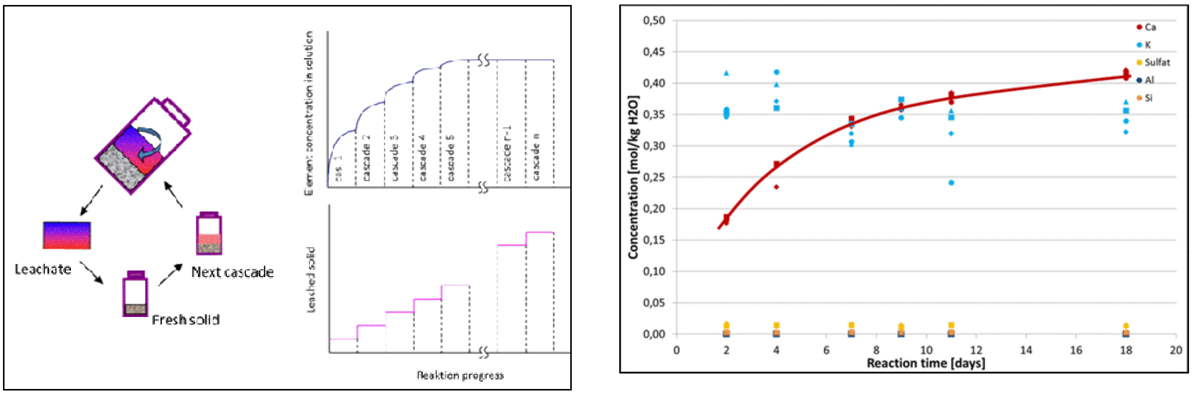


Figure 6.13: Test procedure followed in the GRS cascade experiment (left); Development of solution composition in batch experiments with salt concrete / Mg-rich-solution. Development of Na- and Cl-concentrations is not shown because the concentrations are much higher and remain constant.

The total chemical reaction path of a solution penetrating a geotechnical barrier can be reproduced by the cascade experiment until thermodynamic equilibrium between the original solution and the solid material is attained. In this way, chemical reactions which may occur by an intrusion of brine to a sealing element can be simulated in a short time (Niemeyer *et al.*, 2014). Figure 6.14 shows the XRD-diagrams from Sorel concrete A1 before contact to NaCl-solution (left) and after 11 days of contact (right).

Sorel concrete A1 consists of characteristic Sorel phases (3-1-8-phases), anhydrite ($CaSO_4$) and some halite (NaCl). Chloratinite ($Mg_2(CO_3)Cl(OH)*3H_2O$) is a product of carbonation which results from production of Sorel concrete because samples have not been hardening with consideration of CO_2 elimination.

After contact with NaCl-solution the dissolution of Sorel concrete typical 3-1-8-phases is to identify. It develops secondary phases which are currently unknown. Halite and chlorartinite stays stable. After 11 days a further change in phase composition has not been observed. The analysis of the solution has revealed no significant change in its composition over total testing time.

Consequently, each cascade in the cascade experiment in system Sorel concrete / NaCl-solution needs minimum 11 days. Equilibration time between concrete and NaCl-solution is only valid for a system with powdered concrete, for solid samples a longer equilibration time is expected because of the smaller specific surface.

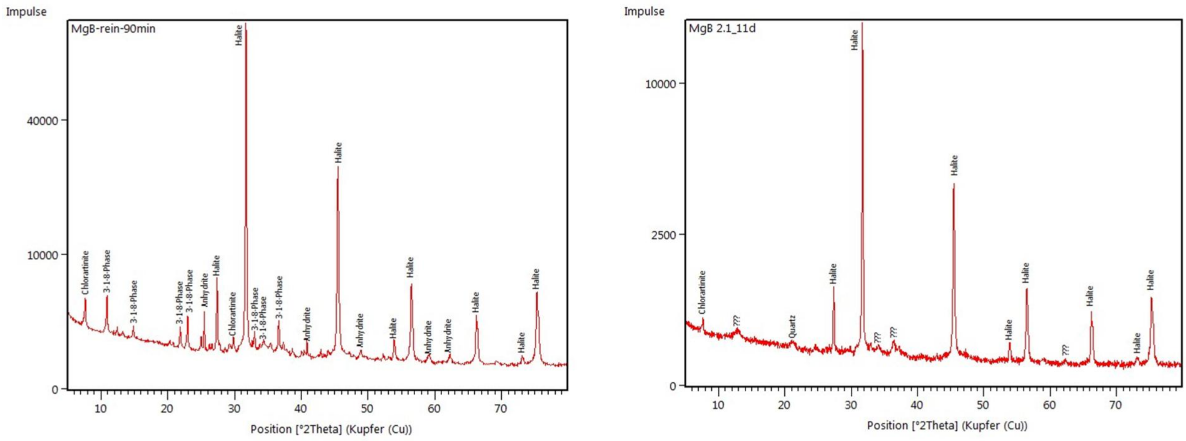


Figure 6.14: Phase composition of Sorel concrete before contact to NaCl-solution (left) and after 11 days of contact to NaCl-solution.

6.5.2 In-diffusion and through-diffusion experiments

In-diffusion and through-diffusion experiments have also been undertaken. For in-diffusion experiments, Sorel concrete samples, coated with Araldite on generate surface and one front side, were placed in tracer-spiked magnesium-rich NaCl-solutions. Literature research and calculations, which were undertaken to derive diffusions coefficients, had shown that the diffusion coefficient in concrete is small (1 x 10^{-13} to 1 x 10^{-15} m^2/s) (Mattigod *et al.*, 2012). Hence, all samples are still being tested to gather more robust intrusion profiles.

Through-diffusion-experiments are executed in special diffusion cells. The concrete sample is installed in the diffusion cell and tracer-spiked brine is passed across the bottom of the sample. A second, non-spiked brine is passed across the top of the sample and is analysed with regard to its development of tracer concentration over time. Because of the concentration difference between brine 1 (spiked) and brine 2 (non-spiked), a diffusional transport of tracer molecules from the bottom to the top of the sample is expected. The diffusion coefficient can be calculated on the basis of these experimental data. The thickness of samples and the duration necessary for saturation prior to starting diffusion experiments is determined in preparatory experiments. Through-diffusion experiments aim furthermore at investigating the kinetics of chemical reactions by diffusive corrosion processes. In principle, two scenarios are conceivable: on one hand a parallel progression of diffusion and corrosion may occur, on the other hand diffusion may occur faster than the process of corrosion. This circumstance will be investigated by analyses of the solid sample using x-ray diffraction.

6.5.3 Advective corrosion experiments

Advective transport is another transport mechanism in porous media which may affect corrosion of sealing elements. Corrosion as a result of advective transport and its consequences for the long-term sealing capacity will be investigated in two types of advection experiments: advection experiments which aim at reaction kinetics similar to diffusion experiments and experiments for investigating the influence of corrosion on porosity and permeability of the sealing material. A concrete sample was loaded with fluid pressure (NaCl- / IP21Mg-rich-solution) on one face in the first type of advection experiments. The effluent brine is collected on the other face. The sample surface is pressure-less cast in araldite in an advection cell. A simplified sketch of an advection cell is shown in Figure 6.15.

In regular intervals, permeability is measured and vessels for collecting brine are substituted. The individual brine samples will be analysed with regard to their composition. Additionally, the composition of each concrete sample will be investigated by decomposition and x-ray-diffraction. A conclusion and better understanding of corrosion mechanisms affected by advection processes in concrete is expected from these experiments.

The installation of the second advection experiment is very similar to the first experiment. The main difference is that the cylindrical concrete samples are surrounded by rock salt, thus exhibiting a circular contact zone. It is assumed that the contact zone is the primary pathway for brine and for the migration of nuclides. For these tests samples of the hollow rock salt cylinders with a salt concrete core described in Section 6.4.3 are used (see Figure 6.10). The samples are exposed to a confining pressure until permeability is minimized. This process simulates salt creep onto an *in situ* sealing element. Afterwards, samples are placed in advections cells in the same manner as described before for concrete samples.

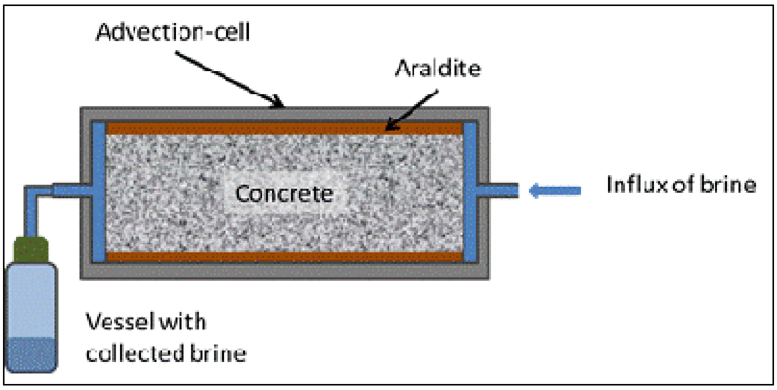


Figure 6.15: Simplified construction of an advection cell (left); Advection experiment in GRS laboratory (right).

A further experiment on combined samples is in progress (preliminary results in Figure 6.16). The sample was tested with a NaCl-solution in the beginning of test. A radial pressure of 5 MPa in the beginning and 10 MPa in further process were brought to the sample. After the contact zone was closed up to a permeability of 10^{-18} m^2/s, radial pressure was reduced to 2 MPa for relaxing the sample. In the next step, NaCl-solution was changed to a Mg-rich-solution (marked by the green line). Permeability increases in the beginning because of the high injection pressure and decreases after pressure was reduced.

After two month of contact to Mg-rich-solution, permeability starts to increase again. This phenomenon results from chemical processes in the salt concrete as former investigations at GRS have shown: If the Mg-rich-solutions is brought in contact to salt concrete, free hydroxide (OH^-) is fixed by magnesium and brucite ($Mg(OH)_2$) is precipitated.

As a result, pores are clogged by brucite and pH decreases to 8-9 (Phase 1). As result of the pH decrease, Portlandite ($(CaO)_2$) becomes unstable and decomposes into Ca- and hydroxide ions. After consumption of all Portlandite, the pH decreases further and stabilizing CSH-phases are dissolved. Now concrete loses its stability and permeability starts to increase (Phase 2) (Niemeyer *et al.*, 2014).

Hence, the dissolution of CSH-phases is also to observe in this experiment composition. But dissolution needs more time compared to the batch- and cascade-experiments with powdered concrete.

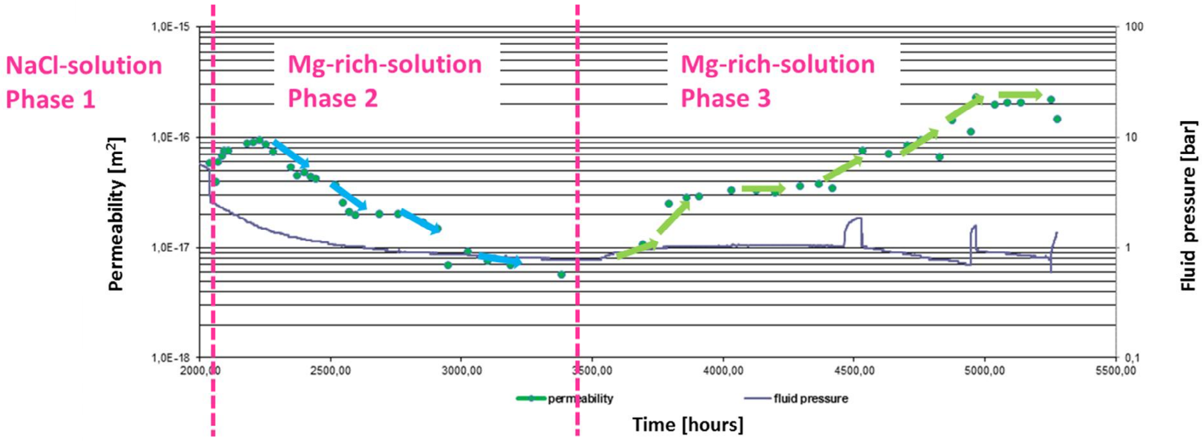


Figure 6.16: Development of permeability in a combined sample of salt concrete and rock salt in contact to NaCl- and Mg-rich-solutions.

6.6 Results of the THM-Ton Experimental Programme

Crushed claystone produced by excavation of repository openings is considered a favourable backfill and seal material for disposal of radioactive waste in clay formations, because of its many advantages such as chemical-mineralogical compatibility with the host rock, availability in sufficient amounts, low costs of material preparation and transport, and no or less occupancy of the ground surface for the excavated claystone. The crushed raw claystone shall be used for backfilling the repository openings and, mixed with bentonite, for sealing boreholes, drifts, and shafts. In the frame of THM-Ton Experimental Programme, GRS has characterised the excavated COX claystone and mixtures with bentonite with regard to the following important properties:

- Compressibility which controls the mechanical stability, interactions with the surrounding rock, and the hydraulic conductivity.
- Water adsorption capacity which determines the water saturation, retardation and the resulting swelling pressure.
- Swelling capacity which is required for sealing gaps between rock wall and seal, for supporting the surrounding EDZ against damage propagation and enhancing the sealing of the EDZ.
- Hydraulic conductivity which dominates water transport and radionuclide migration in the seal.
- Gas migration properties which control development of gas pressure in the repository to prevent the whole multi-barrier system from gas fracturing.

The geotechnical properties of the crushed claystone and mixtures with bentonite were determined also from the previous experiments, e.g., (Zhang, 2014). The most important results are reviewed briefly below.

Crushed claystone (COX) produced by excavation of the Andra's Bure URL drifts was used in the experiments. It is convenient to use the excavated material immediately for backfilling the repository openings without further treatment. Therefore, raw crushed claystone with grain sizes up to a diameter of 10, 20 and 32 mm was tested for its suitability for use as a backfill material. The grain size distribution curves are depicted in Figure 6.17. In addition, fine-grained claystone powder with grain sizes of d <0.5 mm was mixed with the MX-80 bentonite of d <0.5 mm in different ratios. The claystone-bentonite mixture is considered to be used as seal material.

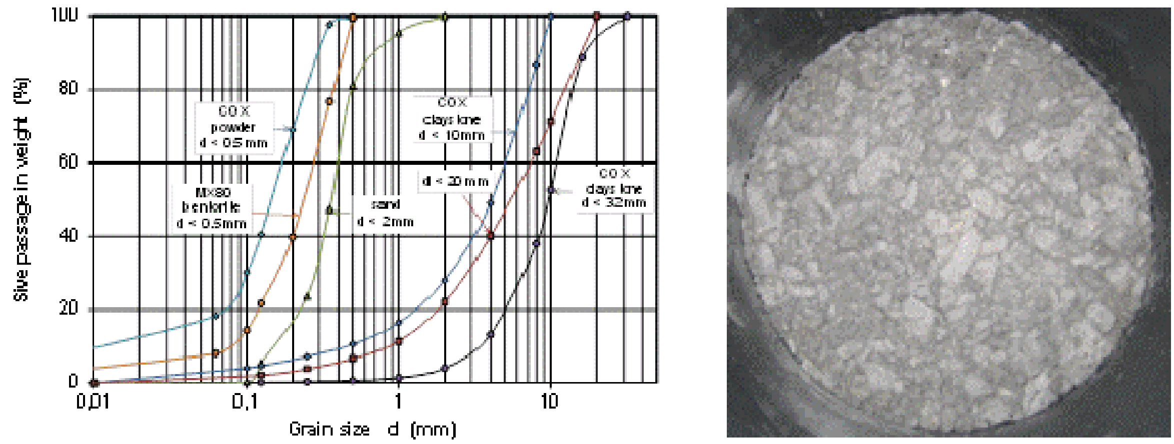
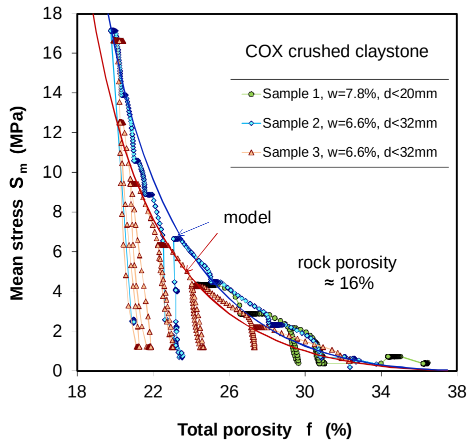


Figure 6.17: Grain size distribution curves of crushed claystone, bentonite and sand (left); Crushed claystone of d < 32 mm (right).

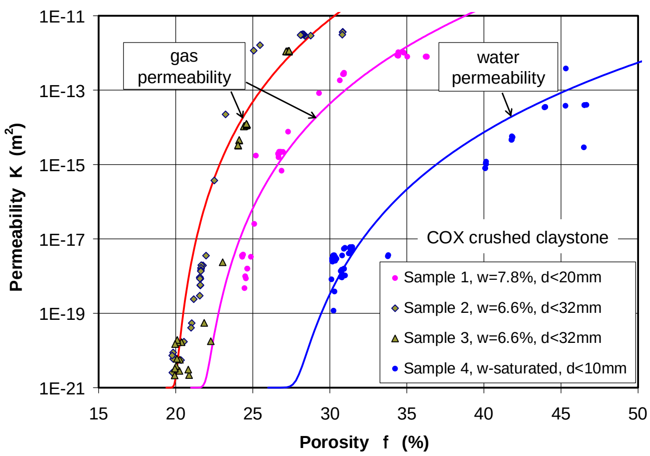
6.6.1 Compaction and permeability of crushed claystone backfill

Large-scale samples of 280 mm diameter and 640-680 mm lengths were prepared with the coarse crushed claystone of grain sizes of d <32 mm and d <20 mm. The initial dry densities reached by hand stamp, vibration and slight compression respectively vary in a range of 1.45-1.82 g/cm³. The samples were compacted in the GRS big triaxial apparatus with measurement of gas permeability.

Figure 6.18 presents the mean stress–porosity and permeability–porosity curves obtained on three samples of d <32 mm and d <20 mm. It is obvious that the stress–porosity curves are close to each other indicating little influence of the grain sizes. The compressibility of the material is relatively high, i.e., its resistance against external load is relatively low. At stresses of 12–16 MPa, corresponding to the overburden pressures at depths of 500–600 m, the backfill can be compacted to a low porosity of 20% which is close to the rock porosities of 14–18%. The compaction leads to a decrease in permeability. The permeability of the coarse-grained backfill is higher than that of the fine-grained material at a given porosity. It is interesting that the permeability measured by flowing water through a sample (d <10 mm) is much lower than the permeability to gas. The low water permeability is attributed to the effects of water-induced swelling and slaking of the clay grains into the pores. The compacted samples exhibited very low water permeability of 10-19 m^2 at a porosity of 30%, while such low gas permeability was observed at lower porosities of 20–25%, depending on the grain size.



Mean stress – porosity relation



Permeability - porosity relation

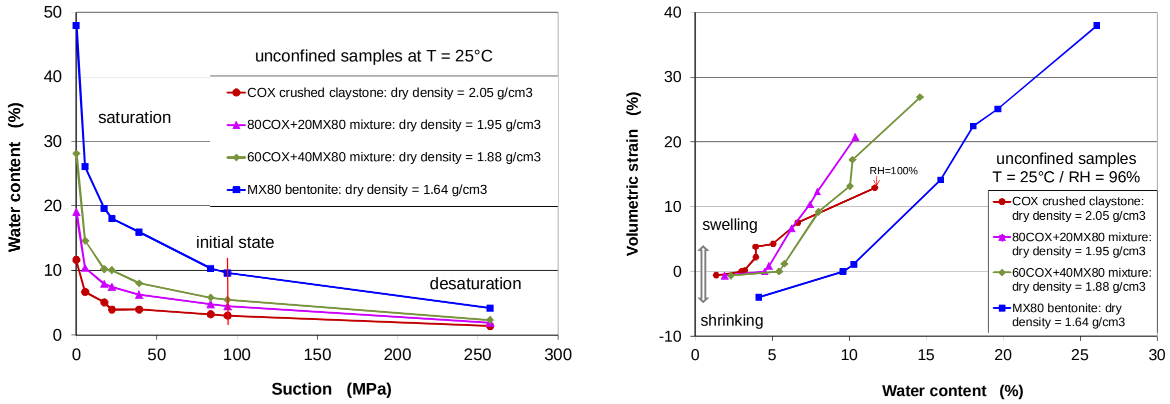
Figure 6.18: Compaction and permeability behaviour of claystone backfill samples.

6.6.2 Sealing properties of compacted claystone-bentonite mixture

As seal material, crushed claystone shall be mixed with bentonite and compacted to certain densities to meet specific requirements for sealing boreholes, drifts, and shafts. Commonly, the seals must have a sufficient supporting capacity against damage propagation of the surrounding rock, a certain swelling capacity for sealing of gaps and interfaces between compacted blocks and the surrounding rock, and a low hydraulic conductivity against migration of radionuclides with fluids.

Fine-grained COX claystone powder (d <0.5 mm) and coarse-grained claystone (d <10 mm) were mixed with MX-80 bentonite (d <0.5 mm) in different ratios of COX/MX-80 = 100/0, 40/60, 50/50, 60/40, 80/20 and 0/100. The claystone-bentonite mixtures were compacted in Oedometer cell to a maximum load of 30 MPa. This leads to different dry densities of the mixtures from 1.56 kg/m^3 at the pure bentonite to 2 kg/m^3 at the pure claystone. The higher the fraction of the crushed claystone in the mixture, the higher density can be achieved by application of the same load.

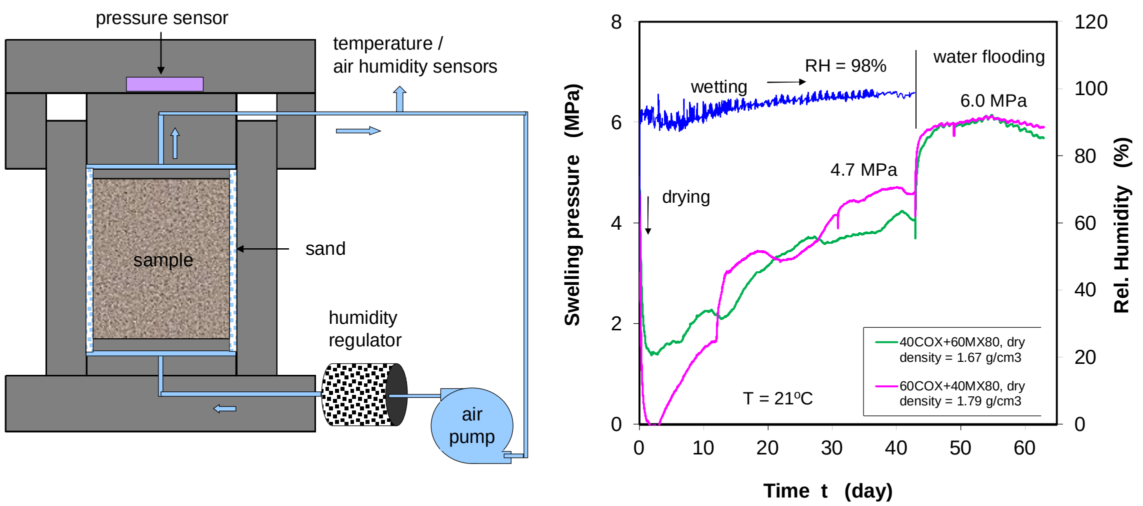
Figure 6.19 shows results of water retention and free swelling obtained on four compacted claystone-bentonite mixtures with dry densities of 1.6 to 2 kg/m^3 depending on the bentonite content. The water content of each mixture increases with decreasing suction or increasing humidity. The moisture uptake at a given suction is proportional to the bentonite content of the mixture. In the wet environment at zero suction or 100% relative humidity, all the mixtures can take up large amounts of water up to 12% for the crushed claystone and 48% for the bentonite. The increase in water content is accompanied by volume expansion. At the high humidity of 96-100%, the compacted claystone can expand to a volume increase of 12%, while the other mixtures expand even more up to 20-40% due to more bentonite content.



Water retention curves | Volumetric expansion with water uptake

Figure 6.19: Water retention and free swelling of compacted claystone-bentonite mixtures.

The swelling capacity of the compacted mixtures was measured in semi-confined conditions as shown in Figure 6.20.



Swelling pressure test on seal samples in semi-confined conditions

Response of swelling pressure to moisture change

Figure 6.20: Evolution of swelling pressures of compacted claystone-bentonite mixtures in semi-confined conditions during air drying/wetting and water flooding.

The annulus between sample and cell was filled with fine-grained quartz sand. The samples were pre-loaded to an axial load of 5 MPa and then fixed. Response of the axial stress was recorded to drying and wetting. The measurement shows that drying leads the axial stress dropping down to zero and wetting contrary causes a rapid increase in the stress to high levels: 3 MPa in the compacted pure claystone; 4.5–4.7 MPa in COX/MX-80 mixture in ratios of 60/40 and 40/60, and 7.6 MPa in the compacted pure bentonite. Flooding the compacted mixtures with synthetic pore water can increase the swelling pressure further to higher levels of 6–7 MPa. The observations suggest high swelling capacities of the compacted claystone-bentonite mixtures.

The water permeability of the compacted claystone-bentonite mixtures was determined by flowing synthetic clay water through the samples in Oedometer cells. Figure 6.21 presents the evolution of measured water permeability. It is obvious that all the compacted mixtures exhibited very low water permeabilities: $K_w = 2\times10^{-19}$ m^2 at 80COX+20MX-80, $K_w = 3\times10^{-20}$ m^2 at 60COX+40MX80, and $K_w = 2\times10^{-20}$ m^2 at 40COX+60MX-80. In case of the crushed claystone with grains of d <10 mm, the water permeability becomes very low, too, $K_w < 1\times10^{-19}$ m^2, as the porosity is below 30% (Figure 6.18). The very low water permeabilities of the compacted mixtures are close to that of intact rock ($K_w < 10^{-20}$ m^2).

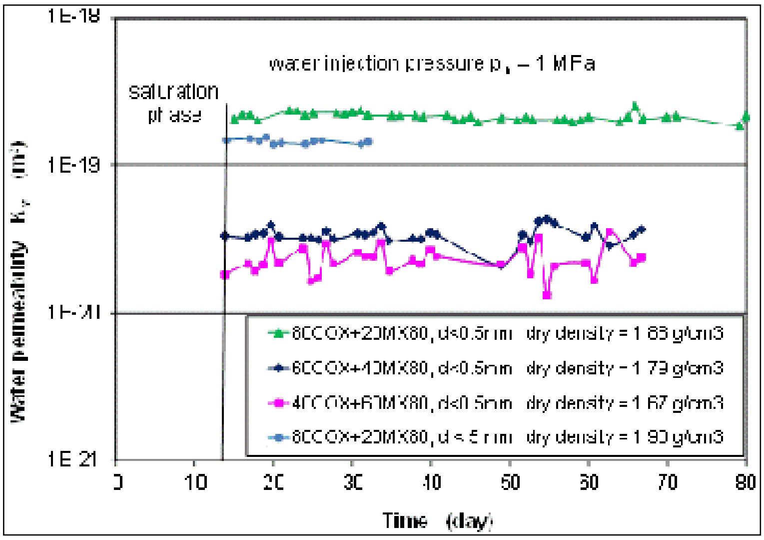


Figure 6.21: Evolution of water permeability of compacted claystone-bentonite mixtures.

Excavated raw claystone as backfill material and compacted claystone-bentonite mixtures as seal material have been comprehensively investigated. All the materials exhibit favourable geotechnical properties with respect to their barrier functions to prevent the release of radionuclides from a repository into the biosphere.

6.7 Lessons Learned

From the *in situ* and laboratory experiments performed so far during the ELSA Project, several methods and materials have been identified that can significantly improve the sealing capabilities of the long-term sealing element that forms one part of the current reference shaft sealing concept for repositories in salt dome structures. When using pure crushed salt for constructing the sealing element, as in the current reference design, a long time is required to achieve the full sealing abilities through porosity and thus permeability reduction in response to compaction by rock convergence. Using an admixture that includes fine clay, the initial porosity directly after installation can be significantly reduced. In addition, owing to inflowing brine the clay admixture starts to swell leading to a pore clogging, and therefore to an additional reduction in porosity and permeability. Generally, it can be stated that the time which is necessary to develop the full sealing abilities is reduced significantly.

The multi-layer bitumen seal “hard shell – soft core” has been tested successfully in the Sondershausen salt mine. The permeabilities have been measured to be $\leq 1 \times 10^{-21}$ m^2 which is close to the permeability of the salt host rock. The installation of this kind of seal has been proven to be quite easy without significant problems.

MgO concrete has been proven to be a suitable material for use as a sealing material. The *in situ* test was successful and the pressure development during hardening was higher than assumed leading to a good sealing of the contact zone to the salt rock.

7. Achievements, Lessons Learned and Future Challenges

This chapter provides an integrated discussion of the achievements of, and the lessons learned from, the DOPAS Project, in relation to the design and construction of full-scale experiments for plugs and seals, and for full-scale experiments in general. Achievements and lessons learned from the DOPAS Project experiments are discussed in seven sections:

- Section 7.1 discusses good practice for successful planning of full-scale experiments.
- Section 7.2 discusses the strengths and weaknesses of the different locations used for conducting the full-scale experiments in the DOPAS Project.
- Section 7.3 discusses the achievements and lessons learned with respect to design and installation of concrete components of plugs and seals.
- Section 7.4 discusses the achievements and lessons learned with respect to design and installation of bentonite components of plugs and seals.
- Section 7.5 discusses the achievements and lessons learned with respect to excavation of the experiments.
- Section 7.6 presents some of the challenges encountered during installation of the experiments and how these were overcome.
- Section 7.7 summarises logistical issues encountered during the design and construction of the experiments.
- Section 7.8 summarises health and safety issues addressed during the design and construction of the experiments.

Evaluation of the monitoring and measurement systems used in the DOPAS Project experiments, including their installation, is included in WP4 of the DOPAS Project and reported in Deliverable D4.4 (DOPAS 2016b).

7.1 Good Practice for Successful Experiment Planning

The design and construction of all the DOPAS Project experiments has been successful. All four of the full-scale tests planned for construction within the project have been successfully designed and constructed within the timeframe of the DOPAS Project, allowing pressurisation and monitoring, or dismantling to commence (where planned), so that the response, or at least the initial response of each plug/seal to pressurisation, and the findings from the dismantling process can be evaluated within the project. In addition, a successful programme of *in situ* tests and laboratory experiments has been undertaken in support of the development of shaft sealing concepts in Germany. Success of the design and construction activities has been underpinned, in part, through the use of good planning, through application of systematic design processes, by taking a stepwise approach involving studies at multiple scales, and through a mix of experienced and less-experienced staff.

There were some delays experienced by the experiments. For example, the POPLU experiment suffered a delay following a decision to change the rock excavation method from wire sawing to a method based on wedging and grinding, and EPSP suffered a delay following a decision for staff from CTU to undertake some of the construction work, rather than relying on contractors. However, both of these changes had benefits. Use of the wedging and grinding method to construct the POPLU experiment niche allowed a practical comparison to be undertaken with the wire sawing method used for construction of the

DOMPLU slot. The use of CTU staff to be more directly involved in the construction work of EPSP allows the experience and expertise so developed in house to be more readily utilised in future projects. These changes illustrate the need for flexibility to be incorporated in the plans for full-scale demonstrator tests. In the case of POPLU and EPSP, the changes could be implemented because contingencies had been allowed for in the experiment plans and schedules.

The DOPAS Project experiments have all benefitted from the development of comprehensive and detailed design bases, as recorded in the DOPAS Design Basis report (D2.1) (White *et al*., 2014). Each experiment had clear objectives, which allowed the project team to have a clear and common vision of how activities related to the global objective:

- The FSS experiment was built to prove the technical feasibility of constructing a seal at full scale, and was not saturated or otherwise pressurised to check the swelling pressure and hydraulic conductivity (these processes are examined in parallel experiments such as the REM experiment which is part of WP5 of the DOPAS Project).
- The objectives of EPSP were to test materials and technology, extending laboratory experience to the underground environment and to full scale, and to build the practical expertise of personnel.
- The DOMPLU experiment was part of an on-going SKB testing and demonstration programme. The overall objective of the test was to reduce uncertainties in the long-term performance of deposition tunnel plugs and in the description of their initial state. The DOMPLU experiment design represented a detailed iteration of the reference design rather than a fundamental change.
- The POPLU experiment investigated an alternative to the reference deposition tunnel plug design (which is similar to the DOMPLU plug design). Should the POPLU experiment ultimately be successful, including the analysis of the performance of the plug, which is included in D4.4 (DOPAS 2016b), there may be two options for the deposition tunnel plug available during the implementation stage, and, possibly, the wedge design might replace the dome design as the reference design. This experiment is also a first, from a regulatory point of view, as it was carried out in a future repository site identified as a nuclear installation, under scrutiny of the Finnish Nuclear Regulatory Authority (STUK).

All of the full-scale experiments were supported by a series of mock-ups undertaken at a range of scales to support the upscaling of the design from the laboratory to the full scale. For example, Posiva used mock-up castings of the concrete wedge and the tunnel back wall to fine tune the maximum aggregate size and admixture dosages of the preferred SCC mix (this underpinned a change of the aggregate maximum grain size in the upper and lower parts of the concrete wedge, where most of the reinforcement was located, from 32 mm to 16 mm), and Andra rearranged the transfer system used to emplace the bentonite pellet and powder mix based on metric-scale testing of the backfilling machine. Mock-up tests may be required by the regulator prior to implementation of a process within an operating repository; the experience from the DOPAS Project has illustrated the general benefit from undertaking such activities.

Another successful aspect of the organisation of the experiments was the incorporation of experienced staff within the project. In terms of experienced staff, the DOMPLU experiment benefitted from a senior member of SKB staff who had been involved in all of the full-scale plug and seal tests previously undertaken by SKB (since initial testing of plug designs in the

Stripa Mine) and CTU benefitted from the availability of a senior professor who had been involved in the construction of the concrete plugs in the Háje underground gas storage facility (see White *et al*., 2014 for a discussion of previous full-scale tests and other experience).

The DOPAS Project also benefitted from cooperation between staff from different waste management organisations. This is particularly demonstrated through the application, in the POPLU experiment, of lessons learned during installation of the DOMPLU experiment. These included lessons related to lead-through designs, instrumentation/monitoring plans and development of SCC.

7.2 Location of the Demonstrators

A range of different types of facility were used for the full-scale demonstrations:

- FSS was undertaken in a surface building.
- EPSP was conducted in a research facility in a former mine (the Josef URC and underground laboratory).
- DOMPLU was constructed in a purpose-built generic URL.
- POPLU was constructed in an underground rock characterisation facility at the site of a future spent fuel repository.

Locating the FSS experiment at the surface provided the benefit of flexible access to the test box for both monitoring of the construction and for the ultimate dismantling activities. The additional space in the building meant that it was feasible to store materials and ensure their availability for the filling operations. Although the experiment was not undertaken underground, it was feasible to control the ambient temperature and relative humidity so that the conditions were suitably representative.

The former mine used for EPSP is leased from the Czech Government by CTU and, although there are certain restrictions imposed by the lease (for example regarding the removal of rock from the facility), this arrangement allows significant flexibility in operating experiments within it. As noted above, one of the objectives of the EPSP experiment was to build or raise the practical expertise of personnel. In part, this was achieved because Josef is also used for teaching undergraduate and graduate courses and the students could gain experience of EPSP during this work. In addition, the DOPAS Project Training Course was held at Josef, which allowed further use of the EPSP demonstrator in training of staff.

The use of Josef for EPSP did have some drawbacks. As noted in Section 3.6, the Josef facility tunnel cross-sections are small, and only small volumes of concrete could be delivered for each spraying operation in EPSP, which slowed the construction of the concrete plugs. However, this time was utilised for maintenance and installation of the monitoring system, so was not critical. Also, the rock in the Josef facility is relatively weak. Significant ground improvement works were required and the response of the rock to pressurisation will have to be monitored carefully during the active stage of the experiment.

DOMPLU is located in the deepest sections of the Äspö HRL. This location was chosen to best represent the hydraulic pressures and stress conditions likely to be encountered in the SKB repository at Forsmark. The benefit of using a generic URL compared to other types of facility is that there is greater flexibility in using additional materials (e.g., organic superplasticisers in concretes), in order to speed up certain aspects of the emplacement process.

In contrast, POPLU was constrained as it was located in a repository site. This aspect was exploited by Posiva, who used the demonstrator experiment to test the RSC methodology for locating deposition tunnels and deposition tunnel plugs, and also used the POPLU experiment to test working practices related to working in an operating repository in which regulatory approval and strict compliance, such as with foreign materials acceptance, were required at key stages in the process.

In summary, all of the types of facilities used for the DOPAS Project demonstrator experiments have strengths and weaknesses, and the location for full-scale tests must be chosen to be compatible with the project objectives, as done in the DOPAS Project.

7.3 Design of Concrete Components of Plugs and Seals

A range of low-pH concrete mixes have been developed, tested and installed in the DOPAS Project concrete structures. Shotcretes and SCC have been used. Laboratory testing of these mixtures has demonstrated that they have the required curing temperature, hydraulic conductivity, shrinkage characteristics, strength, water interaction (pH of pore water or leachate) and rheology/segregation characteristics for application in repository plugs and seals.

All of the low-pH concrete mixes used in the DOPAS Project used a common approach to provision of the low pH. This included the substitution of cement used in high-pH concretes with silica fume and fly ash, with the addition of filler. Aggregates were locally sourced. All of the cement mixes needed to be accounted for the properties of the specific components, such as locally-sourced cements and aggregates, and had to be tailored to the boundary conditions of the experiments (e.g., dimensions and positioning of other structures such as reinforcement and monitoring systems).

Amongst the key properties of concrete used in plugs and seals are compressive strength, pH of leachate and curing temperature. High compressive strengths of ~40-60 MPa could be achieved for concrete mixes applied in all of the experiments. In FSS, the shotcrete used had a relatively low compressive strength of ~24 MPa, whereas the concrete used in EPSP, which incorporated glass fibres for additional strength, had a compressive strength of ~44 MPa. The compressive strength of the concrete used in POPLU was provided by a combination of the concrete and the reinforcement.

Challenges encountered during the emplacement of the concrete mixes have been overcome:

- The approaches used in DOMPLU allowed for addition of superplasticiser to the mixture following acceptance testing in order to ensure the appropriate rheological characteristics for the concrete.
- In FSS, the preparation of concrete mixes at the concrete plant had to be scrutinised and monitored to ensure compliance with the mix specification, for example to check the homogeneity of the dry material mixture.
- Posiva were able to tailor the design of the low-pH concrete mix used in POPLU to the specific conditions of the experiment, especially the nature of the reinforcement used in the concrete wedge and the type of superplasticiser.

However, there is further work to be undertaken on low-pH concrete mixes. Additional work should be done to evaluate the sensitivity of the concrete performances to marginal variations in component percentages and infer, by so doing, how robust a given mix can be. The role of certain additives e.g., plasticisers should also be further explored to see how dependant on the ratio between organic and mineral components a concrete mix composition can be. This is necessary to better assess the robustness of the safety demonstration (i.e. determine the role of complexing products in long-term safety). All of the concrete mixes used in the DOPAS Project experiments are novel materials, and, therefore, are not yet compliant with existing standards. Qualification of the concrete mixes is required, and this will be discussed further in WP4 of the DOPAS Project.

Testing of the B200 mix used by SKB has indicated that the shrinkage of the concrete may be less than expected. This may mean that it will not be necessary for the concrete to fully release from the rock for curing to occur without significant crack formation. The monitoring of the DOMPLU experiment concrete dome will contribute to further understanding phenomena related to concrete shrinkage and performance and also help to determine whether further work is needed on the shrinkage characteristics and release requirements.

The DOPAS Project experiments have demonstrated that reinforced and non-reinforced low-pH SCC can be emplaced successfully in repository-like conditions. Further evaluation of the poured concrete characteristics during the analysis work in WP4 will consider the performance of these different concretes. Nonetheless, the design and construction work undertaken in WP3 of the DOPAS Project has identified that a range of low-pH concrete mixes can be considered during the design of repository plugs and seals.

7.4 Design of Bentonite Systems for Plugs and Seals

A range of different bentonite systems have been introduced in the full-scale demonstrators, for example:

- In FSS, a mixed pellet and crushed pellet bentonite system is used to provide the swelling clay core, which has an internal diameter of 7.6 m. Although the original bentonite dry emplacement density requirement was 1620 kg/m^3, evaluation of the "dry density linked" swelling pressure for the bentonite undertaken in parallel with material testing showed that the required swelling pressure could be achieved with a dry emplacement density of 1500 kg/m^3.
- In EPSP, a composite bentonite seal composed of a lower section of high-density bentonite pellets further compacted *in situ* and an upper sprayed layer (shotclay), fills the experimental niche which has an approximate cross-section of 3.6 x 3.6 m. The bentonite target emplacement dry density is 1400 kg/m^3, which would provide a swelling pressure of 2 MPa and a hydraulic conductivity of 10^{-12} m/s.
- In DOMPLU, a bentonite block and pellet system is used for a backfill transition zone in which the swelling pressure of the backfill transition zone is reduced from approximately 6 MPa to 2 MPa, and for the bentonite seal, in which the installed dry density is dimensioned for a final swelling pressure of 2 MPa.

All of the bentonite systems have approaches to manage the heterogeneity in the system, particularly the vertical heterogeneity, which results in a lower density being achieved towards the top of the bentonite layers, i.e. close to the tunnel roof. In principal, this is achieved by exceeding the design specification in the lower parts of the layer to compensate for lower densities in the upper parts of the layer. In these cases, the overall emplaced dry density of the system meets the design specification. Furthermore, at small scales, it is observed that homogenisation of the saturated material appears with time.

7.5 Challenges during Excavation and Construction of the Experiment Site

The three DOPAS Project experiments that were conducted underground and therefore required excavation of rock were all situated in crystalline rock environments. Therefore, in the DOPAS Project, lessons learned regarding the excavation of sites for plugs and seals are mainly focused on crystalline rock concepts.

The underground full-scale experiments used a range of techniques for excavation of the plug/seal location. These techniques focused on in particular methods for reshaping and final reworking of the rock mass:

- In the EPSP experiment, the experiment niche was already excavated but had to be reshaped by excavation of the plug slots. This was undertaken using both hydraulic wedge splitting and pressure disintegration techniques.
- In the DOMPLU experiment, the slot for the concrete dome was excavated using a wire sawing method.
- In the POPLU experiment, the wedge excavation for the concrete mass was constructed using a combined wedging and grinding technique.

The excavation works undertaken in the DOPAS Project have demonstrated that a range of suitable techniques are available. Each technique has its own particular strengths and weaknesses. For example, wire sawing has the potential to deliver extremely smooth surfaces rapidly, but excavation methods need to be suitable for local *in situ* stress conditions, and wedging and grinding can be undertaken without the need for safety structures to be fixed into the rock, but may require repeated assessment to demonstrate that the requirements on the excavated surface have been met. The extent to which these strengths and weaknesses could influence the selection of an approach depend on local (site-specific) factors such as the nature of the rock mass and the regulatory regime under which the facility is operating (especially with respect to operational safety).

In both DOMPLU and POPLU, challenging criteria were set for the excavation of the plug slots. Scanning of the DOMPLU experiment slot demonstrated that the wire sawn cut deviated from the ideal plane by several centimetres in places. The experience with excavation of the POPLU experiment slot was similar. Development of the excavation techniques (e.g., development of methods to release stress prior to cutting of the rock for the wire sawing technique) may improve the performance of the excavation techniques. This issue may also be addressed through reconsideration of the requirements on the shape of the excavated slot and by undertaking the final design of the plug once the slot has been excavated and the final shape is known accurately.

As noted above, in FSS, rock excavation was not carried out as the experiment was located in a surface facility. Partial dismantling of the drift concrete liner in Cigéo will be an issue of its own, for which scale 1:2 tests are planned by Andra in its Bure URL in the years to come. These tests will provide further information on the potential for break-outs in drift and ILW vault seal locations, and provide a basis for excavation methodology development.

Work in the DOPAS Project has allowed testing of procedures for siting plugs and seals, including testing of the RSC by Posiva and testing of deposition hole characterisation techniques by SKB. Characterising crystalline rocks at the small scale can be particularly challenging, owing to the complexity of the fracture systems on a small-scale, and further work is required to match plug/seal requirements to parameters of the rock that can be characterised / predicted to a sufficient level of confidence. This may include further work to develop criteria related to the EDZ at the location of plugs and seals, and methods for detecting water-bearing fractures.

7.6 Challenges during Installation

Installation of the experiments was successfully completed within the timeframe of the DOPAS Project. However, there have been challenges associated with installation of materials close to the rock face (especially the roof) and with installation of the instrumentation.

This was found to be an issue in several experiments:

- Installation of the components in all of the experiments required a certain amount of manual labour, for example the use of chiselling and manually-operated compaction machines during the installation of EPSP. This introduces the potential for human error in the installation process and operator skill becomes crucial.
- All of the experiments required modified procedures for emplacement of material close to the roof space. Future design improvements should include better processes for installation of materials in this area.
- FSS was conducted in a surface facility, obviating the need for certain activities, such as removal of the tunnel lining before installation of the swelling clay core. Procedures for removal of the tunnel lining will need to be developed in the future.

Installation of the instrumentation required modifications to the design, for example use of LECA® pellets rather than LECA® blocks close to the rock lead-through flanges in the POPLU experiment. Installation of monitoring equipment is complex and time consuming, and specific monitoring developments were required to overcome the pressures expected to develop in the experiments.

In FSS, the main challenge was the pellet breakage which appeared with the progressive wearing of the augers (screws) installed inside the bentonite pellets conveyor pipes: this breakage contributed to a poor effective dry density of the emplaced material (a phenomenon due to the bridging effect created by the broken parts of the pellets preventing the bentonite powder from fillings the voids). This situation led to a change of screw steel characteristics to minimize the wearing effects.

Nonetheless, the DOPAS experiments demonstrated good application of quality control and construction procedures during the installation of the experiment components. This included, for example, meeting the requirements of the regulator during the installation of the POPLU experiment.

7.7 Logistics

Although all of the experiments were successfully installed within the course of the DOPAS Project, logistical problems were faced by all experiments. Of particular note were delays associated with machine availability. Delays were encountered, for example, when excavation machines required regular maintenance. Delays in one aspect of installation can have knock-on effects, especially when novel machinery is scheduled to be used elsewhere.

Delays were experienced by the FSS experiment when the screw augers required replacement parts, and, during later stages of the project, replacement screws and electric motors were made available. In the DOMPLU experiment, a replacement chiller was made available, but was not needed in the end.

Logistical issues slowed the construction process down for all experiments, examples include:

- Construction of the formwork for the DOMPLU experiment.
- The challenges with installing the bentonite tapes in the POPLU experiment.
- The manufacturer of the FSS bentonite mixture backfilling machine was bankrupted during the project and this caused an additional delay in the experiment implementation.

In addition, all experiments relied on contractors to mix concrete, which was appropriate, but delivery was challenging, for example the need to exchange the concrete mix from large trucks to smaller trucks in EPSP.

7.8 Health and Safety

The experience of the DOPAS Project has shown how health and safety concerns can slow and/or require changes to the installation of experiments:

- Consideration of health and safety led to a change from wire sawing to wedging and grinding for the POPLU experiment. For the DOMPLU experiment wire sawing was judged feasible and safe enough, as the method used incorporated a safety beam structure with netting that was arranged before the work began to protect workers from falling stones. However, it was judged that the methods proposed to provide safety in the POPLU experiment would require further work, in particular the method proposed for removal of the beam and associated nets following completion of the excavation work. The decision to adopt the wedging and grinding technique for the POPLU experiment also provided an opportunity to test an alternative to wire sawing.
- Management of dust was an issue for FSS. All workers had to wear face masks and the content of dust in the air had to be monitored to follow the workers' exposure. This was caused, in part, because the mining ventilation installed turned out to be insufficient.
- The manual nature of some of the installation processes used in the experiments introduces additional health and safety hazards into the underground. These include manual sawing of bentonite blocks and manual construction of formwork close to the experiment site. These hazards should be removed from the installation process as far as is practicable in the future industrial activities.

8. Conclusions

Within WP3 of the DOPAS Project, four full-scale experiments of plugs and seals have been designed and constructed. These include the FSS experiment in France, the EPSP experiment in the Czech Republic, the DOMPLU experiment in Sweden, and the POPLU experiment in Finland. In addition, a series of *in situ* tests and complementary laboratory investigations have been completed in Germany leading to improvements of individual sealing elements of shaft sealing concepts.

Design and construction of the experiments contribute to the readiness level of plug and seal installation in repositories in the near future. The completion of the experiment design and construction represents a successful collaboration between WMOs, research institutes and consultants.

The design and construction of the DOPAS experiments has demonstrated that plugs and seals are more challenging and complex sub-systems of the repository than previously recognised. The challenges and complexities of plug/seal design and construction can be met through available technology, methods and procedures currently available.

Common approaches to design and construction of plugs and seals have been developed and implemented by the WMOs responsible for different disposal programmes. These include, in crystalline rocks, the excavation of a slot through which the plug/seal can be keyed into the rock. In clay and salt host rocks, benefit is drawn from the creep properties of the rock to provide an effective seal when operating in conjunction with engineered features.

Common approaches also include the use of low-pH concrete and/or bentonite systems as the primary components of plugs and seals. Significant work on low-pH concrete and bentonite pellet, tape and block systems has been undertaken within WP3 of the DOPAS Project. Contact grouting is a common feature to all four of the full-scale tests.

The host rock can significantly impact the installation of plugs and seals. Weak rock, the presence of water-bearing fractures and formation of break-outs can be challenging, but approaches have been developed and demonstrated in WP3 of the DOPAS Project to overcome these challenges.

Logistics is a significant issue for plugs and seals. There may be multiple components requiring installation and appropriate time must be allowed for these materials to be installed and evolve prior to installation of the next component. There may be issues associated with manpower and machinery availability (and performance). Therefore, contingency planning, such as the provision of back-ups and spares may be necessary. Contingencies also need to be built into project plans and schedules.

Application of the lessons learned from the DOPAS experiments and feedback to reference designs will be considered in WP4 of the DOPAS Project and reported in Deliverable D4.4. This will include an analysis of further work required to develop designs so that they are ready for implementation in repositories in the near future.

9. References

Andersson, M., Ervanne, H., Glaus, M. A., Holgersson, S., Karttunen, P., Laine, H., Lothenbach, B., Puigdomenech, I., Schwyn, B., Snellman, M., Ueda, H., Vuorio, M., Wieland, E. and Yamamoto, T. (2008). Development of Methodology for Evaluation of Long-term Safety Aspects of Organic Cement Paste Components. Posiva Working Report 2008-28.

Andra (2005). Dossier Argile 2005 Andra Research on the Geological Disposal of High-level Long-lived Radioactive Waste. Results and Perspectives.

Andra (2016). FSS Experiment Summary Report. DOPAS Work Package 4 Deliverable D4.8.

Börgesson L., Sandén T., Andersson L., Johannesson L-E., Goudarzi R. and Åkesson M. (2015). End plug for Deposition Tunnels – Preparatory Modelling and Tests of the Sealing and Draining Components. Clay Technology. SKB R-Report, R-14-25.

Bosgiraud, J.-M. and Foin, R. (2013a). FSS Tunnel Model Design Report. DOPAS Work Package 3 Deliverable D3.2.

Bosgiraud, J.-M. and Foin, R. (2013b). FSS Drift Model Construction Report. DOPAS Work Package 3 Deliverable D3.10.

Bosgiraud, J.-M. and Foin, R. (2014). Test Report on FSS Cast in Box Concrete with Low pH Concrete Formulas for FSS and Laboratory work on the Performance of Low-pH Concrete for FSS. DOPAS Work Package 3 Deliverable D3.8.

Bosgiraud, J.-M.; Foin, R. and Bethmont, S. (2014). Report on FSS Cast Concrete Plug Construction. DOPAS Work Package 3 Deliverable D3.11.

Breidung, K.P. (2002). Forschungsprojekt Schachtverschluss Salzdetfurth Schacht II. Abschlussbericht - Kurzfassung. Hg. v. K+S AG. Bad Salzdetfurth (in German).

Cristescu, N. D. and Hunsche, U. (1998). Time Effects in Rock Mechanics. John Wiley and Sons, Chichester.

Czaikowski, O. and Wieczorek, K. (2016). DOPAS Work Package 3, Deliverable D3.31. Final technical report on ELSA related testing on mechanical-hydraulical behaviour - LASA, 29 February 2016.

DOPAS (2016a). WP2 Final Report: Design Basis for DOPAS Plugs and Seals. DOPAS Work Package 2, Deliverable D2.4.

DOPAS (2016b). WP4 Integrated Report. Summary of Progress on Design, Construction and Monitoring of Plugs and Seals. DOPAS Work Package 4 Deliverable D4.4.

DOPAS (2016c). WP5 Final Integrated Report. DOPAS Work Package 5 Deliverable D5.10.

DOPAS. (2016d). DOPAS Final Project Summary Report. DOPAS Work Package 6 Deliverable D6.4.

EC-JRC (2008). The Eurocodes: Implementation and use. Booklet B1, Joint Research Centre (JRC) and Enterprise and Industry Directorate General (DG ENTR), European Commission.

Flansbjer M. and Magnusson J. (2013a). Creep Properties at High Stress Levels of Concrete for Deposition Tunnel Plugs. SKB P-Report, P-13-37.

Flansbjer M. and Magnussin J. (2013b). Mechanical Properties of Rock-concrete Interface at the Concrete Plug. SKB P-Report, P-13-38.

Glaubach, U., Hofmann, M., Gruner, M., Königer, F., Emmerich, K., Schuhmann, R., Grafe, F. & Wils-nack, T. (2016). Teilbericht zum Arbeitspaket 4: Halbtechnische Versuche zu den Arbeitsschritten 2.1 bis 2.6 Im Rahmen des Vorhabens „Schachtverschlüsse für Endlager für hochradioaktive Abfälle“ ELSA-Phase 2: Konzeptentwicklung für Schachtverschlüsse und Test von Funktionselementen von Schachtverschlüssen, Technische Universität Bergakademie Freiberg, Institut für Bergbau und Spezialtiefbau (in German).

Grahm, P. and Karlzén, R. (2015). System Design of Dome Plug. Experiences from Full-scale Wire Sawing of a Slot Abutment for the KBS-3V Deposition Tunnel Plug. SKB Report R-14-24.

Grahm, P.; Malm, R.; Eriksson, D. (2015). System Design and Full-scale Testing of the Dome Plug for KBS-3V Deposition Tunnel. Main report. SKB Technical Report TR-14-23. Content identical to DOMPLU Summary Report; DOPAS Work Package 4 Deliverable D4.3.

Haaramo, M. and Lehtonen, A. (2009). Principle Plug Design for Deposition Tunnels. Working Report 2009-38.

Herold, P. (2011). Diplomarbeit. Auswertung der Bauausführung bisheriger Asphalt- und Bitumendichtungen in Schachtverschlüssen im Salinar und Schlussfolgerungen für zukünftige Dichtsysteme auf Basis von Asphalt und Bitumen. Hg. v. Institut für Bergbau und Spezialtiefbau TU Bergakademie Freiberg (in German).

Herold, P. and Müller-Hoeppe, N. (2013). Safety Demonstration and Verification Concept – Principle and Application Examples. Technical Report (Extraction from Kudla *et al*., 2013), DBE TECHNOLOGY, Peine.

Hilar, M. and Pruška, J. (2011). Fiber Concrete – Construction Material of Underground Structures. *fib* Symposium Prague, 2011.

Holt, E. (2014). POPLU Concrete Test Memorandum Concrete Mix Design and Performance Study. DOPAS Work Package 3 Deliverable D3.27.

Holt, E. and Dunder, J. (2014). Detailed Design of POPLU Deposition Tunnel End Plug. DOPAS Work Package 3 Deliverable D3.24.

Holt and Koho, P. (2016). POPLU Experiment Summary Report. DOPAS Work Package 4 Deliverable D4.5.

Imbert, C., Billaud, P., Touzé, G., Villar, M.V., García-Gutiérrez, M. and Mingarro, M. (2005). RESEAL-II. A Large-scale *In Situ* Demonstration Test for Repository Sealing in an Argillaceous Host Rock – Phase II. Contract N° FIKW-CT-2000-00010 Final Report on Laboratory Tests (WP1). Hg. v. 5th Euratom Framework Programme 1998-2002 Key Action: Nuclear Fission European Commission.

Itasca Consulting Group, Inc. (2012). FLAC3D – Fast Lagrangian Analysis of Continua in Three-Dimensions, Ver. 5.0, Minneapolis, Itasca.

Jantschik, K. and Moog, H. (2016). DOPAS Work Package 3, Deliverable D3.29. Final technical report on ELSA related testing on chemical-hydraulical behaviour – LAVA.

Jobmann, M. (2013). Requirements for Shaft Seals for HLW Repositories in Germany – Technical Report, DBE TECHNOLOGY GmbH, Peine, Germany.

Kappei, G. and Eikmeier, V. (2006). Verfüll- und Verschlusskonzept für die Schließung der Tagesschächte 2 und 4 des Bergwerkes Asse. (Schachtverschlusskonzept). - Bericht der GSF - Forschungszentrum für Umwelt und Gesundheit. 16/ERV/GH/BZ/0001/00, 15.11.2006 (in German).

Köhler, S., Weber, H., Garitte, B. and Müller, H. (2015). Backfilling a Horizontal Emplacement Tunnel with Granular Bentonite – Machine Development, Pre- and Mock-up Tests and Application at the Mont Terri URL. Proceedings of the LUCOEX Conference and Workshop, Oskarshamn, Sweden, 2nd June 2015.

Köster, R. (2009). Das Verschlusskonzept für das Endlager Konrad. Fachgespräch Verschlussysteme. Bundesamt für Strahlenschutz. Freiberg, Oktober (in German).

Kosunen, P. (2014). URCF RSC Work Memorandum (POPLU). DOPAS Work Package 3 Deliverable D3.26.

Krauke, W. and Fliß, T. (2008). Stilllegung ERAM - Konzeptplanung und Nachweisführung für ein Abdichtungsbauwerk im Hauptanhydrit aus Magnesiabinder, BfS (in German).

Kudla, W., Dahlhaus, F., Glaubach, U., Gruner, M., Haucke, J., Hofmann, M., Wasowiecz, B. (2009). Diversitäre und redundante Dichtelemente für langzeitstabile Verschlussbauwerke, TU BAF Institut für Bergbau und Spezialtiefbau, Freiberg (in German).

Kudla, W., Schreiter, F., Gruner, M., Jobmann, M., Bollingerfehr, W., Müller-Hoeppe, N., Herold, P., Freyer, D., Wilsnack, T. and Grafe, F. (2013). Schachtverschlüsse für Endlager für Hochradioaktive Abfälle – ELSA Teil 1–, TU Bergakademie Freiberg (TUBAF) und DBE TECHNOLOGY, Freiberg, Peine (in German).

Kudla, W., Gruner, M., Herold, P. and Jobmann, M. (2015). Schachtverschlusskonzepte für Zukünftige Endlager für Hochradioaktive Abfälle für die Wirtsgesteinsoptionen Steinsalz und Ton. Mining Report Glückauf 151 No. 1, Herne, Germany (in German).

Kudla, W., Glaubach, U., Gruner, M., Herold, P., Hofmann, M., Jobmann, M. (2016). Shaft Sealings for High-level Waste Repositories – Final Synthesis Report ELSA Phase 2–, TU Bergakademie Freiberg (TUBAF) and DBE TECHNOLOGY, Freiberg, Peine.

Malm, R. (2012). Low-pH Concrete Plug for Sealing the KBS-3V Deposition Tunnels. SKB Report R-11-04.

Mathern A. and Magnusson J. (2014). Experience of Low-pH Concrete Mix B200 – Material Properties from Laboratory Tests and Full-scale Castings. SKB P-Report, P-13-38.

Mattigod, S.V., Wellman, D.M., Bovaird, C.C., Parker, K.E., Recknagle, K.P., Clayton, L. and Wood, M.I. (2012). Diffusion of Radionuclides in Concrete and Soil, USA.

McEwen, T. (ed.); Aro, S.; Kosunen, P.; Mattila, J.; Pere, T.; Käpyaho, A. and Hellä, P. (2012). Rock Suitability Classification – RSC 2012. Posiva Report 2012-24.

Meyer, T., Herbert, H.J. and Schmidt-Döhl, F. (2003). Endlager Morsleben – Korrosion von Salzbeton Durch Salinare Lösungen, BfS P 180 (in German).

Müller-Hoeppe, N., Engelhardt, H.-J., Lerch, C., Linkamp, M.; Buhmann, D.; Czaikowski, O., Herbert, H.-J., Wieczorek, K. and Xie, M. (2012). Integrität Geotechnischer Barrieren Teil 1: Vorbemessung. Bericht zum Arbeitspaket 9, Vorläufige Sicherheitsanalyse für den Standort Gorleben, GRS-287, Gesellschaft für Anlagen- und Reaktorsicherheit, Köln (in German).

Niemeyer, M., Wilhelm, S., Hagemann, S. and Herbert, H.J. (2014): Stilllegung ERAM - Zusammenfassende Auswertung von Experimenten und Modellrechnungen zur Korrosion von Salzbeton mit IP21-Lösung, BfS (in German).

Nold, A.L., Zuidema, P. and Blümling, P. (2002). Konzept für die Anlage und Projekt Opalinuston, Konzept für die Anlage und den Betrieb eines Geologischen Tiefenlagers, Entsorgungsnachweis für Abgebrannte Brennelemente, Verglaste Hochaktive Sowie Langlebige Mittelaktive Abfälle. Technischer Bericht 02-02, Nationale Genossenschaft für die Lagerung radioaktiver Abfälle. Wettingen (in German).

Olivella, S., Gens, A., Carrera, J. and Alonso, E.E. (1996). Numerical Formulation for a Simulator (CODE-BRIGHT) for the Coupled Analysis of Saline Media. Eng. Comput., 1: 87– 112.

Parkkinen, I. and Mäntynen, A. (2013). Concrete Wedge Plug Design Report (Betonitulpan Laskentaraportti), Finnmap Consulting Oy, Memo to Posiva Oy, 23.5.2013 (in Finnish).

Posiva (2012a). YJH-2012. Nuclear Waste Management at Olkiluoto and Loviisa Power Plants: Review of Current Status and Future Plans for 2013-2015. Eurajoki, Finland: Posiva Oy, 370 p.

Posiva (2012b). Backfill Production Line 2012. Design, Production and Initial State of the Deposition Tunnel Backfill and Plug. Posiva Report POSIVA 2012-18.

Raivio, P. and Hansen, J. (2007). Technical Performance of Cementitious Grouting Materials for ONKALO: Laboratory Tests 2006. Posiva Working Report 2007-78.

Ranta-Korpi, R., Karttunen, P. and Sievänen, U. (2007). R20 programme: Field Testing of Grouting Materials. Posiva Working Report 2007-102.

Rauche, H., Sitz, P., Lukas, V., Rumphorst, K., Lippmann, G., Wagner, K. and Teichert, T. (2003). Nachweisführungen zur Langlebigkeit, zur Tragfähigkeit und zur Gebrauchstauglichkeit der Schachtverschlüsse ERA Morsleben. Hg. v. Bundesamt für Strahlenschutz. Erfurt, Freiberg, Kassel, zuletzt geprüft am 19.09.2011 (in German).

Rautioaho, E., Börgesson, L., Åkesson, M., Kristensson O., Valli, J., Hakala, M., Pintado, X. and Koskinen, K. (2016). Assessment of Watertightness and Mechanical Integrity of POPLU Plug. DOPAS Work Package 5 Deliverable D5.11.

Rübel, A., Buhmann, D. and Kindlein, J. (2014). Status Report on Conceptual and Integrated Modelling Activities. DOPAS Work Package 5 Deliverable D5.6.

Sainsbury, B., Pierce, M. and Mas Ivars, D. (2008). Simulation of Rock Mass Strength Anisotropy and Scale Effects using a Ubiquitous Joint Rock Mass (UJRM) Model. Proceedings First International FLAC / DEM Symposium on Numerical Modelling, 25–27 August, Minneapolis, US. Minneapolis, Itasca.

Schmidt, M.W., Fruth, R., Stockmann, N., Birthler, H., Boese, B., Storck, R. *et al*. (1994). Schachtverschlüsse für untertägige Deponien in Salzbergwerken. - Vorprojekt -. Band 2: Anhang. Hg. v. GmbH GSF-Forschungszentrum für Umwelt und Gesundheit. Braunschweig (in German).

Sitz, P. (2001). Langzeitstabile Verschlussbauwerke in Strecken und Schäcten. In: Bergbau (11), S. 520–526 (in German).

SKB (2010a). Design, Production and Initial State of the Backfill and Plug in Deposition Tunnels. SKB Technical Report TR-10-16.

SKB (2010b). Design, Construction and Initial State of the Underground Openings. SKB Technical Report TR-10-18.

Studer, J., Ammann, W., Meier, P., Müller, Ch. and Glauser, E. (1984). Verfüllen und Versiegeln von Stollen Schächten und Bohrlöchern. Technischer Bericht 84-33. Nationale Genossenschaft für die Lagerung radioaktiver Abfälle (in German).

SÚRAO (2012). Update of the Reference Project of a Deep Geological Repository in a Hypothetical Locality. Accompanying Report. Report EGP 5014-F-120055.

SÚRAO and CTU (2016). EPSP Experiment Summary Report. DOPAS Work Package 4 Deliverable D4.7.

Svoboda, J., Vašíček, R., Smutek, J. and Šťástka, J. (2015). Detailed Design of EPSP Plug. DOPAS Work Package 3 Deliverable D3.15.

Trpkošová, D., Dobrev, D., Gondolli, J., Havlová, V. and Hokr, M. (2013). Methodology of Complex Safety Assessment of the Deep Repository. Final report, project no. FR-TI1/362. [Research report]. Řež u Prahy: ÚJV, 153 s. (in Czech).

Vašíček, R., Svoboda, J., Večerník, P. and Dvořáková, M. (2013). Testing Plan for EPSP Laboratory Experiment. DOPAS Work Package 3 Deliverable D3.16.

Vašíček, R., Levorová, M., Hausmannová, L., Šťástka, J., Večerník, P., Trpkošová, D. and Gondolli, J. (2014). Interim Results of EPSP Laboratory Testing. DOPAS Work Package 3 Deliverable D3.17.

Villar, M. V., 2007. Water Retention of Two Natural Compacted Bentonites. Clays and Clay Minerals, vol. 55, No. 3, 311-322.

Vogt, C., Lagerblad, B., Wallin, K., Baldy, F. and Jonasson, J-E. (2009). Low-pH Self-compacting Concrete for Deposition Tunnel Plugs. SKB R-09-07.

White, M.J., Doudou, S. and Neall, F.B. (2014). Design Bases and Criteria. DOPAS Work Package 2 Deliverable D2.1.

Wilsnack, T., Sitz, P., Heinemann, K.-H., Rumphorst, K. and Hunstock, F. (2008). Flüssigkeitsdichte Verwahrung von Schächten. In: Kali und Steinsalz (3), S. 24–35.

Zhang, C.L. (2014). Characterization of Excavated Claystone and Claystone-Bentonite Mixtures as Backfill/Seal Material. PEBS International Conference on the Performance of Engineered Barriers, February 6-7, 2014, Hannover.

Zhang, C-L. (2016). DOPAS Work Package 3, Deliverable D3.32. Final report on the sealing behaviour of fractured claystone and seal materials (THM-Ton).