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Recent Advances in Concrete Technology

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V. M. Malhotra



American Concrete Institute®
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Eighth CANMET/ACI International Conference on Recent Advances in Concrete Technology

Editor
V.M. Malhotra



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PREFACE

The Canadian Centre for Mineral and Energy Technology (CANMET) of Natural Resources Canada, Ottawa, Canada, has played a significant role in Canada for over 40 years in the broad area of concrete technology. In recent years, CANMET has become increasingly involved in research and development dealing with supplementary cementing materials, high-performance normalweight and lightweight concretes, and alkali-aggregate reactions. As part of CANMET's technology transfer program, an international symposium on Advances in Concrete Technology was sponsored jointly with the American Concrete Institute and other organizations, and held in Athens in May 1992. In June 1995, CANMET, in association with the American Concrete Institute and other organizations in Canada and the United States, sponsored the Second CANMET/ACI International Symposium on Advances in Concrete Technology in Las Vegas, Nevada, U.S.A. For the Athens symposium, the CANMET publication "Advances in Concrete Technology" constituted the proceedings of the symposium; for the Las Vegas symposium, the proceedings were published by the American Concrete Institute as ACI SP-154.

In August 1997, CANMET, in association with the American Concrete Institute and other organizations in Canada and New Zealand, sponsored the Third CANMET/ACI International Symposium on Advances in Concrete Technology in Auckland, New Zealand. The symposium brought together representatives from industry, universities, and government agencies to present information on the latest advances and to explore new areas of needed research and development. Thirty-three refereed papers from 15 countries were presented. In addition to the refereed papers, more than 20 other papers were presented and distributed at the symposium. The refereed proceedings were published as ACI SP-171.

In June 1998, CANMET, in association with the American Concrete Institute, Japan Concrete Institute, and several other organizations in Canada and Japan, sponsored the Fourth CANMET/ACI International Conference on Recent Advances in Concrete Technology in Tokushima, Japan. More than 80 papers from 20 countries were received and reviewed in accordance with the publication policies of the American Concrete Institute. Sixty-one refereed papers were accepted for presentation at the conference and for publication as ACI SP-179. In addition to the refereed papers, more than 30

other papers were presented and distributed at the symposium.

In July-August 2001, CANMET, in association with the American Concrete Institute and several organizations in Singapore, sponsored the Fifth CANMET/ACI International Conference on Recent Advances in Concrete Technology in Singapore. Over 100 papers from more than 25 countries were received and reviewed. Forty-six refereed papers were accepted for presentation at the conference and were published as ACI SP-200. In addition to the refereed papers, more than 25 other papers were presented and distributed at the conference.

In June 2003, CANMET, in association with the American Concrete Institute and several organizations in Romania, sponsored the Sixth CANMET/ACI International Conference on Recent Advances in Concrete Technology in Bucharest, Romania. More than 40 papers presented at the conference were distributed “as received,” and no formal ACI special publication was published.

In May 2004, CANMET, in association with the American Concrete Institute and several organizations in the U.S.A., sponsored the Seventh CANMET/ACI International Conference on Recent Advances in Concrete Technology in Las Vegas. Seventeen refereed papers from more than 10 countries were presented and distributed at the conference. The proceedings, consisting of the refereed papers, were published as ACI SP-222. In addition to the refereed papers, 20 other papers were presented and distributed at the conference.

In May 2006, CANMET, in association with the American Concrete Institute and several organizations in Canada and the U.S.A., sponsored the Eighth CANMET/ACI International Conference on Recent Advances in Concrete Technology in Montreal, Canada. The proceedings of the conference consisting of 17 refereed papers, were published as ACI SP-235. In addition to the refereed papers, more than 20 other papers were presented and distributed at the conference.

During the conference, a special symposium was held in honor of retired Professor Marc-André Berubé of Laval University, Quebec City, Canada, for his outstanding and sustained contribution in the broad areas of alkali-aggregate reactivity. The proceedings of this symposium, consisting of more than 20 papers, have been published as a separate publication by CANMET.

Thanks are extended to the members of the CANMET/ACI paper

review panel who met in Budapest, Hungary, in May 2005 to review the papers. Without their prompt review and constructive comments, it would not have been possible to bring out the ACI special publication for distribution at the conference in Montreal. The cooperation of the authors in accepting reviewers' suggestions and in revising their manuscripts accordingly is greatly appreciated. Particular thanks are extended to Dr. Pawan Gupta, G.D. Brearley, and Colleen Mansfield-Joiner for their help in the processing of draft manuscripts.

The help of Messrs. A. Bilodeau (Chair), and B. Fournier and R. Chevrier (Members) of the audio-visual review panel is greatly appreciated.

Rebecca Hartford, Editor, ACI Special Publications, and her staff deserve a very special mention for their assistance in getting this publication ready on time for distribution at the conference.

V.M. Malhotra, P.Eng.

Editor

Chair, Eighth CANMET/ACI International Conference

on Recent Advances in Concrete Technology

May 2006

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Influence of Capillary Porosity on the Transport Properties of Self Compacting Concrete

by K. Audenaert, V. Boel, and G. De Schutter

Synopsis: Self compacting concrete (SCC) is a new kind of concrete that combines a high flowability and a high segregation resistance obtained by a large amount of fine particles or the presence of a viscosity modifying agent and the use of superplasticizers. As self compacting concrete does not need external compaction, the pore structure, and more specifically the amount of capillary pores, is not influenced by the compaction method. These capillary pores play a very important role in the transport of water and gases in concrete and are of major importance for the understanding of degradation mechanisms.

In order to verify the correlation between the transport properties and the capillary pores, tests were carried out. Water and gas permeability, capillary absorption, carbonation and chloride penetration tests were performed on 11 self compacting concrete mixtures and 1 traditional concrete mixture. The selection of the mixtures is made in order to consider some important parameters like the cement/powder and water/cement ratio, the amount of water, the amount of powder and the type of filler (limestone filler with two different grading curves).

The amount of capillary pores was calculated by the method of Powers. The calculated values were compared with the test results and gave very good correlations.

Keywords: capillary absorption; capillary pores; carbonation; chloride penetration; gas permeability; water permeability

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G. De Schutter is professor at the Magnel Laboratory for Concrete Research, Ghent University Belgium. His main research interests are early age concrete, massive concrete, durability of concrete structures and self-compacting concrete.

INTRODUCTION

Self compacting concrete (SCC) is a recently developed type of concrete that combines a high flowability and a high segregation resistance obtained by a large amount of fine particles or the presence of a viscosity modifying agent and the use of superplasticizers. In this research, no viscosity modifying agents were used. As the conception of SCC is totally different from traditional concrete (TC) and no external compaction is needed, the pore structure, and more specifically the amount of capillary pores, will be influenced. These capillary pores play a very important role in the transport of water and gases in concrete and are of major importance for the understanding of degradation mechanisms.

In order to verify the correlation between the transport properties and the capillary pores, tests were carried out. In this project, water and gas permeability, capillary absorption, carbonation and chloride penetration tests were performed on 11 self compacting concrete mixtures and 1 traditional concrete mixture. The selection of the mixtures is made in order to consider some important parameters like the cement/powder and water/cement, the amount of water, the amount of powder and the type of filler (limestone filler with two different grading curves).

The amount of capillary pores was calculated by the method of Powers. The calculated values were compared with the test results and gave good correlations.

EXPERIMENTAL PROGRAM

Concrete composition

At the Magnel Laboratory for Concrete Research, 11 self compacting concrete mixtures (SCC) and 1 traditional concrete mixture (TC) were investigated. In the first 5 mixtures a constant amount of powder (cement and filler) is considered: 600 kg/m^3 , as well as a constant amount of water, sand and gravel, respectively 165 kg/m^3 , 853 kg/m^3 and 698 kg/m^3 . In the first four mixtures, the cement/powder is varied. In the next mixture, a limestone filler with a finer grading is used (Figure 1). In the next three mixtures, the amount of powder is varied (500 kg/m^3 , 700 kg/m^3 and 800 kg/m^3). In the following three mixtures, the

amount of water is varied (144 kg/m³, 198 kg/m³ and 216 kg/m³). In the traditional concrete mixture, the same water/cement and amount of cement is considered as for SCC1. The cement used in this research is CEM I 42.5 R, which is a portland cement with a C₃S, C₂S, C₃A and C₄AF content of respectively 58.2, 12.7, 8.2 and 9.1%.

The amount of superplasticizer, which is a modified polycarboxylether, was determined in order to obtain a suitable flowability without segregation. The flowing time in the V-funnel was also measured (values between 5s and 10s), air content (values between 1% and 3%) and the U-box requiring self levelling. In Table 1, the mixture composition is given together with the compressive strength at 28 days measured on concrete cubes with a side of 150 mm.

Test method

Water permeability - For the determination of the water permeability, Darcy's law for laminar flow through porous materials is used:

$$\frac{dq}{dt} \frac{1}{A} = \frac{K' \rho g h}{\eta L}$$

with dq/dt the flow velocity of the fluid [m³/s], A the cross section of the test specimen [m²], h the pressure head over the specimen [m], L thickness of the specimen [m], η the dynamic viscosity of the fluid [Ns/m²], ρ the mass density of the fluid [kg/m³] and g the gravity constant m/s². K' is the intrinsic permeability coefficient in m², which is a material constant independent of the fluid. In the tests described in this article, only water is used which leads to the formula of the water permeability coefficient K [m/s]:

$$K = \frac{K' \rho g}{\eta}$$

When water is used, ρ = 1000 kg/m³ and η = 1 · 10⁻³ Pa.s at 20 °C. When the pressure head is not constant in time, the formula for K becomes:

$$K = \frac{L}{t_2 - t_1} \ln \left(\frac{h_2}{h_1} \right)$$

with h₁ and h₂ the pressure head at time t₁ and t₂ respectively. When the steady state flow is reached, K could be directly determined with this formula.

At the Magnel Laboratory for Concrete Research a new testing device was developed in order to determine the water permeability of concrete and is based on test methods found in literature for low pressure heads. Detailed information is given in ⁽¹⁾. The test specimen with a diameter of 80 mm and a height of 25 mm is sawn from the middle of a concrete cube 150 mm and is glued in a PVC tube with epoxy. The specimens are tested at a concrete age between 55 and 666 days, due to the long duration of the test and the limited amount of testing devices. Until the testing date, they are stored in a climate room at 20 °C ± 2 °C and at least 90% R.H.. Before the start of the test, the test specimens are vacuum saturated following the Belgian standard NBN B05-202. The pressure head is approximately 500 mm. For each concrete mixture, at least 3 specimens are tested.

Gas permeability - The gas permeability is experimentally determined by means of the Cembureau-method according to ⁽²⁾. For each mixture, one prism 400 x 400 x 100 mm³ is made and stored in a climate room at 20 °C ± 2 °C and more than 90% R.H.. At the age of 28 days, three cores of 150 mm diameter were drilled for each concrete mixture and

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from the center of these cores, samples with 50 mm height were taken. Until the beginning of the test, at the age of three months, the samples were stored in a climate room at $20\text{ }^{\circ}\text{C} \pm 2\text{ }^{\circ}\text{C}$ and more than 90% R.H. In order to avoid a chemical reaction between the gas and the concrete, oxygen is used. The saturation degree of the samples is of major importance. The test is performed at three different levels of inlet pressure, namely 3, 4, and 5 bar. In this article, only test results performed on dry specimens (dried at $105\text{ }^{\circ}\text{C}$ for 1 week) and at an inlet pressure of 3 bar are discussed.

The apparent gas permeability $K\text{ [m}^2\text{]}$ can be calculated based on the Hagen-Poiseuille relationship for laminar flow of a compressible fluid through a porous body with small capillaries under steady-state conditions ⁽³⁾:

$$K = \frac{4,04P_2QL \cdot 10^{-16}}{A(P_1^2 - P_2^2)}$$

with Q the volume flow rate of the fluid measured during the test with a bubble flow meter [ml/s], L the thickness of the specimen in the direction of the flow [m], A the cross-sectional area of the specimen [m^2] and P_1 and P_2 the inlet and outlet pressure of oxygen [bar]. This formula is valid for tests performed at $20\text{ }^{\circ}\text{C}$ for which the dynamic viscosity of oxygen is $2,02 \cdot 10^{-5}\text{ Nsm}^{-2}$.

Capillary suction - From the mixtures described above, cubes with a side of 150 mm were manufactured. These concrete cubes were stored in a climate room at $20\text{ }^{\circ}\text{C} \pm 2\text{ }^{\circ}\text{C}$ and more than 90% R.H.. At the age of 28 days, these concrete cubes were stored for a minimum of 2 and a maximum of 3 weeks (the tests for capillary suction were started on monday) at $40\text{ }^{\circ}\text{C} \pm 5\text{ }^{\circ}\text{C}$ and $40 \pm 3\%$ R.H. Then, the cubes were placed on stable supports in water, so that the water level is $5 \pm 1\text{ mm}$ above the lower face of the specimen. Before the test and after 3, 6, 24, 72, and 168 hours the mass of the specimen is determined.

The test was performed in an environment at $20\text{ }^{\circ}\text{C} \pm 3\text{ }^{\circ}\text{C}$ and $60 \pm 3\%$ R.H. These tests were carried out on three specimens for each concrete mixture. First, the water will penetrate the concrete at the surface. This surface zone is more porous than the concrete core. Therefore, the mass increase will be strong in the beginning. After a certain time, as the water penetrates the core, the mass increase will proceed more slowly. To model this behavior, two capillary absorption coefficients are determined by linear regression: W based on the measurements during the first 24 hours and S based on the measurements at 72 and 168 hours. In this article only the values of S are discussed.

Carbonation - From the mixtures described above, 100 mm cubes were made. These concrete cubes were stored in a climate room at $20\text{ }^{\circ}\text{C} \pm 2\text{ }^{\circ}\text{C}$ and more than 90% R.H. At the age of 28 days, an epoxy coating was applied to all faces, except on the face that will be exposed to CO_2 . The concrete cubes were stored permanently in a carbonation room at $20\text{ }^{\circ}\text{C}$, 60% R.H. and 10 vol.% CO_2 . At regular times (8, 12, 16, 20, and 24 weeks) the carbonation depth was examined experimentally. At each time, a slice with a thickness of 1 cm was sawn from each specimen. This slice was sprayed with a phenolphthalein solution in order to determine the carbonated zone. This pH indicator changes from colourless to purple as the pH rises from 8.3 to 10.0. The test indicates the depth to which the calciumhydroxide has been depleted. After sawing the slices, the remaining concrete specimen was covered again with the carbon resisting epoxy coating and the carbonation was continued.

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The carbonation of concrete can be considered as a Fickian problem and is governed by the following equation:

$$x = A\sqrt{t}$$

in which x is the carbonation depth, t is the exposure time and A is a constant depending on the diffusion resistance of the material.

After sawing a slice of the concrete cube, the carbonation depth is determined at 10 points and the mean value is calculated. For each composition, two concrete cubes are made in order to obtain a mean carbonation depth. With the carbonation depths at several times, and with the above mentioned equation, a regression value for A can be determined.

Chloride penetration – 150 mm cubes were manufactured and stored in a climate room at $20\text{ }^{\circ}\text{C} \pm 2\text{ }^{\circ}\text{C}$ and more than 90% R.H. in order to determine the chloride penetration. At an age of 21 days, three cores with a diameter of 100 mm and a height of 50 mm were drilled from each cube. Afterwards the concrete cores, were placed back in the climate room until the age of 28 days. On these cores, a non-steady state migration test was performed following the method of Tang et al.⁽⁴⁾. Firstly, the specimens are vacuum saturated with a saturated $\text{Ca}(\text{OH})_2$ solution. Afterwards an external electrical potential (for the tests described in this paper between 25V and 40V) is applied across the specimen for 24 hours and forces the chloride ions to migrate into the specimens. Afterwards the specimen is axially split and a silver nitrate solution is sprayed on to the freshly split sections. The chloride penetration depth can then be measured on each section at 7 points from the visible white silver chloride precipitation, after which the chloride migration coefficient can be calculated from these penetration depths with:

$$D = \frac{RT}{zFE} \frac{x - \alpha\sqrt{x}}{t}$$

with: $E = \frac{U - 2}{L}$ and $\alpha = 2\sqrt{\frac{RT}{zFE}} \operatorname{erf}^{-1}\left(1 - \frac{2c_d}{c_o}\right)$

and D the non steady state migration coefficient (m^2/s), z the absolute value of ion valence, for chloride z equals 1, F the Faraday constant: $9.648 \times 10^4 \text{ J}/(\text{V} \cdot \text{mol})$, U the absolute value of the applied voltage (V), R the gas constant: $R = 8.314 \text{ J}/(\text{K} \cdot \text{mol})$, T the average value of the initial and final temperatures in the anolyte solution (K), L the thickness of the specimen (m), x the average value of the penetration depths (m), t the test duration (s) and erf^{-1} the inverse of the error function. c_d and c_o are respectively the chloride concentration at which the color changes, $c_d = 0.07 \text{ N}$ for portland cement concrete and the chloride concentration in the catholyte solution, $c_o = 2\text{N}$.

Test results

From the test results, the following could be concluded (with SCC1 as reference):

Effect of cement/powder with the same amount of water (Figure 2) - If the amount of cement and powder is changed, but the used amount of water and powder is maintained constant, a lower W/C and a higher C/P are leading to lower transport properties (SCC2, SCC1, SCC3, SCC4). Due to lower transport properties, the carbonation and chloride penetration will decrease. If a higher W/C is used, more water is present to hydrate the cement, but there is also more water that remains unused in the concrete matrix. A higher

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amount of remaining water, leads to a higher amount of pores in the internal concrete structure, which results in higher transport coefficients.

Effect of type of filler (Figure 3) - Replacing limestone filler P2 by limestone filler S (SCC5) with a finer grading curve, does not change significantly the transport properties and durability behavior.

Traditional concrete – self compacting concrete (Figure 3) - The difference between SCC and TC with the same amount and type of cement and the same W/C concerning transport properties is very small except for the gas permeability and carbonation. The high gas permeability of the traditional concrete can be explained based on the differences in pore structure and the transition of laminar to turbulent flow. This is explained in Reference 5. The lower carbonation of traditional concrete, even if the concrete strength is lower than for self compacting concrete, will be studied in further research.

Effect of the amount of powder (Figure 4) - If the W/C and the C/P are maintained constant, an increase in the amount of powder includes an increase in the amount of water. In these mixtures (SCC6, SCC1, SCC7 and SCC8), a higher amount of water than needed for the hydration of the cement (for portland cement, the W/C needed for complete hydration is approximately 0.42) is used. An increase in the amount of water with constant W/C leads to a higher amount of water that is not needed for the hydration and is resulting in more pores. Moreover, an increase in the amount of powder leads to an increase in the amount of paste for the same amount of concrete. Due to the presence of the pores in the paste, more paste leads to more pores for the same amount of concrete. Therefore, the mixture with the highest amount of powder will also have the highest amount of pores, resulting in the highest water transport. This is corresponding with the test results. The carbonation and chloride penetration are also increasing with an increasing amount of powder.

Variation of the amount of water (Figure 5) - If the amount of water increases, the W/C increases, leading to more pores and an increase of the transport properties and carbonation and chloride penetration (SCC9, SCC1, SCC10, and SCC11).

DISCUSSION

In literature, it is written that the transport properties of concrete are mainly determined by capillary pores (6,7). Therefore, the model of Powers is used to determine the capillary porosity to verify the possible link between the capillary pores and the transport properties.

V_{cap} = capillary pores + free water = capillary pores + water – gel water – bounded water

$$\begin{aligned}
 &= 0.185 \frac{Ch}{\rho_c} + \frac{W}{\rho_w} - \frac{0.28}{0.72} \left(\frac{Ch}{\rho_c} (1 - 0.185) + \frac{0.23Ch}{\rho_w} \right) - \frac{0.23Ch}{\rho_w} \\
 &= -0.1319 \frac{Ch}{\rho_c} + \frac{1}{\rho_w} (W - 0.3194 Ch)
 \end{aligned}$$

with V_{cap} the volume of capillary pores [m^3], C the amount of cement [kg], W the amount of water [kg], h the degree of hydration [-], ρ_c and ρ_w the mass density of cement and water [kg/m^3], respectively.

$$V_{concrete} = V_{water} + V_{cement} + V_{coarse\ aggregate} + V_{sand} + V_{filler}$$

$$= \frac{W}{\rho_w} + \frac{C}{\rho_c} + \frac{A + S + F}{\rho_{agg}}$$

$$capillary\ porosity = \frac{V_{cap}}{V_{concrete}}$$

with $V_{concrete}$ the volume [m^3], A the amount of coarse aggregate [kg], S the amount of sand [kg], F the amount of filler [kg] and ρ_{agg} the mass density of aggregate [kg/m^3].

The parameters W, C, A, S and F are known from the mixture proportions. For the mass densities, a value of $1000\ kg/m^3$ is used for water, $2625\ kg/m^3$ for the aggregates, sand and filler and $3115\ kg/m^3$ for portland cement.

The test specimens are stored until the testing age in a climate room at $20\ ^\circ C \pm 2\ ^\circ C$ and at least 90% R.H. for at least 28 days. This means that the degree of hydration will not strongly differ from the ultimate degree of hydration that could be determined, by the Mill formula (8):

$$h_{ultim} = \frac{1.031\ W/C}{0.194 + W/C}$$

The calculated values for the capillary porosity are given in Table 1. The correlation between water permeability (Figure 6), gas permeability (Figure 7) and capillary absorption (Figure 8) and capillary porosity is good (R^2 is respectively 0.92, 0.81 and 0.91). For carbonation (Figure 9) and chloride penetration (Figure 10), the correlation is not as good as the correlation for the transport properties (R^2 is 0.79 and 0.71 respectively). This leads to the conclusion that not only transport properties determine the durability behavior. This could be explained by the chemical reactions taking place during carbonation and chloride penetration.

CONCLUSIONS

In this article, the water permeability, gas permeability, capillary absorption, carbonation and chloride penetration of 11 self compacting concrete mixtures and of 1 traditional concrete mixture is discussed. The selection of the mixtures is made in order to consider some important parameters like type of filler (limestone filler with two different grading curves), the cement/powder and water/cement, the amount of water and the amount of powder (cement and filler). The results of these tests are discussed and compared with the capillary porosity calculated by the method of Powers et al. The following conclusions were made:

- Using a filler with a finer grading has no influence on the transport properties and durability behavior.
- If the W/C and the C/P are maintained constant, an increase in the amount of powder leads to an increase in transport properties, carbonation and chloride penetration.

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- The transport of water and gas is strongly influenced by the water/cement and cement/powder. Decreasing one of these ratios or both is leading to a slower transport.
- The difference concerning transport properties between SCC and TC with the same amount and type of cement and the same W/C is small except for the gas permeability.
- The transport properties could be modelled based on the capillary porosity. For carbonation and chloride penetration, other influencing factors are also important.

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Table 1-- Mixture Composition

	CEM 142.5 R [kg/m ³]	limestone filler S [kg/m ³]	limestone filler P2 [kg/m ³]	water [kg/m ³]	sand 0/5 [kg/m ³]	gravel 4/14 [kg/m ³]	superplasticizer [l/m ³]	W/C [-]	C/P [-]	compr. strength [MPa]	Capillary porosity [%]
SCC1	360		240	165	853	698	2.3	0.46	0.60	57.3	7.33
SCC2	300		300	165	853	698	2.2	0.55	0.50	46.5	8.51
SCC3	400		200	165	853	698	2.9	0.41	0.67	64.2	6.61
SCC4	450		150	165	853	698	3.0	0.37	0.75	68.7	5.76
SCC5	360	240		165	853	698	2.8	0.46	0.60	56.9	7.33
SCC6	300		200	137	923	755	3.4	0.46	0.60	60.1	6.16
SCC7	400		300	192	782	640	2.6	0.48	0.57	55.9	8.78
SCC8	450		350	220	712	583	2.7	0.49	0.56	50.9	10.07
SCC9	360		240	144	865	707	3.6	0.40	0.60	68.7	5.63
SCC10	360		240	198	835	683	1.8	0.55	0.60	46.6	10.03
SCC11	360		240	216	825	675	2.0	0.60	0.60	40.3	11.51
TC1	360			165	640	1225		0.46	1.00	48.6	7.11

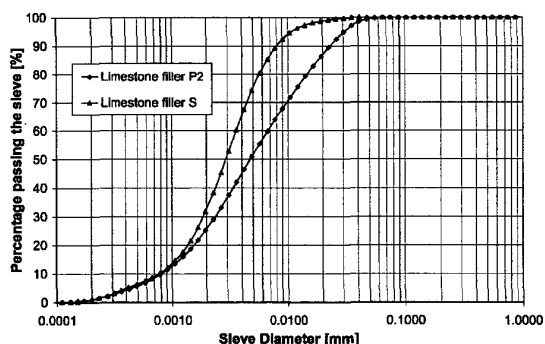


Fig. 1 -- Particle size distribution of limestone fillers

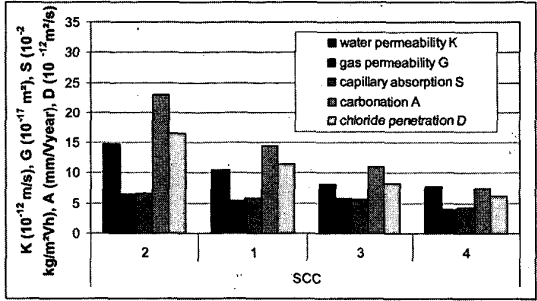


Fig. 2 -- Effect of cement/powder on the properties of SCC

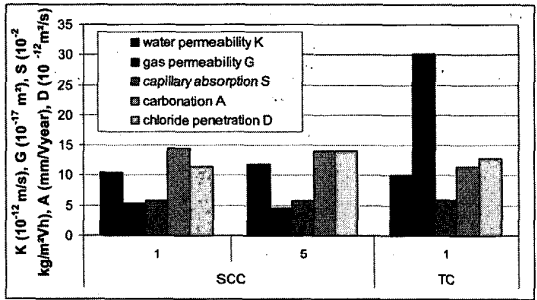


Fig. 3 -- Effect of type of filler on the properties of SCC – comparison SCC-TC

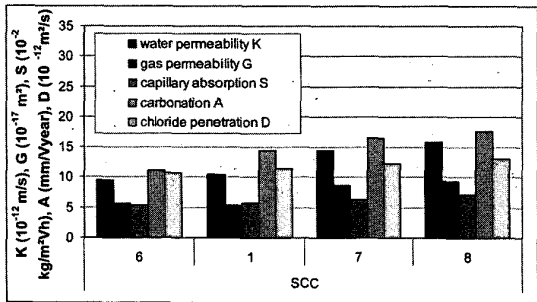


Fig. 4 -- Effect of the amount of powder on the properties of SCC

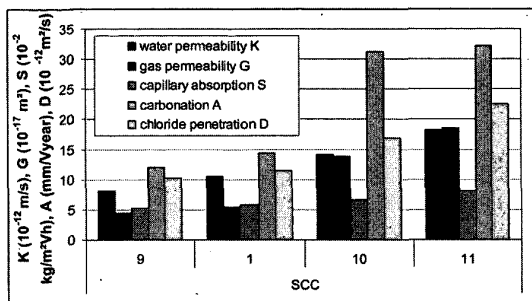


Fig. 5 -- Effect of the amount of water on the properties of SCC

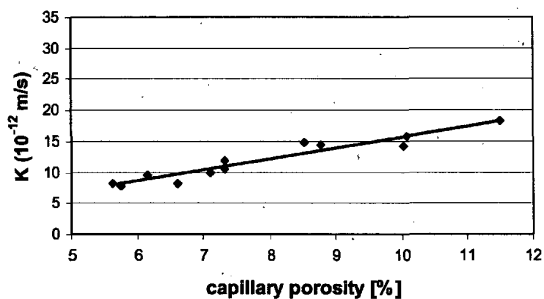


Fig. 6 -- Water permeability versus calculated capillary porosity

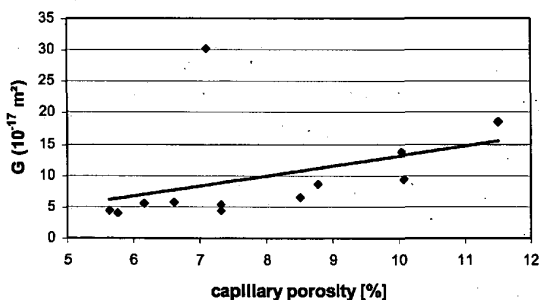


Fig. 7 -- Gas permeability versus calculated capillary porosity

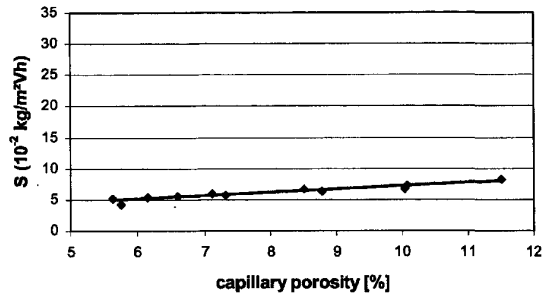


Fig. 8 --Capillary suction versus calculated capillary porosity

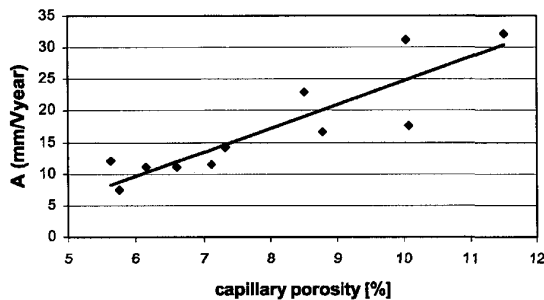


Fig. 9 -- Carbonation versus calculated capillary porosity

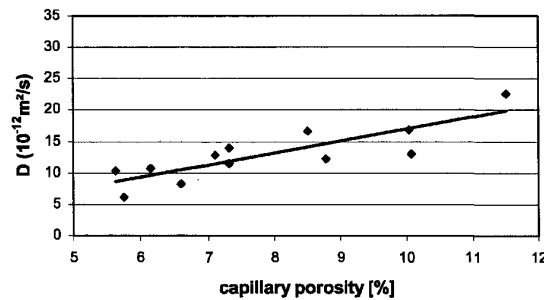


Fig. 10 -- Chloride penetration versus calculated capillary porosity

Development of Shrinkage Reduced Self Compacting Concrete

by **A.B. Eberhardt and J. Kaufmann**

Synopsis: It is well known that, in comparison with normal (usual) concrete, self compacting concrete (SCC) shows higher shrinkage because of the higher volume of binder, inevitable for achieving high fluidity and a good cohesiveness of the fresh concrete. It is necessary to avoid crack formation caused by drying or autogenous shrinkage as cracks could serve as flow paths and ingress zones for gases and salts or favour leaching.

In order to diminish this negative effect of SCC's, mixture concepts for the formulation of a shrinkage reduced SCC were developed. Different factors influencing the shrinkage behaviour are discussed. The effect of replacing cement by high volume of pozzolanic or inert additives as well as the effect of different shrinkage reducing admixtures (shrinkage reducing agents - SRA) was examined. Furthermore an optimization of the grain size distribution was evaluated.

The measuring setup consisted of different methods, most important the measurement of free and restrained shrinkage (ring test) on mortar and concrete specimens. It could be shown, that through a significant reduction of the cement amount, accompanied by an effective shrinkage reducing admixture a massive shrinkage reduction of SCC's in the range of 50% to 60% can be achieved.

Keywords: drying shrinkage; free shrinkage; restrained shrinkage; ring test; self-compacting concrete; shrinkage reducing admixture

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INTRODUCTION

Self compacting concrete (SCC) is mainly characterised by its special rheological properties. Without any vibration effort this concrete can be placed and compacted under its own weight. Thereby SCC has to possess good cohesiveness to avoid segregation of aggregates or bleeding of cement paste. Whereas SCC differs in its fresh concrete attributes from normal vibrated concrete it is desired to have similar properties as normal concrete in the hardened state. In comparison to normal concrete some modifications in mixture design are necessary to achieve the required rheological properties of SCC. Besides the addition of superplasticizers (SP) and viscosity enhancing agents a higher volume of paste ensures the discrete distribution of aggregate within the paste matrix and avoids internal sliding friction. Because of the higher volume of paste SCC shows higher shrinkage, some researches revealed shrinkage strains in the range of 0.5 to 0.8 mm/m^[1-3]. Owing to strength requirement, SCC is often fabricated at low W/C and hence shrinkage is increased^[4-6]. Cracking caused by drying or autogenous shrinkage is an important factor for durability as cracks serve as flow paths and ingress zones for gases and salts or might permit leaching.

For this reason SCC may be discarded in many practical applications. Investigations on normal and high performance concrete revealed three main concepts for shrinkage and crack prevention. In the recent years new shrinkage reducing admixtures (SRA) were introduced to counteract drying shrinkage.

According to some reports, these SRA furthermore can reduce plastic and autogenous shrinkage. SRA lower the surface tension of water in capillary pores. During drying of concrete, the water filled capillary pores in the range of 2.5-50 nm loose moisture initiating the formation of curved menisci. The surface tension of water pulls together the walls of the pores. With the lower surface tension of water the resulting force is reduced which leads to reduced shrinkage^[7-9]. Furthermore SRA is able to reduce crack risk and crack width of concrete under restrained conditions^[10].

Another concept to reduce shrinkage is to lower paste volume by optimizing packing density of aggregates. A third concept is to avoid large amounts of cement in the binder system causing shrinkage and cracking, by replacing a part of the cement by less reactive or inert additives such as fly ash or limestone.

In our study we followed these main concepts, applying them on SCC without changing the rheological properties. The measuring setup consisted of different methods, most important the measurement of free and restrained shrinkage.

Binder

In this study portland cement CEM I 42,5 N according to European Standard EN-197-1 was used. Furthermore limestone filler and fly ash replaced cement as a part of the binder system. These pozzolanic or inert additives were chosen so that their particle size distribution was close to that of the cement. Under this condition and at constant volume ratio of solids/liquids in the mortar suspensions it was supposed to achieve similar flow properties. Figure 1 shows the different particle size distributions of the cement and the several additives. Densities of used materials are given in Table 1.

Aggregates

It is well known that with different grain size distributions one will need different volumes of cement paste to achieve self compacting property of mortar or concrete. To obtain different grading curves five sieve fractions of aggregate were added together for mortar (0-4 mm) and seven fractions for concrete (0-16 mm).

Admixtures

To adjust flow properties of mortar and concrete a superplasticizer (polycarboxylate type; solid content 25 M.-%) was added at a maximum dosage 2 wt.-% of cement. In our investigations, we used two liquid shrinkage reducing agents (SRA) from two different producers. The main active ingredients in both cases are dipropylene glycol, dipropylene glycol di-tert-butyl ether and propanol [2-(1,1-dimethylethoxy)-methylethoxy]. For SRA 1 the producer's guideline outlines dosages in the range of 1% wt. to 5 wt.-% of mixing water, for SRA 2 a maximum dosage of 2 wt.-% of cement.

METHODS

Composition of binders and Mixture proportions for mortar

The flow properties of self compacting mortars (SCM) were adjusted using the slump test and V-funnel test for mortars ^[1]. Criteria for self compacting property are shown in Fig. 2.

The influence of paste volume -- was investigated on mortars with different grading curves (M1; M2; Fig. 3.), M1 having a higher amount of fine sand. To achieve self compacting property for mortars with grading M1 it was necessary to adjust a paste content from 53 vol.-% to 55 vol.-%. For the coarser grading curve M2 self compacting property was reached with a paste content of 46 vol.-%. In this test series, the content of SP (1.4 wt.-% of cement content) as well as the ratio of solid/liquid (respectively W/C) were kept constant. Table 2 outlines the mixture proportion, rheological properties and denomination of the mortars.

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The influence of cement replacement -- was observed on mortars with the coarser grading curve M2. When replacing cement by additives like fly ash or limestone it is necessary to keep the ratio of solids/liquids in the paste constant: Therefore the replacement was done on a volumetric basis. As the particle size distributions of cement and additives were similar, it was possible to achieve self compacting property for all mortars with a constant solid/liquid ratio. All mortars consist of a total volume of 23 vol.-% liquid phase and 77 vol.-% total volume of solid phase. Except the mortar with 100 vol.-% cement as binder (C100) with a SP-content of 1.4 wt.-% of cement, the mortars contain 2 wt.-% SP. Composition, rheological properties and denomination of mortars for investigation of the influence of replacement cement are outlined in Table 3.

The influence of different shrinkage reducing agents (SRA) -- was investigated on mortars containing different dosages of the two different SRA's. The range of dosages was defined according to the producer's guideline. By adding the different amounts of SRA (density~1g/cm³) the water content was reduced by an equivalent amount. It was possible to keep the ratio of solids/liquids in the paste constant. Therefore all mortars showed rheological properties in the range of SCM. Composition, rheological properties and denomination of mortars used for study of shrinkage reducing agents (SRA) are outlined in Table 3.

Composition of binders and Mixture proportions for concrete

The flow properties of self compacting concretes (SCC) were adjusted using the slump test and L-box test for concrete ^[12, 13]. Criteria for self compacting property are shown in Fig. 2.

Self compacting concretes were developed according to the main boundary condition defined in Fig. 2. The solid/liquid ratio was kept constant.

Three different grading curves (C1, C2, C3; Fig. 3.) were used in order to study the influence of paste volume. Table 4 outlines the different paste volumes necessary for adjusting self compacting property as well as composition of binders and denomination.

Isothermal heat conduction calorimetry

The heat release during cement hydration was measured on pastes by conductivity calorimetry (conduction calorimeter - TAM Air; Thermometrics). The samples were equilibrated to a reference temperature of 20 °C. Measurement on pastes was performed with W/C = 0,4. The W/C value was chosen to ensure a complete hydration and to avoid large amounts of water that may change the composition of pore solution, hydration kinetics etc. To produce a homogeneous paste the samples were stirred while injecting liquid (mixing water including admixture) into the reaction vessel.

Free shrinkage

Measurement of free drying shrinkage was on mortar specimen 40x40x160 mm³ and 120x120x360 mm³ for concrete. 24 hours after casting the specimens were demolded. Then two plug gauges were attached on two opposite length sites of the specimen. The reference length was 100 mm for mortar specimen and 250 mm for concrete.



Resolution of the used gauges was for both mortar and concrete 0.1 μm . Until 7 days samples were cured at 20 °C and 95% relative humidity (Storage A) and then stored at 20 °C and 70% relative humidity (Storage B).

Restrained shrinkage

The restrained shrinkage setup is shown in Fig. 4. The sample geometry is adapted from [10]. Degree of restraining is more than 75% up to 3N/mm² mortar tensile stress. Samples were cured in two different ways on the one hand for 2 days at 20 °C/95% relative humidity (Storage A) and on the other hand for 7 days at Storage A, afterwards stored at 20 °C and either 70% RH (Storage B) or 35% RH (Storage C). One day after casting the specimen were demolded and supplied to the described curing respectively storage program.

The water loss was measured by weight. Both the time and width of the first crack were observed.

RESULTS AND DISCUSSION

Isothermal heat conduction calorimetry

Conductivity calorimetry revealed that cement hydration is retarded by both shrinkage reducing agents (SRA) and superplasticizer (SP) (Fig. 5). Retardation increases with increasing content of chemical admixtures.

In the range of added dosages of SRA (1 to 3 wt.-% of cement), the start of acceleration period as well as the main peak of heat release are displaced, up to half an hour for the acceleration period and from 1 hour up to 3,5 hours for the main peak of heat release, increasing with increasing amount of added SRA. The added amount of SP is related to the dosage used in our mortar and concrete observations and shows a strong retarding influence. The retarding effect is about 10 hours for the start of the acceleration period and about 17 hours for the main peak of heat release. For pastes with the maximum dosages of SRA and SP the beginning of acceleration period is retarded up to 16 hours and the main heat release is delayed for about 26 hours. Figure 6 displays the total heat release up to 190 hours of hydration. The degree of hydration of the paste with the maximum dosage of SRA (3wt.-% of cement content) is hardly affected compared to the reference containing no SRA (after 190 hours). A strong retarding effect is observed between 6 and 36 hours within the paste containing the admixture combination (SP+SRA) at maximum dosage, decreasing the degree of hydration to 85% after 190 hours compared to the one without additives. At this time of hydration the heat release curves still show different behavior and seem to merge at a later term. The retardation of hydration also has implications on the measurement of shrinkage. Attaching the plug gauges 24 hours after casting, the calorimetry result shows that different ratios autogenous shrinkage / drying shrinkage might influence the values for total shrinkage. This is relevant in case of specimen containing different amounts of both admixtures, SP and SRA.

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Free shrinkage of mortar

Influence of paste volume -- The first part of the investigation was on series of mortars with grading curve M1 and different paste volumes varying from 49 vol.-% up to 55 vol.-% (P49_M1,..., P55_M1; Fig. 7). The measured total shrinkage increases with increasing paste volume. At the end of the observation period (180 d) the self compacting mortars P53_M1 and P55_M1 showed a total shrinkage of 0,92 mm/m and 0,98 mm/m, respectively. SCM P46_M2 with the coarser distribution of aggregates M2 required a paste content of 46 vol.-% because of the better packing density. The shrinkage strain after 180 days was 0,76 mm/m.

To quantify the shrinkage reducing effect of an improved particle size distribution of aggregates the mortars with equivalent flowability (P53_M1, P55_M1 vs. P46_M2; Fig. 7) are compared. A shrinkage reduction in the range of 17% up to 22% is observed.

Influence of cement replacement -- The shrinkage reducing effect was evaluated by comparing self compacting mortars containing pure cement in binder (reference C100) to blended cement mortars. 50 vol.-% of cement were replaced through 50 vol.-% fly ash (mortar F50), 25 vol.-% fly ash plus 25 vol.-% limestone (mortar F25_L25) or 50 vol.-% limestone (mortar L50). Results of total free drying shrinkage are plotted in Fig. 8. In comparison to the reference mortar C100, the highest shrinkage reduction was obtained for the mortar L50 (limestone) with a reduction of 28% after 300 days followed by F25_L25 with 25% and F50 with 13% after 300 days, outlined in Table 5.

Influence of Shrinkage reducing agent (SRA) -- The influence of SRA on free drying shrinkage was tested on modifications of mortar L50 (reference mortar). Following the producer's guideline different amounts of SRA1 were added. 1.5 wt.-% (min. SRA1). 2.5 wt.-% (med. SRA1) and a maximum dosage of 5 wt.-% (max. SRA1) of mixing water were replaced. Results of total free drying shrinkage are displayed in Fig. 8. The effect of shrinkage reduction increases with increasing amount of added SRA and was in the range of 9% (min SRA1), 14% (med. SRA1) and 24% (max. SRA1) after 300 days (Table 5). The effect of shrinkage reduction is time dependent and decreases with progressing storage time. To evaluate the effectiveness of different shrinkage reducing agents the reference mortar L50 was additionally modified with a second agent SRA2 in the valid range of dosages (producer's guideline). Because of the different dosaging procedures (SRA1 refers to water content of paste; SRA2 refers to cement content) the SRA dosages outlined in Fig. 9. were transferred and correspond now to the cement content of the series L50_X.

For both agents the shrinkage reduction increases with increasing content of admixture. The effect of reduction decreases with time (Table 5). There is only a small difference in shrinkage for comparable dosages of SRA1 and SRA2.

Free shrinkage of concrete

Influence of cement replacement -- The influence of replacing cement on free drying shrinkage was evaluated by comparing the concrete C3_L50 (grading curve C3; binder 50 vol.-% limestone) to the concrete containing 100 vol.-% cement in the binder (reference C3 C100). Results of total free drying shrinkage are displayed in Fig. 10.

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In comparison to the reference concrete C3_C100, the shrinkage reduction for concrete C3_L50 was 40% after 28 days decreasing within time to 31% after 180 days (outlined in Table 6.).

Influence of paste volume -- The influence of paste volume on free drying shrinkage was studied by applying three different grading curves (C1, C2, C3; Fig. 3.) on the binder system L50 and by adding the shrinkage reducing agent SRA1 at a dosage of 3 wt.-% of cement content.

Under the condition of self compacting property the differences in paste volume vary from 32.9 vol.-% (C1_L50; C1_L50_SRA1) to 38.3 vol.-% (C3_L50; C3_L50_SRA1). The mixture proportions of the concretes are outlined in Table 4. In Fig. 10 the results of total free drying shrinkage are plotted.

With increasing paste volume the measured total shrinkage increases. The shrinkage reduction decreases with longer storage time (outlined in Table 6). The potential of shrinkage reduction through an improved grading curve in the observation period of 180 days was in the range of 12% up to 29 %.

Influence of Shrinkage reducing agent (SRA) -- The influence of SRA on free drying shrinkage was studied by adding SRA1 up to the maximum dosage (5 wt.-% of mixing water) to concretes with the binder system L50 and three different grading curves (reference concretes C1_L50; C2_L50; C3_L50). Results of total free drying shrinkage are given in Fig. 10 and Table 5. The achieved shrinkage reduction through shrinkage reducers is in the range of 9% (min. dosage) to 41% (max. dosage) after 180 days.

With increasing amount of SRA, the shrinkage deformation is decreasing. The reduction is decreasing with time. These results are similar to those obtained on mortars.

Furthermore, investigations revealed an increasing shrinkage reduction potential of SRA with increasing volume of paste. However, the shrinkage reduction decreases with time (Table 5). The lowest total shrinkage deformation of 0.18 mm/m (after 180 days) was reached by applying the dense packing of aggregates (grading curve C1).

Putting together the three different concepts, the shrinkage reduction of self compacting concrete was in the range of 65% after 180 days.

Restrained shrinkage on mortar

Restrained shrinkage was studied on mortars with aggregate size distribution M2 and the binder systems C100 and L50. Additionally a maximum dosage of SRA1 of 5wt.-% of mixing water (replaced) was used. The mixture compositions are given in Table 3 (M2_C100; M2_L50; M2_L50_SRA). Because of the different bulk densities of the specimen water loss was calculated by volume percent. Figure 11 displays the calculated water loss of the specimen stored in different climate. Table 7 gives water loss at crack time as well as crack width for the mortars which showed cracking.

Influence of replacing cement -- Independent from storage program mortars crack in the order M2_C100 and then M2_L50, thereby showing high differences in water loss at cracking time.

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For mortars containing binder L50 (50 vol.-% limestone) the crack initiating water loss was 5 (7d – 20°C/95% RH) to 6 times (2d – 20°C/95% RH) higher in comparison to binder system C100 (100 vol.-% NPC; W/C = 0,3).

The replacement of cement by 50 vol.-% limestone (binder L50) lowers the sensitivity to autogenous shrinkage. Because of the constant ratio of solids to liquids in all of the tested mortars, the amount of water available for the cement in the binder L50 (W/C=0,6) is twice as high as for C100. Autogenous shrinkage is reported to appear up to W/C= 0.4 [4]. For this purpose early age crack formation caused by autogenous shrinkage is unlikely for binder system L50.

Cracking appearing after 21d (after a storage at 20°C/95% RH for 7d, high degree of hydration) can be attributed to drying shrinkage.

Influence of storage condition -- The storage program has a strong influence on the crack width of the mortars (Table 7). The longer the mortars are stored at 95% RH (storage A) the broader is the crack width (crack width {7d at 95%RH}> crack width {7d at 95%RH}), given in Table 7. It is supposed that the degree of hydration at starting time of storage under drying conditions (70%RH - storage B) is the main reason for the observed behavior.

The mechanical properties of mortars (compressive strength, tensile strength, modulus of elasticity) depend on the degree of hydration, increasing with increasing degree. Drying shrinkage starting with a lower degree of hydration reduces partially restraining tensions by creep, increasing with decreasing degree of hydration. The mortars with a higher degree of hydration and therefore a higher modulus of elasticity are bearing higher tensions from restrained shrinkage. The higher the residual tension at cracking time the wider the crack width will be.

Referring to the results of isothermal heat conduction calorimetry, the degree of hydration of the studied mortars can be estimated. After 2 days curing at 20 °C/95% RH the binder systems without SRA show a degree of hydration of about 80% (254 J/g) referring to heat release after 7 days (325 J/g). For mortars with SRA and SP (retarded cement hydration), the calculated degree of hydration after 2 days at 20 °C/95% RH is about 50%, leading to a lower crack width of about 70% (Table 7).

In storage with short curing at 20 °C/95% RH and for mortars containing SP and SRA the reduction of tensions by creep has a great influence on crack width.

Influence of shrinkage reducing agent (SRA) -- Mortars containing SRA (5 wt.-% of water content) did not crack in the applied storage condition during the observation period and may reach equilibrium humidity without cracking (35%RH - storage C; Fig. 11).

The reduction of shrinkage by SRA reduces tension stress under restrained conditions, so that the critical splitting tensile stress respectively the tensile strength is not reached.

Creep might have influenced the positive behavior of SRA-mortars under restrained conditions. Until now the influence of SRA on creep is barely investigated.

CONCLUSIONS

In this paper the influence of different mixture parameters of self compacting mortar and concrete on free drying shrinkage and restrained shrinkage were tested. The potential for the reduction of shrinkage and crack risk was estimated following three main concepts.

- Replacing cement by high volume of pozzolanic additives reduces free drying shrinkage. The replacement of 50 vol.-% of inert limestone into the binder system is reducing drying shrinkage in the range of 30% (long term) for both mortar and concrete, and additionally lowers the risk for cracking.
- Reducing paste volume by optimizing packing density of aggregates reduces drying shrinkage in the range of about 20% for mortar and about 30% for concrete.
- Application of shrinkage reducing admixture heavily reduces crack risk under restraint condition and reduces drying shrinkage. The reducing effect of these admixtures decreases within time. The potential of reduction after 28 days is 40% for mortars and 50% for concrete. Long term (180 days) reduction potential is in the range of 20% for mortar and 40% for concrete.

Following these three main concepts a shrinkage reduced SCC was developed. A total shrinkage reduction of about 65% (0,52 mm/m up to 0,18 mm/m after 180 days) was obtained.

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Table 1 – Densities of materials

Material	bulk density	density
	[g/cm ³]	[g/cm ³]
cement	3.15	-
fly ash	2.31	-
limestone	2.73	-
aggregate	2.63	-
SP	-	1.05
SRA1	-	0.98
SRA2	-	1.00

Table 2 – Mixture proportions and flow properties of mortar

mixtable 1		volume ratio	mass				flow properties		
mortar		paste volume	water	cement	SP	W/C excl. SP	slump	flow time	sec-criterion
	grading curve	[vol.-%]	[kg/m ³]	[kg/m ³]	[kg/m ³]		[mm]	[s]	[yes/no]
P49 M1	M1	48,9	237	770	15,4	0,31	220	16,0	no
P51 M1		50,7	247	795	15,9	0,31	265	10,0	no
P53 M1		52,6	257	821	16,4	0,31	270	7,0	yes
P55 M1		55,3	270	866	17,3	0,31	275	5,5	yes
P46 M2	M2	45,9	223	729	10,2	0,31	275	7,0	yes

Table 3 – Mixture proportions and flow properties of self compacting mortar /constant total volume of liquid phase (water, SP, SRA) in all mortars

mortar	volume ratio; binder			admixtures						flow properties	
	cement	limestone (L)	fly ash (F)	SP	SRA1		SRA2			slump	flow time
	[Vol.-%]	[Vol.-%]	[Vol.-%]	[kg/m ³]	[kg/m ³]	[wt.-% o.w.]	[kg/m ³]	[wt.-% o.c.]	[mm]	[s]	
C100	100	0	0	10,2					275	7	
F50	50	0	50	7,3					270	7	
F25 L25	50	25	25	7,3					275	7,5	
L50	50	50	0	7,3					265	8	
L50 min SRA1	50	50	0	7,3	3,3	1,5			270	7	
L50 med SRA1	50	50	0	7,3	5,6	2,5			275	7	
L50 max SRA1	50	50	0	7,3	11,1	5,0			280	6,5	
L50 min SRA2	50	50	0	7,3			1,8	0,5	265	8	
L50 med SRA2	50	50	0	7,3			3,6	1,0	270	8,5	
L50 max SRA2	50	50	0	7,3			7,3	2,0	285	8,5	

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Table 4 – Mixture proportions of self compacting concrete; for all concretes SRA-content 5 wt.-% of mixing water; SP-content 2 wt.-% of cement content; volume ratio of liquid/solid in paste $l/s = 0.99$

concrete	grading curve	paste volume	volume ratio binder		flow properties	
			cement	limestone (L)	slump	flow time
		[vol.-%]	[Vol.-%]	[Vol.-%]	[mm]	[s]
C3 C100	C3	38,4	100	0	630	5
C3 L50	C3	38,4	50	50	675	4,5
C3 L50 SRA	C3	38,4	50	50	730	4
C2 L50	C2	35,6	50	50	700	4
C2 L50 SRA	C2	35,6	50	50	720	5
C1 L50	C1	32,9	50	50	710	5
C1 L50 SRA	C1	32,9	50	50	675	9

Table 5 – Concepts, conditions and potential for shrinkage reduction of self compacting mortar (SCM)

concept	condition	shrinkage reducing effects on SCM				at time of drying
		reduced [%]				[d]
reduction of paste volume	from 55 vol.-% to 46 vol.-%	22	from 53 vol.-% to 46 vol.-%	18		28
		27		23		90
		22		18		180
replacement of cement in the binder	50 vol.-% limestone	31	25 vol.-% limestone + 25 vol.-% fly ash	22	50 vol.-% fly ash	5
		29		24		11
		28		23		11
		28		25		13
shrinkage reducing agent (SRA1)	content 0.9 wt.-% of cement	17	content 1.5 wt.-% of cement	26	content 3 wt.-% of cement	39
		13		18		30
		12		14		28
		9		14		24
shrinkage reducing agent (SRA2)	content 0.6 wt.-% of cement	13	content 1.0 wt.-% of cement	49	content 2 wt.-% of cement	32
		8		14		21
		5		11		15
		5		11		15

Table 6 -- Concepts, conditions and potential for shrinkage reduction of self compacting concrete (SCC)

concept	condition	shrinkage reducing effects on SCC					at time of drying
		reduced [%]					[d]
replacement of cement	50 vol.-% limestone	40					28
		31					90
		31					180
reduction of paste volume	from 33.4 vol.-% to 32.9 vol.-%	33	from 38.4 vol.-% to 35.6 vol.-%	21	from 35.6 vol.-% to 32.9 vol.-%	15	28
		30		20		13	90
		29		19		12	180
shrinkage reducing agent (SRA)	0.9 wt.-% of cement, related to paste volume	23					28
		16					90
		9					180
	1.5 wt.-% of cement, related to paste volume	23					28
		20					90
		13					180
	3 wt.-% of cement, related to paste volume	35					28
		36					90
		29					180
	paste volume [vol.-%]	32.9					

Table 7 -- Water loss of specimen at cracking age / restrained shrinkage; water loss referring to initial volume of specimen

7 days at Storage A (20°C/95% RH)					2 days at Storage A (20°C/83% RH)			
Storage B (20°C/70%RH) up to the 21th day then Storage C (20°C/35% RH)								
mortar	water loss	time	cracked	crack width	water loss	time	cracked	crack width
	[vol.-%]	[d]	[yes/no]	[mm]	[vol.-%]	[d]	[yes/no]	[mm]
M2_C100	1,3	25	yes	0,95	1,5	24	yes	0,65
M2_L50	6,3	26	yes	0,49	10,0	59	yes	0,33
M2_L50_SRA	7,8	91	no	-	7,7	91	no	-
Storage C (20°C/35% RH)								
mortar	water loss	time	cracked	crack width	water loss	time	cracked	crack width
	[vol.-%]	[d]	[yes/no]	[mm]	[vol.-%]	[d]	[yes/no]	[mm]
M2_L50_SRA	8,7	91	no	-	9,1	26	no	-

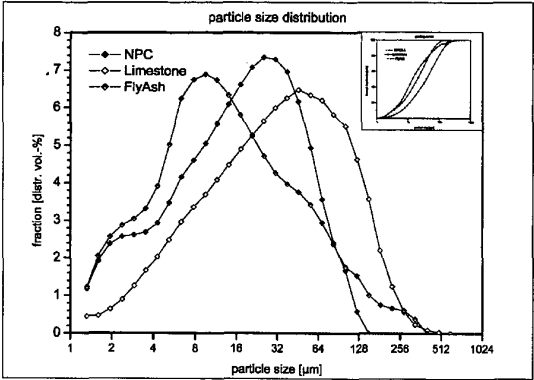


Fig. 1 -- Particle size distributions of cement and additives

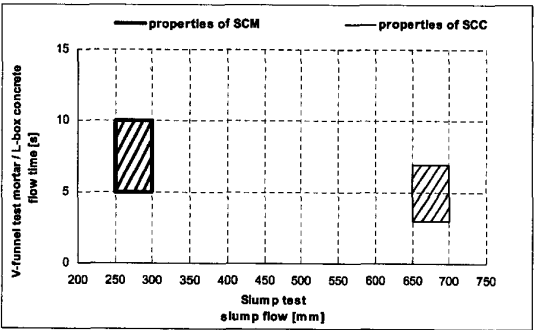


Fig. 2 -- Criteria for self compacting property of mortar and concrete

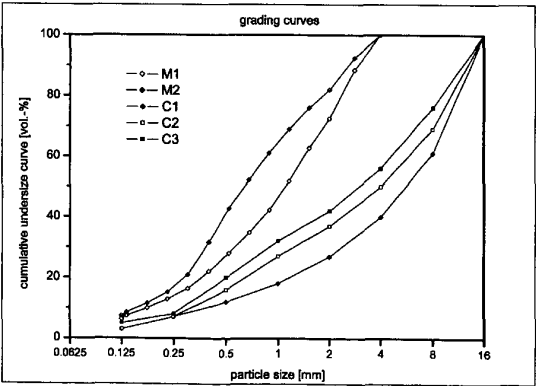


Fig. 3 -- Particle size distributions of aggregates

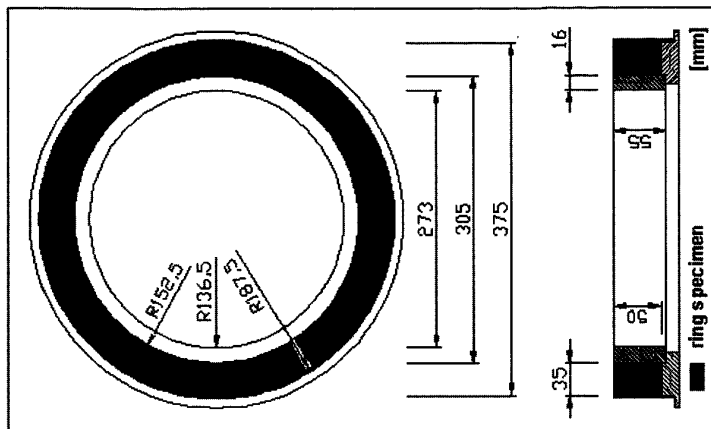


Fig. 4 -- Setup for restrained shrinkage measurement / ring-test

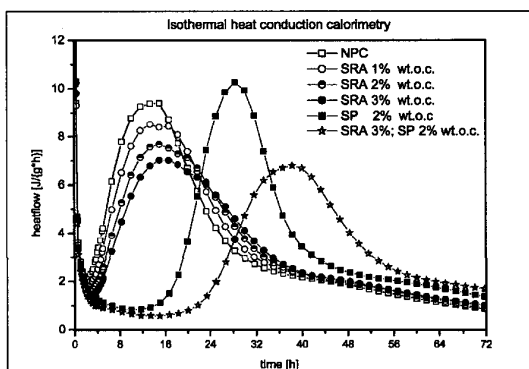


Fig. 5 -- Isothermal heat conductivity calorimetry; heat flow of cement paste; influence of additives

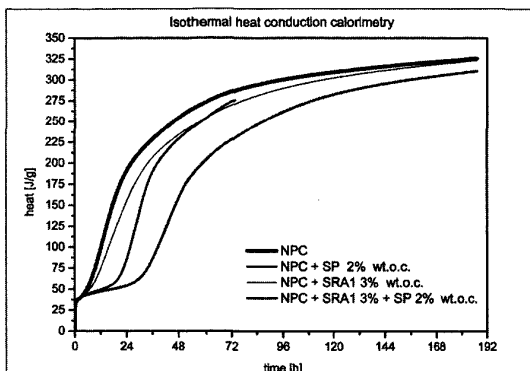


Fig. 6 -- Isothermal heat conductivity calorimetry; heat release of cement paste; influence of additives

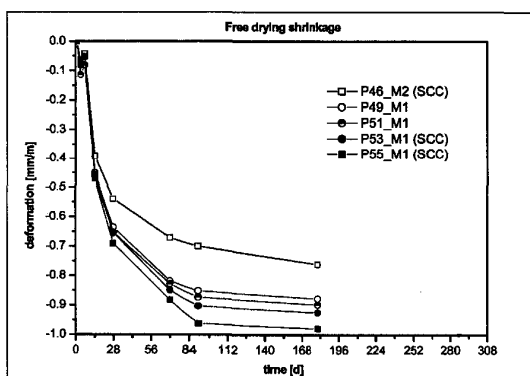


Fig. 7 -- Influence of paste volume on shrinkage of mortar

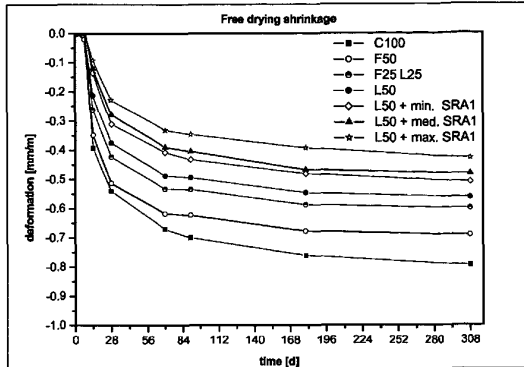


Fig. 8 -- Influence of cement replacement / effect of dosage of SRA on shrinkage of self compacting mortar

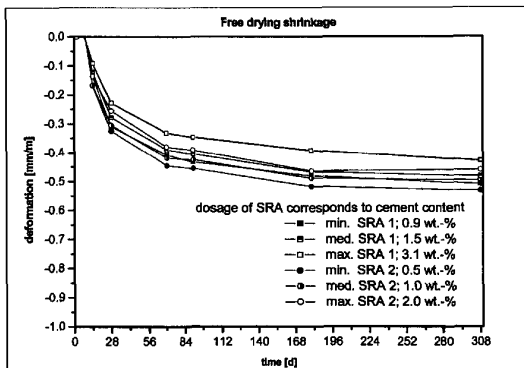


Fig. 9 -- Effect of the dosage of different SRA on shrinkage of self compacting mortar

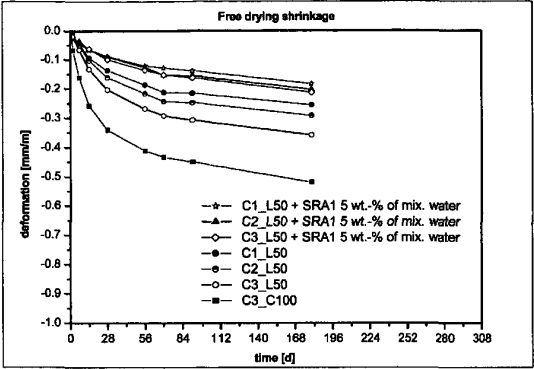


Fig. 10 -- Effect of paste volume; cement replacement; SRA on shrinkage of self compacting concrete

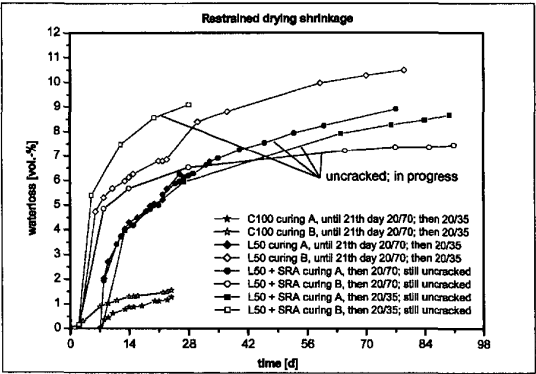


Fig. 11 -- Water loss of specimen and cracking time; water loss referring to initial volume of specimen measurement stopped after cracking

State-of-the-Art Radar Tomography for Concrete Structure Assessment

by B. Giroux, P. Rivard, E. Gloaguen, and M.C. Chouteau

Synopsis: The knowledge of the concrete condition and monitoring of on-going deterioration processes are key factors for the management of concrete structures. Traditional assessment often remains limited and qualitative. This paper presents a new methodology based on radar tomography to estimate the water distribution in concrete, and therefore to assess indirectly the concrete condition. It aims at providing effective nondestructive procedures for engineers and structure managers. Work was conducted on a hydraulic structure suffering from alkali-aggregate reaction. Six boreholes were drilled to allow for core recovery and radar downhole measurements. Borehole radar antennas of 100 MHz central frequency were used for crosshole tomography. Radar data pointed out a high attenuation (high electrical conductivity), corroborated with the core testing. Both travel time and amplitude tomography needed to be carried out for an accurate determination of the dielectric permittivity. The Complex Refractive Index Model (CRIM) was used to relate the permittivity to the volumetric water content. The tomographic calculations were done using an algorithm based on geostatistical methods and stochastic simulations, allowing to draw probability maps of the volumetric water content. Results indicate that the overall volumetric water content is moderately high (0.15), and that some zones have high water content (above 0.2). These results will be used in the maintenance program of the structure.

Keywords: alkali-aggregate reaction; attenuation; radar tomography; structure assessment; velocity; water content

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INTRODUCTION

The importance of water content is critical regarding the expansion rates in concrete suffering from alkali-aggregate reaction^{1,2,3}. In fact, with the alkali levels and aggregate reactivity, the water content will govern the evolution of the alkali-aggregate reaction (AAR). The knowledge of the moisture distribution in an AAR-affected structure is then a key factor for the evaluation of the undergoing expansion rate and the duration of the residual expansion if the alkalis and reactive silica have not been consumed. However, in a large concrete structure, the moisture levels may vary significantly within the concrete body, depending on the accessibility to external water sources, and on the temperature and geometry of the different members.

The moisture distribution in a concrete structure remains hard to assess. Humidity probes provide point information and the accuracy of the measurements is sometimes questionable, especially at high moisture levels. Over the past years, electromagnetic methods have been used to determine the water content in various media.

Ground penetrating radar (GPR) is a technique, which employs radio waves to draw images of the ground or man-made structures⁴. Although the first attempts to use radio waves to map underground features occurred in the 1960s and 1970s, commercial GPR systems appeared in the 1980s and their uses increased in the 1990s. Nowadays, GPR systems are used routinely in many fields of applications such as mining, hydrogeology, archaeology, glaciology, forensic investigations or geotechnical investigations to name a few. GPR imaging owes its success to the high resolution of the

pictures that it provides, and to the various scales of applications it allows (from reinforcement bar mapping to glacier stratigraphy studies).

The propagation of radio waves is governed by three physical properties of the medium, namely the dielectric permittivity ϵ , the electrical conductivity σ and the magnetic permeability μ . At typical GPR frequencies (10 MHz to 1.5 GHz), the radio wave velocity is mainly a function of the permittivity, or dielectric constant κ^1 , of the medium. More specifically, the velocity is inversely proportional to $\sqrt{\kappa}$. In concrete, the main substance affecting the effective dielectric constant is water. In the radar frequency range, free water has a dielectric constant of about 79 to 81 depending on temperature, while the dielectric constant of most aggregates is about 5 and the dielectric constant of air is approximately 1. Hence, the relative proportion of water governs the dielectric constant of the concrete. Therefore, an increase in water content translates into a decrease of the wave velocity. Empirical and theoretical relations exist to infer water content from dielectric constant measurements^{5,6,7}. This fact has long been recognized, and radio wave velocity measurements have been used for years to estimate the amount of water in the probed material. This fact also serves as the basis of the present study.

Although GPR provides useful information for many applications, the technique suffers from one serious limitation: its restricted penetration depth in electrically conductive materials. This has precluded its use in environments such as clay-rich terrains, or in porous materials when the water of imbibition is electrically conductive. One way to attain greater depths of investigation is to use borehole antennas. Borehole radar systems appeared in the late 1980s for mining or geological applications such as fracture mapping in crystalline rock environments⁸. Since then, their use has widened to hydrogeological and geotechnical investigations^{9,10}. Borehole systems are mainly used for tomographic surveying. Crosshole travel time tomography consists of computing the velocity field between two holes, from measures of the arrival time of the direct wave that traveled from transmitters to receivers. The same can be done for the attenuation field, in which case the wave amplitude decay rate is used instead of the travel time. This paper deals with methods employing both velocity and attenuation results to infer accurate estimates of the water content in the concrete of a hydraulic structure suffering from alkali-aggregate reaction.

METHODOLOGY

Radar tomography principles

Tomographic radar measurements are accomplished by using a transmitting antenna (Tx) located in a borehole to emit a high frequency electromagnetic pulse. The radiated pulse propagates within the medium with velocity v , and is eventually recorded at a receiving antenna (Rx) located in another hole a known distance away. The recording, referred to as a *trace* in the geophysical literature, is the voltage at the receiver

* $\kappa = \epsilon / \epsilon_0$, where ϵ_0 is the dielectric permittivity of vacuum.

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as a function of time. Hence, the trace contains the transmitted wavelet arriving at time t , as well as other waves produced by eventual reflections or refractions of the emitted pulse. One approach to tomographic inversion is to retrieve the arrival time of the direct wave and use it to obtain the velocity between the transmitter and receiver by considering that the wave traveled along a raypath. Another approach is to consider all the information contained in the trace, i.e. the whole waveform, to reconstruct the physical properties distributions between the transmitter and receiver¹¹. Waveform-based algorithms require extensive computational resources. Hence, in this paper, only the former will be considered, i.e. the ray-based approach.

Following the theory of geometrical optics, to each transmitted wavelet can be associated a raypath l , which is a curve describing the trajectory of the wave between the Tx and Rx antennas. The travel time is related to the slowness s , i.e. the reciprocal of velocity, and raypath by

$$t = \int_{ray} s(l)dl. \quad (1)$$

During a survey, the recording process is repeated for various Tx and Rx positions along the holes, in order to have the raypaths cover the whole area between the two holes. After completion of the survey, the arrival times of the direct waves are picked from each trace, and these data are used to perform the travel time tomography calculations. For n_o traces, this data takes the form of a $n_o \times 1$ matrix called $\mathbf{t}$. The aim of the tomography is to obtain the velocity field, or its inverse the slowness, between the two boreholes. To perform the calculations, the slowness field must be discretized into n_p cells, yielding a matrix of $n_p \times 1$ unknown slownesses $\mathbf{s}$. This discretization implies that the raypaths are split into segments (see Fig. 1). Each segment has a length equal to the distance the ray travels in that cell. The travel time associated with a raypath becomes the sum of each segment times the slowness of the corresponding cell:

$$t = \sum_{i=1}^{segments} s_i l_i. \quad (2)$$

Gathering all measured travel times and unknown slownesses, a system of equations of the form

$$\mathbf{Ls} = \mathbf{t} \quad (3)$$

can be built, with $\mathbf{L}$ being a sparse $n_o \times n_p$ matrix. The objective of the tomographic calculation is to solve this system for $\mathbf{s}$. Note that the problem is nonlinear because, by virtue of Fermat's principle, the raypaths are function of the slowness field. Fermat's principle states that the path of a ray between two points is the path that minimizes the travel time. Consequently, rays tend to "bend around" low velocity zones and "concentrate" in high velocity zones. This nonlinearity implies that the tomographic problem must be solved iteratively.

Until recently, variants of least-square algorithms were the exclusive approach to solving the tomographic problem. However, because this problem is inherently ill-posed, least-square algorithms rely on so-called regularization techniques to stabilize the solution¹². Very often, the results of regularized least-square algorithms are sensitive to regularization parameters, which cannot easily be adjusted automatically. To circumvent this problem, an approach based on geostatistical analysis of the travel time data has been developed by Gloaguen et al.¹³. It can be shown that the covariance model of the slowness field can be modeled from the travel time data. This covariance model describes the spatial characteristics of the slowness field. Using this covariance model and the fact that slowness and travel time are linearly related, it is possible to cokrig the slowness field using the travel time data. Cokriging is an interpolation technique allowing using a more intensely sampled covariate (the travel time) in the estimation of values for a related variate (the slowness)¹⁴. The procedure is said to be self-regularized because regularization is supported entirely by the data¹³. It is also easy to go further and to impose slowness gradients or even any kind of linear constraint to the solution. Another advantage of this technique is that, once the slowness covariance model is known, stochastic simulations can be carried out to measure the robustness of the slowness model. This is done by performing a large number of simulations and storing each realization of the slowness field. The realizations are all different and independent, but equally consistent with the measured data¹⁴. The probability that the slowness falls within a given range of values can then be drawn from the set of realizations.

So far the focus has been on travel time tomography, and it is common practice to perform travel time tomography alone to compute the dielectric constant within the probed medium. This approach is valid when electrical loss is low, i.e. when the condition $\sigma/\omega\epsilon \ll 1$ is satisfied. Here, ω is the angular frequency equal to $2\pi f$, f being the frequency of the radio wave. Recall that the water content is obtained from the dielectric constant κ . Under the low loss condition, the velocity is

$$v = \frac{1}{\sqrt{\mu\epsilon}}. \quad (4)$$

Note that unreinforced concrete is non magnetic, and its magnetic permeability can safely be assumed to be equal to the permeability of free space μ_0 . Therefore, the dielectric constant is obtained from

$$\kappa = \frac{1}{v^2 \mu_0 \epsilon_0}. \quad (5)$$

However, the above expression yields inaccurate results if the electrical conductivity σ of the concrete is close to, or higher than, the product of permittivity ϵ and ω . In such circumstances, the dielectric constant must be evaluated from both the velocity and attenuation¹⁵. With the attenuation α in Np/m, the dielectric constant is

$$\kappa = \frac{1}{\mu_0 \varepsilon_0} \left[\frac{1}{v^2} - \left(\frac{\alpha}{\omega} \right)^2 \right] \quad (6)$$

The attenuation within the medium can be obtained from the ratio of the emitted amplitude to the received amplitude. In this case, the received amplitude must be processed so that it can be linearly related to the attenuation. This can be done either in the time domain, i.e. with the amplitudes recorded on the radar traces, or in the frequency domain, i.e. with the Fourier transform of the radar wavelet¹⁶. In this study, we chose to process the amplitudes in the time domain. In short, the amplitude measured at the receiver A_m is an exponential decay of the initial amplitude at the source A_0 times a correction term taking into account the radiation patterns of the antennas (Θ_{Tx} and Θ_{Rx}) and the geometrical spreading of the wave:

$$A_m = A_0 \exp \left(- \int_{ray} \alpha(l) dl \right) \frac{\Theta_{Tx} \Theta_{Rx}}{l} \quad (7)$$

The terms A_0 , Θ_{Tx} and Θ_{Rx} cannot be easily known and in most cases are approximated. This makes amplitude tomography somewhat less robust than travel time tomography. Details and discussion on amplitude data reduction can be found in¹⁷ and¹⁸. However, once A_0 , Θ_{Tx} and Θ_{Rx} are estimated, taking the natural logarithm of Eq. (7) leads to

$$\tau = \int_{ray} \alpha(l) dl; \quad (8)$$

where $\tau = \ln(A_0) + \ln(\Theta_{Tx} \Theta_{Rx}) - \ln(A_m l)$. This equation has the same form as Eq. (1). This means that the algorithms developed to solve Eq. (1) are applicable for Eq. (8).

Field procedure

Field work was carried out on a portion of a hydraulic structure located in Eastern Canada and that was built in the late 1950s. Alkali-aggregate reaction was detected about twenty years after the construction. Six holes were drilled along the structure for radar and seismic downhole measurements as well as for core recovery. Petrographic examination conducted on cores confirmed that the concrete is affected by AAR. Numerous petrographic features associated with AAR were observed, all along the boreholes, such as gel filling cracks and air voids, cracks running through the aggregates and extending into the cement paste, etc. All holes penetrated the whole concrete structure and reached the black shale bedrock. From these holes, eight GPR tomographic panels were surveyed. A panel refers to the plane defined by the bounding boreholes. The hole positions were surveyed and are shown in Fig. 2 along with the arrangement of the tomographic panels. Each panel was surveyed in crosshole mode, as well as in vertical radar profiling mode (VRP), i.e. with the transmitter on the surface and the receiver in one hole. Note that the holes are on average 14 meter deep and that the

transmitter Tx and receiver Rx were moved 0.3 m between the measurements. This yields between 2500 and 3000 traces per panel. The whole survey took 5 days to complete.

École Polytechnique's RAMAC system was used for data acquisition. The system, built by Malå Geosciences, comprises a CU-II console linked to a laptop where the data are displayed in real time. The system was operated using antennas with a nominal frequency of 100 MHz (Fig. 3). During the survey, every Rx measurement position is scanned successively for each Tx position. The measurements are performed automatically at a regular spatial interval by using, at the receiver, a mechanical encoder linked to the console. Two measurements, one before and one after the scans, are performed with both antennas lifted in the air to control instrument drift.

At survey time, the boreholes were filled with water. The water level ranged between 1.44 and 1.64 meter below the structure surface. The water level did not change significantly during the survey, and the drilling report also indicates stable levels. References 18 and 19 have shown that the presence of water in the borehole affect the radiative properties of the antennas, especially at the air-water interface. This makes the approximation of Θ_{Tx} and Θ_{Rx} weaker, and limits the validity of the attenuation reconstruction. For this reason, as well as others discussed below, the results from the surface to depths of 2 to 3 meters might be questionable.

RESULTS

Qualitative interpretation

A first appreciation of the data can be obtained from zero offset profiles (ZOPs), which are analogous to cross-hole sonic logging. These profiles can be built from a crosshole dataset by extracting the traces corresponding to transmitter and receiver located at the same depth in their respective hole. These traces are plotted as a function of depth, as shown in Fig. 4. Such a zero offset profile allows determining if significant variation of velocity is present in the probed medium. Similarly, the power spectra of the traces can be computed and plotted, which allow appraising the attenuation through the medium. Two examples are shown in Fig. 4. Consider first the amplitude data ZOPs (leftmost images). For both profiles, a zone of perturbation is observed around depths of 2 meter, as well as earlier arrival times at the bottom. The zone of perturbation is at a depth corresponding approximately to the water level. The earlier arrival times at the bottom are attributed to the transmission of the radar wave through the bedrock, which is less porous than concrete and consequently contains less water. In the power spectra ZOPs (rightmost images), it is first observed that the energy at the receiver has a dominant frequency around 50 MHz, which is much lower than 100 MHz, the nominal frequency of the transmitting antenna. This is an indicator of strong attenuation. Secondly, the zones of higher energy are at the top and bottom of the holes, as indicated by the darker bands.

Overall, examination of the zero offset profiles revealed strong attenuation, stronger than was at first expected. This implies that the concrete is relatively electrically

conductive. Indeed, DC resistivity measurements on core samples, which were kept in water at least 48 hours before testing, resulted in conductivity values ranging from 5 to 16 mS/m² with an average of 10 mS/m. Typical values for concrete are in the range 1-3 mS/m. Possible causes explaining this are: AAR which have induced cracking over the years, concrete made of high W/C which increases the porosity and pore connectivity, and high alkali concentration in pore solution which increases its conductivity.

Quantitative interpretation

Determination of the dielectric constant – Qualitative interpretation and core sample measurements pointed out that the concrete in the survey area is relatively electrically conductive. Knowing that the dominant frequency is around 50 MHz and that the average value of conductivity is 10 mS/m, the loss term $\sigma/\omega\kappa\epsilon_0$ equals 0.3 for an assumed value of $\kappa=12$, which is a conservatively high value. For higher conductivity and lower dielectric constant encountered, the loss term increases to nearly 1. Clearly, the low loss approximation cannot be used with the data, and Eq. (6) must be used to compute the dielectric constant.

The calculation of the dielectric constant is done in three steps. First, travel time tomography is performed. This results in a map of the velocity field and the knowledge of the raypaths associated with each measured travel time. The second step consists in the tomographic inversion of the reduced amplitude data to obtain the attenuation map. Finally, Eq. (6) is applied.

Before proceeding with travel time tomography, the covariance model of the slowness field must be determined. The procedure described in ¹³ is followed for that purpose. Once the covariance model is determined, the travel time inversion starts with a homogeneous concrete model and straight rays. After a first velocity model is obtained, it is used to compute the curved rays. Note that in this paper, the velocity above the surface is constrained to 0.3 m/ns (value in air) during all travel time inversions. In contrast, the average velocity in concrete is about 0.1 m/ns. Therefore, the air/concrete interface is sharply contrasted. By virtue of Snell's law, waves incident at the air/concrete boundary at angles greater than $\sin^{-1}(0.1/0.3)$ will be critically refracted. Curved ray tracing allows to properly take into account waves critically refracted at the surface. After computation of the curved rays, the matrix **L** is updated and the system of Eq. (3) is solved for a second time. Usually, one or two curved ray iterations suffice to obtain satisfactory results. Two examples of velocity tomograms and associated ray coverages are shown in Fig. 5 and 6. Knowing the raypaths, the system of Eq. (8) is then put together. Note that all rays refracted on the surface, including the VRP data, are not used for amplitude tomography due to the difficulty in correcting the amplitude²⁰. The covariance model of the attenuation must also be determined before performing amplitude tomography. This is done in the same manner as with the travel time data. Besides, since the raypaths are not affected by attenuation, iterating is not necessary

²Units of conductivity are millisiemens per meter, where 1 siemens equals 1 ohm.

during amplitude tomography. Two examples of attenuation tomograms are shown in Fig. 5 and 6, together with the dielectric constant maps obtained from the v and α models. For each tomogram, 128 geostatistical simulations were carried out. The results of the simulations will be used in the next section to compute the water content probability maps.

Determination of the volumetric water content – There exists a number of theoretical models to compute the effective permittivity of a composite medium from the relative proportions of the constituents. The most common are the Hanai-Bruggeman and Maxwell-Garnett models²¹. These models require some level of information about the medium (i.e. the shape of the inclusion), and have not been studied extensively for concrete²². In this study, the complex refractive index model (CRIM) was used because of its simplicity. The CRIM formula relates the effective dielectric constant of the medium κ to the dielectric constant of the solid phase κ_h , to the dielectric constant of water κ_w , to the dielectric constant of air κ_a , to the porosity ϕ and to the saturation S ⁶

$$\sqrt{\kappa} = \phi S \sqrt{\kappa_w} + (1 - \phi) \sqrt{\kappa_h} + \phi(1 - S) \sqrt{\kappa_a}. \quad (9)$$

The porosity is defined here as the ratio of pore volume to the total volume, and the saturation is the ratio of volume of water to pore volume. The volumetric water content θ , defined as the ratio of volume of water to the total volume, is therefore equal to the product ϕS .

In concrete, the solid phase includes the cement paste and the aggregates. At the survey site, the relative proportions and dielectric properties of these two constituents are not known, and it is therefore not possible to explicitly calculate the effective permittivity of the cement paste and aggregate mixture. Published data are also very scarce. ²³ measured the dielectric constant on dried samples of concrete with 13.3% porosity and on mortar with 24.3% porosity. He obtained values of 5.1 for concrete and 4.4 for mortar. ²⁴ obtained a value of 4 for unsaturated concrete. In the remaining, we assume that κ_h is equal to 5.

The profile of saturation is also unknown, although we assume that the concrete is saturated below the water level recorded in the boreholes. In order to compute θ in the unsaturated concrete (the top 1.5 m of the structure), we assume that the porosity is homogeneous, and use the mean value of θ in the saturated part as an estimate for the porosity (ϕ). Note that this is a weak assumption, and the validity of the results in the unsaturated zone might be questionable. Nevertheless, Eq. (9) is then used to estimate the saturation:

$$S = \frac{\sqrt{\kappa} - (1 - \phi) \sqrt{\kappa_h} - \phi \sqrt{\kappa_a}}{\phi (\sqrt{\kappa_w} - \sqrt{\kappa_a})}. \quad (10)$$

Using a value of 80 for the dielectric constant of water, 1 for air, and the values determined above, maps of the volumetric water content are built from the dielectric

constant tomograms. The two examples in Fig. 5 and 6 are used to produce the maps shown in Fig. 7 and 8. As mentioned above, 128 stochastic simulations were performed for each travel time and each amplitude datasets. A set of 128 water content maps is built from these simulations results. Recall that geostatistical simulations are techniques used to sample random fields with given spatial properties. One run of a particular simulation algorithm yields one realization of the random field. Additional runs yield as many additional realizations, which are all different, independent, but equally consistent with the measured data. Since the realizations are independent, the probability that the field exceeds a given value can be drawn from the set of realizations. Hence, at a given point x in the water content map, the probability of $\theta(x)$ being above a given threshold θ_i is

$$P(\theta(x) > \theta_i) = \frac{1}{N} \sum_{i=1}^N I(\theta_i(x) > \theta_i); \quad (11)$$

with $\theta_i(x)$ being each realization i at point x and $I(\theta_i(x) > \theta_i)$ the indicator taking value 1 if $\theta_i(x) > \theta_i$ and 0 otherwise. N is the number of realizations (or maps of θ). Three probability maps, shown in Fig. 7 and 8, are built for θ_i equal to 0.1, 0.15 and 0.2.

These figures show that, except in a few zones, the probability that $\theta > 0.2$ is very low. They also indicate an important variability in moisture content. One particularity is the high water content zone in Fig. 8 near Hole 4, between 3 and 4 m depths. This zone correlates with the boring log data of Hole 4, which indicate fracture zones at 3.5, 4 and 4.5 m, and large cracked aggregates at 3.5 m. Another intriguing high moisture zone is observed in the top 1.5 m, above the water level, between Holes 2 and 3 (Fig. 7A). Boring log data indicate a high concentration of voids with silica gel above the water level in Hole 3, which could partly explain higher moisture content. It might also be an artifact due to the assumption of homogeneous porosity in the unsaturated zone, or an artifact due to the low ray coverage near the surface (Fig. 5B).

CONCLUSION

A new methodology based on radar tomography to estimate the moisture distribution in concrete is presented. Specifically, the novel aspects are: the use of borehole antennas, the use of a self-regularized geostatistical approach allowing to draw probability maps of the water content, and the use of constrained slowness models and curved ray-tracing to properly take into account waves refracted at the surface. The methodology was applied to data collected on a hydraulic structure suffering from alkali-aggregate reaction. Results indicate that the overall volumetric water content is moderately high (0.15), and that some zones present high water content (above 0.2) suggesting that AAR expansion rates may be higher in these zones. Work is under way to correlate the radar tomography results with the seismic tomography results and laboratory measurements in order to refine the interpretation and validate the results.

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