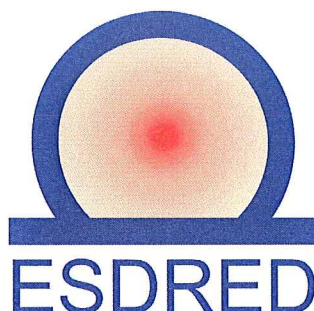




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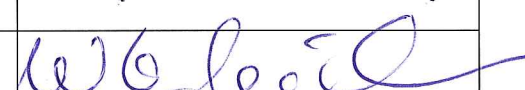
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(Contract Number: FI6W-CT-2004-508851)

## DELIVERABLE 8.1 OF MODULE 4 WORK PACKAGE 3.2

### Report on Short Plug Test Results

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1/47



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# CONTENTS

<b>1</b>	<b>EXECUTIVE SUMMARY .....</b>	<b>4</b>
<b>2</b>	<b>INTRODUCTION AND SCOPE .....</b>	<b>6</b>
<b>3</b>	<b>TEST LAYOUT.....</b>	<b>7</b>
<b>4</b>	<b>SHORT PLUG CONSTRUCTION .....</b>	<b>8</b>
4.1	PRELIMINARY TESTS .....	8
4.2	SITE PREPARATION .....	9
4.3	CONDITIONING OF THE WATER CHAMBER .....	9
4.4	CONSTRUCTION OF THE SHORT PLUG .....	11
4.4.1	Preliminary shotcrete test (above ground).....	11
4.4.2	Short plug construction.....	12
4.4.3	Short plug hardening .....	13
4.4.4	Auxiliary system installation .....	13
<b>5</b>	<b>SHORT PLUG TEST .....</b>	<b>15</b>
5.1	INITIAL ASSUMPTIONS AND SCOPING CALCULATIONS.....	15
5.2	FIRST TEST SERIES (February 2006).....	16
5.3	SECOND TEST SERIES (March 2006).....	17
5.4	THIRD TEST SERIES (March 2006) .....	20
5.5	TEST RESULTS .....	22
<b>6</b>	<b>SHORT PLUG DISMANTLING.....</b>	<b>24</b>
6.1	PURPOSE AND APPROACH.....	24
6.2	DESCRIPTION OF DISMANTLING OPERATION .....	24
6.2.1	Preliminary works.....	24
6.2.2	Plug front inspection and core sampling.....	24
6.2.3	Excavation of an access to the water chamber .....	25
6.2.4	Water chamber inspection .....	26
6.2.5	Plug demolition.....	30
6.2.6	Final works .....	33
6.3	PLUG SAMPLING AND ANALYSIS .....	34
6.3.1	Horizontal cores.....	34
6.3.2	Inclined boreholes and resin injection .....	35
6.3.3	Sampling of concrete in contact with rock .....	37
6.3.4	Vertical sampling.....	37
6.3.5	Analysis of samples .....	38
6.4	SUMMARY OF THE DISMANTLING OBSERVATIONS .....	42
<b>7</b>	<b>INTERPRETATION AND CONCLUSIONS.....</b>	<b>44</b>
7.1	LOADING TEST BACK-ANALYSIS AND INTERPRETATION .....	44
7.2	CONCLUSIONS .....	46
<b>8</b>	<b>REFERENCES .....</b>	<b>47</b>



# 1 EXECUTIVE SUMMARY

This report is the last in a series of documents describing the work carried out and results obtained in the research and demonstration project entitled *Temporary Sealing Technology*, performed within the framework of the Integrated Project *Engineering Studies and Demonstration of Repository Designs* (IP ESDRED). The Temporary Sealing Technology project was devised to study the feasibility of formulating cementitious materials of low-pH (below 11) that might be used in the construction of an underground repository in accordance with a set of Functional Requirements previously defined for each of the applications envisaged.

The prime application investigated was the formulation of low-pH concrete that might be used in the construction of shotcrete plugs, designed to efficiently substitute (time and cost efficient) the conventional mass concrete plugs - keyed in the rock – used to close the repository disposal galleries.

The formulation of a low-pH concrete that might be pumped for shotcrete applications was successfully achieved and reported in **DELIVERABLE D 2.1 & D 3.1 Design of low-pH concrete for the construction of shotcrete plugs**.

The next step was to construct and test a plug under realistic conditions (repository conditions). This report describes the construction of a short low-pH shotcrete plug; the testing of its structural behaviour when loaded up to failure; the dismantling and sampling of the plug; the analysis of the data gathered and the conclusions reached.

Prior to the construction of the plug in a 1.85 m diameter gallery excavated by the push boring technique in the Äspö underground laboratory (Sweden), a series of field tests was performed to check that the low-pH concrete formula designed complied with the functional requirements established. A one meter long horizontal plug was, therefore, constructed at Äspö, together with the auxiliary system needed to perform the loading test and monitor its behaviour.

A period of some three months was allowed for hardening of the shotcrete before starting the loading test. The axial loading of the plug was attained by pressurising a water chamber at the back end of the plug. Although the water chamber was not sealed tight as expected, and significant water leakage developed in the system, sufficient mechanical pressure was developed in the plug to induce the failure of the locked-in forces existing between the plug and the rock. Failure occurred at an applied pressure of 27.09 bars, with a non-recovered displacement of the plug of 0.4 cm. Four additional loading cycles in two episodes were applied to the plug after the failure episode to gain more insight into its behaviour. In each of the two loading cycles of the first episode, the plug moved as a solid body when the axial pressure applied was approximately 24 bars. During the second episode the plug moved again when the pressure applied, in each one of the cycles, was about 20 bars. The total displacement measured in the plug was 3.5 cm.

A sampling (drill coring) and dismantling operation of the plug was devised, in search of evidence of the plug failure mechanism. The lack of evidence of water circulation throughout the periphery of the plug at its contact with the rock and the presence of slickensides, developed both in the concrete and rock at the interface, demonstrates the tightness of the shotcrete plug, even when axially loaded after failure.

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Mod4-WP3.2-D8.1– Report on Short Plug Test Results

Dissemination level: **PU**

Date of issue of this report: **Revision 1 on 29 May 2007**

4/47



It is concluded from the research work performed that low-pH concrete may be formulated and used for the construction of shotcrete plugs under repository conditions. The concrete formula selected did not, as expected, present either significant shrinkage or temperature increase phenomena due to the very low cement content in the formula and the high moisture content of the emplacement site. The failure mode of the plug under axial loading corroborated the hypothesis established in the design: that the interlocking effect (represented in the scoping calculations by the dilation angle) plays a crucial role in the shear strength of the confined rock-shotcrete interface. The results of the failure test allow very accurate predictions of plug performance under loading to be performed by back calculation.

The demonstration of the feasibility of utilising shotcrete plugs in combination with highly compacted bentonite for the permanent sealing of disposal galleries represents an improvement and a step towards the optimisation of the geological disposal concept design.

## 2 INTRODUCTION AND SCOPE

Low-pH shotcrete may be required in the construction of underground repositories. Although the behaviour of standard shotcrete in conventional construction works is well known, there is no experience of the workability or the performance of shotcrete formulated to obtain a final low-pH product.

The experience gained during the construction of a three meter long shotcrete plug, based on OPC cement, in the Febex “In situ” Test at Grimsel [i] provides evidence that when using shotcreting as a construction method a better contact (bonding) between the plug and the host rock might be achieved and costs significantly optimised. A full demonstration test, based on that experience but using low-pH cements in the shotcrete formulation, was proposed as part of the ESDRED Integrated Project, to check the construction feasibility and performance of this type of plug under realistic conditions.

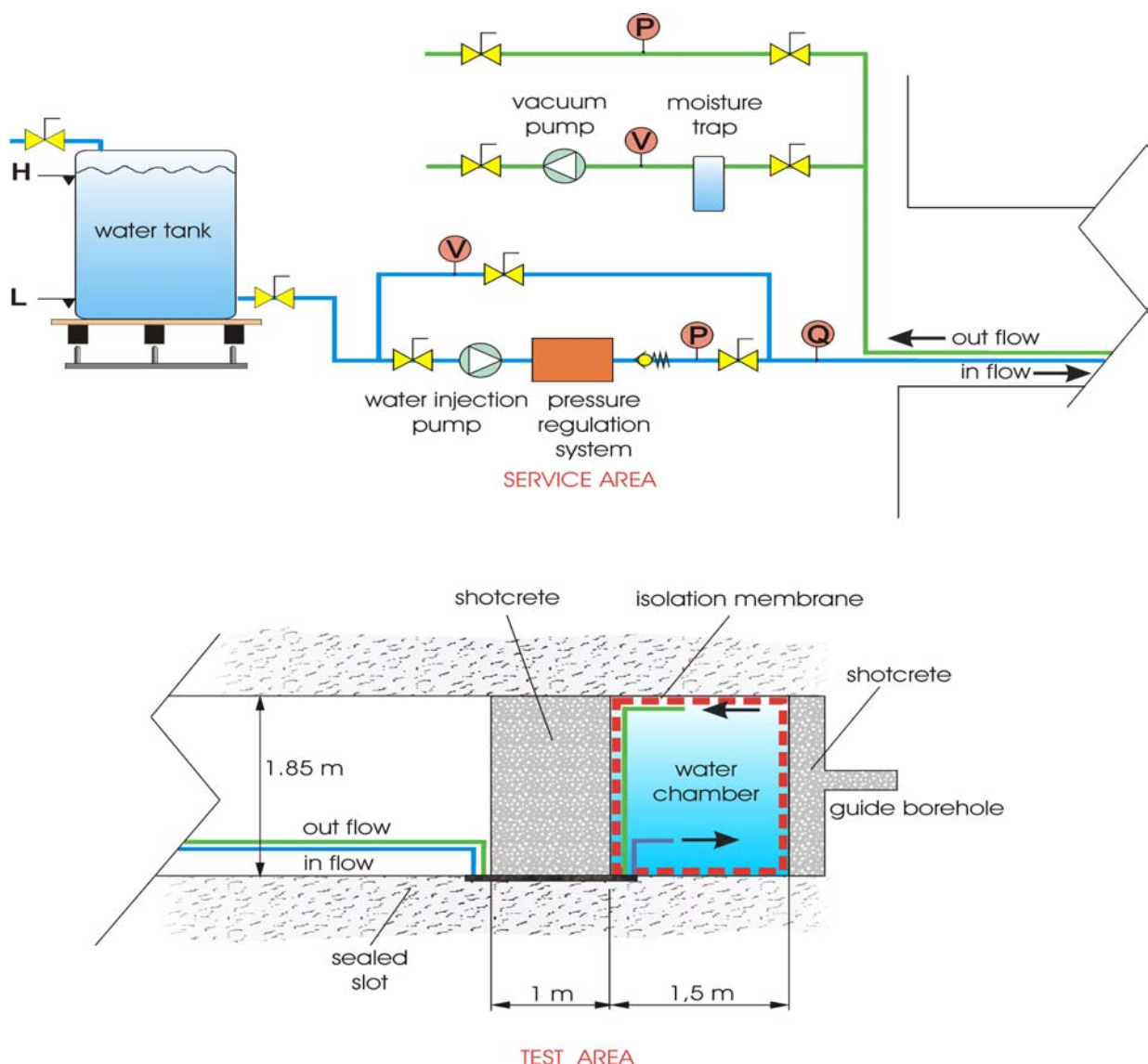
As a result of the preliminary design work performed during the initial stages of the project it was clearly established that the bearing capacity of this type of plug is a key issue that requires a specific load test. Some parts of the project approach were, therefore, modified in order to ensure that a complete understanding of the plug mechanical performance under realistic conditions was going to be achieved. The modifications introduced, with respect to the initial proposal, have been as follows:

- The shortening of the plug length, to facilitate failure under a reasonable loading pressure.
- The application of only mechanical (not hydraulic) loading inside the plug mass.

The document describes the construction of a 1 m long low-pH shotcrete plug, the loading tests performed to understand the mechanical performance of the plug under pressure, the dismantling and sampling operations performed on the plug after the loading test, the interpretation of the results obtained and the conclusions drawn. The work was carried out in the Äspö Hard Rock Laboratory (Äspö HRL) as part of Module 4 program of the Integrated Project ESDRED.

### 3 TEST LAYOUT

The layout of the short plug test, carried out in a gallery (15 m long by 1.85 m in diameter) excavated in granite, may be seen in Figure 1. There is a water chamber at the back end of the gallery, designed to be water tight by means of an isolation membrane covering the rock walls and the rear face of the plug. The water chamber is connected to a water injection system by inflow and outflow lines in order to provide the desired mechanical load. After filling the chamber with de-aired water, it was pressurised by means of a water pump via the inflow line and the injection pressure registered by a pressure transducer. Some sensors (extensometers, pressure cells and acoustic sensors) were installed in the plug and in the rock to measure different parameters related to the behaviour of the plug during loading.



*Figure 1: Test layout*

## 4 SHORT PLUG CONSTRUCTION

### 4.1 PRELIMINARY TESTS

The low-pH short shotcrete plug and ancillary structures were constructed in the -220 m level niche of the Äspö HRL during the months of September, October and November 2005.

BYGGS was the Swedish company selected to construct the plug under the supervision of AITEMIN and with the attendance of SKB. A preliminary test designed to check the shotcrete equipment with the formulation provided was successfully performed on September 14, 2005 at some surface facilities owned by BYGGS in Tumba (Sweden). The manual mixing of the formula in a mixer truck, as foreseen during the construction, was also checked. It was observed that the weighing of the aggregates directly into big bags using a dynamometer was time consuming, as a result of which it was decided that a pre-weighing of the aggregate needed for each batch would speed up the process.

A Meyco SUPREMA pump was used with the following configuration: straight reduction at the pump outlet followed by a flexible hose and nozzle (Figure 2).

The dry spraying of the isolation membrane Masterseal 345 from Degussa was also successfully tested using a Meyco PICCOLA pump.



*Figure 2: Shotcrete test in Tumba*

## 4.2 SITE PREPARATION

The preparation of the site for the construction of the short plug consisted of the following tasks:

- Site preparation

The work, carried out by SKB staff, consisted of the geological mapping of the gallery, grouting of the pre-existing boreholes at the end of the gallery, construction of the access to the gallery and the supply of electrical power, light and ventilation to the test gallery, as described in the Test Plan [ii]

- Total pressure cells installation

Three total pressure cells (TPC) were installed in the rock at the mid section of the designed plug to measure the pressure transmitted by the plug to the rock. The excavation of the slots in the rock needed to house the cells and that required for the water injection pipes was carried out by SKB and the TPC's were installed (Figure 3), as described in the Test Plan [ii].



*Figure 3: Installation of a total pressure cell*

## 4.3 CONDITIONING OF THE WATER CHAMBER

The water injection pipes were installed in the drilled slot and sealed with a resin based concrete [2]. The watertight membrane was then applied along the entire length of the water chamber rock wall (Figure 4). The construction of the water chamber was completed by installing the wooden panel needed to construct the plug and by spraying the watertight membrane over it (Figure 5).



*Figure 4: View of the surface of the rock corresponding to the water chamber*



*Figure 5: Water chamber completed*

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Mod4-WP3.2-D8.1– Report on Short Plug Test Results

Dissemination level: **PU**

Date of issue of this report: **Revision 1 on 29 May 2007**

10/47



## 4.4 CONSTRUCTION OF THE SHORT PLUG

### 4.4.1 Preliminary shotcrete test (above ground)

The construction of the plug was scheduled for October 12, 2005. A preliminary test at Äspö was scheduled for the day before (Figure 6). Due to a failure of the pump previously tested in Tumba, a Meyco SUPREMA concrete pump mounted on a truck was used, with a straight reduction at the pump outlet, followed by a flexible hose and the nozzle. The concrete was mixed as initially planned using a mixer truck. The shotcreting test above ground was carried out successfully, although it was necessary to set the pump at full pressure.



*Figure 6: Shotcrete surface test at Äspö*

An attempt to construct the short plug at the -220m niche was made the following day using the equipment described above but with a different configuration - steel pipe at the pump outlet with a 90° elbow, followed by a reduction, a flexible hose and the nozzle - due to the lack of sufficient space in the niche to properly house the mixer and pump trucks (Figure 7). Several attempts were unsuccessfully made to pump the concrete. Time after time the concrete became stuck at different points along the pump line, as a result of which it was not possible to construct the plug.

After analysing the problems encountered it was decided to perform a new attempt on November 2, 2005, but this time using the same stand-alone Meyco SUPREMA pump and arrangement used in the preliminary test at Tumba. The mixing of the concrete was improved by using a mixer truck with a lower capacity drum and a higher rotation speed.



*Figure 7: Arrangement during the failed spraying attempt*

#### **4.4.2 Short plug construction**

With the correct set-up the plug was successfully constructed in four steps. Three layers of 21, 22 and 35 cm were constructed on November 2, 2005. The last layer needed to construct the 1m long plug was sprayed the following day.

All the activities described were documented by photo and video recording [iii].



*Figure 8: Short plug construction*

#### **4.4.3 Short plug hardening**

After construction the plug was left to harden for more than 90 days, up to the end of January 2006.

#### **4.4.4 Auxiliary system installation**

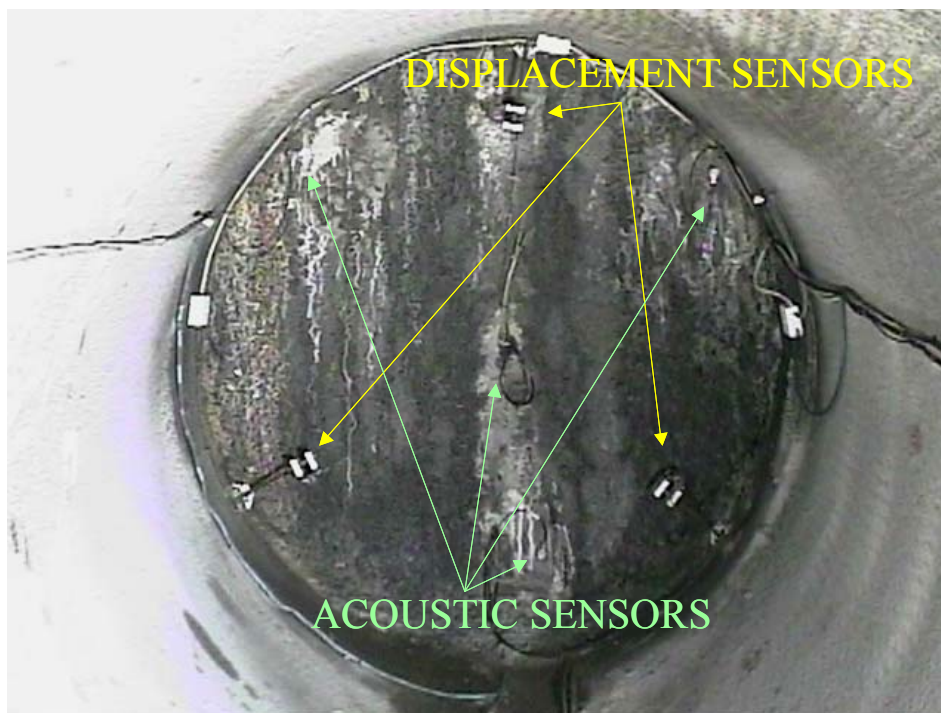
The auxiliary system required to perform the test was installed during the first week of February 2006, namely the water injection system, the remaining sensors (displacement, acoustic, and camera) and the data acquisition and control system.

The water injection system was installed at the drift entry. Three pumps have been used: the initially foreseen flow controllable piston pump (pump 1) with an inflow and pressure capacity of 155 ml/min and 140 bar; an HPLC type pump (pump 2) with electronic flow and pressure regulation, capable of providing a water flow of 1 l/min and up to 50 bar of pressure; and a bigger piston pump (pump 3) with manual pressure regulation, capable of providing a flow of 25 l/min and up to 220 bar of pressure (Figure 9).



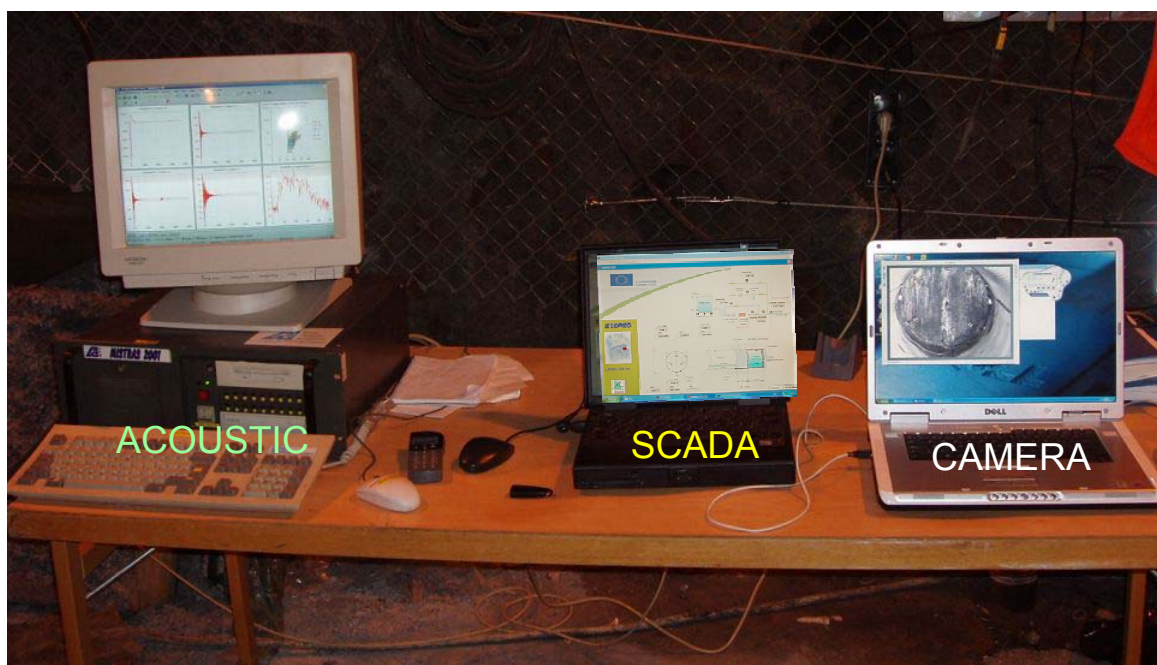
*Figure 9: Water injection system*

Three linear displacement sensors were placed on the plug outer face, in a 120° circular array, at a distance of 25 cm from the gallery wall. Four acoustic emission sensors were also installed; three of them on the plug face, in a 120° array, interpolated with the linear displacement sensors. The fourth was installed inside the plug at about 0.5 m in a horizontal borehole in the centre of the plug. In addition, a video recording camera was installed in the gallery, facing the plug (Figure 10).



*Figure 10: Instrumentation layout at the plug front*

Finally, the data acquisition system for control of the test and monitoring of sensor variation was installed close to the gallery entry (Figure 11).



*Figure 11: Layout of the Data Acquisition System*

## 5 SHORT PLUG TEST

### 5.1 INITIAL ASSUMPTIONS AND SCOPING CALCULATIONS

The testing by loading up to failure of the low-pH shotcrete short plug was based on a set of mechanical scoping calculations. In these calculations, performed using the FLAC code, reasonable values of the mechanical properties of the rock, the shotcrete mass and of the rock-shotcrete interface were assumed [ii]:

#### Granite parameters:

- Density ( $\gamma$ ) = 2 700 kg/m<sup>3</sup>
- Porosity (n) = 0.003
- Young's modulus (E) = 55 000 MPa
- Poisson's ratio ( $\nu$ ) = 0.25
- Friction angle ( $\phi'$ ) = 41°
- Cohesion ( $c'$ ) = 16 MPa
- Tensile strength (T) = 14 MPa

#### Shotcrete parameters:

- Density ( $\gamma$ ) = 2 250 kg/m<sup>3</sup>
- Porosity (n) = 0.15
- Young's modulus ( $E_s$ ) = 10 000 to 20 000 MPa
- Poisson's ratio ( $\nu$ ) = 0.25
- Friction angle ( $\phi'$ ) = 38°
- Cohesion ( $c'$ ) = 3 MPa
- Tensile strength (T) = 1.5 MPa

#### Ubiquitous joint parameters:

- Friction angle ( $\phi'_j$ ) = 38°
- Cohesion ( $c'_j$ ) = 0.7 to 1.0 MPa
- Tensile strength ( $T_j$ ) = 0.7 MPa
- Dilation angle ( $\psi_j$ ) = 5°

Also, a fundamental assumption taken was to set the dilation angle equal to zero for interface displacements greater than 0.5 mm.

There was confidence regarding the friction angle adopted ( $38^\circ$ ), in the knowledge that this was probably conservative. On the contrary, less experience exists regarding the appropriate values of the “true” cohesion (usually values ranging from 0.5 to 1.0 MPa are adopted for concrete interfaces with good quality rocks), tensile strength and, especially, dilation angle. The adoption of a value of  $5^\circ$  for this last parameter was felt to be clearly conservative (values of  $12^\circ$  for concrete and even  $15^\circ$  for very dense sands are reported in the literature) but adequate for a scoping calculation phase, due to the general uncertainty regarding the mechanical behaviour of a confined rock-shotcrete interface.

In fact, the main objective of the short-plug loading test (up to “failure”) was to reduce this uncertainty, gaining better insight into the mechanical behaviour of plugs, specifically at their interface with the rock. This enhanced knowledge might then be of real benefit in the final design of the repository plugs.

The results of the scoping calculations [ii] showed that the 1 m long plug might fail under applied pressures of 3 to 4 MPa, the calculated variation depending mainly on the assumed range of the cohesion (0.7 – 1.0 MPa). The Young’s modulus of the shotcrete also ranged between 10 000 and 20 000 MPa, but this parameter (if greater than 10 000 MPa in any case) has little relevance for plug failure calculation.

## **5.2 FIRST TEST SERIES (FEBRUARY 2006)**

On February 8, 2006, the water chamber was filled using both pump 1 and the vacuum pump to avoid air from becoming entrapped inside the water chamber. Subsequently, a first pressure test was carried out using pump 1 to check for water tightness and the functioning of the instrumentation and injection systems. A pressure of 6 bars was reached without problems. When the pump was shut off, the pressure dropped very rapidly, indicating that the chamber was not watertight.

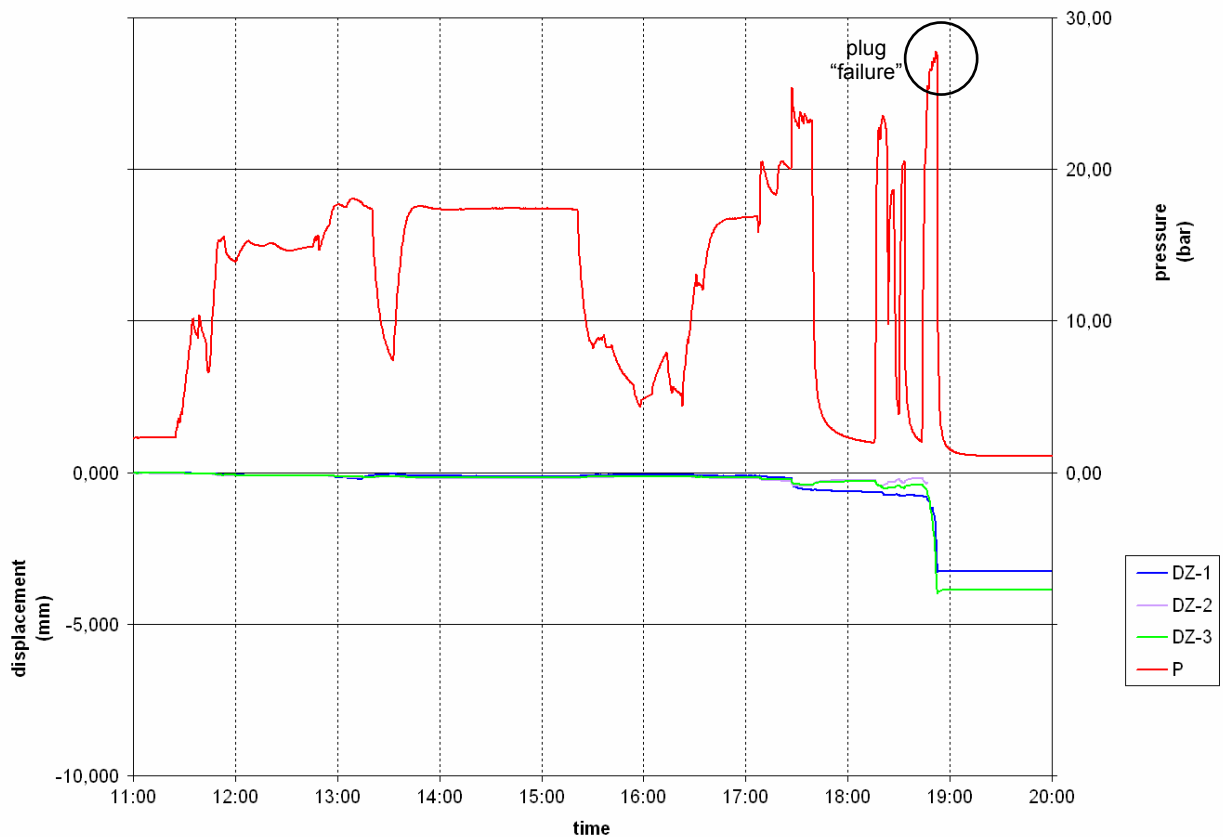
On February 9, another pressure pulse was carried out to check the capacity of pump 1 to increase the pressure in spite of the water leakage detected. At 9 bars the pressure increase rate slowed down so much that more than three hours were necessary to increase by one additional bar. This indicated that the pump did not have sufficient capacity to pressurise the system. At this point, it was decided to stop the test until a new pump system with a higher water flow capacity was available (pumps 2 and 3).

During the test the displacement sensors registered a deformation of the plug of only 0.080 mm at 10 bar. When the system was depressurised a remnant deformation of 0.024 mm was recorded. No relevant information was obtained at this time either from the total pressure cells or from the acoustic emission sensors.

### 5.3 SECOND TEST SERIES (MARCH 2006)

The two new pumps (2 and 3) were installed to continue with the plug testing. On March 2, the following test with the HPLC pump was carried out (Figure 12). The objective was to reach at least 20 bar, taking into account that the minimum calculated “failure” pressure (scoping calculations) was 30 bar:

- At 11:24 the pressure test was started. The pressure was increased in a few minutes up to 10 bar (value reached during the first test) and the plug deformation increased from the 0.024 mm remaining after the previous test to 0.046 mm.
- The pressure was increased again up to 15 bar, resulting in a plug deformation increase up to 0.100 mm. The pressure was maintained at that level for almost an hour with a water injection rate of 0.375 l/min (the water leakage at that injection pressure was about 37% of the full capacity of the pump). During this test, at constant pressure, no variations were registered in the displacement sensors, as would have been the case if there had been a plug “failure” instead of deformation (practically elastic).
- At 13:00 the pressure was increased again up to 17.7 bar, for which the output of the pump had to be increased (the water leakage increased). The deformation increased from 0.100 mm to 0.170 mm. The pressure was then maintained constant at that level for a further 20 minutes, again with no increase in deformation.



*Figure 12: Loading test, March 2, 2006*

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Mod4-WP3.2-D8.1– Report on Short Plug Test Results

Dissemination level: **PU**

Date of issue of this report: **Revision 1 on 29 May 2007**

17/47



At this point it was decided to stop the test in order to introduce a tracer in the water, to identify the water flow path (through the plug, through the rock or both?). A red colour tracer, namely Rodhamine WT, was used. The test then continued as follows:

- After a pressure release down to 7.3 bar, with a deformation recovery from 0.170 mm to 0.120 mm, the pressure was increased once more, now with the doped water being injected through the inflow pipe. With the HPLC pump at its full capacity of 1 l/min, only 17.5 bar was reached. The doped water was seen to be coming only from a narrow zone at the very bottom of the plug (Figure 13). The leakage was roughly measured as about 1 l/min.



*Figure 13: Red doped water flowing out from the plug base*

Since the HPLC pump was not capable of reaching the target minimum pressure, it was decided to continue the test by also installing the larger pump:

- At 16:22 a new pressure test was started, first with the HPLC pump, a pressure of only 16.8 bar being reached after approximately 45 minutes.
- At this point (17:06), with the system at a pressure of 16.8 bar, the injection was continued with the larger pump. The pressure increased to 20.5 bar in three minutes. At this point, the deformation was 0.232 mm. No signs of microcracking were recorded at this pressure.

During this episode with the larger pump the water leakage in the bottom part of the plug increased significantly, which led to reasonable doubts regarding the capability of the inflow system to reach the expected “failure” pressures (at least 30-40 bar). Since this was critical for the test performance, it was decided to continue checking the system pumping capacity. It is also worth mentioning that all the testing performed up to this point seemed to indicate that the system was far from the “failure” point. The test continued as follows:

- The pressure was then (17:28) increased to 24 bars and maintained at that level for ten minutes. The deformation increased to 0.420 mm during the pressure increase and remained stable at that pressure (Figure 12). Again, no microcracking was recorded. At this point, the pump was set at full capacity and the leakage area in the bottom part of the plug widened to about 60 cm (Figure 14).
- Several short loading cycles were carried out to confirm that it was not possible to increase the water pressure above 24 bar.



*Figure 14: Maximum extent of doped water flowing out from the plug base*

Before cancelling the test, it was decided to check that the flow provided by the pump was correct. Only 16 l/min were being measured while the nominal value was 25 l/min. The pump was revised and it was found that the water filter was partially clogged, this being the cause of the observed flow reduction. Once the problem was solved it was decided to resume the test:

- At 18:43 the pressurisation started again, a pressure of 23.85 bar being reached in three minutes, with plug deformation reaching 0.554 mm.
- At 18:47 a pressure of 25.5 bars was reached and the deformation was 0.723 mm. The water leakage continued to increase and there was no indication of plug “failure” from the acoustic emission sensors.

- At 18:48 the pressure was at 26.6 and the deformation reached 1.077 mm. At this point, a significant amount of acoustic “noise” was suddenly registered.
- At 18:49, the rate of displacement increased suddenly, along with the number of acoustic hits per second. At that moment the pressure was 27.09 bar and it was considered that the plug had “failed”. Over the next three minutes the pressure increased very little, up to 27.6 bar, and the displacement reached 3.839 mm.
- The pump was then shut off and the pressure dropped very rapidly, to 1.2 bar at 19:05 without displacement recovery.

A visual inspection of the plug face revealed no cracks in the plug mass. The only cracks observed were located (outside the plug itself) in a thin film of shotcrete that was adhered to the rock walls in the 15 cm to 25 cm closest to the plug face. The movement of the plug pushed this thin film, which bent the support of one of the displacement sensors (Figure 15).



*Figure 15: View of the displacement sensor bent by the plug movement*

The information provided by the acoustic emission sensors correlated well with the plug movement but did not clearly anticipate the “failure” episode.

#### **5.4 THIRD TEST SERIES (MARCH 2006)**

On March 3, a new test series was planned to gain further insight into the plug “failure” event. More tracers were added to the water chamber to check the water leakages. The tracer was introduced this time both through the outflow and inflow pipes, to guarantee that all the water in the chamber was doped.

The larger pump was then disconnected from the inflow pipe and connected to the outflow pipe to introduce the tracer. The operation was carried out as follows (Figure 16):

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**Mod4-WP3.2-D8.1**– Report on Short Plug Test Results

Dissemination level: **PU**

Date of issue of this report: **Revision 1 on 29 May 2007**

20/47



- At 10:34 the pump was started and water with tracer was introduced into the water chamber.
- Three minutes later the pump was shut off to allow the tracer to spread throughout the chamber; only 0.1 mm of deformation was recorded.
- At 10:41 the pump was restarted to introduce the remaining tracer and at 10:43 an unexpected plug displacement was registered. At the same time many hits per second were registered by the acoustic emission sensors.
- At 10:44 the pump was shut off. The total displacement had increased from 3.856 mm to 8.419 mm.

At this point the system was revised. It was found that, by accident, a valve in the outflow line had remained closed. Therefore, no pressure could be observed on the monitoring screen or via the manometer on the outflow pipe during this episode. However, the evolution of the pressure in the water chamber had been registered throughout the entire episode by the sensor on the inflow line (Figure 16). According to this data, this second out-of-equilibrium movement episode started at 25 bar of applied pressure. It should also be emphasised that the leakage of doped water was detected only at the bottom part of the plug.

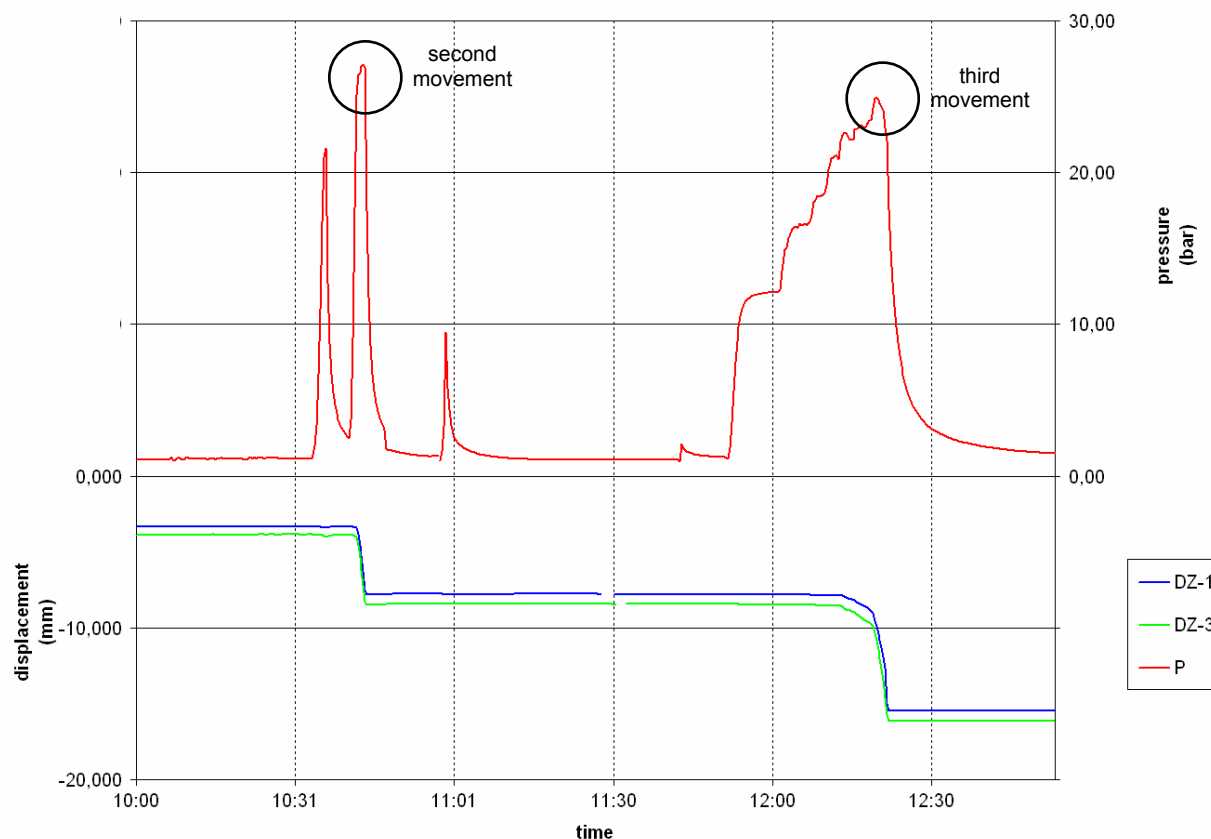
The results were considered sufficiently relevant to warrant the performance of another test. The pump was connected again to the inflow line and the two displacement sensors that remained operative re-positioned. Also, for visual checking of future displacements of the plug, several reference points were marked by hand along the gallery wall.

The test was performed as follows:

- At 11:52 the pump was connected, using doped water, and the pressure increased in a step-wise controlled manner (Figure 16).
- The third out-of-equilibrium movement of the plug took place at an applied pressure of 24.9 bar (at 12:19). Many hits per second were again registered by the acoustic emission sensors during this displacement.
- The pump was shut off at 12:22 (three minutes later) when the plug movement had increased from 8.419 mm to 15.718 mm (7.3 mm).
- At 12:55 the pressure had decreased to 1.48 bar.

The manual measuring along the perimeter of the plug confirmed that the movement of the plug was uniform. The accumulated movement of the plug measured at this point was 16.117 mm.

Despite the plug movements recorded during the loading tests described above, two additional hydraulic pulses were performed coinciding with a technical meeting held at Äspö during 28-29 March 2006. In each one of the pulses the plug moved again, when the hydraulic pressure reached about 20 bars, with a total displacement of 19 mm, which added to the 16 mm recorded in the previous movements gives a total uniform displacement of the plug of more than 3.5 cm.



*Figure 16: Loading and displacements during March 3, 2006*

## 5.5 TEST RESULTS

The test results indicate that after breaking of the “true” cohesion between the concrete and the rock at an applied pressure of 27.09 bars, the plug remains intact, functioning like a rigid body, at least in most of its mass, and still withstanding pressures of about 25 bar.

With the exception of the bottom part of the plug, no water leakage was detected at the rock-plug interface even after plug “failure”, including a significant movement of about 1.5 cm. It may be stated that no significant retraction of the concrete has taken place. The water leakage observed at the bottom part of the plug was most likely channelled through heterogeneities in the plug due to construction rebound.

The information provided by the acoustic emission sensors correlated well with the plug movement but did not anticipate the “failure” episode.

In addition to the malfunctioning from the very start of one of the three cells, the analysis of the data recorded from the total pressure cells points to the possibility of installation problems having caused the noisy signals and unexpected overpressures registered. The data registered might be interpreted as the result of cells working under anomalously high shear stresses in relation to the normal stresses applied (which is not the case under the standard conditions for

which these types of cells have been devised). These abnormal working conditions might have significantly affected both the contact between the cell and the filling mortar and the contact between this mortar and the plug (surely this was quite different from the rock-plug contact in shape, roughness and resistance), creating punctual stress concentrations not representative of the overall behaviour of the perimeter.

However, the pressure increase applied to the rock registered by the cells correlates fairly well with the pressure applied in the water chamber, even after the plug displacement events. Furthermore, the cells registered an increase in the rock tension after each displacement, which might be explained as lateral dilatation-confinement in the plug (concrete mass jammed in the gallery and especially in the location of the lower resistance cells).

The database of the tests may be found in [iv].

## 6 SHORT PLUG DISMANTLING

### 6.1 PURPOSE AND APPROACH

The detailed dismantling of the plug sought to gain information on the properties of the plug (fabric, bonding between concrete batches and with the rock, mechanical and hydraulic properties, etc) and look for any signatures left by the induced failure that might help in test interpretation and in future design and construction processes. To this effect the two core sampling campaigns and a careful “manual demolition and visual inspection” were planned. The dismantling was planned in the following steps:

- Preparatory works
- Plug front inspection and sampling
- Excavation of plug penetration
- Water chamber inspection and sampling
- Plug demolition and sampling
- Final works

### 6.2 DESCRIPTION OF DISMANTLING OPERATION

#### 6.2.1 Preliminary works

The dismantling started with the removal of the water injection system and of the three displacement and three acoustic emission sensors installed at the plug front. All the components of the Data Acquisition and Display System were also removed.

#### 6.2.2 Plug front inspection and core sampling

The plug sampling and dismantling process started with a detailed inspection of the plug face (Figure 17). The thin layer of shotcrete adhered to the rock at the border of the plug was removed in order to better observe the shotcrete-rock contact along the plug perimeter. No evidence or sign of any gap between the shotcrete and the rock was observed.

To further investigate the concrete/rock interface, four boreholes were drilled from the plug surface at an angle to intercept the rock at different points. Besides the logging of the cores, these boreholes were used to inject resin at the concrete/rock interface to analyse the bonding and/or potential detachments between the two surfaces. Also, nine horizontal boreholes were drilled to study the concrete fabric and properties. The location of the boreholes, description of the sampling and results are described in **6.3 PLUG SAMPLING AND ANALYSIS**.



*Figure 17: Initial status of plug after failure test*

### **6.2.3 Excavation of an access to the water chamber**

A penetration passing through the plug to the wooden support of the water chamber was excavated by drilling two horizontal channels measuring 500 mm in diameter. The first was drilled in the upper part of the plug at 4 cm from the rock, and the second immediately below (Figure 18). The connection between them was hammer drilled. To access the water chamber the wooden support was cut with a jig saw. This opening allows for both personnel access to the water chamber and the subsequent inspection of the behaviour of the system during the test and a detailed mapping of the inner part of the plug along its entire length.



*Figure 18: Drilling of penetration (left) and open access (right)*

The first observation made was that the contact between the four layers of concrete used for the construction of the plug could be clearly identified, mainly those between layers 1 and 2 and between 3 and 4 (Figure 19). The contact between layers 2 and 3 is not so obvious. This clearly reflects the fact that layers 2 and 3 were constructed with the same batch of concrete

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and very little time lag between them (only 35 minutes). The contact between layers 3 and 4 could be clearly observed but with no detachments along the entire surface.

A gap measuring some 20 mm in thickness was found between the wooden support and the concrete (gap 1 in Figure 19). It is not possible to state when this gap might have occurred, either during the loading of the plug or the drilling of the horizontal boreholes, since (for reasons of design) the wooden frame had the potential to move forward during pressurisation of the water chamber and, therefore, might in this case have also moved backwards during the drilling of the entry to the water chamber. Another gap measuring between 20 mm and 40 mm in width was found along the surface of the penetration at a distance of about 200 mm from the rear end of the plug, approximately at the contact between concrete layers 1 and 2 (gap 2 in Figure 19). From visual inspection this gap appears to extend across the entire surface of the plug. The concrete in the layers 2 and 3 is very homogeneous and the contact between them difficult to observe.



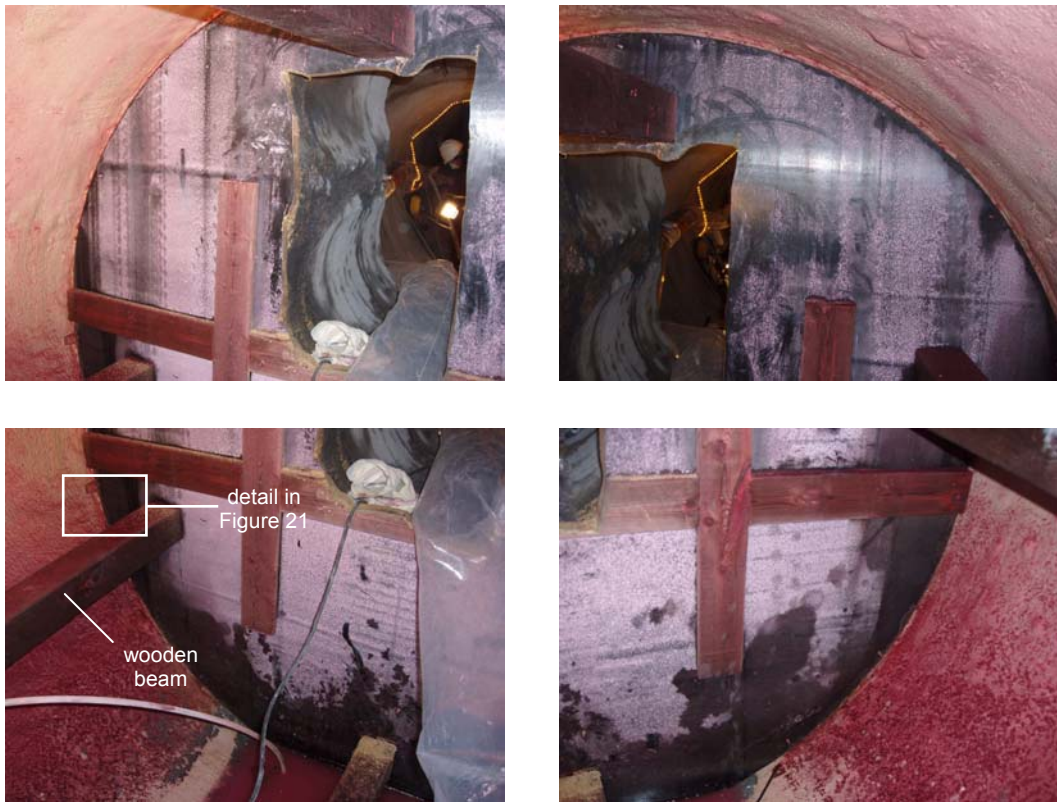
*Figure 19: Appearance of shotcrete layers (left) and detail of gap between layers 1 and 2 (right)*

## 6.2.4 Water chamber inspection

### 6.2.4.1 Initial inspection

The following observations were made during the inspection of the water chamber:

- The contact between the wooden support and the watertight membrane seemed to be intact, except for the lower left-hand part facing towards the entrance. In this zone the membrane presented a fissure (Figure 21).
- No signs of displacement could be observed at the anchoring points of the four beams that supported the wooden frame when these were removed.
- The membrane seemed to be intact and well bound along the entire rock surface of the chamber.



*Figure 20: Water chamber as found (view from inside)*



*Figure 21: Detail of fissure in the water membrane (lower left section facing towards the entrance)*

#### 6.2.4.2 Membrane and concrete inspection.

After removal of the wooden support, the membrane at the rear end of the plug was left exposed (Figure 22). In general, it presented a homogeneous appearance across the entire concrete surface, with a thickness of around 5 mm. A crack in the membrane that extends into the concrete (Figure 21) was observed almost along the whole surface (Figure 23).

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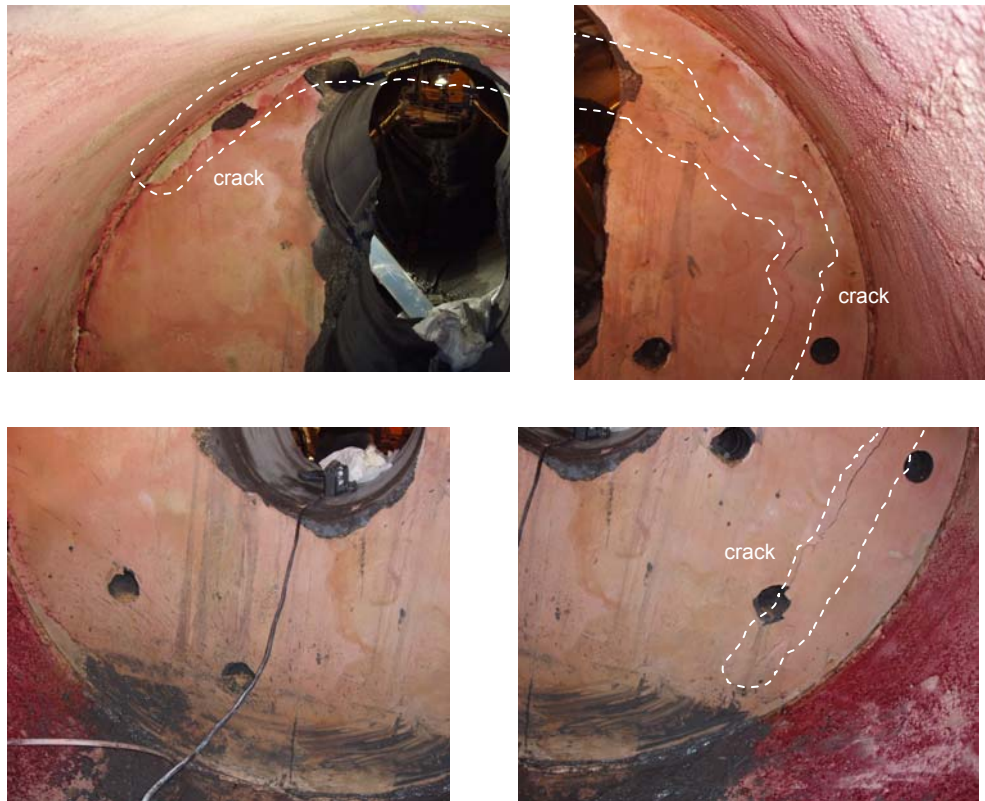
Mod4-WP3.2-D8.1– Report on Short Plug Test Results

Dissemination level: **PU**

Date of issue of this report: **Revision 1 on 29 May 2007**

27/47





*Figure 22: Appearance of membrane after removal of the wooden support*

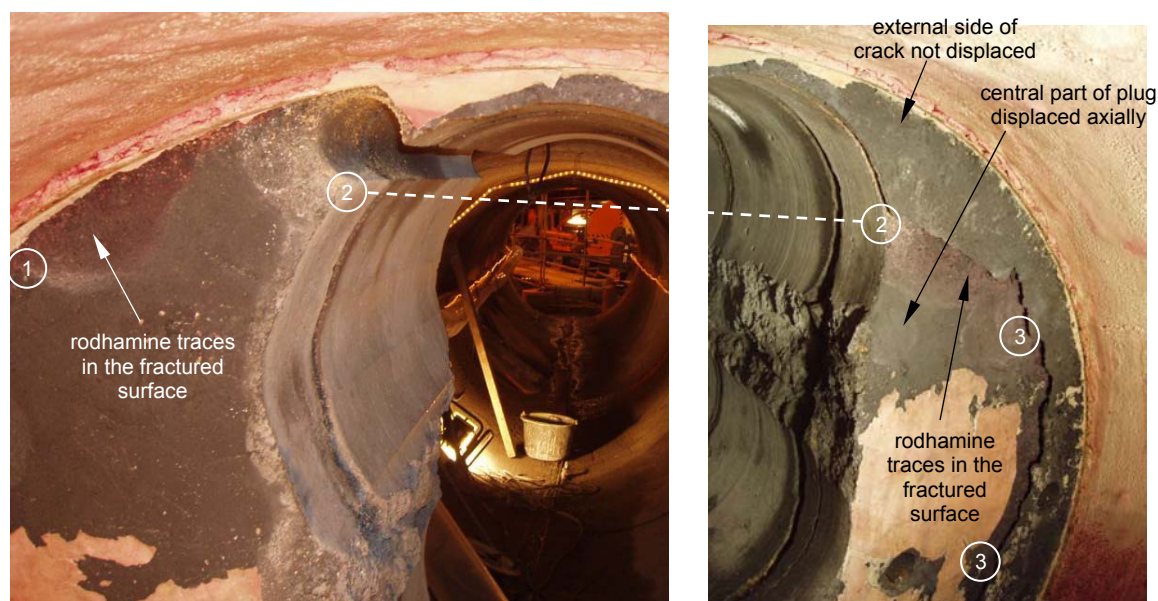


*Figure 23: Detail of the crack (side view from penetration)*

The inspection continued with the removal of the membrane. The concrete at the rear end showed a solid appearance, without pores. The crack could now be clearly seen, starting close to the rock, at 45° upper-left side (1 in Figure 24) facing the entrance, running through the pass-through (2 in Figure 24) and down to the lower right-hand side along the perimeter of the section (3 in Figure 24). A gap of 15 mm to 20 mm was measured in the axial direction of the gallery between the two sides of the crack. It was noticed that the side of the concrete on the crack in contact with the rock had not undergone axial displacement, so this gap was due to a displacement of the central part towards the entrance. The fractured surface of the concrete of

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the crack was coloured with Rodhamine, so it is clear that the water penetrated through this crack into the gap between layers 1 and 2.



**Figure 24: Concrete appearance, crack and traces of Rodhamine on fractured surface**

The concrete in contact with the rock above the penetration opened in the plug was carefully dismantled to study the interface and it was found that slickensides had developed along the entire surface as a consequence of the movements of the plug (see 6.3.3).

The inspection finished with the removal of a concrete piece from layer 1 (Figure 25). The penetration of the Rodhamine doped water may be clearly observed in the rock (Figure 26), as well as on the exposed surface of the concrete.



**Figure 25: Concrete piece removed from layer 1**



*Figure 26: Detail of Figure 25; side view of the rock surface following removal of the rock piece showing the penetration of Rodhamine*

## **6.2.5 Plug demolition**

### **6.2.5.1 First part of the demolition**

The gap found between layers 1 and 2 of the plug gave rise to a change in the demolition plan. The initial idea was just to leave the bottom part of the plug, up to 0.5 m in height, then to extract 9 vertical cores from it and to finish the demolition. The plan was modified to include the careful demolition of the upper part of the plug, using a hydraulic splitter, if possible leaving layers 1 and 2 intact with the gap in between, followed by the extraction of only six vertical cores and the careful dismantling of layer 2, mapping the gap during the operation, and finally completion of the demolition.

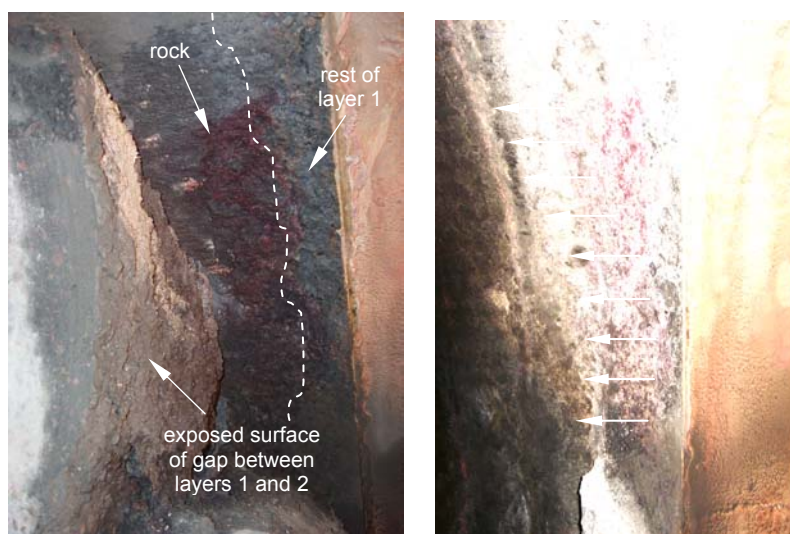
Even with this controlled demolition method it was not possible to leave layer 2 intact (Figure 27). The demolition continued until only layer 1 and the bottom part of the plug were left in place. Then the vertical drilling was carried out as described in paragraph 6.3.4.

After removal of the concrete, numerous slickenside surfaces were observed in the granite in contact with the concrete. The slickensides were produced by the displacement of the plug, which indicates a tight concrete-rock contact, even after the failure episode during the successive horizontal movement of the plug.

After dismantling concrete layer 2, it was observed that the penetration of rodhamine affected the fractured zone in layer 1, but it did not progress further along the concrete-rock contact, indicating that there was no gap between concrete and rock beyond layer 1 (Figure 28). This was observed more clearly along the entire periphery after the completion of the demolition (see 6.2.5.2).



*Figure 27: Demolition of upper part of the plug*



*Figure 28: View of penetration front before (left) and after (right) demolition of layer 2*

#### 6.2.5.2 Second part of the demolition

The plug demolition was completed by hydraulic splitting of the bottom part (Figure 29).



*Figure 29: Demolition of the bottom part of the plug with hydraulic splitter*

No noticeable variations in the porosity or noticeable heterogeneities in the bottom part of the concrete were observed either during breaking with the splitter or upon close inspection of the pieces of concrete removed. No clear signs of flow paths with Rodhamine were observed in the bottom part of the plug. Figure 30 shows the marks of Rodhamine along the entire contact between layer 1 and the rock, which did not progress through layer 2.



*Figure 30: View of rodhamine marks, along only the layer 1-rock contact*

### 6.2.6 Final works

The total pressure cells were removed from the slots where they were embedded in mortar (Figure 31). It was observed that the slots had been well filled with mortar, with no air bubbles.

Some damage was found in TPC-2 (right-hand side of the gallery) - the cable was separated from the electronics.



*Figure 31: Removal of TPC-3 at left-hand side*

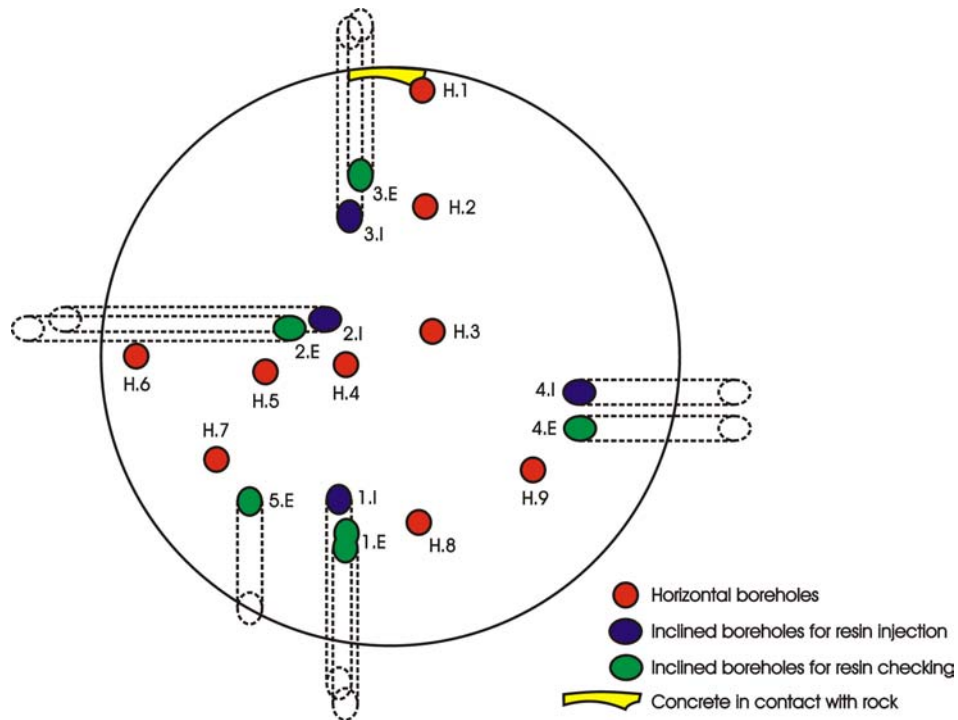
The dismantling finished with the cleaning and removal of waste from the gallery (Figure 32)



*Figure 32: Final status after dismantling*

### 6.3 PLUG SAMPLING AND ANALYSIS

This section describes the sampling carried out during plug dismantling. Figure 33 shows a layout of the samples taken. More information on the sampling may be found in [v].



*Figure 33: Layout of low-pH short plug sampling*

#### 6.3.1 Horizontal cores

Nine horizontal boreholes measuring 75 mm in diameter (H.1 to H.9) were drilled (Figure 34).



*Figure 34: Horizontal sampling*

Core H.3 was analysed to determine density, compressive strength and elastic modulus (paragraph 6.3.5).

[ESDRED]

Mod4-WP3.2-D8.1– Report on Short Plug Test Results

Dissemination level: **PU**

Date of issue of this report: **Revision 1 on 29 May 2007**

34/47



### 6.3.2 Inclined boreholes and resin injection

Four inclined boreholes measuring 75 mm in diameter were drilled from the plug surface to intersect the rock at different points. Mechanical packers were installed at the bottom of the boreholes (Figure 37) to isolate the concrete/rock interface and allow for the injection, at low pressure, of Sikadur 53 resin. The goal was to identify potential gaps between the concrete and the rock by drilling nearby boreholes parallel to the injected resin sections, which might have been intruded by the resin. The location of the boreholes is shown in Figure 32.

Boreholes 1.I and 3.I (bottom and top of the plug, respectively) were drilled to intersect the rock in the middle of the plug, whereas boreholes 2.I and 4.I (horizontal on left and right-hand sides) intersect the rock at some inner and outer points, respectively.



*Figure 35: Inclined sampling*

The core extracted from the top of the plug showed slickensides both in the concrete and in the rock (Figure 36). This is a clear indication of the tight contact existing between the concrete and rock, at least in the central part of the plug.



*Figure 36: Detail of core 3.I from the top of the plug; rock core (left) and concrete core (right) both with slickensides*

[ESDRED]

Mod4-WP3.2-D8.1– Report on Short Plug Test Results

Dissemination level: **PU**

Date of issue of this report: **Revision 1 on 29 May 2007**

35/47



It was not possible to check the characteristics of the interface cut by the other three boreholes due to the poor quality of the cores extracted. The core of borehole H.3 was used to analyse density, compressive strength and elastic modulus (paragraph 6.3.5).



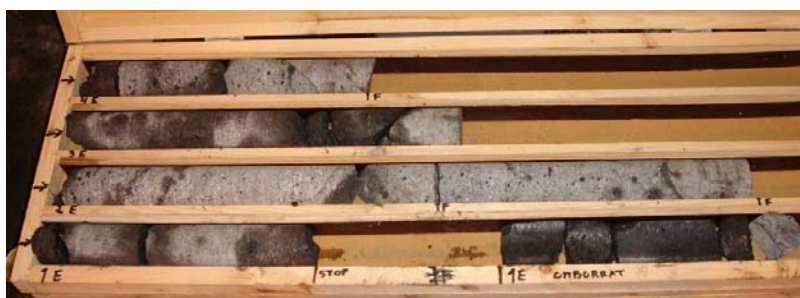
*Figure 37: Introduction of packers into the inclined boreholes*

Resin was injected in the inclined boreholes 1.I to 4.I. After three days of resin hardening, boreholes 1.E to 4.E (parallel and very close to I holes) were drilled and cored in search for traces of resin intruded in potential gaps at the rock/plug interface. No resin was found in any of the cores extracted.

Four additional small boreholes (15 mm in diameter) were drilled at a different position to check the validity of the resin injection procedure. The sections where the holes intersected the rock were isolated with minipackers and again resin was injected at low pressure in one of them. After a hardening period this borehole was overcored and, as previously, no resin could be seen at the interface.

The absence of resin in any of the cores extracted cannot be interpreted as an indication of the absence of gaps, since it was observed that the space sealed with the packers was not completely filled by resin, probably due to the high density of the resin used, thus hindering the injection under pressure of resin in any possible existing gap.

Although no conclusions could be drawn from the resin testing, the slicked surfaces found in some of the cores extracted, as well as in pieces of concrete in contact with the rock, are clear evidence of the tight contact existing between the plug and the surrounding rock.



*Figure 38: Inclined cores for resin checking*

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Mod4-WP3.2-D8.1– Report on Short Plug Test Results

Dissemination level: **PU**

Date of issue of this report: **Revision 1 on 29 May 2007**

36/47



### 6.3.3 Sampling of concrete in contact with rock

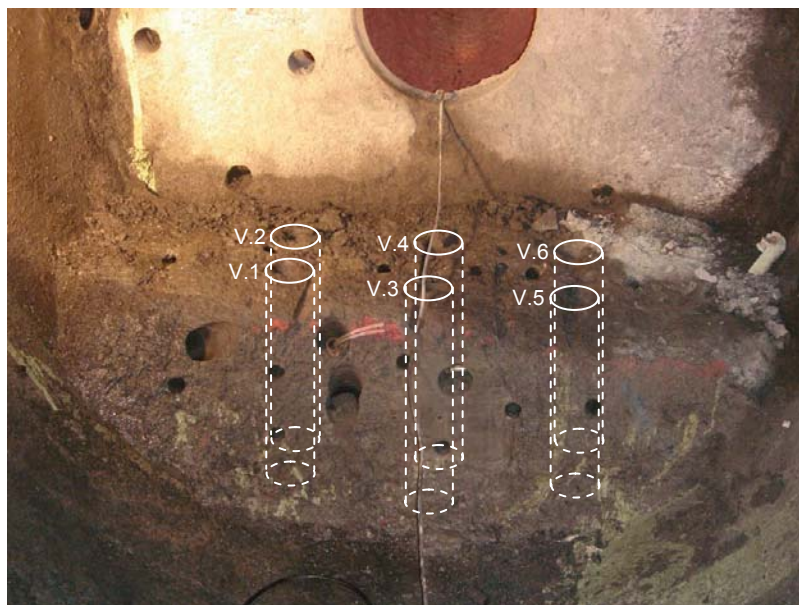
The contact between the rock and the concrete above the penetration was studied by cutting a rectangular piece of the concrete (600 mm long by 400 mm wide) using a saw. Again, the entire surface of the concrete in contact with the rock showed slickensides (Figure 39).



*Figure 39: Concrete in contact with rock at top of the gallery*

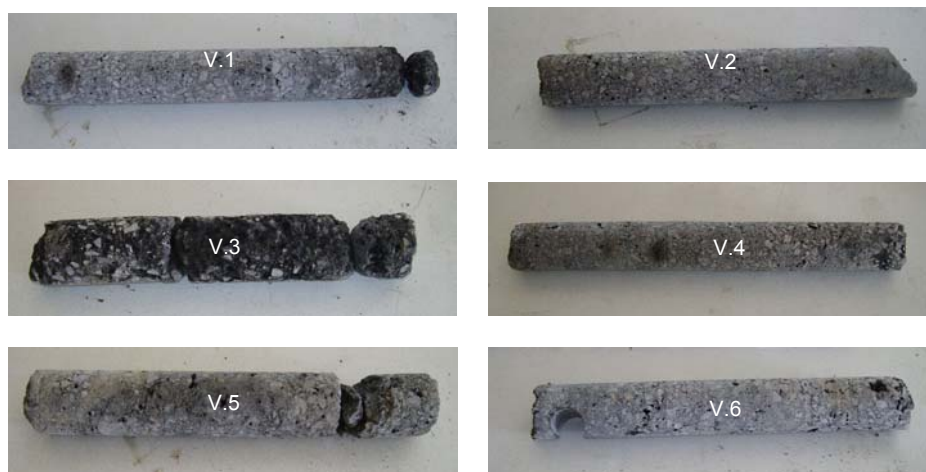
### 6.3.4 Vertical sampling

The vertical sampling was carried out by drilling six boreholes measuring 75 mm in diameter in order to check the fabric and properties of the concrete at the bottom part of the plug (Figure 40). Six cores (V.1 to V.6) were extracted.



*Figure 40: Vertical sampling*

Boreholes V.1, 3 and 5 were drilled in concrete layer 3, while V.2, 4 and 6 were drilled in concrete layer 2. In the cores from layer 2 a more heterogeneous and porous zone was observed at a few centimetres from the rock. Apart from this, no significant variation may be observed between the bottom and top parts of the cores.

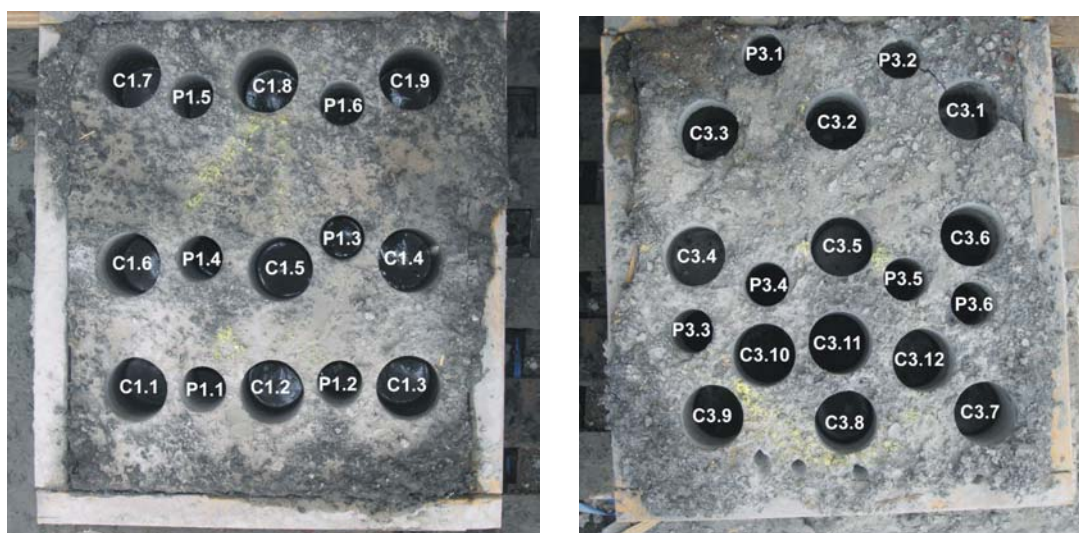


*Figure 41: Vertical cores*

### 6.3.5 Analysis of samples

In addition to the samples described in the previous sections, cores were extracted from the sample panels shotcreted during the construction of layers 1 and 3 of the plug (Figure 42). Cores coded Cxy and Pxy (where x stands for the layer number and y is a correlative index) were used to study the mechanical properties, and for pH and permeability determinations. Two cores extracted from the plug were also analysed. The horizontal core H3 and inclined core 2I were also analysed.

All the analyses were carried out by the Torroja Institute, unless otherwise stated.



*Figure 42: Cores from sample panels of layer 1 (left) and layer 3 (right)*

#### 6.3.5.1 pH determination

The pH values measured in the cores from both panels range (Table 1) from 10.4 to 10.7, which complies with the requirement of the pH being below 11

*Table 1: Results on pH measurements of pore fluid*

Core	PH	Mean value
P1.1	10.40	10.61 ± 0.12
P1.3	10.56	
P1.5	10.59	
P3.3	10.76	
P3.4	10.72	
P3.5	10.63	

#### 6.3.5.2 Hydraulic conductivity determination

All the cores analysed, except for one, show a hydraulic conductivity of the order of 1 E-10 or lower (Table 2).

*Table 2: Results for hydraulic conductivity*

Core	Hydraulic conductivity (m/s)	Mean value (m/s)
P1.1	2.97 E-10	7.8 E-10
P1.3	3.61 E-9	
P1.5	4.16 E-10	
P3.3	1.80 E-10	
P3.4	5.23 E-10	
P3.5	1.41 E-10	

#### 6.3.5.3 Porosity determination

Total porosity measurements may be found in Table 3.

*Table 3: Results for total porosity*

Core	Total porosity (%)	Mean value (%)
P1.1	24.4	17.1 ± 3.9
P1.3	15.9	
P1.5	16.4	
P3.3	17.6	
P3.4	12.5	
P3.5	15.6	

#### 6.3.5.4 Determination of mechanical properties

The compressive strength and elastic modulus of the cores from the layer 1 panel are within the required limits (over 10 MPa and below 20 GPa, respectively). Three cores from this panel were analysed also by the Madrid School of Mines, for cross-checking. The values obtained are of the same order as those provided by the Torroja Institute, although one of the values for elastic modulus is over 20 GPa. The results obtained may be found in Table 4.

**Table 4: Compressive strength and elastic modulus determinations from panel of layer 1**

Core	Compressive strength (MPa)	Mean value (MPa)	Elastic modulus (GPa)	Mean value (GPa)
C.1.1	13.8	18.6 ± 3.8	-	12.3 ± 1.3
C.1.2	16.5		11.4	
C.1.4	20.1		13.3	
C.1.5	24.9		-	
C.1.6	16.6		-	
C.1.8	19.9		-	
C.1.3*	12.0	17.1 ± 5.0	4.2	12.5 ± 8.5
C.1.7*	22.1		21.3	
C.1.9*	17.1		12.1	

\*Analyses carried out by Madrid School of Mines

The compressive strength of the cores from the panel of layer 3 showed higher values, over 20 MPa. Given the size of the cores, only one measurement of elastic modulus could be performed. This value was over 20 GPa (Table 5).

**Table 5: Compressive strength and elastic modulus determinations from panel of layer 3**

Core	Compressive strength (MPa)	Mean value (MPa)	Elastic modulus (GPa)
C.3.7	24.9	26.3 ± 1.4	-
C.3.8	27.9		-
C.3.9	28.2		-
C.3.10	25.5		-
C.3.11	26.5		22.7
C.3.12	25.1		-

Compressive strength and elastic modulus determinations were performed with cores H.3 and 2.I (Table 6). The values are of the same order as those from the panels.

**Table 6: Compressive strength and elastic modulus determinations in cores from the plug**

Core	Layer	Compressive strength (MPa)	Mean value (MPa)	Elastic modulus (GPa)	Mean value (GPa)
H.3*	2	29.9	25.6 ± 4.9	14.9	16.4 ± 2.2
	3	20.3		19.0	
	4	26.6		15.3	
2.I*	2	28.2	28.85 ± 0.9	17.2	17.9 ± 1.0
	4	29.5		18.6	

\*Analyses carried out by Madrid School of Mines

Indirect tensile tests were carried out on some of the cores from the panel of layer 3, and in the part of cores H.3 and 2.I corresponding to that layer. The results may be found in Table 7.

**Table 7: Determinations of tensile strength from panel of layer 3 and from cores from the plug**

Core	Tensile strength (MPa)	Mean value (%)
C.3.1*	1.58	1.89 ± 0.39
C.3.3*	1.96	
	1.61	
C.3.5*	2.43	2.52 ± 1.03
H.3 (layer 3)*	1.79	
2.I (layer 3)*	3.25	

\*Analyses carried out by Madrid School of Mines

#### 6.3.5.5 Density determination

The results of the density measurements from the shotcreted panels may be found in Table 8 and Table 9. The cores from both panels were analysed also by the Madrid School of Mines for cross-checking. In both panels, the values obtained were of the same order as those provided by the Torroja Institute, but slightly lower.

**Table 8: Density values measured in a panel from layer 1**

Core	Density (kg/dm <sup>3</sup> )	Mean value (kg/dm <sup>3</sup> )
C1.4	2.22	2.19 ± 0.05
C1.5	2.23	
C1.8	2.13	
C1.3*	2.09	2.12 ± 0.03
C1.7*	2.16	
C1.9*	2.13	

\*Analyses carried out by Madrid School of Mines

**Table 9: Density values measured in panel 3**

Core	Density (kg/dm <sup>3</sup> )	Mean value (kg/dm <sup>3</sup> )
C3.9	2.27	2.21 ± 0.05
C3.11	2.20	
C3.12	2.17	
C3.1*	2.15	2.16 ± 0.04
	2.11	
C3.3*	2.19	
	2.20	
C3.5*	2.22	
	2.13	

\*Analyses carried out by Madrid School of Mines

Density measurements were performed on parts of cores H.3 and 2.I corresponding to all four layers of the plug (Table 10). The results are similar to those from the panels.

**Table 10: Density measurements from cores from the plug**

Core	Layer	Density (kg/dm <sup>3</sup> )	Mean value (kg/dm <sup>3</sup> )
H.3*	1	2.06	2.14 ± 0.06
	2	2.16	
	3	2.20	
	4	2.17	
2.I*	2	2.18	2.19 ± 0.02
	3	2.22	
	4	2.19	

\*Analyses carried out by Madrid School of Mines

## 6.4 SUMMARY OF THE DISMANTLING OBSERVATIONS

The interpretation of the observations made during the dismantling operation provides an explanation of the evolution of the plug during the load test performed as follows:

1. The fissure observed in the membrane at the contact with the plug in its lower left-hand part facing towards the entrance (Figure 20 and 21) most likely allowed water to enter into contact with the concrete at the start of the water chamber pressurisation process, producing the water leakage at the lower part of the plug (probably favoured by the existence of more permeable zones caused by shotcrete rebounds) (Figure 41). This leakage grew with increasing pressure, and perhaps with the wash out of the concrete in that part of the plug,

2. The plug maintained its integrity until the first displacement. This first movement probably caused the long fracture found in the membrane and in the first shotcrete layer along the perimeter (Figure 22 to 24). It most likely also caused a detachment of the wooden support from the plug, because of the water pressure and the lower adherence of the membrane with the wood than with the concrete.
3. The long fracture along the perimeter, probably incipient, widened when performing a new pressurisation of the chamber, until the water entered into contact with the rock, from where it progressed until it reached the interface between layers 1 and 2. This interface was progressively widened with subsequent movements of the plug.
4. It is concluded that, with the exception of the first layer, the plug maintained its integrity throughout the entire loading test.

## 7 INTERPRETATION AND CONCLUSIONS

### 7.1 LOADING TEST BACK-ANALYSIS AND INTERPRETATION

The loading test performed has provided very valuable information on the mechanical behaviour of granite-shotcrete interfaces. The above is based on the following facts observed during the loading test:

1. The plug weighs only about 6 tonnes. In order to start moving the plug for the first time (plug “failure”: the plug suddenly starts to move –not only “to deform”– several millimetres as a rigid body and with no recovery of the displacement when unloaded), a total horizontal load of about 725 tonnes had to be applied to its internal face. This significant horizontal load was taken by the shear strength of the granite-shotcrete interface, which is mainly due to the effects of the “true” cohesion (“adherence”) and of the dilation (interlocking along the interface, even with relatively small rock roughness). This pure friction effect seems less relevant in this case, for the specific geometry of the plug tested, even considering that the compressive isostatic lines in the plug mass are not parallel to the longitudinal axis of the plug and that some normal compressive stresses act on the interface.
2. The maximum relative water pressure on the internal face of the plug, which induced the first plug “failure” (non-stop plug movement), was approximately 2.7 MPa; and the plug movement was about 0.4 cm before unloading. Subsequently, two more loading cycles were performed. During these, plug movement took place again when the applied water pressure was about 2.4 MPa; only about 0.3 MPa less than in the first “failure”; and the final total plug movement was higher than 1.5 cm. It may be assumed that the “true” cohesion of the interface disappears after the first “failure” (movement of about 0.4 cm). Consequently, this parameter is probably relatively low, because even without cohesion the plug was able to withstand applied pressures of about 2.4 MPa in the subsequent loading cycles. At the same time, it may be assumed that **friction and dilation (interlocking) were acting fully in all the loading cycles**.
3. In the three loading series, the “failure” occurred when the recorded extensometer deformations reached about 1 mm. Below this threshold value, plug deformations were quite stable for each load step and partly elastic (deformation recovery when unloading). This observation has been taken into account for the hypotheses of the back-analysis calculations performed after the loading test.
4. The local water leakage at the bottom of the plug was an unexpected and uncontrolled event. Both the scoping and the back-analysis calculations have been performed without hydraulic effects, assuming that no interstitial water pressures develop in the plug mass. Fortunately, it seems that the local water leakage (and possibly some build up of interstitial water pressure) occurred only in a very small zone near the interface; probably with some effect in less than 10% – 20% of the total interface area. Even after the three loading series, and a plug movement greater than 1.5 cm, most of the plug perimeter appeared to be sealed and no other water leakages were observed along the interface.

Taking into account the comments made above, a new set of back-analysis calculations have been performed using the same code (FLAC, two-dimensional axisymmetric) and model (elasto-plastic Mohr-Coulomb with ubiquitous joint at the granite-shotcrete interface) as in the set of scoping calculations. The granite and mass shotcrete parameters adopted are the same, while the relevant changes have been made only for the interface. Specifically, the previous parameters (scoping) and those now adopted (back-analyses) are as shown in Table 11:

**Table 11: Interface parameters**

	<b>Scoping (Interface)</b>	<b>Back-analyses (Interface)</b>
<b>Friction angle</b>	38°	38°
<b>Cohesion</b>	0.7 and 1.0 MPa	0.12 MPa and zero
<b>Tensile strength</b>	0.7 MPa	0.12 MPa and zero
<b>Dilation angle</b>	5°	12°

In the scoping calculations, the dilation angle was set equal to zero when the calculated interface displacement is greater than 0.5 mm. Now, in the back-analysis calculations, the dilation angle is set equal to zero if that displacement is greater than 1.0 mm.

Also, in the scoping calculations values of the Young's modulus of the shotcrete ranging from 10 000 to 20 000 MPa were considered. Now, a single value of 10 000 MPa has been used.

The main results obtained from the back-analysis may be summarised as follows:

- a. In the first calculation run, with cohesion = 0.12 MPa, the plug starts to move (out-of-equilibrium) for an applied pressure equal to 2.8 MPa. This reproduces fairly well the actual first "failure" of the plug.
- b. A second run has also been performed without cohesion at the interface. With this assumption the plug starts to move when the applied pressure is equal to 2.4 MPa. Consequently, this second run also agrees well with the subsequent actual plug "failures", following the first.

The recorded data, back-analyses and interpretation of the loading test performed have greatly improved the knowledge of the mechanical behaviour of confined granite-shotcrete interfaces. It seems reasonable to assume that "true" cohesion (adhesion) at these interfaces is relatively low (0.12 MPa in this case); while the interlocking effect (represented by the dilation angle, equal to 12° in this case) plays a very relevant role in the strength of the confined (by the rock) interfaces. The interlocking effect is normally taken (and used in the standard geotechnical practice) as an "apparent" cohesion, when estimating the peak shear strength of potential failure surfaces.

## 7.2 CONCLUSIONS

From the results and interpretation of the short plug loading tests performed, the following main conclusions may be drawn:

1. Concrete with a pH equal to or lower than 11 may be formulated and successfully used for the construction of plugs in underground small diameter galleries using the shotcrete technique.
2. The shrinkage phenomenon has been negligible with the concrete formulation used.
3. The “failure” (sudden plug movement of several millimetres behaving as a rigid body) of the plug is governed by the mechanical characteristics of the confined rock-shotcrete interface. The loading test has allowed a better estimation of these characteristics to be obtained. The interlocking effect (represented in the calculation model by a dilation angle) plays a very important role in the shear strength of the confined interface.
4. The feasibility of constructing shotcrete plugs in small diameter galleries has been demonstrated, although some improvements might be introduced in the process to avoid or minimise the discontinuities between concrete layers and potential heterogeneities in the bottom part of the plug caused by shotcrete rebounds. The improvements might be convenient for situations in which significant amounts of water are expected to interact directly over the plug. In conventional repository conditions, highly compacted bentonite plugs, interposed between the waste and the shotcrete plug, act as hydraulic sealants, as a result of which the shotcrete plugs will never be exposed to significant amounts of free water.



## 8 REFERENCES

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