Rock Mech Rock Engng (2008) 41: 445–466
DOI 10.1007/s00603-007-0156-5
Printed in The Netherlands

Rock Mechanics and Rock Engineering

Leaching of rock-concrete interfaces

By

O. Buzzi[1], M. Boulon[2], M. Hervé[3], K. Su[4]

[1] Centre for Geotechnical and Materials Modelling, School of Engineering, University of Newcastle, NSW, Australia
[2] Laboratoire "Sols, Solides, Structures", Université Joseph Fourier, Grenoble, France
[3] Laboratoire LMC_2, Electricité de France, Moret sur Loing, France
[4] Agence nationale pour la gestion des déchets radioactifs (Andra), Châtenay Malabry, France

Received July 8 2006; Accepted June 28 2007; Published online November 20 2007

Summary

It has been shown that contacts between host rock and engineered barriers may be critical in the design of deep radioactive waste repositories. Water is expected to reach the interface zone after the resaturation of the geological massive and its presence may lead to concrete leaching. Such a phenomenon could increase the interface transmissivity and compromise the confinement of radioactive waste. This paper intends to investigate the influence of concrete leaching on the hydromechanical behaviour of host rock-concrete interfaces. Some concrete specimens have been subjected to an accelerated leaching process using ammonium nitrate. The hydromechanical response of degraded concrete-rock interfaces has been studied under shearing and compared to that of sound interfaces. Consistent with the results available in the literature on bulk concrete, a loss of mechanical strength has been observed for the degraded interface. Unlike the sound specimens, the degraded interfaces do not dilate when sheared and they tend to be closed, thereby preventing water from flowing.

Keywords: Interfaces, rock, concrete, calcium leaching, degradation, ammonium nitrate, hydromechanical behaviour, nuclear waste disposal

1. Introduction

Deep underground repositories have been proposed in many countries as a possible solution for storing high level long lived radioactive waste. The effectiveness of such repositories in an adequate geological environment relies on both natural and engi-

Correspondence: O. Buzzi, Centre for Geotechnical and Materials Modelling, School of Engineering, University of Newcastle, 2308 Callaghan NSW, Australia
e-mail: olivier.buzzi@newcastle.edu.au

neered barriers. Several Underground Research Laboratories have been developed worldwide to study the feasibility of this concept and the materials involved have been studied for more than twenty years now. However, still very few data are available on the behaviour of contacts between engineered and natural barriers even if it is common knowledge that interfaces or discontinuities have a higher hydraulic conductivity than bulk materials (Barton et al., 1985). Some waterflow is expected in case of a repository site built in granite (as observed by Dixon et al., 2002) whereas, for storage in argillite, water is expected to percolate within the interfaces (Andra, 2005c). In any case, the concrete plug is likely to be leached (Adenot et al., 1992; Naus et al., 1999; Andra, 2005b), which can compromise the long term confinement of the repository.

Indeed, several studies have shown that leaching a mortar or a cement paste tends to increase its porosity and to reduce its mechanical strength (Carde et al., 1996; Gérard, 1996; Le Bellego et al., 2000). So far, the transfer properties of leached concrete (e.g. permeability) have not been widely investigated but they are usually correlated to the porosity by means of empirical formula (Luping and Nilsson, 1992; Ollivier and Massat, 1992). Considering that the porosity of the concrete part of the interface is likely to increase, an augmentation of the interface transmissivity could be expected.

Calcium leaching by water leads to the dissolution of portlandite contained in the cement paste. This phenomenon is well understood even if most of the data come from accelerated chemical reactions. Indeed, natural leaching is a very long process and very few data are available in the literature about long term degradation of concrete by pure water (Tragardh and Lagerblad, 1998). Some accelerated procedures have been set up to solve this issue and to get relevant data in an acceptable time. Two techniques are commonly used: subjecting the concrete specimen to an electrical potential (LIFT test, e.g. in Saito and Deguchi, 2000) or ammonium nitrate leaching (Adenot et al., 1992; Gérard et al., 1995; Carde, 1996; Kamali et al., 2003; Le Bellego et al., 2003).

It is of particular importance to understand the hydromechanical behaviour of degraded interfaces in the context of nuclear waste repository since any concrete component exposed to water is likely to be leached (Bourbon, 2005; Andra, 2005b). Moreover, most of the knowledge on concrete leaching is related to the bulk material but not to interfaces, for which there is an obvious lack of data. Considering these two points, the study presented herein has been undertaken. The paper reports the results obtained after a series of experimental investigations aiming to point out the effect of long term degradation on the hydromechanical response of a rock concrete contact. Leached concrete-rock as well as sound interfaces have been subjected to hydromechanical shear test in order to investigate the effect of the leaching on the mechanical response and on the evolution of the transmissivity of the interface.

2. Experimental details

2.1 Experimental program

Four tests have been performed to highlight the influence of the leaching on the behaviour of the interfaces: two on artificially aged interfaces, one on a naturally aged interface and one on a sound interface (see Table 1 for details).

Table 1. Chemical treatments the specimens have been subjected to for 100 days

Tests	INT	DIST	LIX2	LIX3
Chemical treatment	water plus lime	distilled water	NH_4NO_3 solution	NH_4NO_3 solution

In order to fully investigate the hydromechanical behaviour of the interfaces, the shear tests have been performed in two phases: the specimens are first subjected to compression with increments of load of 1 MPa and then, they are subjected to a constant stiffness shearing until a final tangential displacement of 6 mm is reached. The imposed normal stiffness during the shear phase has been set to 3 GPa/m, which is of the same order of magnitude as the parameters used by Jiang et al. (2004). Actually, a technical problem occurred during the tests, preventing the normal stiffness from being kept constant and from following the intended stress path. This point will be discussed in the results section. The specimens are loaded up to 8 MPa in compression except specimen LIX2, which is loaded up to 5 MPa in order to investigate the behaviour under a lower normal stress. The initial flow rate is set at approximatively 4E-6 m^3/s for a normal stress equal to zero and for all the specimens. As observed by Hans (2002) and Buzzi (2004) for such a value of flow rate, the flow is assumed to be laminar. Pressure and flow rate are not monitored during the test, they evolve only by the effect of compression and shear of the interface.

2.2 Ammonium nitrate leaching

The ammonium nitrate (NH_4NO_3) degradation process has been widely studied so that the keys parameters of the leaching kinetics are identified (e.g. temperature, pH, leaching rate, ammonium nitrate concentration) and their influence are understood (Adenot et al., 1992; Kamali et al., 2003). In particular, it has been shown that the more concentrated the solution, the more accelerated is the leaching process; the maximum concentration (480 g/l or 6 mol/l) is governed by the ammonium nitrate solubility. Herein, the concrete specimens have been subjected to an accelerated leaching for 100 days in a 6 mol/l solution. The leaching process dissolves the portlandite crystals $Ca(OH)_2$ and decalcifies the C–S–H of the cement paste. More details about the ammonium nitrate leaching can be found in Carde et al. (1996). Nitrogen bubbling prevents the carbonation of the specimen, which tends to fill the pores and would reduce the diffusion process of the NH_4NO_3 through the cement matrix and thus limit the concrete degradation. After 100 days of degradation, the specimens are immersed in distilled water in order to prevent the formation of a swelling gel damaging the specimen.

2.3 Hydromechanical shear device BCR3D

The tests have been performed using the direct shear apparatus BCR3D and the associated hydraulic device. This apparatus was designed by Boulon (1995) and developed by Armand (2000) and Hans and Boulon (2003). The direct shear box "BCR3D" can be used for all classical interface compression and shear tests (constant normal

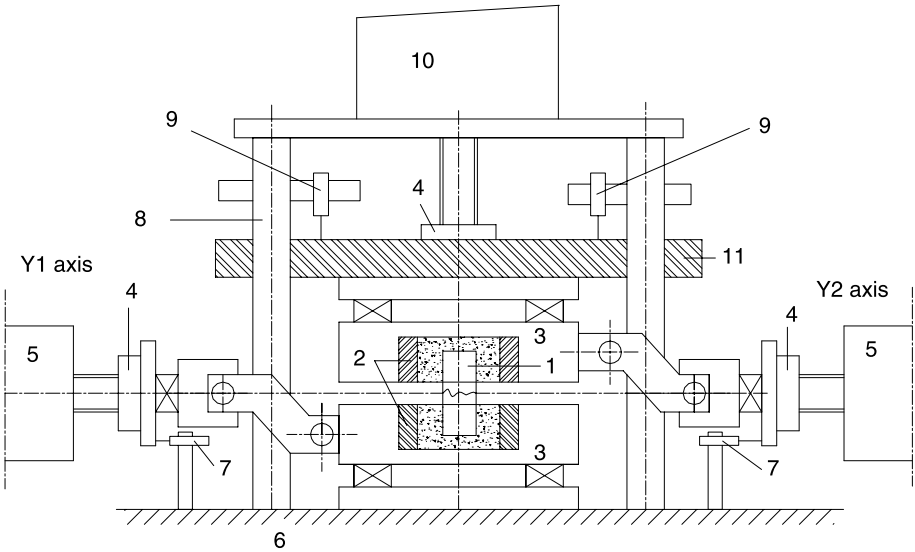


Fig. 1. Front view section along one shear axis of the BCR3D. *1* Interface specimen to be tested, *2* internal removable metallic boxes, *3* external boxes, *4* load cells, *5* horizontal actuators, *6* rigid frame, *7* displacement sensors (LVDT measuring Δy_1 and Δy_2), *8* rigid columns, *9* displacement sensors (LVDT measuring Δz), *10* vertical actuator, and *11* rigid vertically translating structure

stress, constant normal stiffness, constant volume) and has a shear velocity ranging from 0.05 mm/s to 50 cm/s through the use of both quasi static and dynamic electromechanical jacks. The BCR3D has been designed to avoid any relative rotation of the rock surfaces during the shear displacement. Such a relative rotation can greatly affect the quality of the tests (Boulon, 1995). Through the use of two actuators in the horizontal shear axis, the tangential relative displacement of both rock surfaces are symmetrical with respect to the vertical (normal) axis, facilitating the application of the normal force on the joint. One advantage is that the normal force is kept centered on the surface of the joint thus preventing the upper part of the specimen from rotating. Each axis is equipped with one or two actuators (capacity of 100 KN) and with sensors measuring displacements and forces. Two LVDT's are used in the normal load axis to check that there is no parasite rotation during the shear test. A cross section of the apparatus is shown in Fig. 1.

The diagram of the associated hydraulic device is shown in Fig. 2a. A constant water flow Q1 produced by the volumetric pump is injected into the hydraulic circuit which is divided in two branches: the first one (Nr3) is an adjustable discharge. The second branch (Nr2) is connected to the specimen with a continuous measurement of the pressure (P2) and the flow rate (Q2). Both hydraulic gates R2 and R3 are used to drive the test.

2.4 Calculation of the transmissivity

Similar to rock joints, rock-concrete interfaces are networks of voids that can be described as a porous medium of permeability k saturated by water and obeying

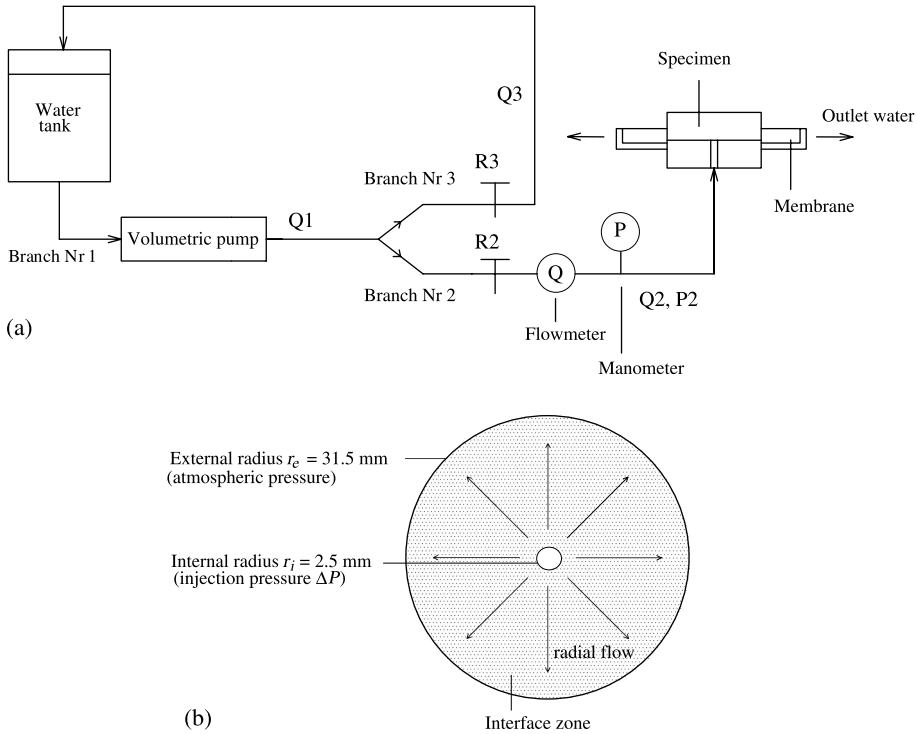


Fig. 2. (a) Diagram of the hydraulic device. The pump injects a constant flow rate (*Q1*) into the circuit which is divided into two branches. One branch (Nr3) is the adjustable discharge whereas the other one (Nr2) goes to the specimen with a continuous measurement of flow rate and of pressure. (b) Schematic representation of the interface zone subjected to internal overpressure ΔP generating a radial flow

Darcy's law. Transmissivity tests are performed on annular specimens of interfaces and the flow is created by applying an injection overpressure ΔP of fluid at the internal radius (r_i) of the specimen while the atmospheric pressure (equal to zero) is applied at the external radius (r_e) (see Fig. 2b). As in many other studies (Esaki et al., 1999; Lee and Cho, 2002; Hans and Boulon, 2003), the tests can be interpreted in terms of isotropic transmissivity rather than in terms of permeability because this choice avoids making a hypothesis about the local hydraulic aperture. For a radial flow, the intrinsic transmissivity can be expressed as follows:

$$T = \frac{\ln(r_e/r_i)}{2\pi} \cdot \mu \cdot \frac{Q}{\Delta P} \tag{1}$$

with:

- T: intrinsic transmissivity [m^3],
- Q: flow rate [m^3/s],
- ΔP: internal injection pressure [Pa],
- r_e: external radius of the annular specimen [m],

- r_i: internal radius [m],
- μ: dynamic viscosity of water [Pa · s] (9.11E-4 Pa · s at 24 °C).

2.5 Materials

The rock used herein is a Callovo Oxfordian Argillite from Bure (East of France) where an underground research laboratory is currently under construction at a depth of about 500 m. The rock specimen used for this investigation has a uniaxial compressive strength of 21 MPa and a Young's modulus of 5600 MPa. With the laboratory still under construction, the roughness of the excavated rock wall constituting the contact between host rock and engineered barrier is not really known. It will depend on the rock fracturation and on the excavation method. So far, there is no real guideline regarding the rock wall surface to use in interface studies. The interface tested herein corresponds to a natural discontinuity of the rock (see Fig. 3). As rock replicas are commonly used in rock joint investigations for the sake of convenience, the contact morphology has been reproduced by a mortar plaster, which has a compressive strength of 35 MPa at 24 hours and of 45 MPa at 28 days.

The concrete subjected to leaching is a material recommended by the Andra (2005a) and its composition is given in Table 2. Such a concrete has a Young's Modulus of 43,000 MPa, a compressive strength of 64 MPa at 28 days and an intrinsic water permeability of 1.5E-19 m^2.

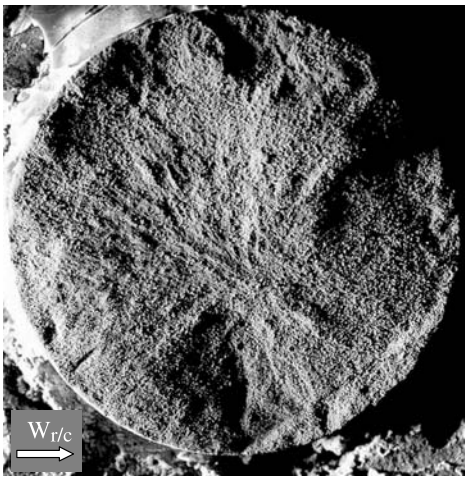


Fig. 3. (a) Argillite specimen (from Bure) used for the rock replica. $W_{r/c}$: Direction of the tangential relative displacement of the rock part with respect to the concrete part. Diameter of the argillite core: 63 mm

Table 2. Concrete components and composition. $W/C = 0.4$

Components	Cement CLC CEM V Calcia – Airvault	Sand 0/4	Aggregates 4/12.5 calcaire du boulonnais	Water	Additive Glénium 27
Amount	430 kg/m^3	800 kg/m^3	984 kg/m^3	180 kg/m^3	10.45 kg/m^3

2.6 Specimen preparation

The specimen preparation is shown in Fig. 4 and it can be summarized in four main stages as follows:

- Repoduction of the rock specimen using a silicon mould in order to make four rock replicas (steps I to III of the procedure shown in Fig. 4),
- making of the concrete part on the interface (steps IV to VIII),
- chemical treatment of the concrete specimen (see Table 1) for 100 days (step VI),
- reconstruction of the contact (steps VII and VIII).

After these four stages, the specimen can be set up in the BCR3D and tested. To better understand the course of the whole experimental procedure, a chronology is given in Table 3.

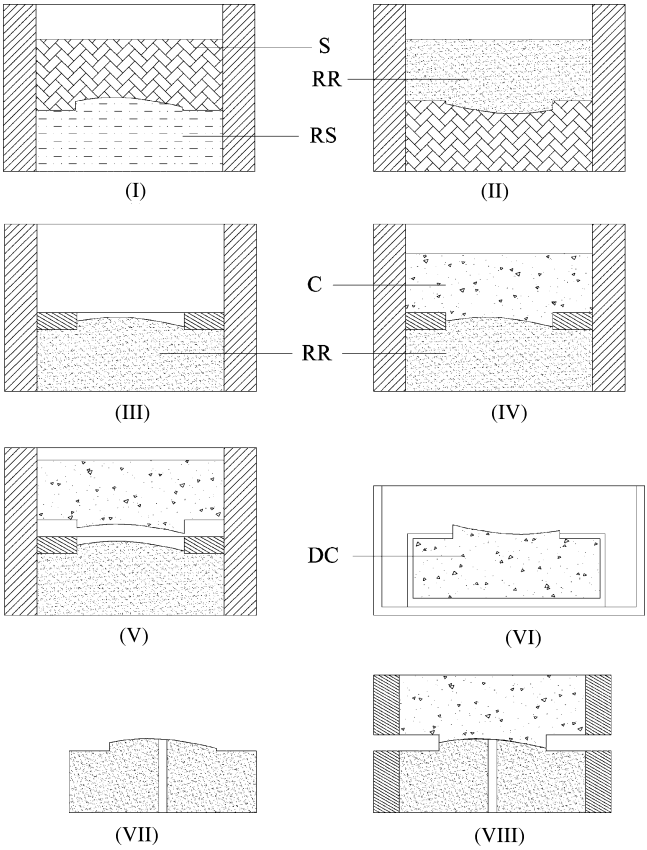


Fig. 4. Preparation steps of the rock-concrete interface specimens. Step I: A negative reproduction of the rock is made in silicon (S). Step II: The mortar is poured on the silicon mold to create the rock replica (RR). Steps III and IV: The concrete (C) is poured on the rock replica (RR) to create the contact. Step V: After 48 hours, the hardened concrete is separated from the rock replica. Step VI: The degraded concrete (DC) is obtained after 100 days in the leaching solution. Step VII: The injection hole (diameter 5 mm) is drilled in the rock replica in order to enable the fluid injection under pressure at the center of the interface. Step VIII: The contact is reconstituted and both parts are sealed in the metallic removable boxes

Table 3. Chronology of experimental program from specimen preparation to testing

Course of experimental program	Starting day	Duration (days)
Making of 4 rock replicas	$d_0 - 1$	1
Making of 4 concrete specimens	d_0	1
Concrete curing in water	$d_0 + 1$	30
Chemical treatment	$d_0 + 31$	100
Rinsing phase in distilled water	$d_0 + 131$	30
Preparation of specimen DIST	$d_0 + 187$	1
Preparation of specimen INT + test DIST	$d_0 + 188$	1
Preparation of specimen LIX3 + test INT	$d_0 + 189$	1
Preparation of specimen LIX2 + test LIX3	$d_0 + 190$	1
Test LIX2	$d_0 + 191$	1

3. Results and discussion

3.1 Degradation

Figure 5a–c show a cross section of LIX2, LIX3 and DIST specimens. The degraded zone, about 2 mm deep, and the portlandite dissolution front can clearly be seen in the LIX2 and LIX3 specimens. As shown in Fig. 6, the LIX3 specimen appears to to be slightly more degraded than LIX2 considering the higher final concentration of Ca^{2+}. No evidence of degradation is visible on specimen DIST, which is then considered sound as is specimen INT. It has to be emphasized that the leaching process in the laboratory is quite different to the in situ phenomenon in terms of conditions i.e. water percolation and low flow rate can not be compared to immersion in a chemical solution. According to the hydrogeological conditions in situ, 40 cm of significant chemical perturbations in 300,000 years are expected on the edge of an engineered barrier made of CEM V concrete (Bourbon, 2005). Hence, 2 mm of degradation, obtained in a few weeks under laboratory conditions, could be compared to less than twenty years degradation in natural saturated conditions for steady state diffusion (Bourbon, 2005).

3.2 Mechanical behaviour

The evolution of normal stress versus normal displacement during the compression phase is shown in Figs. 7 and 8. The normal stiffness of the contact, which is a material property, can be estimated to about 10 GPa/m for all the four tests leading to the conclusion that the stiffness is not affected by the degradation. Two reasons could explain this observation. First, considering the concrete specimen as a two layer material (see Fig. 9) enables one to estimate the effect of the degradation on the normal stiffness. With u_d (or u_i) being the relative normal displacement generated by the normal stress σ_n for the degraded (or intact) interface, the difference in normal displacement $\Delta u = u_i - u_d$ can be expressed as:

$$\Delta u = \sigma_n \cdot h_{DC} \left(\frac{1}{E_{SC}} - \frac{1}{E_{DC}} \right) \tag{2}$$

where h_{DC} is the height of degraded concrete (2 mm), E_{SC} is the Young's modulus of the sound concrete (43,000 MPa) and E_{DC} is that of the degraded concrete. Kamali

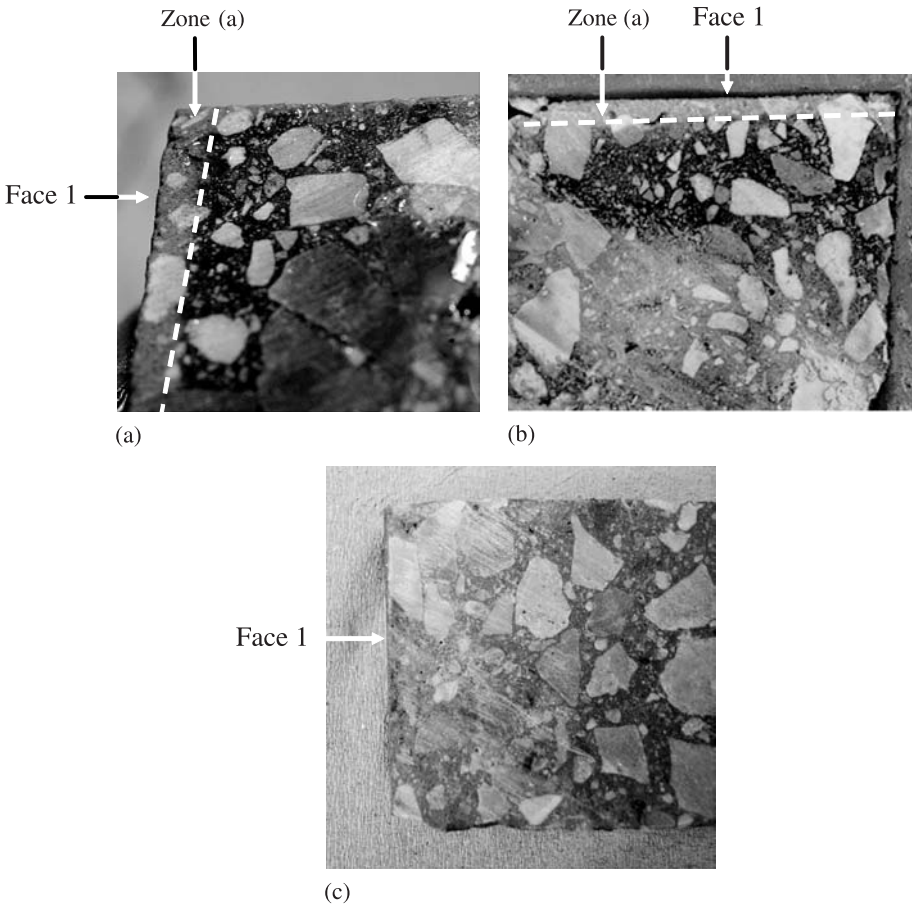


Fig. 5. Section of concrete specimens LIX2 (a) and LIX3 (b) leached on Face 1 by an ammonium nitrate solution (thickness of the degraded zone (a) 2 mm (± 0.5 mm)). (c) Section of specimen DIST leached on Face 1 by distilled water

et al. (2005) have shown that the loss of Young's modulus can reach 50% after 98 days of leaching and this value has been considered herein i.e. $E_{DC} = E_{SC}/2$. Then, for a normal stress $\sigma_n = 8$ MPa, Eq. (2) leads to Δu of about 4E-4 mm, a very small value, explaining that degraded and intact specimens seem to behave the same in compression. The other plausible explanation is that some aggregates are found close to the interface (see Fig. 5) and predominantly contribute to the normal stiffness of the degraded zone. With the stiffness of the aggregate not much affected by the leaching, the stiffness of the degraded contact is very close to that of the sound one.

However, the effect of leaching appears clearly when considering the evolution of normal displacement with tangential displacement during compression (see Fig. 10). To make the difference between the tangential displacement due to the compression and the one corresponding to shear phase, the total shear displacement W is divided into W_c (due to compression) and W_s (due to shearing), such that $W = W_c + W_s$. In the

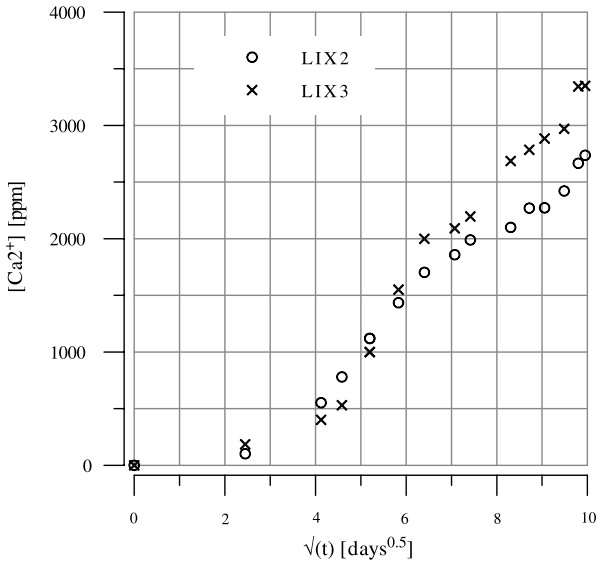


Fig. 6. Leaching kinetics: evolution of concentration of Ca^{2+} ions in the leaching solution versus square root of time

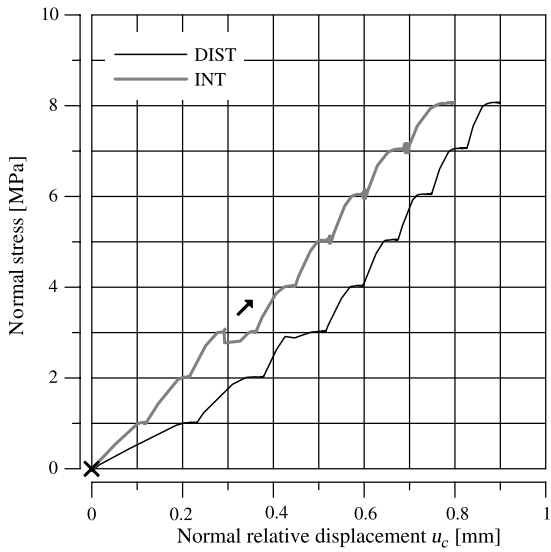


Fig. 7. Evolution of normal stress with respect to normal displacement u_c for intact specimens INT and DIST

same way, the normal displacement is decomposed as: $u_n = u_c + u_s$. If the asperities of both rock surfaces are perfectly interlocked, no tangential displacement is expected and the normal displacement is only due to the compression of the rock (Zhao, 1996). However, for a reconstituted contact, the matching is never perfect and Gentier (1986)

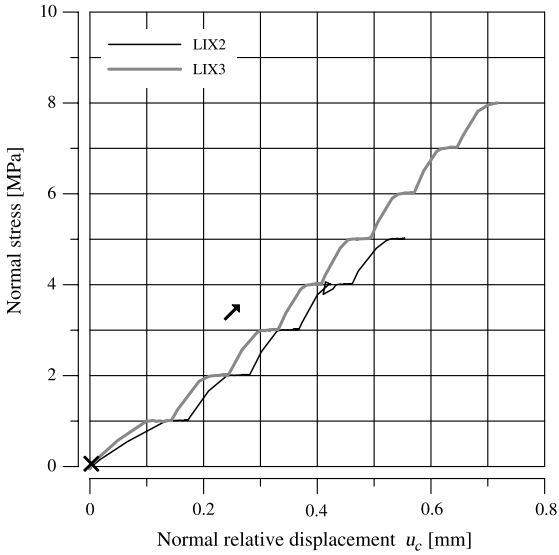


Fig. 8. Evolution of normal stress with respect to normal displacement u_c for artificially aged specimens LIX2 and LIX3

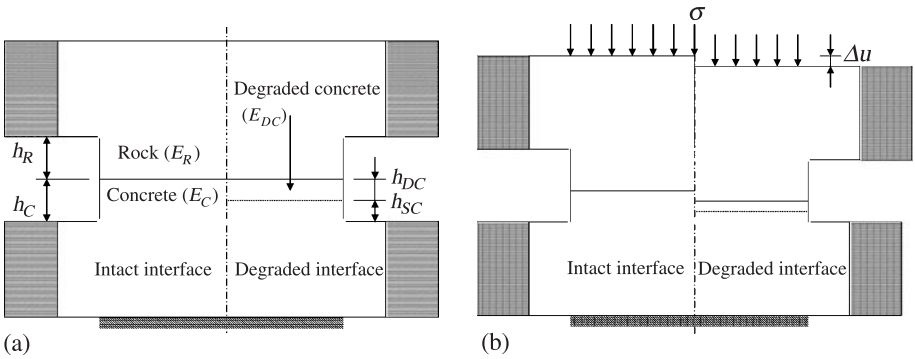


Fig. 9. Serial materials model in compression. (a) Initial configuration. (b) Deformed configuration. h_R: Height of unconfined rock specimen, h_C: height of unconfined concrete specimen, h_{DC}: height of degraded concrete, h_{SC}: height of unconfined sound concrete, E_R: Young's modulus of the rock, E_{SC}: Young's modulus of the concrete, E_{DC}: Young's modulus of the degraded concrete and Δu: difference in normal displacement

has developed a protocol to obtain a better matching of the rock surfaces before testing. Herein, the tangential displacement is not constrained during compression so that both rock surfaces can slightly move along the shear axis. This phenomenon is due to the progressive matching of both rock surfaces which are not perfectly interlocked initially (see schematic representation in Fig. 11). The same phenomenon leads to an initial contraction when shearing a rock joint (Olsson and Brown, 1993). In case of degraded interfaces, the asperities of the concrete wall have lost a part of their mechanical strength (as for any concrete, see Carde et al. (1996)). As a consequence,

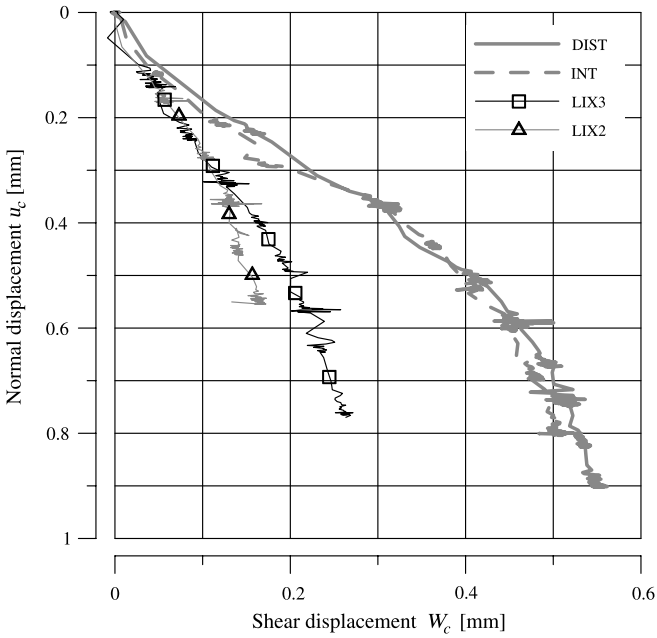


Fig. 10. Evolution of normal displacement u_c with respect to shear displacement W_c during the compression phase

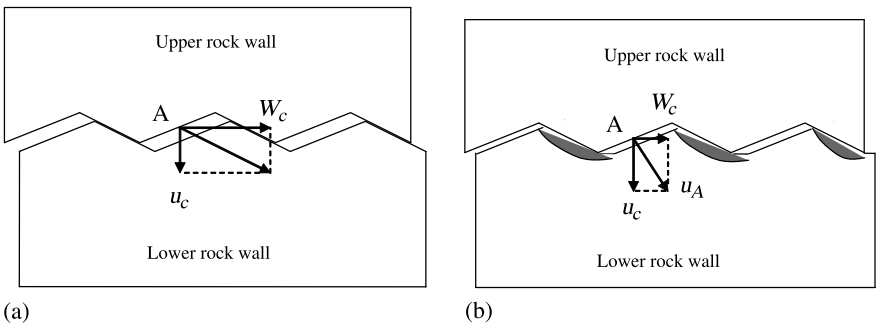


Fig. 11. Schematic representation of the matching process occurring during compression. (a) Intact concrete: the upper surface tends to slide on the lower surface and the total displacement in point A depends on the asperities' inclination. (b) Degraded concrete: the asperities are crushed during compression (hatched zone) so that the upper surface does not slide as much as before. The tangential displacement W_c induced by the compression is then reduced

the asperities are more easily crushed during compression and the sliding phenomenon is reduced. It can be seen in Fig. 10 that the final value of W_c for the degraded interfaces is lower than that of sound interfaces.

Short term creep can be noticed for all the tests in Figs. 7 and 8. The normal displacement keeps increasing under a constant normal stress just after having applied a load increment. This effect, studied by Matsuki et al. (2001) in the framework of rock joints, is more obvious in Fig. 12a where normal displacement and normal stress are

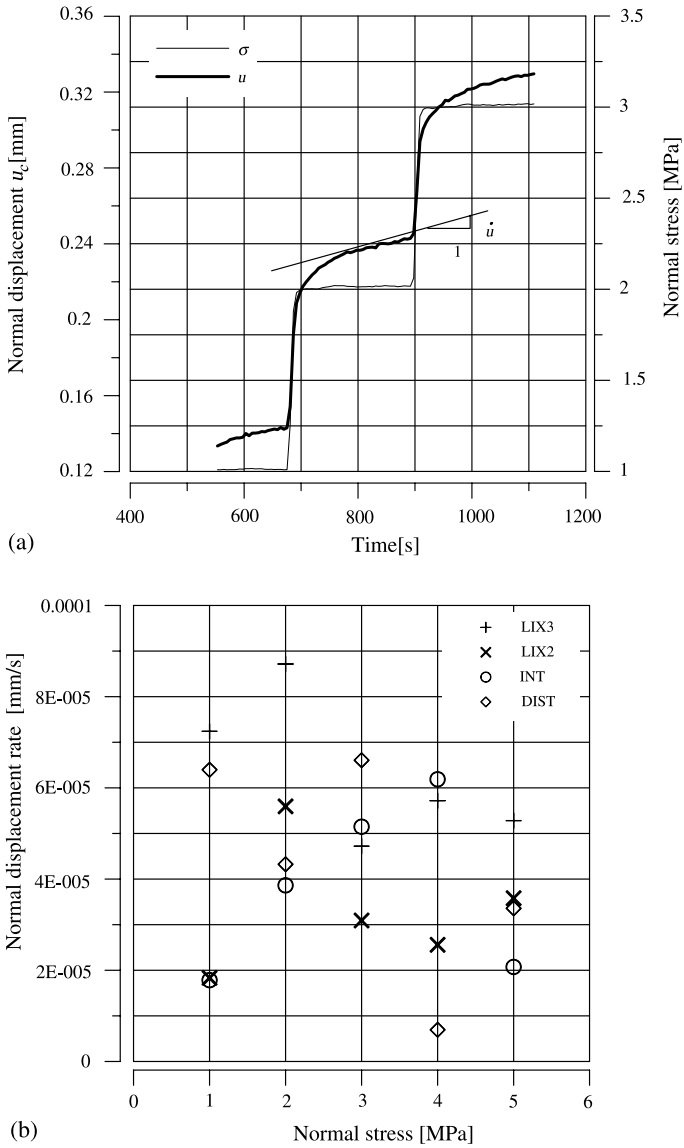


Fig. 12. Short term creep: (a) Evolution of normal stress and normal displacement u_c with time for part of test LIX3. (b) displacement rate $\dot{u}$ at different normal stresses for all tests

plotted with respect to time. Bernard et al. (2003) have studied the creep of leached concrete at the microscopic level, and they have shown that the characteristics of the short term creep, governed by the interfacial transition zone, is preserved in case of leached material. Herein, the creep kinetics $\dot{u}$ of all four specimens is plotted as a function of normal stress in Fig. 12b and, consistent with the observations by Bernard et al. (2003), no significant difference can be made between sound and leached interfaces.

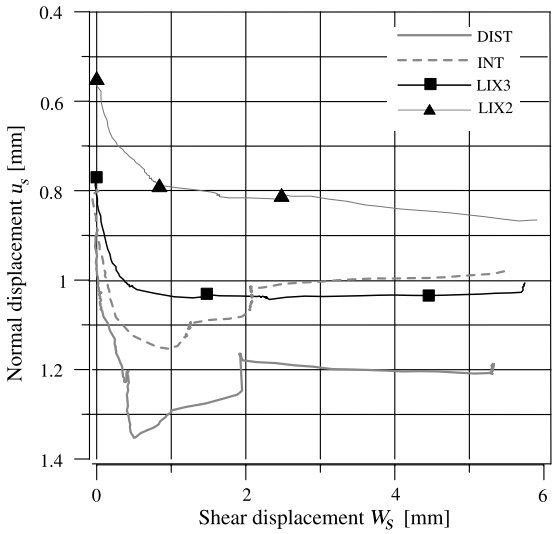


Fig. 13. Evolution of normal displacement u_s with respect to shear displacement W_s during the shearing phase

The degradation produces a drastic change in behaviour when shearing the interface. Figure 13 shows the evolution of the normal displacement u_s with respect to the tangential displacement W_s during the shear phase. As shown by Zhao (1996), a perfectly matched contact tends to dilate when sheared but the typical response of a rock joint is to first contract and then dilate (Barton et al., 1985). As mentioned by Barton et al. (1985), a joint starts to dilate when the roughness is mobilised and the dilation results from the interaction of the two surfaces riding over one another. Consistent with the results of the technical literature, the sound interfaces in our research are successively contracting and dilating. On the contrary, the degraded interfaces are fully contracting under a normal stress of either 5 and 8 MPa. Reducing the value of normal stress before shearing from 8 MPa to 5 MPa does not lead the degraded contact to dilate.

Due to a technical problem on the apparatus, the stiffness could not be kept constant during the shearing. The effective normal stiffness ranged from 3 to 9 GPa/m. However, the qualitative behaviour of the interfaces is not affected by such a variable stiffness. Jiang et al. (2004) have observed little difference in the amount of dilation when changing the normal stiffness but they have not observed any change of behaviour: the interfaces remained contracting/dilating. The change of behaviour observed herein between sound and degraded contacts can then reasonably be considered intrinsic to the interfaces and not dependent on the stress path followed during the shear test.

Heukamp et al. (2003) have observed localized shear bands leading to the collapse of macropores when studying the micro structure of leached mortar subjected to deviatoric tests. Moreover, the degraded specimens displayed a contracting/dilating behaviour. They measured a volumetric strain of about 2% during dilation, which is not sufficiently significant, when applied to a 2 mm thick layer, to produce a visible effect at the macroscopic level. On the other hand, the increase of the macroporosity

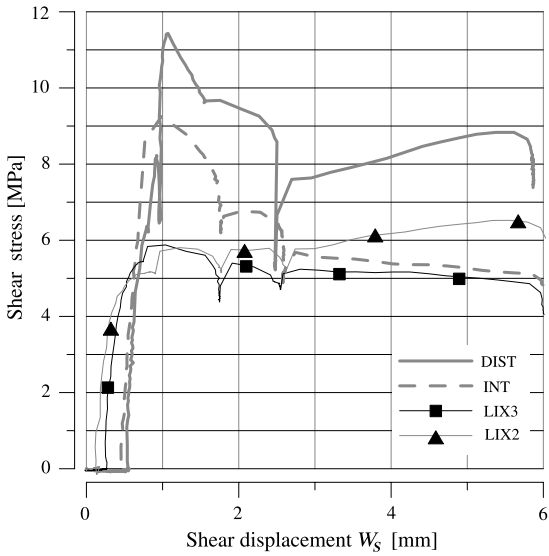


Fig. 14. Evolution of the shear stress with respect to the shear displacement W_s for all the tests

of the cement paste due to the leaching is accompanied by a reduction of mechanical strength as shown by Carde et al. (1996) and visible in the mechanical response in Fig. 14. This indicates that, unlike the sound interfaces, the asperities of the leached contact are worn off easily preventing the dilation. The leached interfaces are then fully contracting.

3.3 Hydromechanical behaviour

The analysis developed in the following is more qualitative than quantitative. Attention is focused on the evolution of the hydraulic parameters rather than on their absolute values. Figure 15 shows a typical evolution of hydraulic parameters versus normal stress. As for rock joints, flow rate drops and pressure rises with the compression because of void space reduction and augmentation of tortuosity. As a consequence, transmissivity is reduced.

The evolution of intrinsic transmissivity for all specimens is shown in Fig. 16. Similar initial values of transmissivity ($T \approx 1E-13\,m^3$) have been obtained by imposing similar initial flow rates for specimens INT, DIST and LIX2 ($Q \approx 4E-6\,m^3/s$). As an increase of transmissivity was expected for the degraded specimen, the initial flow rate has been slightly reduced to $1.6E-6\,m^3/s$ for specimen LIX2, explaining the initial drop of transmissivity. Regarding specimen LIX3, an unexpected lower transmissivity has been observed. Due to the high initial pressure for this test, no attempt to reach an initial flow rate of about $4E-6\,m^3/s$ has been made in order to avoid a breakage of the hydraulic system by excessive injection pressure. Having a similar mechanical behaviour (in terms of normal stress-normal displacement relation), all four specimens have a similar decrease of transmissivity with normal stress even if

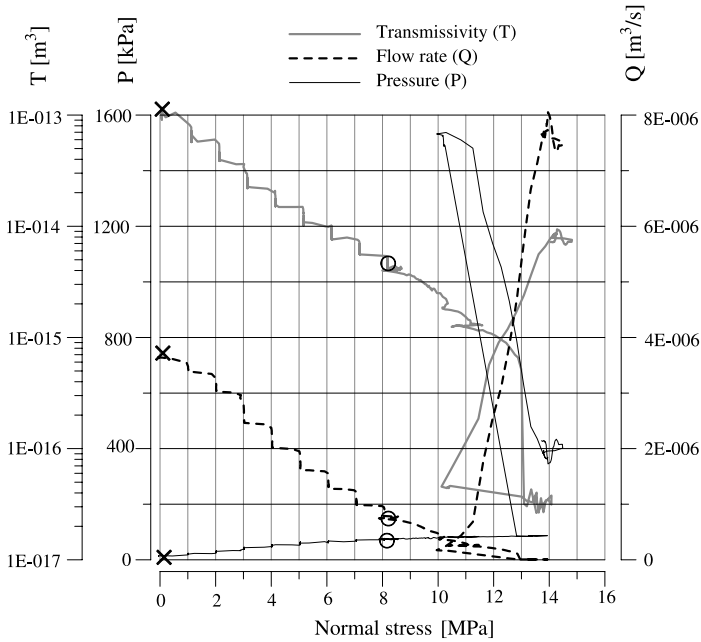


Fig. 15. Typical evolution of hydraulic parameters during the test (specimen DIST). Evolution of intrinsic transmissivity (*T*), pressure (*P*) and flow rate (*Q*) with respect to the normal stress. ×: Beginning of compression phrase. ○: Beginning of shear phase

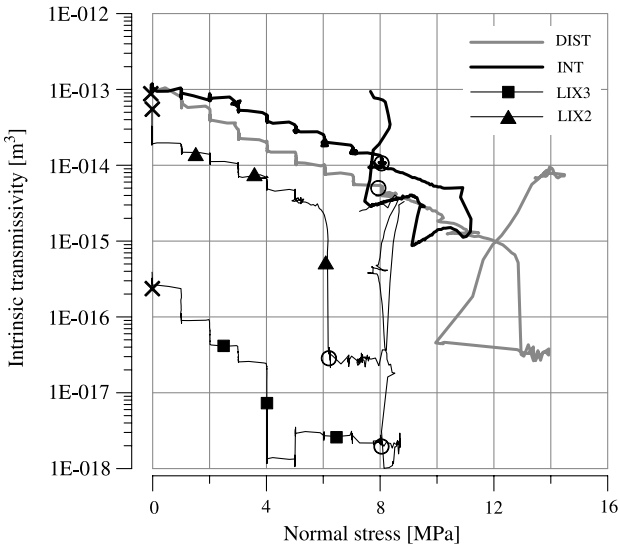


Fig. 16. Evolution of intrinsic transmissivity with respect to normal stress for all tests. ×: Beginning of compression phase. ○: Beginning of shearing phase

LIX3 specimen behaves slightly differently. Indeed, this interface tends to conduct less water: it has a much lower initial transmissivity and it is practically closed at a normal stress of 4 MPa. It has been observed that degraded interfaces produce some gouge material during the test and specimen LIX3 is slightly more degraded than LIX2 (more Ca^{2+} in solution at the end of the chemical treatment). The consequence is that more gouge material is likely to be produced for specimen LIX3. Setting the specimen in the shear box generates slight mechanical loads (normal stress of about 0.1 MPa), which appears to be sufficient to produce some initial gouge material generating a lower initial transmissivity. A time dependent reduction of transmissivity under constant normal stress can be noticed for all the tests. As transmissivity depends on interface closure, this is a consequence of the time dependent phenomena observed in Figs. 7 and 8.

When undergoing shearing, the intact interfaces behave like rock joints (Olsson, 1999; Hans and Boulon, 2003). The evolution of transmissivity closely follows that of normal displacement: it decreases during the first millimeters of shearing and increases when the joint dilates (see Fig. 17). The degraded interfaces are closed under a relatively lower value of normal stress than the sound interfaces but their hydromechanical response in compression is still very similar to that of sound interfaces (see Fig. 18). However, this observation is not consistent with the fully contracting behaviour observed previously. Indeed, as shown in Fig. 19, the transmissivity increases while the joint is still contracting. Actually, the increase of transmissivity at $W = 4.5$ mm and $W = 1.5$ mm is due to the damage of the contact. Indeed, it is obvious in Figs. 20 and 21 that specimens LIX2 and LIX3 are much more damaged by shearing than the sound interfaces. With the damaged zone reaching the injection hole, the transmissivity logically increases but this is not related to the joint dilation.

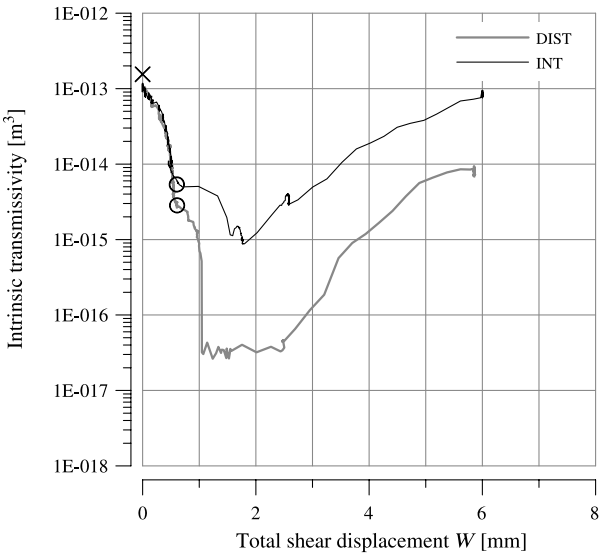


Fig. 17. Tests DIST and INT. Evolution of intrinsic transmissivity with respect to total shear displacement W. ×: Beginning of compression phrase. ○: Beginning of shearing phase

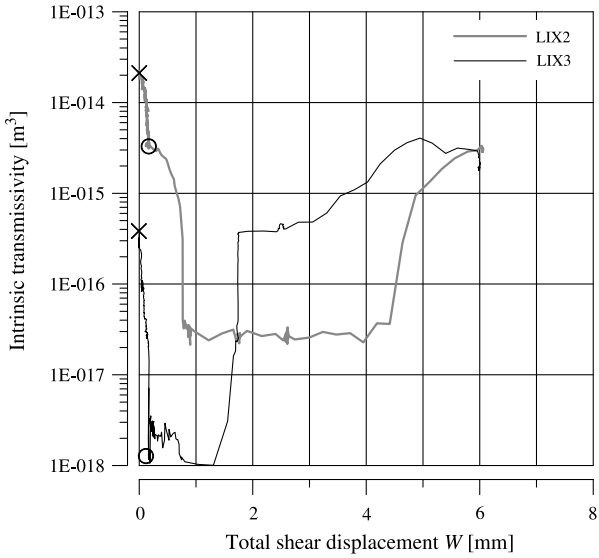


Fig. 18. Tests LIX2 and LIX3. Evolution of intrinsic transmissivity with respect to total shear displacement W. ×: Beginning of compression phrase. ○: Beginning of shear phase

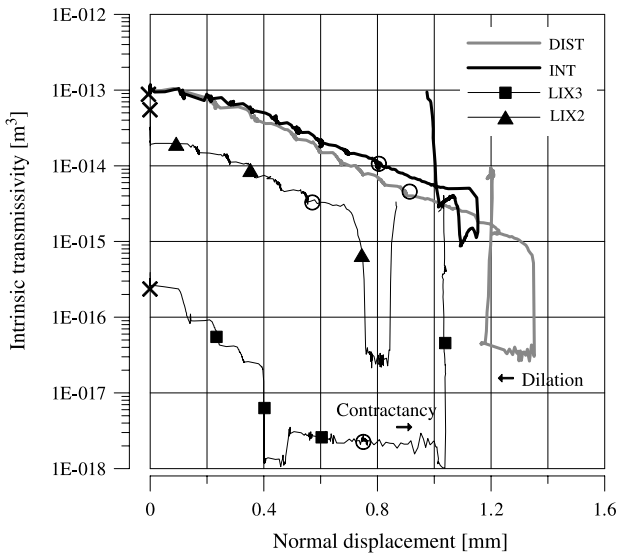
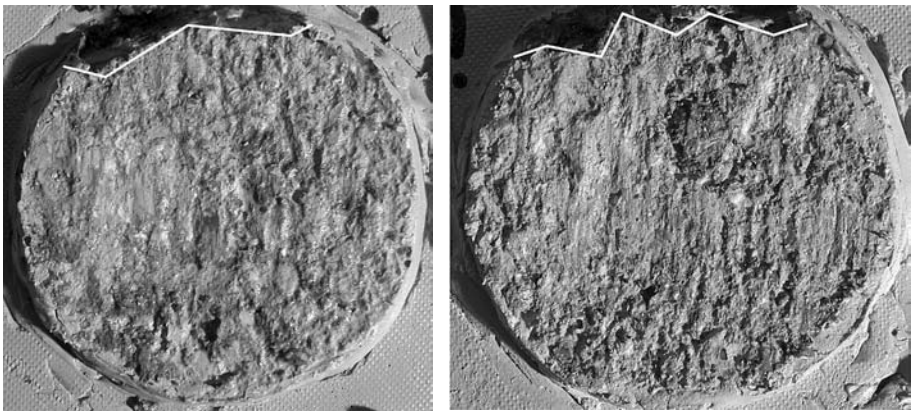


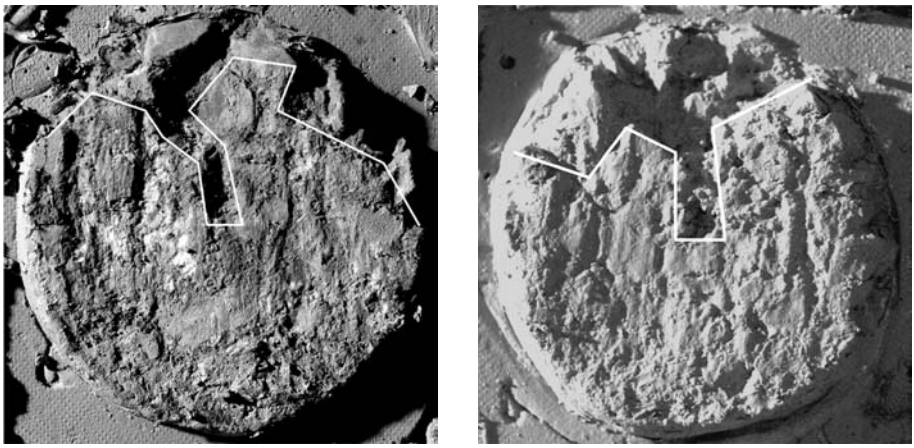
Fig. 19. Evolution of intrinsic transmissivity with respect to normal displacement. ×: Beginning of compression phrase. ○: Beginning of shear phase

As explained in Sect. 2.4, the test results are expressed in terms of transmissivity in order to avoid any assumption regarding the value of hydraulic aperture. The transmissivity of degraded concrete is then required to assess if water flows in the degraded concrete as well as in the interface. While transmissivity of the interface is



(a) (b)

Fig. 20. Photograph of the concrete part of the interface after the hydromechanical shear tests. (a) Specimen INT subjected to an initial normal stress of 8 MPa before shearing, (b) specimen DIST subjected to an initial normal stress of 8 MPa before shearing. The approximate limit of the contact surface is suggested for each specimen



(a) (b)

Fig. 21. Photograph of the concrete part of the interface after the hydromechanical shear tests. (a) Specimen LIX2 subjected to an initial normal stress of 5 MPa before shearing, (b) specimen LIX3 subjected to an initial normal stress of 8 MPa before shearing. The approximate limit of the contact surface is suggested for each specimen

directly measured from the tests performed herein, the transmissivity of bulk degraded concrete is estimated from Andra's data. Specifically the intact concrete has an intrinsic permeability K_{SC} of 1E-19 m^2 (Bourbon, 2005) and the leaching process is expected to generate an increase of the concrete permeability by a maximum of two orders of magnitude (Bourbon, 2005). In this way, the intrinsic permeability of the degraded concrete K_{DC} would be about 1E-17 m^2. For a water injection at the centre of

the interface generating a radial flow, the transmissivity of degraded concrete T_{DC} is given by

$$T_{DC} = K_{DC} \cdot h_{DC} \tag{3}$$

where $K_{DC} = 1\text{E-}17\,\text{m}^2$ and $h_{DC} = 2\text{E-}3\,\text{m}$. Given the order of magnitude of the maximum transmissivity of bulk degraded concrete ($T_{DC} = 2\text{E-}20\,\text{m}^3$) compared to that of the leached interface, it is reasonable to assume that the leaching process does not increase the concrete permeability enough to obtain a flow through it and water flows only within the interface.

4. Conclusions

The influence of calcium leaching on the mechanical properties of concrete or cementitious materials is usually investigated for the bulk materials. No data have been found in the scientific literature on the effect of leaching on the hydromechanical behaviour of interfaces. However, this is a relevant issue in the context of nuclear waste repositories since it can compromise the long term confinement of the waste. Two rock concrete interfaces have been subjected to accelerated leaching using ammonium nitrate for 100 days before being subjected to a hydromechanical shear test. Their behaviour has been compared to that of intact interfaces and it has been shown that the degradation of the concrete wall to a depth of about 2 mm produces a drastic change of behaviour due to the local loss of mechanical strength. From a mechanical point of view, the tangential displacement induced by compression is reduced in case of degraded interfaces compared to intact interfaces. Under shearing, the response of intact interfaces corresponds to the typical behaviour of rock joints namely contraction followed by dilation, while the degraded interfaces display a fully contracting behaviour. Indeed, with reduced mechanical strength, the contact asperities are easily crushed and the roughness can not be mobilised to produce dilation.

Regarding the hydromechanical behaviour of the degraded interface, it has been observed that, as for intact contacts, the transmissivity first decreases when shearing the interface and complete closure is reached under low values of normal stress. However, interface closure is not permanent and water flow recommences. Since the degraded interfaces display fully contracting behaviour, water does not flow as a result of dilation but due to the progressive damage sustained by the concrete surfaces. Without such damage, the increase in transmissivity would be unlikely since the interfaces are closed and still contracting. Finally, the transmissivity of leached interfaces has been compared to that of degraded concrete using Andra's estimate of permeability of leached concrete. It has been estimated that even if the leaching process is known to increase the macroporosity of concrete and thereby its permeability, this is not enough to obtain a flow through the degraded concrete. Thus, water keeps flowing mainly within the interface.

Acknowledgments

The authors would like to thank the laboratory LMC_2 of Electricité de France for their support on the leaching process and Dr. Siham Kamali for her availability for scientific discussion.

References

Adenot F, Gérard B, Torrenti JM (1992) Etat de l'art. In La dégradation des bétons – Couplage fissuration-dégradation chimique. Hermes, pp 19–46

Andra (2005a) Référentiel des matériaux d'un stockage de déchets haute activité et vie longue. Report of Andra n C.RP.ASCM.04.0015

Andra (2005b) Tome evolution phénoménolgique du stockage géologique. Dossier argile. Report of Andra

Andra (2005c) Tome evolution phénoménolgique du stockage géologique. Dossier granite. Report of Andra

Armand G (2000) Contribution la caractérisation en laboratoire et la modélisation constitutive du comportement mécanique des joints rocheux. Ph.D. thesis, University Joseph Fourier, Grenoble, France (in French)

Barton N, Bandhis S, Bakhtar K (1985) Strength, deformation and permeability of rock joints. Int J Rock Mech Min Sci Geomech Abstr 22: 121–140

Bernard O, Ulm FJ, Germaine JT (2003) Volume and deviator creep of calcium-leached cement-based materials. Cement Concrete Res 33: 1127–1136

Boulon M (1995) A 3-d direct shear device for testing the mechanical behaviour and the hydraulic conductivity of rock joints. In: Proc. of the MJFR-2 Conf. Vienna, Austria, pp 407–413

Bourbon X (2005) L'evolution chimique des ouvrages d'un stockage a base de béton: alvéoles de stockage de déchets b, appuis de scellements et de bouchons d'alvéoles, infrastructures des galeries-site de meuse haute -marne. Report of Andra n C.NT.ASCM.03.021

Buzzi O (2004) Hydromécanique du contact entre géomatériaux. Expérimentation et modélisation. Application au stockage de déchets nucléaires. Ph.D. thesis, University Joseph Fourier, Grenoble, France (in French)

Carde C (1996) Caractérisation et modélisation de l'altération des propriétés mécanique due à la lixiviation des matériaux cimentaires. Ph.D. thesis, INSA Toulouse (in French)

Carde C, François R, Torrenti JM (1996) Leaching of both calcium hydroxide and CSH from cement paste: modelling the mechanical behaviour. Cement Concrete Res 26: 1257–1268

Dixon DA, Martino JB, Chandler A, Sugita Y, Vignal B (2002) Water uptake by a clay bulkhead installed n the tunnel sealing experiment at atomic energy of Canadaś underground research laboratory. In: Clays in Natural and Engineered Barriers for Radioactive Waste Confinement. Experiments in Underground Laboratories, Andra, Reims, France

Esaki T, Du S, Mitani Y, Ikusada K, Jing L (1999) Development of shear-flow test apparatus and determination of coupled properties for a single rock joint. Int J Rock Mech Min Sci 36: 641–650

Gentier S (1986) Morphologie et comportement hydromécanique d'une fracture naturelle dans un granite sous contrainte normale. Ph.D. thesis, Université d'Orléans (in French)

Gérard B (1996) Contribution of the mechanical, chemical and transport couplings in the long term behavior of radioactive waste repository structures. Ph.D. thesis, Laval University, Québec, Canada – Ecole Normale Supérieure de Cachan, France

Gérard B, Didry O, Marchand J, Breysse D, Hornain H (1995) Modeling the long term durability of concrete barriers for radioactive waste disposal. In: Scrivener KL, Young FJ (eds) MRS Fall Meeting, in mechanics of chemical degradation of cement based system. E & FN Spon Pubs

Hans J (2002) Etude expérimentale et modélisation numérique multiéchelle du comportement hydromécanique de répliques de joints rocheux. Ph.D. thesis, University Joseph Fourier Grenoble, France

Hans J, Boulon M (2003) A new device for investigating the hydromecanical properties of rock joints. Int J Numer Anal Meth Geomech 27: 513–548

Heukamp FH, Ulm FJ, Germaine JT (2003) Poroplastic properties of calcium-leached cement-based materials. Cement Concrete Res 33: 1155–1173

Jiang Y, Xiao J, Tanabashi Y, Mizokami T (2004) Development of an automated servo-controlled direct shear apparatus applying a constant normal stiffness condition. Int J Rock Mech Min Sci 41: 275–286

Kamali S, Bernard F, Damidot D (2005) Modélisation multi-échelles de la lixiviation des mortiers: effet sur les caractéristiques mécaniques. In: 17me Congrs Franais de Mécanique, Troyes

Kamali S, Gerard B, Moranville M (2003) Modelling the leaching kinetics of cement-based materials-influence of materials and environment. Cement Concrete Res 25: 451–458

Le Bellego C, Gérard B, Pijaudier Cabot G (2000) Chemo mechanical effects in mortar beams subjected to water hydrolysis. J Engng Mech 126: 266–272

Le Bellego C, Pijaudier Cabot G, Gérard B, Molez L (2003) Coupled mechanical and chemical damage in calcium leached cementitious structures. J Engng Mech 29: 453–464

Lee HS, Cho TF (2002) Hydraulic characteristics of rough fractures in linear flow under normal and shear load. Rock Mech Rock Engng 35: 299–318

Luping T, Nilsson LO (1992) A study of the quantitative relationship between permeability and pore size distribution of hardened cement paste. Cement Concrete Res 22: 542–550

Matsuki K, Wang EQ, Sakaguchi K, Okumura K (2001) Time dependent closure of a fracture with rough surfaces under constant normal stress. Int J Rock Mech Min Sci 38: 607–619

Naus DJ, Oland CB, Ellingwood BR, Hookham CJ, Graves HL (1999) Summary and conclusions of a program addressing aging of nuclear power plant concrete structures. Nucl Eng Des 194: 73–76

Ollivier JP, Massat M (1992) Permeability and microstructure of a concrete: a review of modelling. Cement Concrete Res 22: 503–514

Olsson R (1999) Mechanical and hydromechanical behaviour of hard rock joints. Ph.D. thesis, Chalmers University of Technology, Sweden

Olsson WA, Brown SR (1993) Hydromechanical response of a fracture undergoing compression and shear. In: Rock mechanics in the 1990s, Proc. 34th US Rock Mechanics Symp. vol. II, pp 685–688

Saito H, Deguchi A (2000) Leaching tests on different mortars using accelerated electrochemical method. Cement Concrete Res 30: 1815–1825

Tragardh J, Lagerblad B (1998) Leaching of 90-year old concrete mortar in contact with stagnant water. SKB, TR-98-11

Zhao J (1996) Joint surface matching and shear strength. Part A: Joint matching coefficient (JMC). Int J Rock Mech Min Sci 39: 539–553