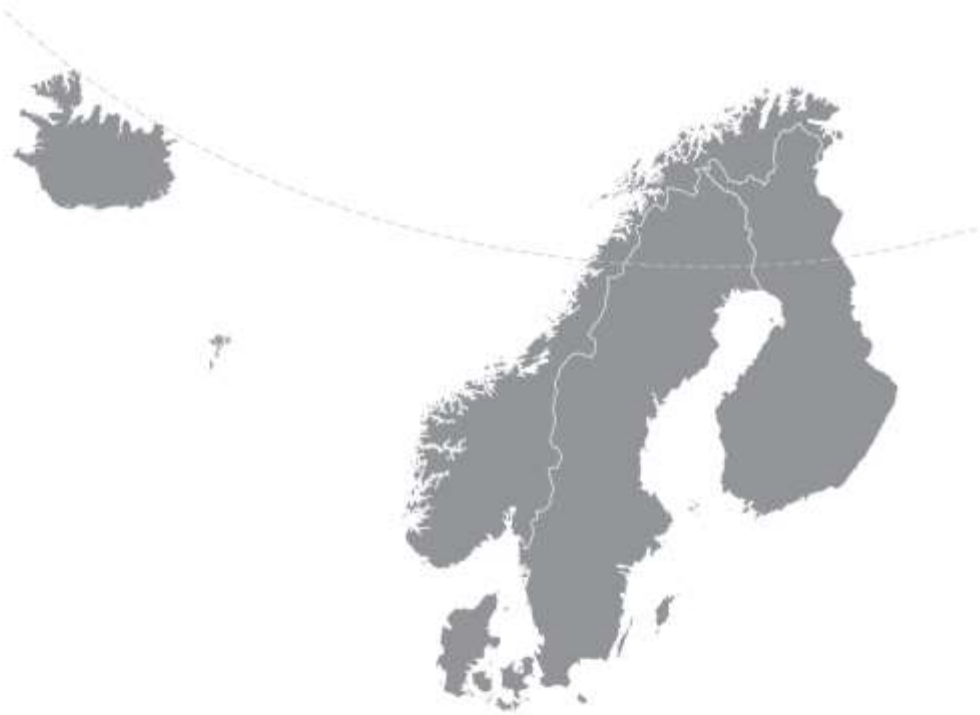
Concrete in arctic conditions

WORKSHOP PROCEEDINGS FROM A NORDIC WORKSHOP

Trondheim – Norway, 18–19 June 2019



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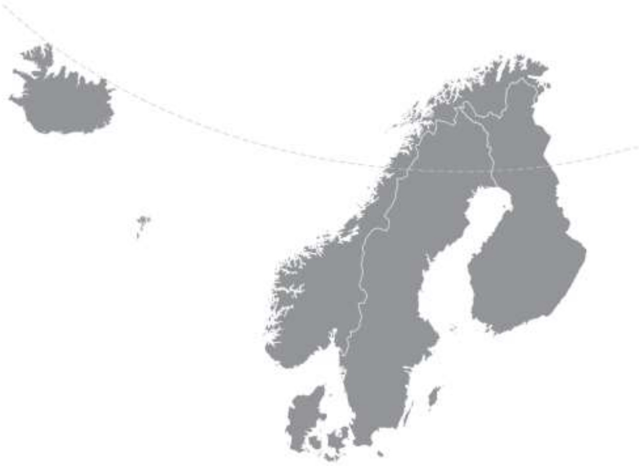

WORKSHOP PROCEEDINGS NO. 16

FROM A

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Trondheim, Norway

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NCF Nordic Concrete Federation

PREFACE

The Nordic Workshop “Concrete in arctic conditions” has been organized under the auspices of the ongoing Norwegian research project; Durable Advanced Concrete Structures (DaCS). The research project DaCS aims to increase the knowledge of sustainable and competitive reinforced concrete structures in harsh environment and is funded by The Research Council of Norway, in addition to several industrial partners. The DACS partners are Kværner AS (project owner), Axion AS (Stalite), AF Gruppen Norge AS, Concrete Structures AS, Mapei AS, Multiconsult AS, NorBetong AS, Norcem AS, NPRA (Statens vegvesen), Norwegian University of Science and Technology (NTNU), SINTEF Byggforsk, Skanska Norge AS, Unicon AS and Veidekke Entreprenør AS.

The workshop is devided into 2 sections: *Ice abrasion* and *Frost resistance*, which are related to the Ph.D. projects under the directive of the aforementioned project: “*Experimental study of concrete-ice abrasion and concrete surface topography modification*” by Guzel Shamsutdinova and “*Production and documentation for frost durable concrete*” by Andrei Shpak.

The event is to be hosted by NTNU, Institute for materials and structures. The main goal is to increase the knowledge within ice abrasion and frost resistance of concrete and to provide exchange of information and further development among the participants. This booklet documents the collection of extended abstracts of most of the given lectures during the workshop. The organizing committee would like to thank all the speakers and contributors at the seminar, and the financial support of the research project DaCS, Mapei AS and Norbetong AS.

Trondheim, June 2019.

Stefan Jacobsen, Kjell Tore Fosså, Guzel Shamsutdinova (ed.) and Andrei Shpak (ed.)

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PROGRAM

DAY 1 – 18 June – «Ice abrasion»

10.30 – 11.00 Registration

11.00 – 12.15 Lunch

12.15 – 12.30

Jan Arve Øverli (NTNU, Norway) – 10 min

Welcome word from a concrete group lead at NTNU

Practicalities by Guzel Shamsutdinova – 5 min

12.30 – 13.50 – Concrete-ice abrasion

Kjell Tore Fosså (Kvaerner AS, Norway) – 15 min + 5

Presentation of the DaCS Project. Ice abrasion relevance for Kvaerner AS

Anne Barker (MUN, National Research Council of Canada, Canada) – 15 min + 5

"IceWear: Improving the knowledge of surface wear and surface friction influences on the ice-induced wear of concrete structures"

Guzel Shamsutdinova (NTNU, Norway) – 15 min + 5

"Concrete-ice abrasion laboratory study"

Tatiana Uvarova (FEFU, Russia) – 15 min + 5

"Application of the hardness test method for evaluating the concrete resistance of ice abrasive"

13.50 – 14.10 ***Lab tour – Abrasion rig***

14.10 – 14.25 *Coffee break*

14.25 – 15.25 – Ice Mechanics

Knut Høyland (NTNU, Norway) – 15 min + 5

"Are ice properties required when estimating ice actions on structures?"

Arttu Polojärvi (Aalto, Finland) – 15 min +5

"Numerical ice mechanics: What is there to learn?"

15.25 – 15.30 *Break*

15.30 – 16.10 – Concrete fracture

Frank Spörel (BAW, Germany) – 15 min + 5

"Hydro-abrasive exposure and damage – appropriate concrete resistance"

Erik Schlangen (DELFT, The Netherlands) – 15 min + 5

"Micro-mechanical testing to support multi-scale modelling."

16.10 – 16.20 Closure

18.30 Dinner

DAY 2 – 19 June – «Frost resistance»

08.45 – 09.00 Coffee

09.00 – 09.15

Stefan Jacobsen (NTNU, Norway) – 10 min

"DaCS WP2 Requirements and recommendations to frost durable concrete"

Practicalities by Andrei Shpak – 5 min

09.15 – 10.15

George Scherer (Princeton, USA) – 30 min +5

Keynote lecture "Low-Temperature Strain in Air-Entrained Mortar"

Jouni Punkki (Aalto University, Finland) – 15 min + 5

"Practical challenges with air-entrained concretes"

10.15 – 10.30 Coffee break

10.30 – 11.30 Mechanisms of frost damage and modelling

Sture Lindmark (LTH / FuktCom, Sweden) – 15 min + 5

"Recapitulation of an old hypothesis on the mechanism(s) of salt frost scaling"

Matthias Müller (Weimar, Germany) – 15 min + 5

"Impact of different liquid uptake processes on salt frost scaling mechanism"

Fuyuan Gong (Zhejiang University, China) – 15 min + 5

"Towards Macroscopic Simulation of Frost Damage in RC Structures"

11.30 – 12.30 Lunch

12.30 – 13.30 Lab testing and field data. Session I

Terje Rønning (Norcem, Heidelberg) – 15 min + 5 min

"*Research needs for Frost/Freeze-Thaw assessment of sustainable concrete on European level"*

Ola Skjølsvold (Sintef (on behalf of SVV), Norway) – 15 min + 5

"Concretes with high volume fly-ash or slag; On Frost Scaling, Air Void Characteristics and Pre-Conditioning"

Andrei Shpak (NTNU, Norway) – 15 min + 5

"Frost testing of HP/HVFA concrete for severe offshore conditions"

13.30 – 13.45 *Lab tour – Frost lab*

13.45 – 14.00 Coffee break

14.00 – 15.00 Lab testing and field data. Session II

Elisabeth Helsing (RISE, Sweden) – 15 min + 5

"Frost-resistance of concrete with supplementary cementitious materials – experiences from testing and field exposure"

Ferreira Miguel (VVT, Finland) – 15 min + 5

"Assessing freeze-thaw performance of concrete - Some considerations"

Frank Spörel (BAW, Germany) – 15 min + 5

"Monitoring of the freeze-thaw attack on concrete"

15.00 – 15.20 Closure

ICE ABRASION

IceWear: Improving the knowledge of surface wear and surface friction influences on the ice-induced wear of concrete structures

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ABSTRACT
Abrasion effects upon concrete in a marine environment could, in the worst case, present a safety hazard to a structure’s load resistance. Design longevity and maintenance costs are also significant concerns. Numerous studies have been performed to examine the abrasive effects of ice on concrete, yet despite the extensive body of literature that has been developed, of field, laboratory and numerical study results, abrasion processes remain not well understood. The Memorial University of Newfoundland IceWear research program has the goal of improving knowledge of surface wear and friction influences on the ice-induced wear of concrete structures in polar marine environments.

Key words: Concrete, ice, adhesion, experiment, laboratory.

1. INTRODUCTION

Maintenance and repair of ice-worn concrete structures in marine environments are ongoing challenges. We still do not really know the relative degrees of abrasion caused by mechanical wear, freeze-thaw cycling, pore water pressure, or seawater chemical effects. Practical solutions for reducing ice-wear for large-scale applications have had marginal success rates to date. The Memorial University of Newfoundland IceWear research program has the goal of improving knowledge of surface wear and surface friction influences on the ice-induced wear of concrete structures in polar marine environments. The program is a multiyear (2017 – 2022), $2.22M CAD endeavour. Research is being executed in four categories of study: ice-concrete interaction laboratory tests; the investigation of ice-concrete contact physics; wear reduction design strategies; and modelling and analysis. A key component of the research is to develop methodologies that allow for the standardization of test samples, in order to facilitate the ease of conducting tests, their repeatability and the ability to compare results across research programs. This document outlines the program's ice-concrete interaction lab tests.

2. PROGRAM COMPONENT RESEARCH – ICE TESTS

The following section presents an overview of the IceWear program's component research, focusing upon describing the ice testing elements of the program. Fundamentally, the research program is examining both intrinsic and extrinsic elements that affect the wear of concrete by ice. The research program is using a suite of methodologies to examine ice-concrete adhesion and crushing. Freshwater ice is being used for all tests, presently.

2.1 Concrete Mix

The concrete mix design being used as the standard mix is a mid-strength concrete, 45 kPa, chosen in order to balance the need to evaluate the effects of a variety of parameters on the concrete in a timely manner, as well as represent a realistic mix design used in a marine environment (Table 1). The same mix is being used for all tests, to facilitate cross-examination of the test results. The mix design also accounts for the variety of standard concrete molds being used, using a small aggregate size, in order to accommodate 100 mm by 50 mm cylindrical molds. Finally, the mix is readily reproducible and inexpensive.

Table 1 Mid-performance concrete mix design

Component	Mix Content
Cement Factor C/F	1.2
Water-to-Binder ratio W/B	0.4
Cement (kg)	400
Silica Fume	0
Coarse Aggregate content C. Agg. (kg/m^3)	112.8
Fine Aggregate content F. Agg (<10mm) (kg/m^3)	94
Water (kg/m^3)	19.2

2.2 Wear

Wear tests were carried out early in the program. Pilot tests were performed that examined the effects of long-duration wear of concrete [1]. The goal of this research was to develop an apparatus that could address "Issues of test duration, realistic pressures and larger contact areas than previous approaches" [ibid], coupled with the ability to allow for both wet and dry experiments. The design of the apparatus was chosen based upon ten design criteria which the author determined could improve over previous experimental designs. The test apparatus is shown in Figure 1. Based upon the results, the author provided suggestions for further refinement of the apparatus design (also Figure 1).

2.3 Stick-Slip

Using a rotating concrete surface with adjustable speed and where a normal load could be applied to the ice, stick-slip behavior of ice has been examined (Figure 2). The laboratory results were used to develop a numerical model that could predict this interaction [2]. The static and kinetic friction coefficients were determined, and related to the normal load, velocity and spring stiffness of the system.

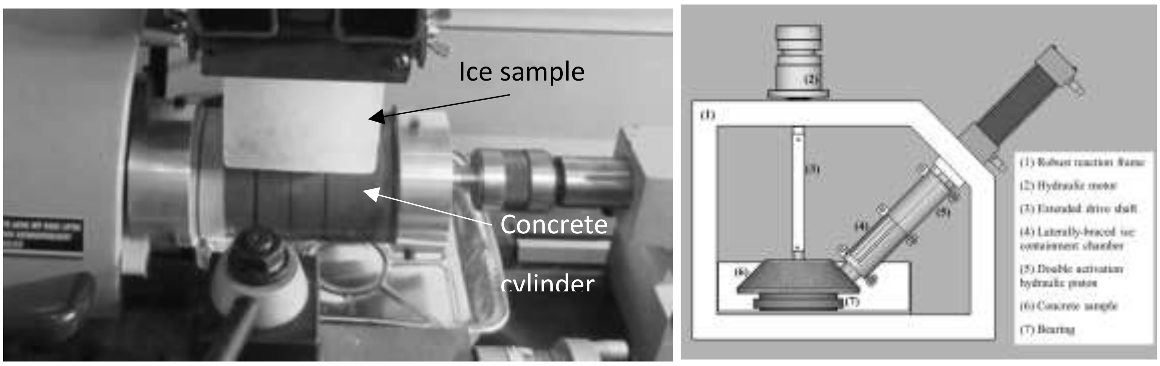

Figure 1 – Test configuration for long-duration, high load wear tests. Left: test apparatus. Right: suggestion for refined apparatus design. Image source: [1].

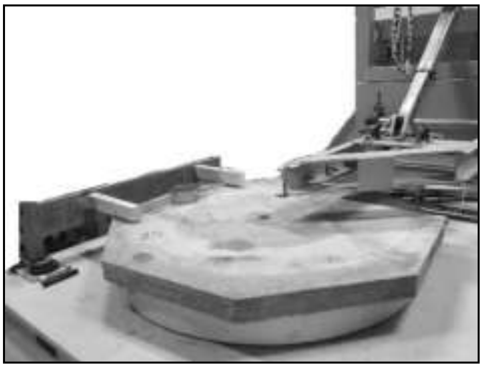
Figure 2 – Test configuration for stick-slip tests. Image source: [2].

2.4 Crushing

In order to advance the refined test apparatus shown in Figure 1, a simplified version of the ice delivery system was constructed for preliminary tests (Figure 3). An ice sample is crushed onto concrete, with varying speed and gap height. It was observed that at low speed, the load developed in four phases: initial saw-toothed loading; linear load build-up where the ice sample underwent physical changes; a plateau of the load as extrusion of the ice began; and a final drop in the load due to an associated drop in friction.

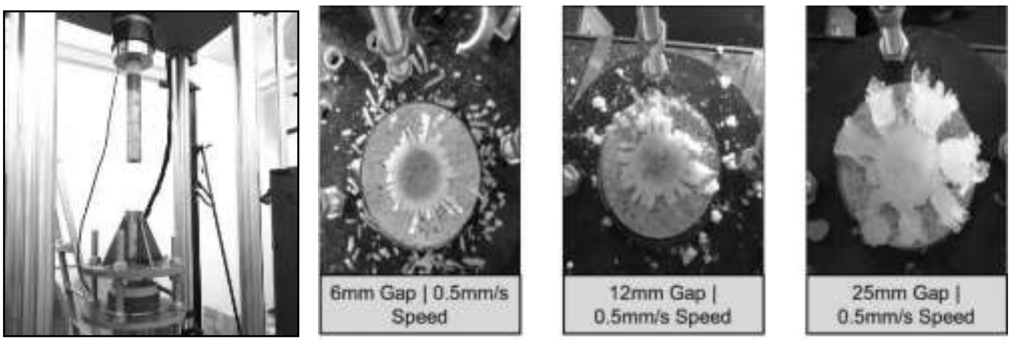

Figure 3 – Test configuration for crushing extrusion tests (left) and patterns of extrusion for tests at low speed, with different gaps.

2.5 Direct Shear

The direct shear apparatus is a modified friction table. Originally designed for friction testing of paint on model marine hulls, the table has been modified by installing a 300 lb S-type load cell. In order to configure the table for adhesion tests, the concrete samples are clamped in place in a jig and braced against moments (Figure 4). The jig can accommodate four samples at one time.

Figure 4 – Test set-up for direct shear tests, using a modified friction table.

2.6 Double Shear

A double direct shear apparatus is in the process of being fabricated. A simplified version is shown in Figure 5. The final apparatus will accommodate standard-sized concrete and ice cylinders, and will be mounted in a Universal Testing Machine (UTM).

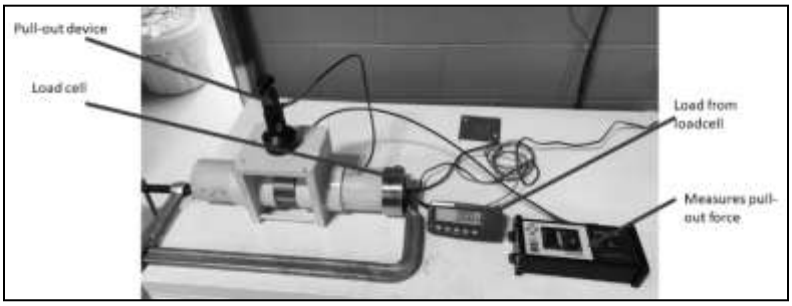

Figure 5 – Simple design concept for double-shear adhesion tests.

2.7 Tension

Tests are underway to examine the adhesion strength between ice and concrete in tension. Freshwater ice with a diameter of 229 mm and a height of 76 mm is frozen and held in a clamping system. The concrete sample is 10 cm by 5 cm. An aluminum clamp holds the concrete sample, and from this is connected to a load cell. Preliminary tests are at -15°C, and are conducted after the ice has been loaded with an 18 kg weight for 24 hours. Figure 6 shows the test set-up.

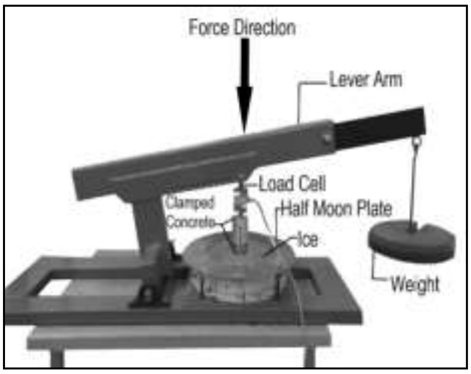

Figure 6 – Test configuration for adhesion strength in tension.

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Ice abrasion testing of high performance concrete for offshore structures

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ABSTRACT

Offshore concrete structures exposed to drifting sea ice have abrasion of the concrete surface caused by the mechanical contact between ice and concrete in the order of 0.1 to 1 mm per year. The concrete-ice abrasion laboratory at NTNU, and results of our recent research of the laboratory simulation of concrete-ice abrasion, showed average abrasion depths of 0.01–0.35 mm for high-performance concrete after 3 kilometres of sliding ice, a severe-mild wear-transition, that abrasion is related to cutting of peaks, formation of valleys, aggregate protrusion by wear of ITZ. The strength – abrasion relation proposed by Huovinen was less clear due to the severe-mild transition.

Key words: Concrete, ice, abrasion, ITZ, surface topography.

1. INTRODUCTION

Research on the concrete-ice abrasion problem has to a large extent been done experimentally in various laboratories in North-America, Europe, Russia and Japan. The experimental methods have varied for different laboratories, but so far most work has been based on the sliding interaction between ice and concrete.

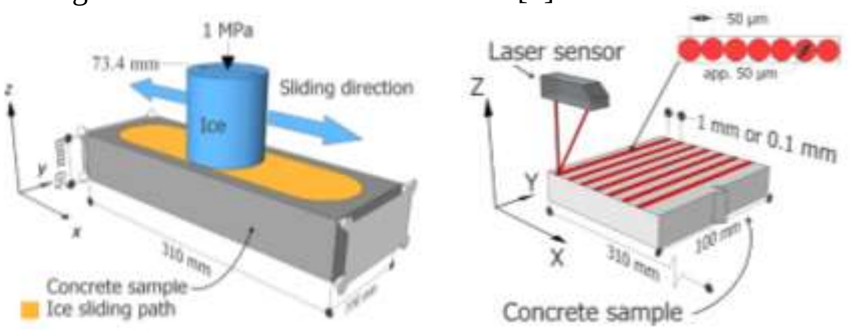
The scope of this research has been to investigate how the tribology parameters friction, surface roughness, topography and wear particle characteristics are affected by ice sliding on various types of concrete and surfaces. The laboratory and test procedures (low-temperature wear machine, ice making, laser scanning of wear) were further developed and a series of high performance off-shore type concretes with different surfaces were made and studied. This paper presents a few excerpts of a PhD project. The entire work is reported in the thesis [1].

2. EXPERIMENTAL SET-UP AND MATERIALS

2.1 Concrete-ice abrasion lab: abrasion rig and laser scanner

The experiments took place in a cold room −10 °C. The lab performs sliding of fresh-water ice samples on concrete surfaces with average pressure of 1 MPa and average sliding velocity of 0.16 m/s, the scheme of the concrete-ice abrasion test is in Figure 1 (a). The effective sliding distance for each concrete sample was 3 km. The temperature of the concrete sample is controlled through the aluminium heating plate below the concrete sample. The heating plate prevents icing on the concrete surface. The temperature of the concrete surface during the test is approximately −2 °C which is sufficient to keep the surface ice free during ice movement. More detailed description of the abrasion rig is given in [1].

The abrasion of concrete was measure with laser scanner that had vertical accuracy of measurements of 16 µm, and gave a surface mesh with measuring point distance 1mm and 50 µm in x- and y- direction respectively. The scheme of the laser scanning is shown in Figure 1 (b).The new laser scanner allows topography studies on different concrete surfaces. The laser scanning method is described in details in [1].



(a) (b)

Figure 1 – (a) The scheme of concrete-ice abrasion test, (b) the scheme of the laser scanning.

2.2 Materials: concrete and ice

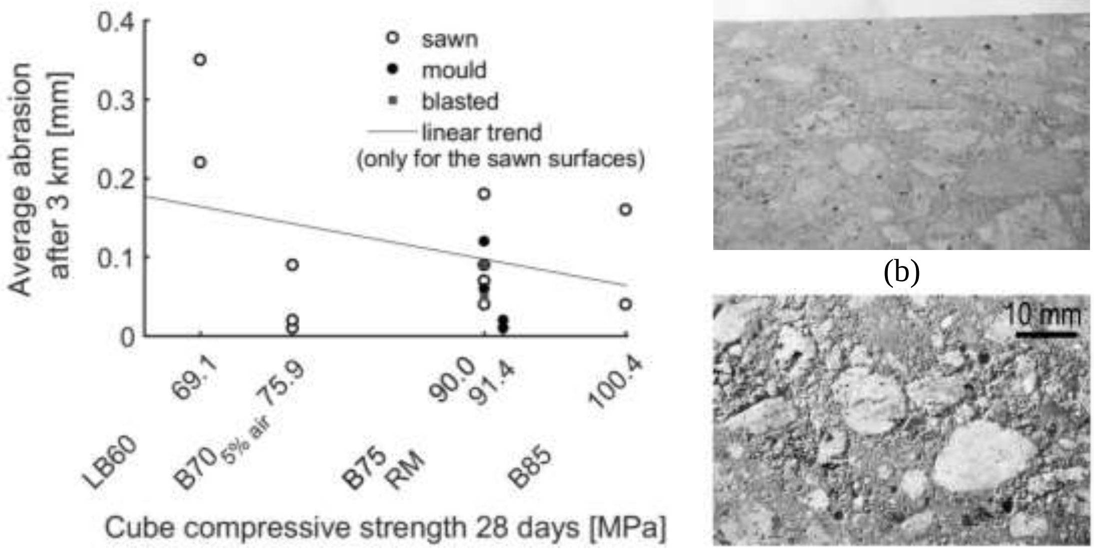
The concrete samples used for the concrete-ice abrasion test were slabs measuring 100 × 310 mm and 50 mm high, which were cured in water at + 20 °C for 11 months before the experiments.

High-performance type concrete for realistic offshore conditions were tested with workability and compressive strength similar to those of offshore concrete. Five concrete mixes were investigated: two Normal Density (ND) concrete mixes with different compressive strength (B75 and B85), a frost durable concrete mix with air entrainment (B70-5% air), lightweight aggregate (LWA) concrete with porous coarse aggregate (LB60), and repair mortar (RM). Besides the quality of the concrete mix the type of concrete surface was varied: sawn, moulded and sand-blasted during the concrete-ice abrasion tests. The test program included 17 samples (two parallel samples of each type).

In the experiments the concrete surface is abraded by fresh-water ice produced by unidirectional freezing. The ice samples had a cylindrical shape with a diameter of 73.4 mm and a height of 180 mm. The density of the ice is 917.0 kg/m^3 and porosity is less than 0.1%.

3. RESULTS AND DISCUSSION

The abrasion of tested concrete samples after 3 km as function of strength is shown in Figure 2. The highest abrasion was found for the sawn lightweight concrete mix and the smallest for the mould sample of the repairing mortar. The protrusion of both lightweight and normal density aggregate was observed on the sawn surfaces. Figure 2 (b,c) shows sawn surface B75 before the test and after with protruded granite aggregate. This happens presumably due to microscale abrasion starting in the interfacial transition zone (ITZ). The abrasion rate of lightweight aggregate is greater than that of normal-weight aggregate.



(a) (b) (c)

Figure 2 – (a) Concrete strength vs average abrasion after 3 kilometres of sliding distance (b) sawn surface B75 before the test, (c)sawn surface B75 after 3 km test.

The abrasion rate was not constant and decreasing with sliding duration. The abrasion rate of tested concrete samples as function of strength is shown in Figure 3 for each sliding kilometer. The maximum wear rate was found during the first kilometer of sliding and thereafter dropping down. This characteristic process of wear rate transition from severe to mild is discussed in [1]. To find the reason for it will require more research but a literature search in [1] revealed that such behavior has been observed also on other materials and wear types.

Topography studies of the laser scan data of the different abraded concrete surfaces showed that it could be primarily depicted as a process of valley formation. The valleys originate from air voids opening and cutting of peaks. Contacts between larger ice-asperities and smaller concrete-asperities can induce contact tensile stress in concrete limited by the ice strength and this is sufficient to fracture the concrete [1]. The roughness of concrete surfaces increases from 0.01 – 0.04 mm up to 0.08 mm.

Another interesting feature observed during testing was that the consumption of ice seemed to vary a lot. This project was aimed to perform concrete-ice abrasion experiments of different surfaces and materials at identical conditions. However, the appearance of ice spallation affected the ice consumption and brought some deviation in test results. During a test of one single concrete specimen, sometimes the consumption of ice samples varied between 10 and 45. This was caused by varying degrees of spallation of ice during the test and increased abrasion depth and coefficient of friction [1].

It was found that the coefficient of friction (COF) was not clearly correlated to the abrasion. It varied in the range 0.005 – 0.013 and 0.008 – 0.085 for kinetic and static COF respectively. Presumably, the stable and low value of the coefficient of friction can be explained by a thin water film in the contact area, which works as a lubricant. The lubricant can support the load if the thickness of the water film is greater than the surface roughness.

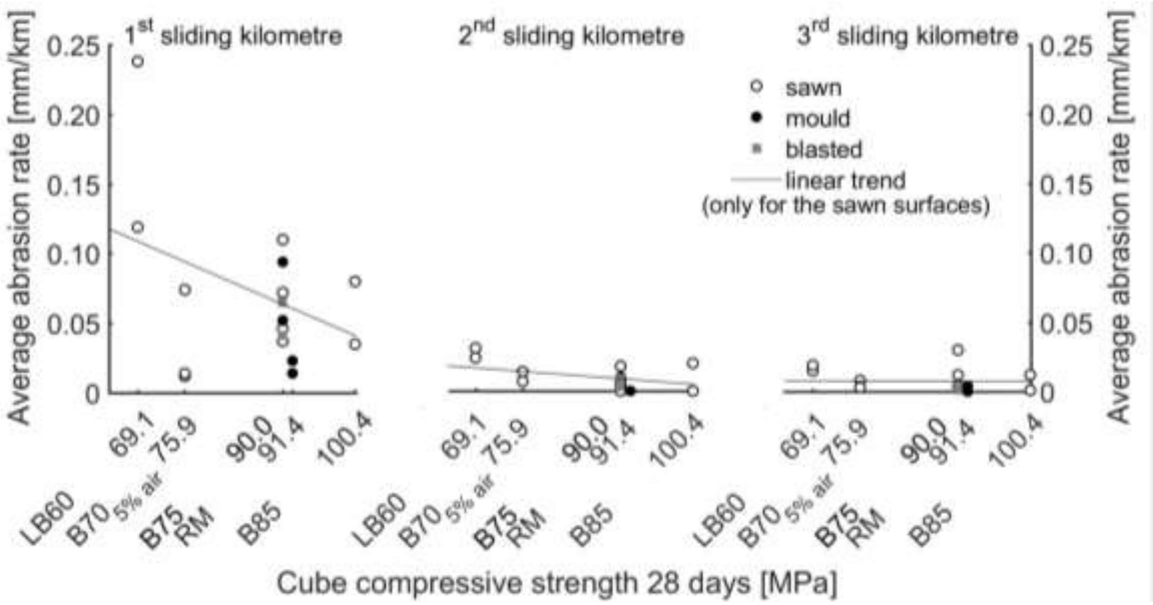


Figure 3 – Concrete strength to wear rate relation for each kilometre of sliding distance

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Application of the hardness test method for evaluating the concrete resistance of ice abrasive

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ABSTRACT
The article substantiates the prerequisites of the hardmetry use for the concrete resistance evaluation to ice abrasion as a method of obtaining an independent direct quantitative resistance index that is correlated with the concrete destruction processes during ice abrasion.

The applying possibilities and adapting existing hardmetry methods testing for concrete as a brittle heterogeneous material have been estimated. The rationale for the optimal method, procedure and algorithm for estimating the concrete hardness is discussed, and the tests results of specimens that prove the determining possibility the integral concrete hardness based on a statistically reliable hardness evaluation of each of its structural elements are presented.

Key words: hardmetry, ice abrasion, concrete, testing method.

1. INTRODUCTION

To predict and regulate the concrete resistance to ice abrasion, it is proposed to identify the relationship between the integral hardness index of the concrete surface and the found depth values of ice abrasion, considering the variation in speed, temperature and ice pressure, performed in the period from 2007 to 2014 on a special unit for studying ice abrasion effects on various types of building materials.

At present, a significant amount of scientific research has been done on the first part of the problem of ice abrasion [8, 9, 10]. As a result, considerable experience has been accumulated in modeling ice impacts and predicting the structures behaviours in various design situations. Experimental studies of the ice abrasion process made it possible to obtain empirical models of ice abrasion concrete resistance [6, 7], reflecting the concrete wear process intensity depending on the parameters of ice impacts.

However, the problem of the abrasion associated with the study and the behaviour forecasting of the materials themselves is currently not being given due attention. Accumulated quantitative data only reflect the concrete strength relationship with the ability to withstand ice abrasion. In particular, for concrete it is assumed that in order to provide sufficient resistance to ice abrasion, it is necessary to regulate its strength (at least 70 MPa) [2]. The standard concrete mechanical characteristics, including strength, are not direct quantitative characteristics of abrasion resistance, since in the testing process, compression failure and bending are reproduced. In the process of abrasion by ice, concrete is destroyed as a result of fundamentally different phenomena. To regulate the ice abrasion resistance, to predict its behaviour, an adequate quantitative resistance index is necessary, on the one hand, correlated with the processes of concrete destruction under ice abrasion, on the other - independent of test conditions and parameters of ice impacts, but determined by the composition characteristics and the material structure.

An analysis of the existing methods complex for evaluating mechanical testing of materials has made it possible to pay attention to hardness as a promising method for solving this problem. However, existing hardness methods have been developed and widely used to evaluate the hardness of metals, i.e. homogeneous materials with their characteristic elastoplastic properties [1, 4]. Concrete is a brittle heterogeneous material, so there is a problem of adaptation for concrete of solid-metal methods used for metals.

In this paper, it is proposed to consider the hardmetry possibilities as a method of obtaining a direct quantitative indicator of the concrete resistance to ice abrasion. There are following reasons for this.

2. MATERIALS, EQUIPMENT, MEASUREMENT PROCESS

Analysis of the ice abrasion process mechanism and the hardness testing process mechanics allowed us to conclude that the hardness characteristics are most adequately correlated with the material destruction mechanism and the ice abrasion loads parameters. Based on this, hardmetry is considered as a method for assessing the concrete resistance to ice abrasion, and hardness is taken as a direct quantitative measure of this resistance.

To study the static hardness of the concrete samples surface, the Qness Q150A+ equipment was used [5]. The hardness measurement in this equipment is carried out automatically in the prescribed trajectory.

As a result, to assess the concrete hardness, the Rockwell method is the best one. The rational parameters of the measurement process are: a normative load of 15 kgf, use of an indenter with a ball tip ∅1.5875 mm, an interval between measuring points of 2.5 mm.

Statistical analysis of the verification tests data has shown the hardmetry method reliability and the possibility of its use for assessing the concrete surface integral hardness and its individual structural elements.

3. DETERMINING THE RELATIONSHIP BETWEEN HARDNESS AND RESISTANCE TO ICE ABRASION CONCRETE

The problem of material resistance to ice abrasion is solved by conducting experimental studies of various materials, on the basis of which the empirical dependence of the intensity of ice abrasion on the main parameters causing the material abrasion (contact pressure, ice temperature, length of the interaction path) is determined. The empirical model of ice abrasion is unique for each material tested (with its corresponding physical and mechanical properties), and characterizes the material's ability to resist ice abrasive effects and reflects the physical processes during the abrasion of the material with ice.

The goal of laboratory testing of the resistance of concrete samples to ice abrasive effects is to obtain empirical dependences of the intensity of ice abrasion of concrete samples of three compositions.

The main parameters monitored during the abrasion test are: contact pressure, ice temperature and air temperature, abrasion depth, abrasion path (relative displacement path when the concrete sample interacts with the ice block during the test).

As a result of statistical processing of laboratory tests, empirical dependences of the intensity of ice abrasion of concrete of 3 compositions were obtained. The research results are shown in Figure 1.

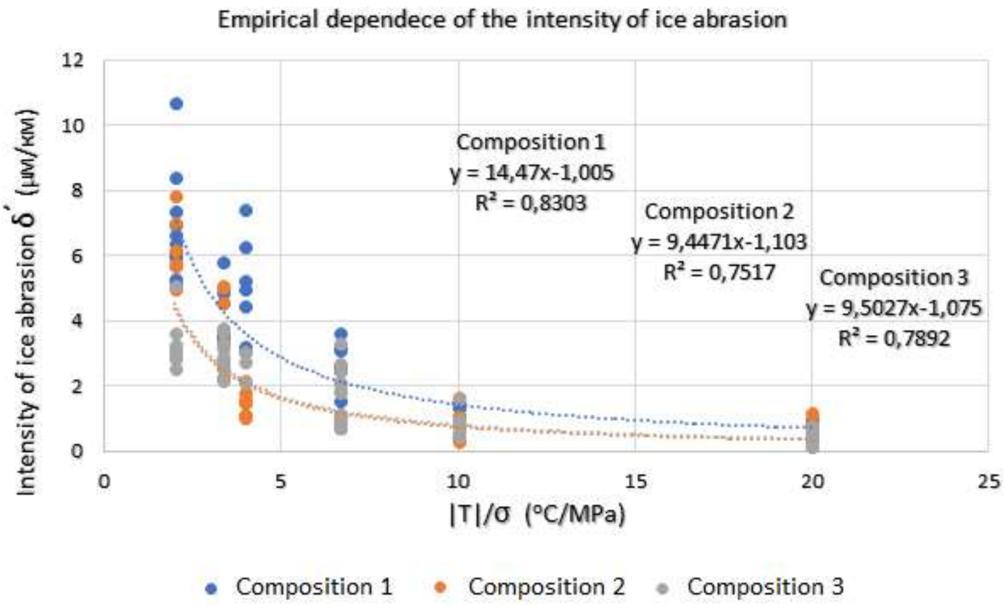

Figure 1 - Empirical dependences of the intensity of ice abrasion for concrete with Compositions 1,2,3.

Empirical dependences of the intensity of ice abrasion for concretes with compositions 1, 2 and 3 have the following form:

Concrete with a composition of 1
$\delta = 14.47 (| T | / \sigma) -1.005$ (1)

Concrete with a composition of 2
$\delta = 9.4471 (| T | / \sigma) -1,103$ (2)

Concrete with a composition of 3
$\delta = 9.5027 (| T | / \sigma) -1.075$ (3)

In accordance with the concept of determining the empirical dependences of the intensity of ice abrasion, the general equation of the model of intensity of ice abrasion can be written as:

$$\delta' = K1\,(|T|/\sigma)^{K2} \quad (4)$$

In order to determine the effect of concrete hardness on its resistance to ice abrasion, in this work functional dependences of coefficients K1 and K2 on the hardness of concrete were obtained.

The results of statistical data processing of concrete hardness and approximation coefficients of the empirical dependences of the intensity of ice abrasion of concrete were obtained by the functional dependences of coefficients K1 and K2 in the model of intensity of ice abrasion formula (4), which are presented in table 3.

Table 3 - Functional dependences of the coefficients in the ice abrasion intensity model

	K1	K2
Aggregate hardness, Ha	K1=0,3163Ha-17,768	K2=0,006Ha-1,6105
Cement stone hardness, Hs	K1=-0,2217Hs+24,002	K2=-0,0037Hs-0,8461
Total hardness, H	K1=-0,3452H+35,798	K2=-0,006H-0,6319

Table 4 - Correlation analysis of the relationship of hardness and physico-mechanical properties of concrete.

Coefficients / Hardness	Aggregate hardness Ha	Cement Stone, Hs	Total Hardness H
K1	0,21056148	-0,926707013	-0,74048974
K2	0,22870433	-0,884654247	-0,736304465

This approach, namely the determination of the functional dependences of the coefficients K1 and K2 in the model of the intensity of ice abrasion in accordance with formula (4) on the hardness of concrete, allows us to estimate the effect of individual structural elements of concrete on the intensity of ice abrasion.

From table 4 it can be seen that the hardness of the cement stone has a high correlation with the coefficients K1 and K2. The hardness of the aggregate is not interrelated with the coefficients. The total hardness shows the average relationship with the coefficients, this is due to the constant hardness among the samples.

Based on the result of the analysis, it can be concluded that tests on the hardness of concrete can be considered as an alternative method, compared to standard methods, tests of concrete on the resistance of ice abrasion.

According to the obtained equations (table 3) according to the formula (4), nomograms of the dependence of the intensity of ice abrasion on hardness were built (Figure 2)

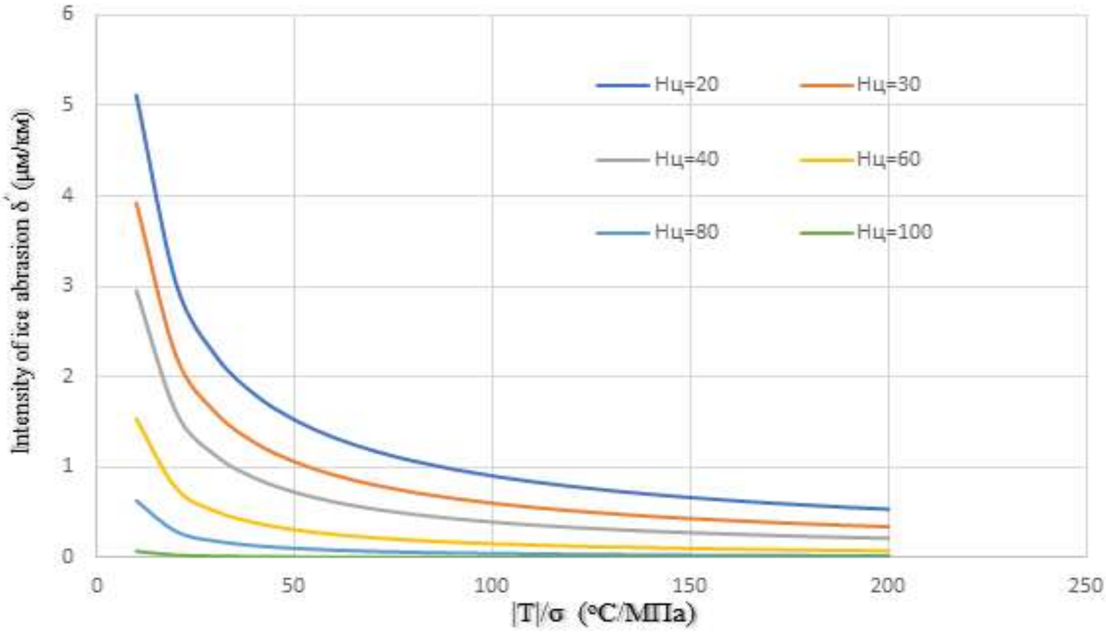

Figure 2- Nomogram of concrete resistance to ice abrasion depending on the total hardness of the sample

These graphs show the dependence of the intensity of ice abrasion on various indicators of concrete hardness and interaction parameters (temperature to pressure). Lines of different colours represent the different hardness of concrete.

To assess the resistance of the ice abrasion material, it is necessary to perform a hardness test of concrete and, based on the test results; determine the intensity of ice abrasion by nomograms.

CONCLUSION

The tests of the hardness of various compositions of concrete allowed a comparative assessment of the hardness and resistance of ice abrasion. As a result, the dependence of the intensity of ice abrasion on various indicators of concrete hardness and interaction parameters (temperature to pressure) was established. In the article a nomogram was proposed for a preliminary assessment of the resistance of concrete to ice abrasion by hardness.

The research development is expected in the following areas:

- obtaining quantitative estimates for the ratio of structural elements hardness of various classes concrete;

- substantiation of the requirements to the method for determining the integral hardness index of concrete taking into account the heterogeneity of its structure;

- analysis of correlation relationships between the integral hardness index of concrete various types and the values of their ice abrasion, depending on the ice impacts parameters;

- analysis of correlation relationships between the integral hardness index of concrete various types and its standard mechanical properties;

- the structure modelling and justification of the requirements for the concrete composition with the ice abrasion resistance optimum value.

The results obtained perspective, in our opinion, consists in introducing the hardness index as an independent indicator in the overall complex of the concrete properties assessments, which must be regulated to ensure its durability when operating hydraulic structures in arctic conditions.

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Numerical ice mechanics: What is there to learn?

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ABSTRACT
Understanding sea ice behaviour and ice loads is important. Ice failure processes are complicated and one effective way to study ice loads is to simulate the failure process and to study the properties of the process. This paper briefly discusses the rationale for numerical ice mechanics and the requirements for numerical models based on recent results from virtual ice load experiments.

Key words: ice loads, numerical ice mechanics, ice-structure interaction, combined finite-discrete element method

1. INTRODUCTION

Understanding sea ice behaviour and ice loads is important as it can be used to design safer and more sustainable Arctic offshore structures and vessels, such as used for marine transportation, offshore wind energy, and offshore drilling. The ice loads arise from a complex and stochastic ice-structure interaction process, occurring as ice, moved by winds and currents, fails against an offshore structure [1–3]. Figure 1 illustrates a simulated ice-loading process where a floating ice sheet is moving against an inclined marine structure and fragments from a solid sheet into ice rubble pile, which will affect the rest of the process.

Due to the complicated ice failure process, an effective way to study ice loads is to simulate the process and to study the properties of the process [2]. Recently it has, thus, become popular to develop rather complicated numerical tools to model ice loading processes. However, the design of offshore structures still often relies on simplified models and empirical load estimates. The true value of the numerical models resides in careful analysis: It is the analysis that yields the insight on ice loads and related mechanical phenomena, not just the simulations tools themselves.

This paper briefly discusses some recent findings that are based on numerical ice mechanics. The findings are discussed together with the requirements that they set on numerical models. The aim is to discuss the use of numerical tools in ice mechanics – what can we learn from them and how?

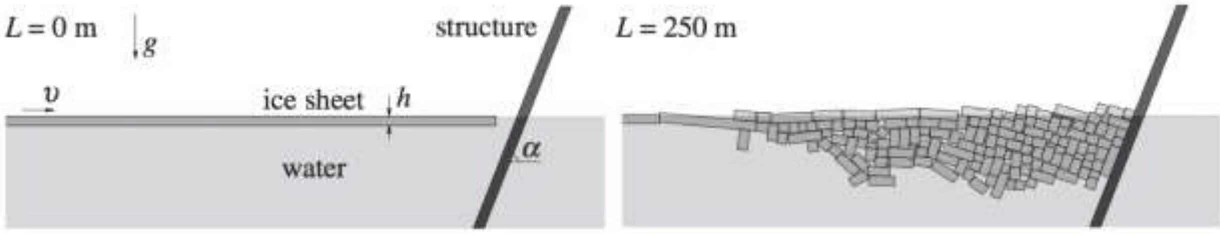

Figure 1 – Two snapshots from different stages of a DEM simulation having an intact ice sheet moving from the left against an inclined structure on the right, further failing and forming a rubble pile [5]. L refers to the amount of ice pushed against the structure.

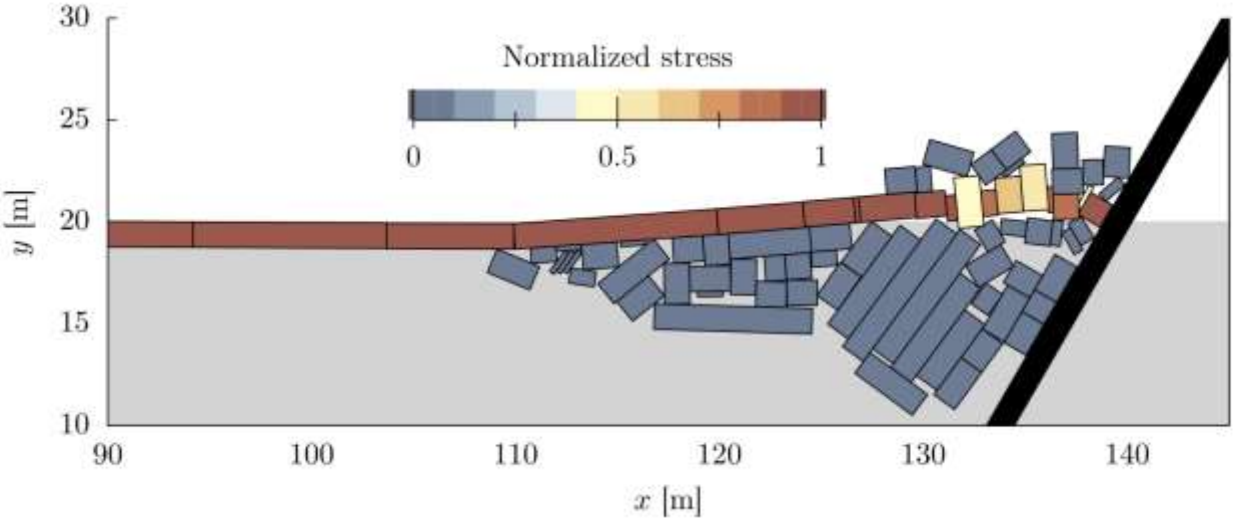

Figure 2 – Force chain, here a chain-like feature consisting of adjacent highly compressed ice blocks, transmitting an ice load from the intact ice sheet to the structure. Stress measure used here is the so-called block stress, and average compressive stress in an ice block.

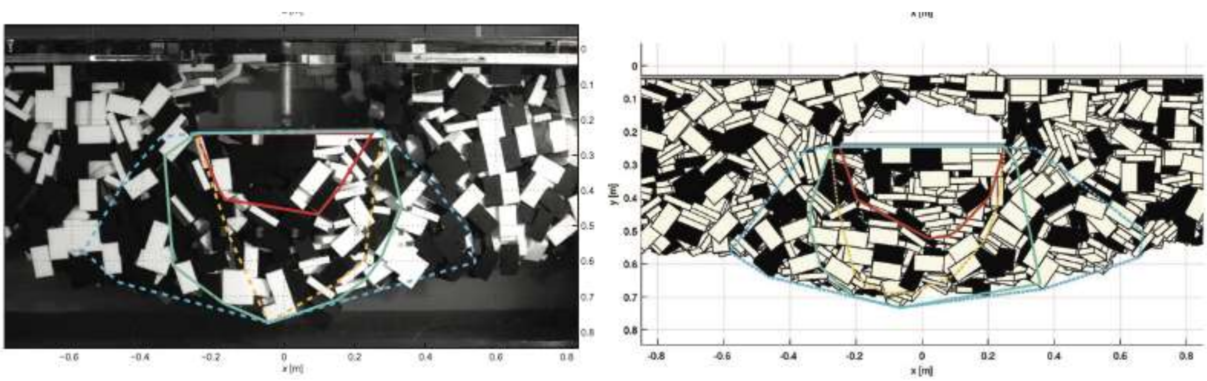

Figure 3 – Comparison between a punch-through experiment (left) and simulation of it (right) on artificial ice rubble [6]. Coloured lines illustrate the amount of deformation within the rubble.

2. MODEL REQUIREMENTS

In order for a model to predict reliable results for analysis of ice loads, there are few conditions that it must meet. One of the most important ones is that the modelling of usual ice action scenarios of engineering interest requires accounting for the individual ice features forming in the interaction process. This requirement can be met when using discrete element method (DEM) simulations, which have been utilized widely in ice mechanics studies [4]. Without this feature, a model cannot predict several of the important phenomena related to ice loads, which makes the applicability of such model for use in ice mechanics questionable.

The above-described requirement can be understood by looking into the snapshots of simulations on ice-structure interaction in Figures 1 and 2. From Figure 1 it appears obvious that the ice rubble blocks breaking of the ice sheet have an effect on the process as they interact with each other, the incoming ice sheet, and the structure. The significance of modelling individual blocks is even more pronounced in the case of Figure 2, which shows a force chain transmitting the load from the intact ice sheet to the structure [4]. Models that do not account for individual ice features render out the force chains and may not correctly describe peak ice load events.

Modelling each ice block is also needed in a case of modelling just ice rubble piles. Figure 3 shows a snapshot from a 3D DEM simulation on artificial ice rubble [5]. While this 3D model of ice rubble may appear complicated, the conclusions drawn from them are strikingly simple: Very simply parameterized DEM models give results, which when using continuum models, can be only reproduced by using unnecessary complicated material models. In addition, here the numerical model importantly reveals, that ice rubble may not always be correctly modelled using continuum models.

Numerical experiments can also be used for studies on parameter effects on ice loads, as the simulations allow full control on the ice and structure parameters. This is never the case in full- or not even in the model-scale experiments. The analysis by Ranta et al. [6] gave insight into peak ice load data in a simulated ice-structure interaction process by showing that only few ice parameters have statistically significant effect on ice loads on inclined structure. The most important are the ice thickness and the inclination angle of the structure, but their importance changes during the process. While being an interesting result itself, this result also sets two major requirements for any ice load model: reliable models should have the ability to account for (1) long interaction processes and (2) the stage of the interaction process.

The importance for the chosen modelling tool to fulfill the last two requirements is further highlighted by Ranta et al. [8, 9], who studied the evolution and the scatter in the peak ice load data. Ranta et al. [8, 9] demonstrate that the numerical ice mechanics gives powerful tools for thorough studies on ice mechanics. The interaction processes are very sensitive to their initial conditions and, thus, any experiment on these processes must be repeated several times. With numerical tools, repeating experiments with small perturbations can be performed with relative ease in order to produce vast amounts of ice load data for statistical studies.

3. CONCLUSIONS

In order to learn from the simulations on ice loading processes, careful analysis and models fulfilling at least the above-described requirements are needed. Once the requirements above are met, studies on ice load statistics and physical mechanisms related to peak ice loads can be carried out. Even if not discussed here, the engineering studies on mechanical behavior of ice may, for example, focus on the limits for peak ice load values on structures [6, 11].

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Micro-mechanical testing to support multi-scale modelling.

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ABSTRACT
This work presents recent developed micro-scale universal testing approaches on cement paste which are designed for the calibration and validation of the micromechanical models. Using different indenter tips, various loading situations were achieved by a nanoindenter. Through a so-called volume averaging up-scaling approach, the outcome from the micromechanical simulation can be further used as input for the fracture modelling at coarser scale. An example about a miniaturized three-point bending test is given, which validates the developed up-scaling approach for the multi-scale fracture modelling.

Key words: Cement paste, micro-scale testing, lattice fracture model, multi-scale modelling.

1. INTRODUCTION

Concrete is fairly heterogeneous ranging from nanometres to millimetres. Therefore, multi-scale analysis of concrete requires validation and determination of input parameters at all length scales. Such experimentally validated multi-scale modelling scheme can be used to link the microstructure to the engineering properties and helps design new materials with desired properties. As micro-scale is the starting point in the multi-scale models, a fundamental understanding of the micromechanical performance is essential for the accurate predictions of multi-scale fracture performance of concrete.

The fracture test at the micro-scale has long been recognised as a challenging task. This is mainly because of the fact that the conventional fracture test in the laboratory lies in the range of a few centimetres in general, which is not capable of testing the material volume smaller than a few millimetres. The instrument that can handle the small sized specimen and record accurately the load-displacement response during the test is still rare. Furthermore, there remains a challenge on the small sized specimen's preparation of cementitious material as the microstructure of such material is sensitive to the environments e.g. temperature and humidity. Therefore, to accomplish the experimentally validated multi-scale modelling scheme, there remains an essential need of developing more advanced techniques for preparation and fracture measurements of the micro-scale sized specimens.

2. MICRO-SCALE TESTING AND MODELLING

100 µm hardened cement paste cubes were made by precision cutting, grinding and micro-dicing. A naoninddenter was instrumented for the universal mechanical test. As developing a quantitatively sound model requires combined experimental and numerical research under a wide variety of loading situations, various types of indenter tip were used to fracture the specimens, see Figure 1 [1, 2]. Microstructure of the micro-cube was determined by X-ray computed tomography and analysed by a discrete lattice fracture model, see Figure 2. The Berkovich loading was used for the model calibration, while the wedge tip splitting and flat end tip compression tests were further used as validation.

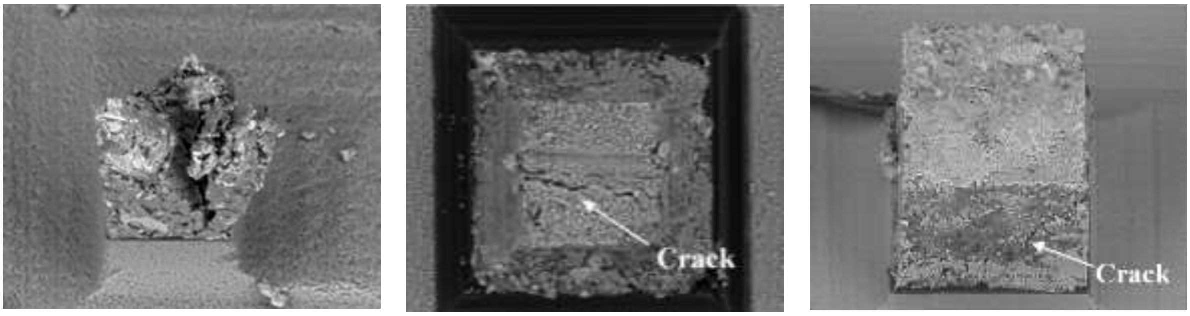

Figure 1 – Experimental observation of 100 µm cube fractured by different tips. Left: Berkovich tip. Middle: wedge tip. Right: flat end tip.

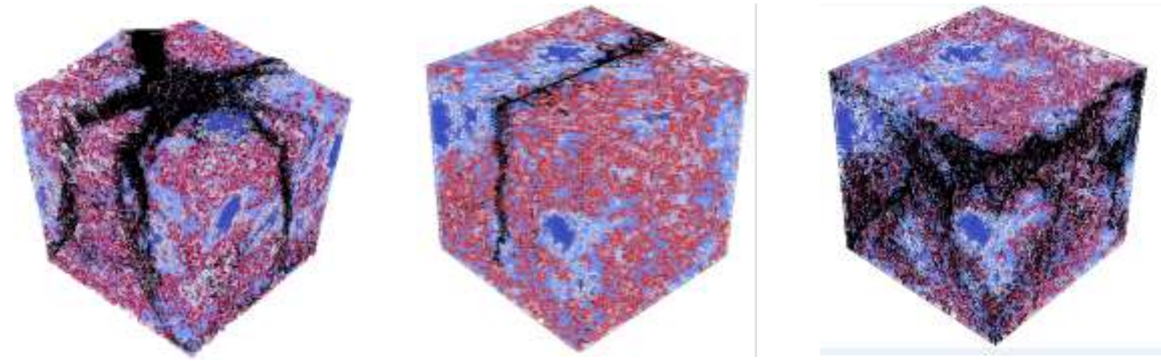

Figure 2 – Modelling results of 100 µm cube fractured by different tips (black-crack). Left: Berkovich tip. Middle: wedge tip. Right: flat end tip.

3 MULTI-SCALE MODELLING AND VALIDATION

As shown in Figure 3, cement paste beams with a cross-section of 500 µm × 500 µm were fabricated using a micro dicing saw and subjected to bending using the nanoindenter [3]. Simultaneously, fracture behaviour of the same size specimen was predicted by a microstructure informed multi-scale lattice fracture model in which input microstructures were obtained by X-ray computed tomography at two scales (micro and meso). Volume averaging up-scaling approach was used to relate the material's mechanical behaviour at meso scale with the microstructures at lower scale, see Figure 4. Figure 5 shows good agreements between experimental and numerical results in terms of flexure strength and modulus.

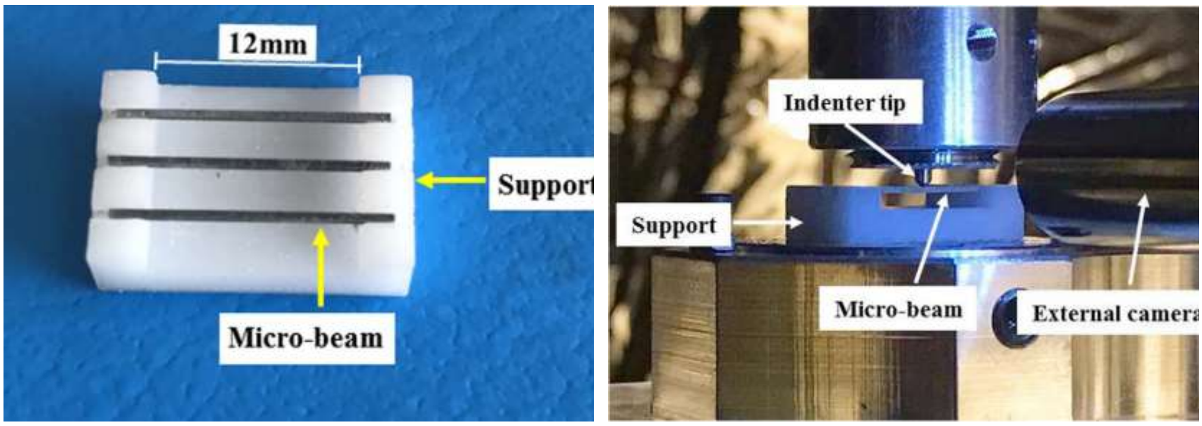

Figure 3 – Miniaturized three-point bending test. Left: micro-beams on the support. Right: testing setup.

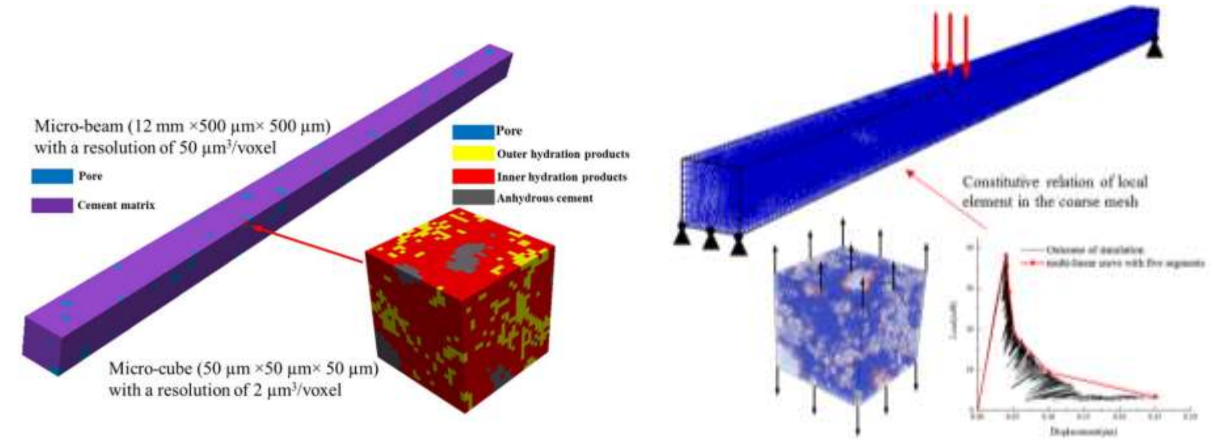

Figure 4 – Schematic view of multi-scale modelling scheme. Left: two levels of material structures. Right: up-scaling constitutive relation from micromechanical modelling.

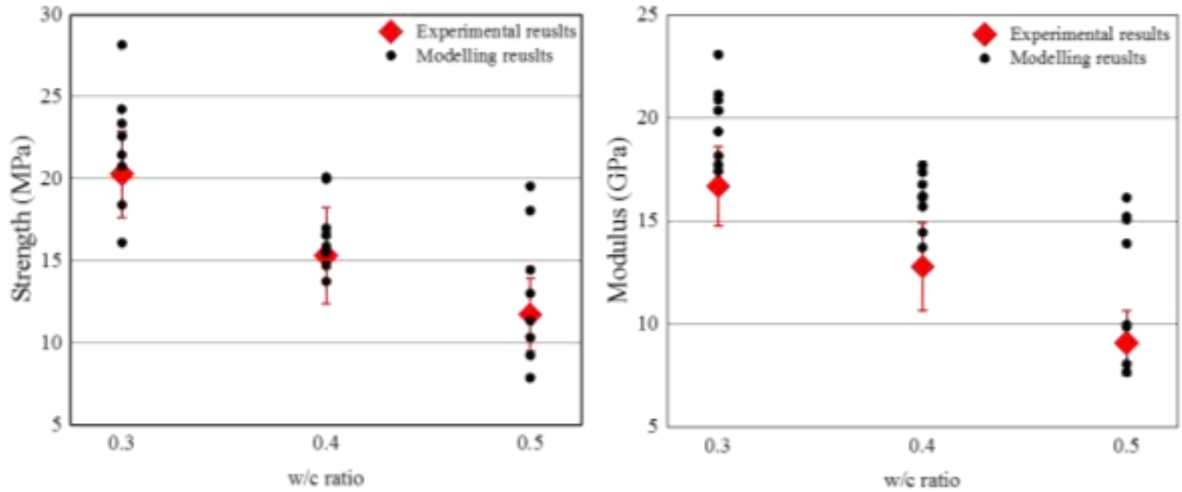

Figure 5 – Comparison between modelling and experimental results. Left: strength. Right: modulus.

4. CONCLUSIONS

Micro-scale testing has been successfully developed and implemented for the calibration of multi-scale modelling. This enables the multi-scale model has fully predictive capabilities at meso-scale.

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Hydro-abrasive exposure and damage – appropriate concrete resistance

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ABSTRACT
In Germany currently about 10 percent of the weirs under the responsibility of the Federal Ministry of Transport and Digital Infrastructure reveal damages caused by hydro-abrasive exposure which impair the serviceability or load-bearing capacity. Unlike as for other exposure such as freeze-thaw attack there are no regulations in European standards which give requirements to assure a sufficient durability against hydro-abrasive exposure. For this reason the BAW has initiated a research project with the aim to classify the exposure of hydro-abrasive attack on concrete structures and to define concrete requirements for a sufficient durability.

Key words: Concrete, hydro-abrasion, simulation, performance tests, aggregate properties.

1. INTRODUCTION

Concrete damage caused by hydro-abrasive exposure is a special topic which is only relevant for certain water exposed structures. International experience on the erosion damage is given in [1] and [2]. In Germany data on abrasion erosion damage of hydraulic structures of the Federal Waterways and Shipping Administration recently has been gathered by an evaluation of the data of the Structural Inspection of the locks and weirs. According to the BAW Code of Practice "Classifying Waterway Construction Damages" [3] the damage is rated on a scale from 1 – 4 (1: good condition, 4: critical condition). Severe abrasion erosion damage (damage rating 3 or 4) was observed on about 10 % of the weirs and 5 % of the locks [4]. The great variety of damage shows Fig. 1.

Figure 1 – Abrasion erosion damage of different degree

2. HYDRO-ABRASIVE EXPOSURE

To design concrete resistance against hydro-abrasive exposure it is essential to know which exposure is expected. Concerning hydro-abrasive exposure not many publications were available [2, 5, 6]. Flow velocity and information on the abrasive load are important parameters to assess the intensity of the exposure as these parameters have a big influence on the abrasion damage [7-10]. As for hydraulic structures like locks or weirs no information was available investigations were conducted by the Technical University of Dresden by order of the BAW [11]. Additional investigations in cooperation with the hydraulic engineers of the BAW were conducted on a severely damaged weir [12]. A Summary of the results was presented in [4, 13]. As a result 3D-hydronumerical-simulations revealed flow velocities with typical ranges on weir sills up to 3.5 m/s, at the stilling basin ground and the ground sill up to 7 m/s respectively 5.5 m/s and up to 6 m/s near strongly damaged baffle blocks. According to [14] these flow velocities are sufficient to move abrasive loads with particle diameters of more than 0.5 m. The abrasive load which was found in structures had diameters up to about 0.3 m.

The period of presence of abrasive material in the structure and its particle size is essential to evaluate the impact intensity. An assessment of the period of presence of abrasive material in the structures currently is not possible as this depends on the flow conditions in the structure and the availability of abrasive material. A classification of the intensity of abrasive impact on concrete structures in the sense of exposure classes presently only seems possible by an assumption of relevant flow velocities and an estimation of the probability of the presence of abrasive load in different parts of a structure.

Furthermore the results concerning the hydro-abrasive exposure of hydraulic structures can be used to evaluate the intensity of laboratory test procedures and their transferability on practical conditions. A summary of the intensity of different laboratory test methods is given in [15]. Comparing the experience of the exposure of hydraulic structures and the intensity of the laboratory tests shows that the tests operate at lower intensity than may be expected under practical conditions.

3. CONCRETE RESISTANCE

For the investigations presented in this paper three test methods were applied in the laboratory of the BAW. Information on the test methods is available in [13]. Fig. 2 shows the test setups. The test procedures operate differently concerning flow velocities, abrasive material (steel balls, gravel) and diameter of the abrasive load. This enables considerations concerning the transferability of test results of different test methods and concerning the transferability to practical conditions at real structures.

Figure 2 – Test setups (ASTM C1138M [16], Bania-method, slab-method)

Differently from other deterioration mechanisms as for example carbonation or chloride ingress where mainly the paste (type of cement, cement content, supplementary cementing materials, water-to-cement ratio) influences the concrete resistance the abrasive attack is a mechanical impact which attacks the whole concrete. Thus the aggregate properties are also important. For that reason for the concrete mix design different aggregates were considered and analysed [13]. The Micro-Deval-coefficient (MD) according to DIN EN 1097-1 [17] seemed a promising parameter as the testing conditions are similar to the hydro-abrasive exposure.

For the mix design the compressive strength was varied in typical ranges covered by the exposure class requirements of DIN EN 206 [18] (C20/25-C35/45). According to DIN 1045-2 [19] the fly ash (72-90 kg/m³) was included in the w/c-ratio with k=0.4. A commercially available blast furnace slag cement CEM III/A 32,5 N LH was used which is a typical type of cement for hydraulic structures in Germany. A cement content of 240 kg/m³ for the C20/25 and cement contents between 285 and 330 kg/m³ for the C35/45 concrete were chosen [13, 15].

The key findings up to now were that the MD-coefficient of the aggregate had a very dominant influence on the concrete resistance [13, 15]. Other aggregate parameters like specific gravity, water absorption or Los-Angeles-coefficient showed no correlation to the concrete resistance against hydro-abrasive impact. At similar strength levels (C20/25) the concrete with the highest MD-coefficient of the aggregate revealed an almost three times higher damage than the concrete with the lowest MD-coefficient of the aggregate. This was observed for all test methods. The ranking of the different concretes was similar for all test methods, too. The compressive strength of the concrete revealed a lower correlation to the abrasive damage than the MD-coefficient. This was still the case when concrete with higher strength (C35/45) was considered [15]. To account for both parameters a new parameter r_{ha} was introduced (equation 1) [15].

$$r_{ha} = \frac{f_c}{MD} \qquad (1)$$

r_{ha} Hydroabrasive resistance parameter
f_c Compressive strength, wet stored cylinder, h/d=2 [MPa]
MD Micro-Deval coefficient, DIN EN 1097-1 [M.-%]

This parameter considers both the influence of the aggregate and the matrix of the concrete which are both affected by the abrasive exposure. Fig. 3 gives an impression of the different abrasion resistance of the matrix and aggregate particles with different properties. The black aggregate grain has remained more or less unaffected by the abrasion exposure during the test whereas the one just below and the matrix have a plain and ground surface.

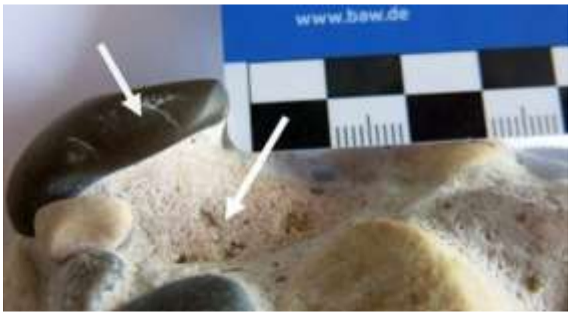

Figure 3 – Specimen detail after abrasion test

Further investigations and evaluations are necessary to define appropriate requirements for concrete exposed to hydro-abrasive exposure.

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FROST RESISTANCE

Low-Temperature Strain in Air-Entrained Mortar

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ABSTRACT
Air-entrained mortars that show no residual strain after freezing to -20 ºC may expand dramatically at lower temperatures, so the conventional freeze/thaw tests are not reliable for materials used under arctic conditions. Dilatometric tests show that unconventional concentrations of air-entraining agents can provide protection even at extreme temperatures. The mechanisms at play are discussed in this paper.

Key words: Air entrainment, frost resistance, dilatometry, crystallization pressure.

1. INTRODUCTION

Air-entraining agents (AEA) protect cementitious materials against frost damage by two mechanisms [1]: (i) the voids provide sinks for pore water displaced by the volume expansion of ice during freezing (avoiding hydraulic pressure), and (ii) ice growing in the voids sucks water from the surrounding mesopores so that the body is pulled into compression (offsetting crystallization pressure from ice in the mesopores). The latter effect is revealed by dilatometric measurements that show contraction upon freezing of cement paste or mortar with adequate air entrainment, whereas non-air-entrained bodies expand [2]. The magnitude of the contraction of bodies containing AEA can be quantitatively predicted using poromechanics [3]. In the present study, dilatometric measurements were performed on mortars prepared with several commercial AEA. With conventional doses of AEA, samples cooled to very low temperatures exhibited a remarkable behaviour pattern: contraction begins after ice nucleates, then below a certain temperature, T_0 (say, -40 ºC), the sample *expands* as the temperature, T, drops further, and continues to expand as it is reheated up to T_0; as T rises above T_0 the sample *contracts* until the ice melts. If a sufficient dose of AEA is used, the re-expansion is small enough to avoid damage to the sample, but the amount required is much more than would be indicated by a conventional freeze/thaw test. The phenomena are illustrated and an explanation is proposed in this paper.

2. EXPERIMENTAL PROCEDURE

The samples used for dynamic mechanical analysis (DMA) were taken from larger mortars that ranged in age from about 2 to 6 months old. Data from samples that were tested after 3 months and 6 months had excellent agreement. Each mortar was made with filtered DI water, Ottawa sand conforming to ASTM C778, Type I/II ordinary Portland cement (OPC), and four commercial AEAs: a saponified rosin, vinsol resin, tall oil, and sulfonate. The total organic carbon in each AEA was found to be 51, 147, 97, and 39 g/L, respectively [4]. Other details on the surfactants, including their critical micelle concentration curves (CMCs) and interaction in cement pore solution are also given in [4]. Three dosages were used for each type of AEA, taken as the maximum recommended dosage provided by the manufacturer, and two and three times this amount. Mortars were made with a water-to-cement ratio (w/c) of 0.4. The sand was taken as 20% of the cement volume, or 8% of the total volume of the mortar.

Dynamic Mechanical Analysis (Perkin-Elmer DMA 7e) was used to measure the length change of mortars when exposed to freezing temperatures. The DMA samples were made by core-drilling a disc that was saw-cut from the 15-cm-tall cylinder. The resulting samples are cylinders that are ~9.8 mm wide and range from 10 to 15 mm in height. These cylinders were placed in a limewater bath until they were removed for analysis, at which point they were rinsed with DI water, and surface-blotted with a tissue. The mortar was then wrapped in tape that had been lightly coated with vacuum grease and sprinkled with metaldehyde, which encourages ice nucleation [5]. During data collection, the sample was immersed in a cup of kerosene to prevent drying and to help with heat transfer. The low temperatures were achieved through a bath of ethanol and dry ice, which reached a minimum temperature of about -65 ˚C. The following temperature program was used: Cool from 20 to 5 ˚C at -1 ˚C / min, isothermally hold for 30 min (or until a stable probe reading is obtained), cool from 5 to -50 ˚C at -0.25 ˚C / min, heat from -50 to -20 ˚C at 0.25 ˚C / min, isothermally hold for 1 hour, heat from -20 to 5 ˚C at 0.25 ˚C / min, isothermally hold for 1 hour.

3. RESULTS AND DISCUSSION

A typical set of dilatometry data is presented in Fig. 1 for samples prepared with the indicated dosage of tall oil AEA; two or three replicates are shown for each dosage. The dash-dot curve (right ordinate) shows the temperature cycle. As T drops, the sample first contracts owing to thermal expansion, then the slope becomes more negative when ice nucleates near -10 ºC. Contraction stops for the sample with the lowest dosage (solid curve) at $T_0 \approx$ -31 ºC, and it expands as the sample is cooled to -50 ºC and then reheated; another strain reversal occurs when T rises again to -31 ºC, after which the sample contracts until the ice melts. The final strain of about +6 x 10^{-4} indicates that damage was done during the expansion. A similar pattern is seen for the sample with twice as much AEA (dashed curve), but the reversals occur at lower T (T_0 $\approx$ -37 ºC) and the residual strain is smaller. For the highest dose (dotted curve), the reversal occurs near -40 ºC, but the strain remains negative and there is no residual strain, so damage was avoided.

Contraction during freezing of an air-entrained mortar is expected [2], owing to suction created by ice trapped in the air voids, and the expansion beginning at T_0 could be attributed to escape of that ice from the voids [6]. However, if ice penetrated the pore in the shell, there would be hysteresis between the freezing and melting points for that ice (because the ice advances into the pore with a hemispherical leading surface, but melts from its cylindrical sides) [7], so the strain reversals during cooling and heating would not occur at the same temperature, T_0. We conclude that ice in the interfacial transition zone surrounding the void blocks the pores at T_0 with hemispherical ice/water menisci, as shown schematically in Fig. 2. The pores in the shell around the void are too small for ice to penetrate, but the menisci bulge into the pore entries on both sides and prevent flow of pore water, so the suction cannot be maintained. Since the ice does not enter the pore, the menisci retain their hemispherical shape, so there is no hysteresis as they advance and retreat. As the temperature rises above T_0, the ice in the ITZ retreats from the pore entries and the suction is gradually restored by the crystals inside the void. The decrease in T_0 with increasing dosage of AEA probably indicates that the shell around the void has finer and/or more numerous pores.

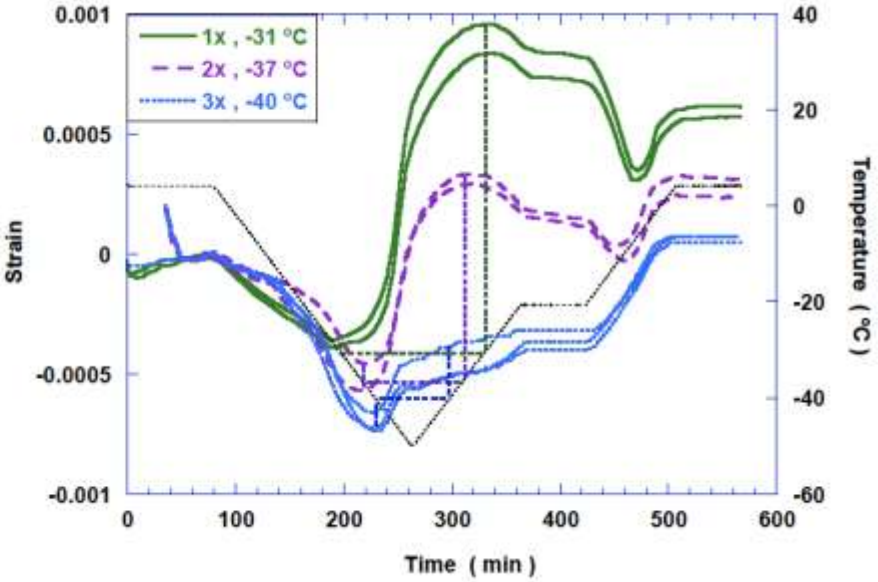

Figure 1 – Strain (left ordinate) during freezing of mortar containing indicated dosage of AEA (nx = n times maximum recommended dosage); thermal cycle (dash-dot curve) corresponds to right ordinate. Strain reversal occurs at indicated temperature.

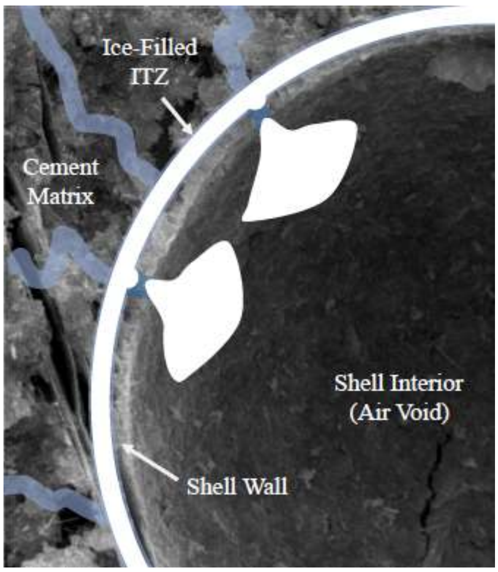

Figure 2 – Schematic of ice crystals superimposed on photo of actual void in mortar. Internal crystals suck water from surrounding mesopores, but below T_0 the pores in the shell are blocked by ice in the ITZ. Menisci bulge from each side into pores in shell, but do not penetrate.

4. CONCLUSIONS

Dosages of AEA that are sufficient for passing conventional freeze/thaw tests will not prevent damage during cooling to very low temperatures. To survive freezing to -50 ºC may require 3 times as much AEA as is needed to avoid damage at -20 ºC. Dilatometry demonstrates that mortar contracts upon cooling below the nucleation temperature, but then re-expands below a temperature that depends on the AEA dosage. Importantly, the strain reverses again when the mortar is heated above the *same* temperature. The absence of hysteresis is best explained by ice formed in the ITZ around the void, which blocks (but does not penetrate) pores in the shell.

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Requirements and recommendations to frost durable concrete – an overview

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ABSTRACT
Requirements and recommendations for frost durable concrete from standards and specifications from Europe, North-America and Asia, various international organizations and construction projects are reviewed, compared and discussed. This is done based on exposure or "load" (wetness/saturation/situation, de-icers, frost, etc.), material or "resistance" (air voids, w/b, binder type, strength etc.), execution (pumping, casting, finishing, curing etc), and tests (air voids, porosity, strength, various frost tests). Finally, some practical examples of the specification together with examples of need of stringency and some occurring peculiarities in testing are given. Also the large variation in how frost durability is perceived in different parties of the decision-, planning-, execution- and commissioning process around the world are discussed and illustrated.
The full report can be downloaded from:
https://ntnuopen.ntnu.no/ntnu-xmlui/handle/11250/2598133

Keywords: concrete, constituent, proportions, execution, frost, standard, testing, performance

1. INTRODUCTION AND SCOPE

Frost deterioration of concrete is seen as progressive scaling, spalling and crumbling of particles from a surface or/and cracking of the volume of the concrete. The cracking can eventually be visible at the surface, or there can be a combination of scaling and cracking. Frost damage can happen with the simultaneous presence of low temperatures and a wet surface, and the surface damage is amplified in the presence of de-icers such as salts during exposure to frost.

With some experience into the topic, it is easy to imagine that agreeing on how to make frost durable concrete structures based on the huge amount of research results available in the literature would be both a vast and likely impossible task. The many experimental results, theories, models and test methods presented over the years have given a large number of data that are difficult to unify for practical purposes. Another approach would be to review the standards, requirements, and recommendations published internationally. Design and building based on a review of the latter three types of documents are, therefore, an easier task than to review experimental results, theories, and models. Design and building are based on loads on and resistance of the structure in question. For frost exposure, this is less clearly defined than for most types of mechanical loads on a concrete structure. Frost durability of concrete is often described as its ability to withstand repeated freeze-thaw cycles throughout a defined life of a structural element while exposed to periodical wetting and drying, often in combination with deicers (road salt, sea water, urea, etc.) It is the country standards, norms and regulations for concrete that stipulate:

- the exposure (wetness/saturation, chlorides, frost, etc.) to base the design on,
- the material requirements (air void requirements, w/b-ratio, binder composition, strength, etc.) to select for that exposure,
- the production techniques and rules (placing arrangements, finishing, curing) to apply for making the concrete frost resistant and
- the test methods (air voids, frost tests, porosity, strength, etc.) for the final product to confirm compliance.

The approach to frost durable concrete varies much from country to country. Numerous committees and unions with their sets of recommendations in addition to national standards have worked with this topic. Selection of the reviewed documents in this paper is limited to standards and recommendations from the USA (ACI, AASHTO, ASTM), Canada (CSA, BNQ), Norway (NS-EN), Sweden (SS, SIS), Danmark (DS), Germany (DIN, ZTV, BAW), Russia (SP, GOST, SNiP) and China (GB/T). The scope is to give an organized and systematic overview of the documents relevant to frost exposure requirements and recommendations including examples for application.

2. DOCUMENTS, EXPOSURE, MATERIALS PROPERTIES, EXECUTION AND TESTS

Based on the available documents reviewed [1] it seemed most convenient to use the following division given in table 1:

Table 1 - List of the main tables in [1]

	Table 2	Overview of the documents included in the review
Load	Table 3a	Classification for freeze-thaw exposure conditions. LOAD
	Table 3b	Summary of exposure classes from the reviewed standards and specifications
Resistance	Table 4	Material requirements. RESISTANCE
Execution	Table 5	Production and execution of concrete works. Requirements and recommendations
Tests	Table 6a	Tests for frost durability – material characterization
	Table 6b	Tests for frost durability – freeze-thaw tests
	Table 7	Overview of requirements for frost durable concrete

2.1 Documents

Table 2 in [1] is a rather large 1-page table, yet limited to only 53 different documents from Europe, North-America and Asia divided into exposure classes, material requirements, production- and execution-standards and recommendations, and freeze-thaw tests. By going into all sorts of details this could, of course, have been a much higher number of documents since such standards, specifications, etc. link to other technical documents. However, the cited 53 documents include the central national standards, requirements from professional organizations such as ACI and large construction owners such as road- and bridge authorities and oil companies. We, therefore, think Table 2 in [1] represents a broad overview and is very useful to the parties of the decision-, planning-, execution- and commissioning process.

2.2 Load (=exposure) and Resistance (=material)

Tables 3a, 3b and 4 in [1] make up a main body of the work. They show the big differences in how frost exposure and material parameters central to obtain durability against frost are seen among different parties in the building process worldwide. Some of the differences, particularly for exposure, pertain to real differences in exposure. A sort of consensus seems to be division onto wetness and to what extent deicers are present. Other differences could be linked to local perceptions and practices for how to obtain frost resistant materials. In general water/binder ratio, strength and air entrainment are basic parameters that there is more agreement on whereas how to use supplementary cementitious materials seems to be more different, difficult or not treated.

2.3 Execution – tests on fresh concrete

Table 5 in [1] is an effort to give an overview of another vast but more practical topic. Our purpose with this has been to help the reader to focus on the main operations of concrete works: mixing, transportation/delivery of fresh concrete, placing and finishing, surface protection and curing. Inevitably, this also includes some tests, eg. fresh concrete sampling and measurements at mixing and delivery/site/after casting and finishing operations are done. The outcome depends on execution, so the question of how to characterize this is obviously difficult, and we have tried to show how for example sampling of air void measurements vary, and the possible methods. Also, some fresh density- and workability tests are listed.

2.4 Tests

Table 6a in [1] is a rather “small” 1-full-page table giving an overview of indirect tests: These include hardened air void content, Protective Pore ratio (PF), i.e. the air void volume as a fraction of total porosity and air void spacing and total fresh air void content. For each country the table lists the relevant performance- (= freeze/thaw-) tests and acceptance criteria for those relative few countries that have such links. Hence, this table shows the testing tools available for requirements and recommendations based on laboratory testing.

Table 6b in [1] is much larger (3 pages) giving an overview of the main frost test methods worldwide. Each of the 3 pages is split in: Samples, Freeze-thaw cycles, Test set-up and Expression of test results. This makes it easier to get an idea about the great difference in how these tests work and how differently they express frost damage. So, in addition to the big difference between frost damage when expressed as surface damage and (internal) cracking, there are large variations in how to characterize these two forms of damage. Scaling can be rated visually, expressed as dried particles lost from the surface or as overall mass loss or -change of remaining specimen. Internal cracking is rated by relative dynamic modulus measured in various ways (resonance frequency, ultrasonic pulse velocity), length change measured in various ways or loss of strength. In addition, we believe there are large differences in how scaling and cracking are provoked in the different tests as well as to what extent the two forms of damage can contribute to each other, for example amplify each other.

3. EXAMPLES OF RECOMMENDATIONS AND REQUIREMENTS

3.1 Frost exposure in Europe - Norway

In Table 2 below an excerpt of the first part of Table 7 in [1] is shown generally for Europe and specifically for Norway of how exposure description and material proportions can be combined to give a specification. Table 7 in [1] is a full 5-page table giving a complete overview of requirements and recommendations worldwide with the information organized for each country into exposure class, material requirement, laboratory tests, and execution.

Table 2 – Excerpt from Table 7 in [1]

Country / Organization, Project	Exposure class, area, decisive parameter		Material requirements										
			Max w/c (effective w/c)	Min cement (binder) content, kg/m³	Max (Min) SF content, %	Max FA content (Max Fa/C ratio), % or kg/m³	Min air content in fresh / hardened** concrete (for aggregate D_{max}, mm), %	Max electric conductivity, Coulombs	Frost resistance, cycles (frost res. class or min. durability factor,%)	Quality of macro-porosity in hardened concrete: Min specific surface, mm²/mm³	Quality of macro-porosity in hardened concrete: Max spacing factor (single results) L, mm	Min concrete comp. strength (strength for cement), MPa or class	Water impermeability class (Durability class)
General Europe EN 206:2013	XF1		0,55	(300)*	11	(33)						C30/37	
	XF2		0,55	(300)*	11	(33)	4					C25/30	
	XF3		0,50	(320)*	11	(33)	4					C30/37	
	XF4		0,45	(340)*	11	(33)	4					C30/37	
Norway NS-EN 206:2013 +NA:2014	XF1		0,60	(250)*		(35)							M60
			0,45	(300)*		(35)							M45
			0,40	(330)*	(6)	(35)							M40
		XF2 XF3	0,45	(300)*		(35)	4						MF45
		XF4	0,40	(330)*	(6)	(35)	4						MF40

Now, to be stringent about the example in Table 2 a few points could be mentioned. Normally, for XF4 to make a concrete that will also pass the severe European EN-TS 12390-9 performance test with 3 % NaCl on sawn surfaces, additional requirements could be needed. More specifically this would be to require air void spacing factor less than approximately 0.2 mm in the finished cast specimen. There could also be cases where concrete mixes pass the severe test without air entrainment. Furthermore, certain SCMs react very differently to carbonation than OPC and other SCMs, simultaneous internal cracking during scaling testing would accelerate scaling and so on. The EN-TS 12390-9 test is known for being very severe, so in practically all cases where concrete passes this test, it will also be durable under similar severe field conditions. Now similar peculiarities can be seen for other countries and other tests, such as for example what is the relevance that scaling occurs sometimes in the ASTM C666 Procedure A test, even when there is no internal damage?

3.2 Frost durability as perceived in different countries

If one should list down the requirements for concrete in a certain exposure from material requirements and testing to execution, quality assurance and handover, the results in different countries will vary in parameters, numbers, and level of detailing, see Figure 1.

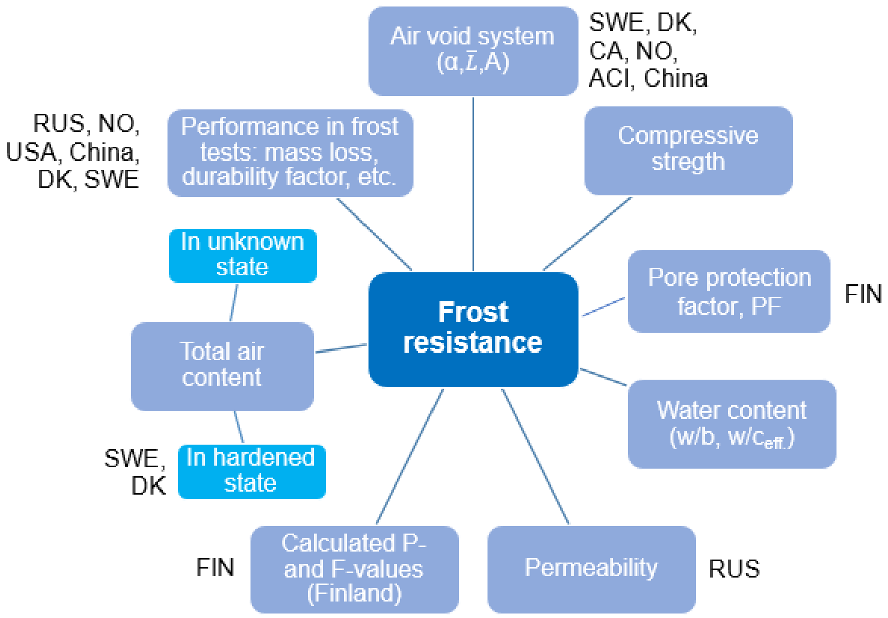

Figure 1 - The meaning of frost resistance in different countries

Figure 1 shows, concerning material requirements, how variable the definition of frost-resistant concrete can be, depending on which country one is going to design and build the structure in.

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Practical challenges with air-entrained concretes

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ABSTRACT

Air-entrained concretes are used extensively in Finland. During the latest years some new problems with air-entrained concretes have been observed. In some cases, the air content has been increasing after initial mixing and causing strength defects in the hardened concrete structures. In addition, the compaction of air-entrained concrete appears to be challenging. Removal of all the compaction air from concrete is difficult. On the other hand, air-entrained concretes may segregate during the compaction, especially when the consistency of concrete is high. This paper summarizes the results of two research projects focusing on the problems with air-entrained concrete.

Key words: Concrete, frost resistance, air content, compaction.

1. INTRODUCTION

Due to climatic conditions, air-entrained concretes are used extensively in Finland. For salt-frost resistance concretes (infra structures), own service life design method (P-factor) was developed already in the 1980's. P-factor is based mainly on the composition of concrete (water-cement ratio and air content) and pre-testing of concretes.

Over the past few years, new problems have been observed with the air-entrained concretes in Finland. In summer 2016, the tensioning anchors sunk into the concrete during the post-tensioning work at Kallanvaara railway bridge in Kemijärvi. The compressive strength of the concrete was clearly lower compared with the requirements. It was revealed that the reason for the strength defects was the elevated air content of concrete. The Kallanvaara bridge was demolished and re-built. Also some other strength defects due to elevated air contents were found also in Finland.

When a number of concrete structures have been analysed more accurately, it has been noticed that the density of hardened concrete in structure is often smaller compared to the laboratory specimens even though the air content of concrete has been on normal level. Density differences up to 100 kg/m^3 between structural concrete and laboratory test specimens have been observed. The density difference was assumed to be due to inadequate compaction of the concrete structures.

This paper summarises the results of two project carried out at Aalto University, Department of Civil engineering. The target of *Robust Air* project [1] was to clarify the causes for the elevated air contents of concrete. The project was carried out in 2017. *Good vibrations* project [2] focused on the compaction of concrete, especially the degree of compaction and segregation of concrete with air-entrained concretes. The project was carried out in 2018.

2. ELEVATED AIR CONTENT OF CONCRETE

Elevated air content of concrete is connected to use of polycarboxylate ether (PCE) based super-plasticizers. The PCE-superplasticizers tend to increase the air content and therefore a foam killer is added into admixture.
In the laboratory tests, both the effects of concrete composition and admixtures (superplasticizer and air-entraining agent) were analysed. Air content of fresh concrete was measured immediately after mixing, 30 min and 60 min after mixing and finally 75 min after mixing. Before the measurements at 30 and 60 min, the concrete was mixed for 1 min. After the 60 min measurement, superplasticizer was added to compensate the workability loss and an additional measurement was carried out at 75 min after the ininitial mixing. In addition, the air content was continuously monitored during the mixing using CIDRA AIRtrac measurement system (acoustic sensor).
Clear increases in air contents were observed in the laboratory tests. Several factors were affecting the increase of air content, but the consistency of the concrete played the most important role. In case of S3 Slump class, the air content was rather stable for most of the admixture combinations, whereas with F5 Flow class the air content increased in most of the cases. Additionally, differences between the different admixtures were observed. The CIDRA AIRtrac measurements indicated that air content is increasing up to the 5 min during mixing. With several admixtures 2 min mixing time with normal AEA dosage gave approximately the same air content as 5 min mixing time with 50% AEA dosage.

2.1 Air content potential

Based on the experiments, the increase of the air content after the initial mixing was explained with *Air content potential* (Fig. 1). Air content potential is the maximum air content the concrete can achieve. Air content potential depends on the admixture combination, but it also depends on the concrete composition such as water-cement ratio and cement content and type. In addition, the consistency of concrete affects the Air content potential. Relatively long initial mixing time is needed to achieve the Air content potential. Consequently, the mixing times used in the ready-mix concrete industry for air-entrained concrete (60… 90 s) are not necessarily long enough.
If the Air content potential is not reached during the initial mixing, there is a risk that the air content will increase later for example during the transportation when the concrete is mixed in

the truck. It is also possible that pumping and casting may increase the air content when the initial mixing process has not been effective enough. Therefore, it is recommended to avoid short mixing time with air-entrained concretes

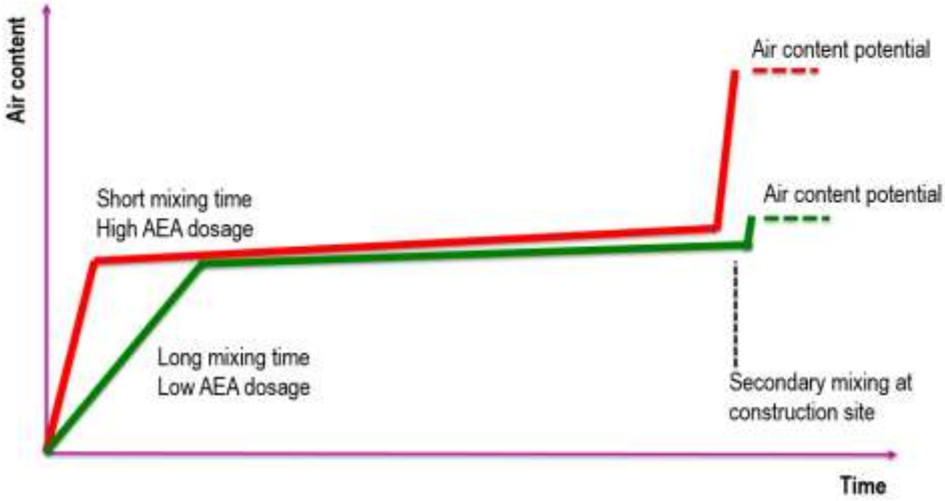

Figure 1 – Illustrated effects of the mixing time (mixing efficiency) and the AEA dosage on the increase of the air content after mixing.

3. COMPACTIBILITY OF CONCRETE

The effects of concrete composition as well as vibration time on the compactibility were investigated using the test structures presented in Fig. 2. The degree of compaction as well as the risk for segregation during the compaction were analysed.

3.1 Degree of compaction

The degree of compaction was analyzed by comparing the densities of the drilled specimens (⌀ = 100 mm, h = 100 mm) with the density of the laboratory test specimens (⌀ = 150 mm, h = 300 mm). Degree of compaction of 100% indicates that the density of the structure is the same as the laboratory specimens.

The test results showed that the laboratory specimens are generally compacted clearly better compared with the concrete structures. With normal vibration times (app. 200 s/m^3) degree of compaction of 0,96…0,99 was achieved (Fig. 2).

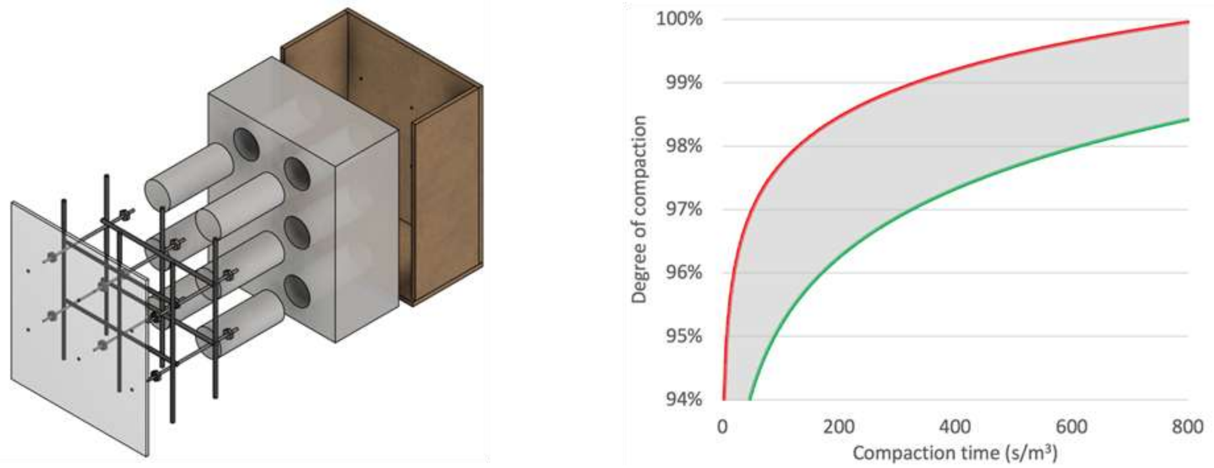

Figure 2 – Left: Test structures used for analysing the compactibility of concrete. The dimensions are: l = 500 mm, h = 600 mm, d = 250 mm. Right: Simplified model for the degree of compaction as a function of vibration time.

The measured degrees of compaction indicate that the test structures contained about 1…4% more compaction air than the laboratory test specimens. In compressive strength this can mean app. 5…20% reduction in strength. The degree of compaction was depending on the vibration time, the other factors investigated were not significant.

3.2 Risk of segregation

The risk of segregation during the compaction was analysed with help of density differences of the test structures. Several concretes with different air contents, consistencies and water-cement ratios were prepared. The vibration time varied between 140 and 800 s/m^3. Standard deviations of six drilled specimens (see Fig. 2) were determined. Theoretically, density differences can be caused if aggregate moves downwards in concrete and compaction air moves upwards.

The density difference increased with the vibration time, but it also depended on the consistency and air content of concrete. The tests showed that concrete without air-entraining is not sensitive for segregation during the vibration. On the other hand, air-entrained concrete segregates relatively easily especially when fluid consistencies are used. In case of normal vibration times (app. 200 s/m^3) the risk of segregation remains reasonable.

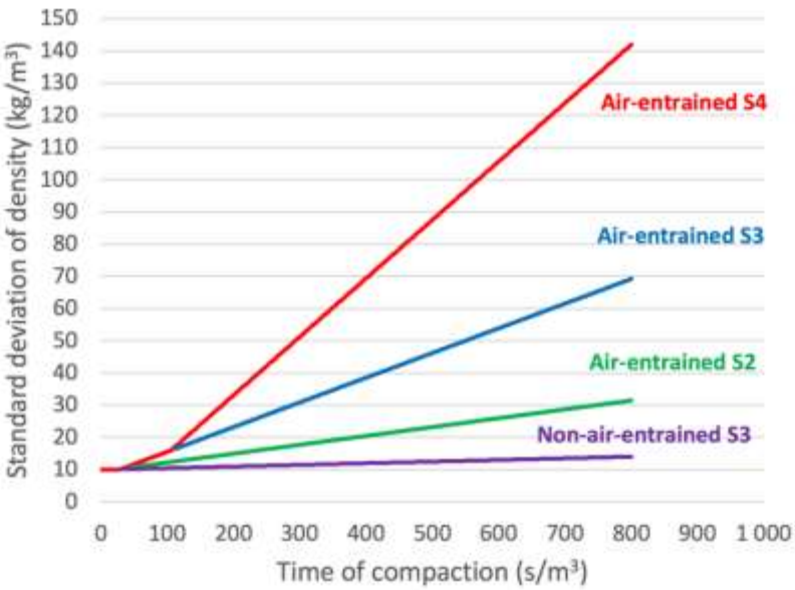

Figure 3 – Average density variations of the drilled specimens as a function of concrete consistency and vibration time. S2, S3 and S4 are slump classes.

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Recapitulation of an old hypothesis on the mechanism(s) of salt frost scaling

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ABSTRACT
A hypothesis for a major mechanism of salt frost scaling on concrete and similar materials is reviewed. Some phenomena of salt frost scaling, reported in various publications, are described. The hypothesis is based on other mechanisms previously described by other authors, and was published in 1998 [1]. This paper is intended to give a quick description of the principles of the hypothesis. Results from a proposed test of the hypothesis is presented.

Key words: Concrete, ice, salt frost resistance, surface scaling

1. INTRODUCTION

Frost damage on concrete is usually divided into two categories; inner damages and surface scaling. While the former is detected as interior cracks and loss of E-modulus and strength, the latter means that thin flakes, up to a few tenths of a millimetre thick, are scaled off from the concrete surface. For a reasonably good, air-entrained concrete quality, surface scaling is usually not an issue as long as the surface is exposed to air or pure water. If the surface is exposed to a saline solution during freezing, surface scaling may be very severe and rapid. This deterioration process is usually called salt frost scaling. Early studies of salt frost scaling were published in the 1940s and 1950s [2, 3].

2. REPORTED PHENOMENA

As described in [1], the following phenomena have been repeatedly reported and may be seen as typical of salt frost scaling:

1. Scaling reaches a maximum for a moderate de-icer concentration in the outer solution: Higher as well as lower concentrations will yield less scaling.
2. Surface scaling almost never appears in the absence of an outer solution which to some extent remains liquid at temperatures lower than the normal freezing point of the pore solution of HCP. It may even be noted that in the two reports in which scaling is reported when the outer solution was pure water, either that outer water was rich in naturally occurring ions [4] or the specimen surface had been exposed to a salt solution prior to testing so that the pore system did contain some salt [5].
3. Portland cement-bound materials with proper air void systems are able to withstand combined salt and frost attack, at least in laboratory tests.
4. The chemical composition of the de-icing agent seems to be of no importance.
5. Without actual freezing temperatures, no scaling occurs.

6. Use of a lower minimum temperature will produce more scaling, at least for minimum temperatures in the interval 0°C $>\theta>$-20°C.
7. Coarse porous materials are more sensitive to salt frost attack than dense materials.
8. Purely chemical mechanisms are of little importance in comparison to physical mechanisms.

3. HYPOTHESIS

The hypothesis presented in [1] is based on the mechanism of osmotic ice lens growth, as described by Taber [6], Powers and Helmuth [7] and Everett [8]: In a pore system consisting of pores of various sizes, the first ice crystals will form in the largest, water containing pores. On subsequent cooling, the free energy of these crystals will drop more than that of water remaining in the smaller pores. Thus there will be a potential for driving moisture from the narrow pores, to the ice crystals. As temperature continues to drop, this process will proceed until either the ice pressure becomes so large or the pressure in the remaining liquid becomes so low, that the difference in free energy between liquid and ice is eliminated. Another way of establishing thermodynamic equilibrium between the ice crystals and the remaining liquid is to add a de-icer to the pore liquid, since this, too, will reduce its free energy.

A pressure in the ice will arise if/when the ice crystal grows so much that it completely fills the pore in which it grows. The ice will tend to penetrate into more narrow pores, but this will bring about an ice pressure that may be detrimental to the pore walls [8]. A reduced pressure in the remaining pore liquid will be established as the narrow pores are drained of liquid (due to the transfer of water from the narrow pores to the ice crystal).

For a concrete sample exposed to air, this means that if the initial degree of saturation is sufficiently low, the early ice formation will drain the narrow pores to such an extent that ice crystals cannot grow sufficiently to exert pressure on the pore walls.

For a concrete sample exposed to pure water, ice will form in that water before any ice forms inside the sample. The external ice will be the place of lowest free energy, and liquid from pores close to the surface will be transported out of the sample. Thus the surface will be successively emptied and thus, to some extent, protected from detrimental ice pressure. (At larger depths, ice may grow from the remaining pore liquid and cause inner deterioration.)

When a moderately concentrated de-icer solution is cooled below 0°C, ice formation will not be initiated until a certain temperature is reached (as seen in phase diagrams). At lower temperatures, the de-icer solution is turned into a mixture of ice and de-icer solution of increased concentration. Thus, there will be some volume of liquid left in the external mixture until the eutecticum of the de-icer solution is reached. The free energy of this mixture of ice and brine will be equal to that of bulk ice at the prevailing temperature. For NaCl, this is about -21°C. Provided that the de-icer has not penetrated into the pores of the concrete, ice formation inside the concrete will proceed as usual. But when ice crystals grow and consume water from nearby, more narrow pores, the liquid will be replaced with new liquid from outside. Thus, the reduced liquid pressure which is required in order to prevent ice crystals from growing, is not established (at least not to a sufficient extent). Instead, it will be possible for the ice crystals to grow and exert pressures in a way that is not possible in a concrete exposed to air or pure water. (Note that it is not primarily water from the outside source that joins the crystals and make them grow, but rather the water in very nearby narrow pores. This pore water is then replaced with liquid from outside.) After a while, the concentration of the external solution will be established even where crystals grow, and further growth of the ice crystal may seem impossible. However, on further temperature reduction, some more water from the brine in the pores will become freezable, and this water may be used for growing the crystals.

If the external solution is very dilute, the amount of remaining brine will be small, and the suction in to the pores of the concrete may be obstructed. In such a case, some reduced pressure will be established in the pore liquid, ice crystals close to the surface will not grow freely, and thus will not exert pressures on the pore walls. Thus, weak salt solutions will not promote surface scaling very well.

If instead the external solution is initially highly concentrated, a lot of salt may penetrate in to the pore system (primarily during the thawed stages) and this will strongly reduce the amount of ice that forms inside the concrete. This, too, will reduce surface scaling.

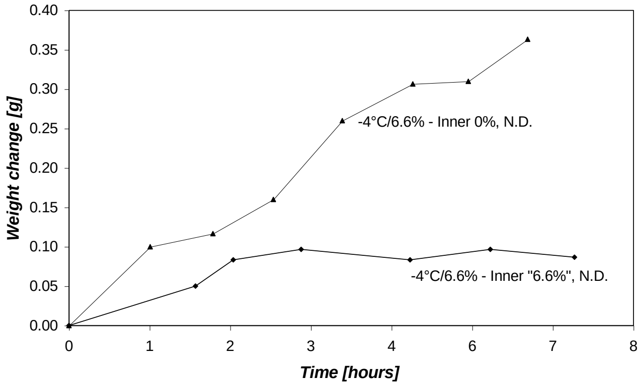
Since the external salt concentration may vary widely under real conditions, it follows from this hypothesis that surface scaling may proceed very differently in laboratory experiments as compared to reality.

4. PROPOSED TESTS OF THE HYPOTHESIS

From the hypothesis, a number of phenomena may be predicted. By performing various tests designed from such phenomena, it is possible to test the entire hypothesis and see whether it needs modification or maybe complete rejection. Some possible tests are mentioned in [1].
One predictable phenomenon is that samples should absorb liquid during a surface scaling test. Indeed, this has been reported in scaling tests on several occasions, e.g. [9].

5. RESULTS FROM PERFORMED TESTS

According to the proposed hypothesis, as a result of ice body growth in the pore system of a specimen, moisture uptake from the outer brine should occur during salt frost testing. This should cause weight gain. To test this and the qualitatively predicted dependence of such weight changes on temperature and salt distributions, discs of Portland cement mortar were frozen by submerging them in pre-cooled salt solutions and then the weight changes occurring at constant sub-zero temperatures were observed. Mortar specimens of *w/c* ratios 0.40, 0.55 and 0.65 were tested. Also mortars of *w/c* ratios 0.40 and 0.65 with additional air-entrainment were tested. Specimens were pre-stored in lime water or strong salt solutions (6.6, 14.4, 19.7% b.w.) for 12 weeks. In addition, a set of specimens were pre-stored in weak salt solutions (1, 3, 5% NaCl b.w.) for 19 days. Tests were run at -4°C, -10°C, -16°C, and at room temperature. Other reasons for weight changes during this kind of test were examined. From the hypothesis, it was predicted that larger weight increases would be observed the lower the temperature and the lower the concentration of salt in the pore system. The predicted results were fulfilled. In addition, two unexpected results were obtained. The results are not interpreted as evidence against the proposed mechanism. The complete test is described in [1].
The figures below show the set up and some results. The most striking result was that a crack was discovered in the centre of one sample, indicating that ice crystals had been growing where the salt concentration was at the lowest.



Left: Cooling bath and discs placed

Right: Effect of (intended) inner salt solution concentration 6.6% on weight gain at -4° for w/c ratio 0,65 submerged in 6,6% NaCl solution. Each line represents three specimens. (N.D: Never-dried before test).

Crack in the center of w/c ratio 0.65 specimen stored in 3% NaCl solution for 19 days prior to test at -10°C, bath salt concentration 14.4%. (Two views). Crack is assumed to be due to ice crystals growing at the centre of the specimen, where the salt concentration was at the lowest.

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Impact of different liquid uptake processes on salt frost scaling mechanism

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ABSTRACT

Currently there are two different models, the glue spall model and the cryogenic suction model, which have the potential to describe the mechanism for salt frost scaling correctly. The cryogenic suction model attributes scaling to the uptake of unfrozen, highly concentrated brine at sub-zero temperatures. Therefore, it cannot fully account for the pessimum de-icer concentration in salt frost scaling tests (~3 wt.% NaCl). The consideration of further liquid uptake processes during a salt frost attack might offer a plausible extension to the cryogenic model, which could help to explain the cause for the pessimum de-icer concentration.

Key words: Concrete, salt frost scaling, damage model, cryogenic suction, micro ice lens pump, liquid uptake, de-icer concentration.

1. INTRODUCTION

Though salt frost attack on concrete has been a research topic for many decades, there is still no consensus on the actual mechanism. However, many aspects of the phenomenology are generally acknowledged. Two main characteristics are, that (a) salt frost attack causes scaling (in contrast to pure frost attack) and that (b) most intensive scaling in freeze-thaw tests occurs at moderate de-icer concentrations (~3 wt.% NaCl) [1]. Two rather recent models claim to describe the scaling mechanism correctly, considering all relevant characteristics of the attack. These models - the glue spall model [2] and the cryogenic suction model [3, 4] - follow different approaches. This paper discusses the cryogenic suction model and how it could be possibly be extended.

2. CRYOGENIC SUCTION

The cryogenic suction model was published by Liu and Hansen [4]. It is based on an earlier work by Lindmark [3], who combined the theory of micro ice body growth [5] with aspects of frost heaving of soils [6], thus considering the uptake of de-icer solution in concrete at sub-zero temperatures. Basis of the theory is the eutectic behaviour of salt solutions.

When a saline solution freezes (on concrete), pure ice crystals begin to form, as the salt ions are not incorporated in the crystal lattice. The salt concentration in the remaining unfrozen solution increases and its freezing point decreases. As long as the temperature stays above the eutectic temperature of the solution, the ice contains pockets and/or channels of unfrozen brine [2].

Ice lenses are formed in coarse pores inside the concrete microstructure during the freezing process. The pore solution in very small pores remains unfrozen because of surface forces and growth constraints [5]. The proportion of frozen pore solution depends on the concentration of solutes and on the pore size distribution in the concrete. As unfrozen pore solution possesses a higher free energy than ice, moisture is drawn towards ice lenses from the surrounding hardened cement paste, until a thermodynamic equilibrium is reached. In the cryogenic suction model, unfrozen brine from the outer ice layer acts as an additional moisture reservoir. The brine is drawn towards surface near ice lenses in the concrete, which now can grow further. The resulting stresses and increased saturation of the surface layer cause scaling according to the theory. The cryogenic suction process was verified in several experimental setups [3, 4].

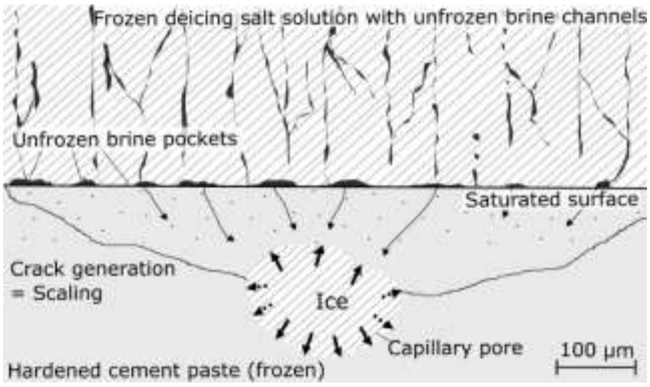

Figure 1 – Schematic representation of the cryogenic suction process, after [3, 4]

The salinity of the attacking de-icer solution is an important parameter in the model. On the one hand, salinity determines the amount of unfrozen brine at respective sub-zero temperatures. At low salt concentrations too little unfrozen brine remains to cause scaling. On the other hand, ingress of salt lowers the freezing point of pore solution in concrete. This effect is meant to become dominant, if de-icer solutions with high concentrations are used, which then hinder ice formation in concrete [3]. That is how the cryogenic suction model tries to account for the pessimum de-icer concentration of about 3 wt.-% in salt frost scaling tests.

However, on a closer look, the diminishing effect of high salt concentrations cannot be fully explained. As the core of the cryogenic suction model is the uptake of unfrozen, highly concentrated brine, scaling should be intensified with increasing de-icer concentrations. This indicates that additional processes for liquid uptake must be considered.

3. PROCESSES FOR LIQUID UPTAKE IN FROST/ SALT FROST ATTACK

3.1 Liquid uptake in frost resistance tests

A test for frost resistance of concrete typically consists of two phases. First, a pre-saturation phase, where concrete is covered with water (slab test) or is placed upside down in water (CIF test). The concrete absorbs water mainly by isothermal capillary suction. Second, the freeze-thaw phase, where concrete is repeatedly frozen and thawed while being in contact with water. Figure 2 shows an example for a comparative freeze-thaw test with water and de-icer solution.

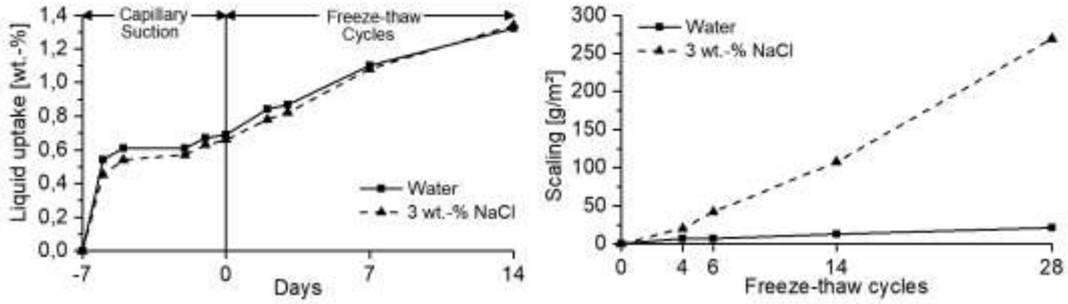

Figure 2 – Liquid uptake (left) and scaling (right) in CIF/CDF test on an identical concrete.

Liquid uptake in the freeze-thaw phase can for example be explained with the micro ice lens pump model from Setzer [7]. In freezing concrete, a negative pressure is created in unfrozen solution in smaller pores to satisfy the conditions of thermodynamic equilibrium. The solid skeleton of cement paste contracts, so pore solution must move to ice lenses in larger capillaries, where it freezes. At temperatures below 0 °C, no liquid uptake is possible, as external water is completely frozen. Cryogenic suction cannot take place. Only weak scaling occurs (Figure 2 – "Water").

During heating thermodynamics requires the process to be reversed, but pore solution remains "trapped" at ice lenses up to temperatures near the melting point of bulk ice. Instead already melted water on the concrete surface is sucked up by the concrete. Though the process starts at the concrete surface, moisture is transported several centimetres into the concrete, as the moisture follows the receding ice front [7]. This intensive saturation of the concrete in subsequent freeze-thaw cycles causes internal damage in non-resistant concretes.

3.2 Liquid uptake in salt frost scaling tests

In salt frost scaling tests de-icer solution is used as test solution. The pre-saturation phase is either carried out with water (e.g. slab test) or with test solution (CDF test). The freeze-thaw phase is always carried out with test solution. Unfrozen brine remains in the test solution, when temperature is below 0 °C, but above the eutectic temperature of the de-icer solution. Thus, the cryogenic suction process can occur and cause scaling (Figure 2 – "3 wt.-% NaCl").

However, the saturation process during the melting phase also takes place. Consequently, three distinct liquid uptake processes must be considered in a model for salt frost scaling – capillary suction, cryogenic suction and suction by the micro ice lens pump process. Total liquid uptake can be quite similar for concrete tested with water or de-icer solution (Figure 2, left – "Water" vs. "3 wt.% NaCl"), so the proportion of liquid uptake due to cryogenic suction should be rather low.

4. IMPACT OF LIQUID UPTAKE PROCESSES ON SALT FROST SCALING

Regarding the cryogenic suction model and the pessimum de-icer concentration, the impact of different liquid uptake processes can be considered as follows:

Capillary suction during the pre-saturation

The initial concentration of the test solution dictates the de-icer concentration in the pore solution of the concrete over a depth of several centimetres before the freeze-thaw phase (Figure 3).

Cryogenic suction at sub-zero temperatures during the freeze-thaw phase
Cryogenic suction can take place, when ice formation in the concrete has begun. It should be strongest at the minimum temperature in a freeze-thaw cycle. De-icer concentration of unfrozen brine mainly depends on the type of solute and on the minimum temperature. E.g. the concentration of NaCl in the unfrozen brine is 22.4 wt.% at a minimum temperature of -20 °C. The initial concentration of the de-icer solution only influences the amount of unfrozen brine. Brine uptake is probably limited to the immediate surface layer of the concrete (< 1 mm), which will to a certain extent be removed from concrete by scaling (Figure 3).

Suction by the micro ice lens pump
This process occurs during the melting phase of a freeze-thaw cycle. Due to the heating gradient outer test solution is melting first, so it resumes its initial concentration, before it is sucked into the concrete. Thus, in this process the initial concentration of de-icer solution dictates how much salt is taken up by the concrete. Assuming, that the major part of liquid uptake occurs during melting, suction by the micro ice lens pump should account for reduced scaling intensity, when test solutions with > 3 wt.% NaCl are used (Figure 3).

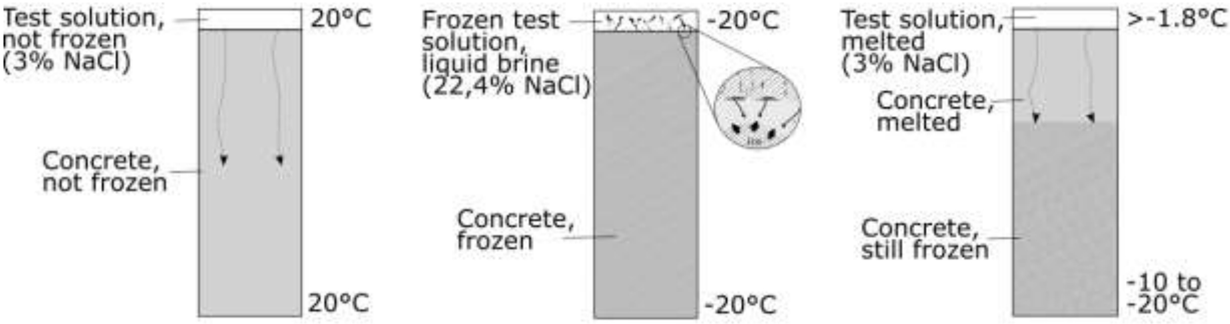

Figure 3 – Capillary suction (left), cryogenic suction (middle) and micro ice lens pump (right)

5. CONCLUSIONS

The cryogenic suction model attributes scaling to the uptake of unfrozen, highly concentrated brine at sub-zero temperatures. It can thus not fully explain, why de-icer solutions with moderate concentrations cause the most intensive scaling. An extension of the model with further, well known processes for liquid uptake during a salt frost scaling test might lead to a more comprehensive description of the scaling process, which can account for the pessimum effect regarding de-icer concentration.

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Towards Macroscopic Simulation of Frost Damage in RC Structures

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ABSTRACT
This paper presents an FEM-based simulation scheme for the frost damage process in Reinforced Concrete (RC) structures, which integrates the micro-/meso- thermodynamic and poromechanical events to the macro- nonlinear mechanical performance. Based on this multi-scale platform, basic frost damage characters of concrete material are analysed at first, then, considering the global temperature field and water motion in cracked concrete, the non-uniform frost damage accumulation process in RC structures can be simulated. Finally, the effect of reinforcing bar on the damage profile is discussed in detail.

Key words: reinforced concrete, frost damage, simulation, multi-scale.

1. INTRODUCTION

The material-level damage mechanisms have been well investigated in the past decades from physical, thermodynamic and poromechanical aspects. In regard to the RC structural level, the non-uniformity of the temperature and moisture as well as the rebar confinement becomes essential to the frost damage process. The past researches on the frost damage at the structural level were mainly phenomenon-based studies [1-4], which were difficult to make the general predictions considering different temperature & moisture boundaries, rebar arrangement and material mixers.
To solve the problem above, the author has developed a multi-scale simulation program [5-6], aiming to simulate not only the non-uniform frost damage accumulation, but also the residual structural quality indexes (capacity, stiffness, ductility, etc.), the structural destruction process and the failure modes. In this paper, the multi-scale simulation scheme, frost damage models and poromechanical coupling of the skeleton deformation and water motion will be introduced in brief. Then, the deformation of concrete cube during freezing-thawing cycles under open moisture condition will be simulated and discussed. Finally, the structural level frost damage profiles considering environmental boundary conditions and rebar arrangements will be numerically investigated.

2. MULTI-SCALE SIMULATION SCHEME

The multi-scale integrated simulation scheme has been briefly summarized in Fig. 1. Starting from the hydration stage at nano-scale, the formation of inner and outer products of CSH cement hydrates is simulated around cement powder particles based on the hydration kinetics [6-7]. Then, the hydration products together with the cement and mineral powder's particles form the micro pore structures, within which the moisture transport and its thermodynamic equilibrium's requirement are formulated as governing theorem. Based on the computed moisture states in micro pores and the local temperature, the ice formation can be quantified. At the meso-scale, the mechanical interaction between expanded ice and surrounding concrete skeleton is taken into account for both non-cracked and cracked conditions. Finally, the meso-scale stress-strain relations are integrated to the macro-scale in considering time-dependent nonlinearity of cracked reinforced concrete [8], for which the reinforcing bars are smeared within finite elements assigned to RC domains. Besides, the calculated damage and cracking will also affect the poro-mechanical equilibrium and water motion at meso-scale reciprocally.

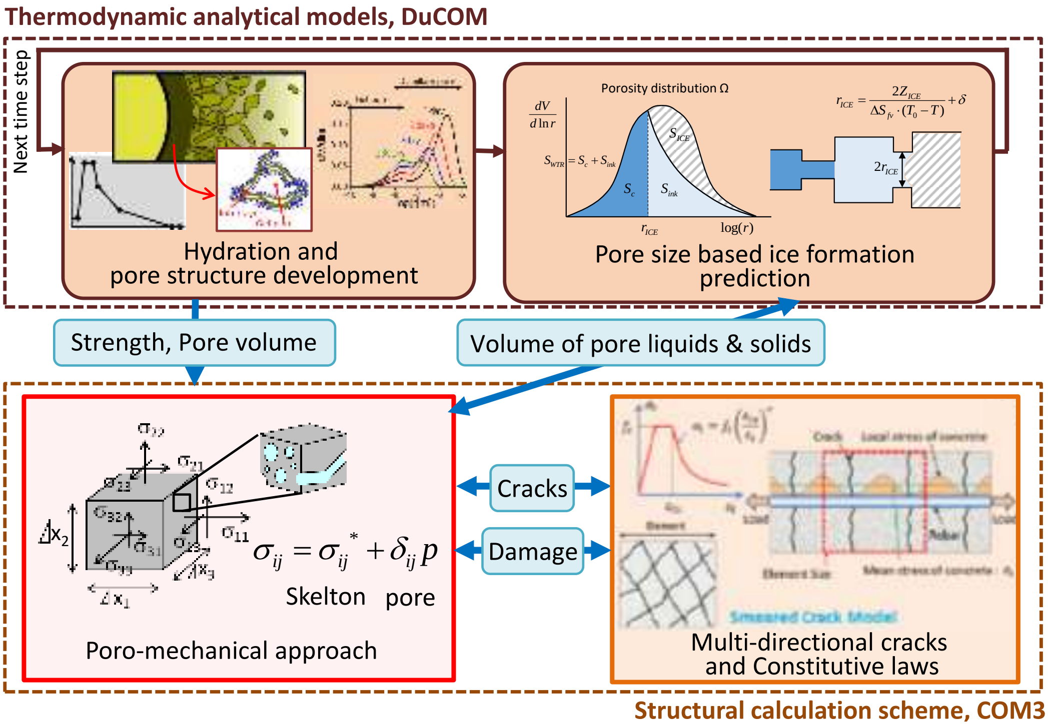

Figure 1 – Multi-scale simulation scheme of structural frost damage

3. COUPLED SKELETON DEFORMATION AND WATER FLOW

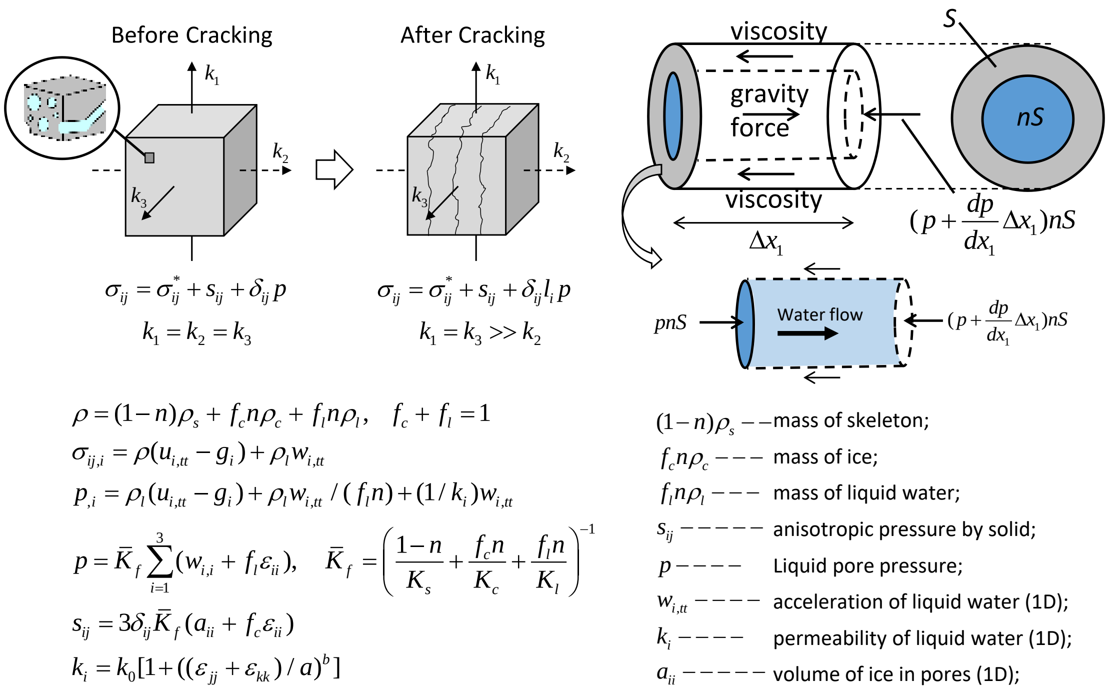

Figure 3 – Calculated ice freezing-melting curve based on empirical pore size distribution

For the coupling effect during freeze-thaw cycles, the detailed formulation has been presented in previous studies [5], here just a brief summary is shown, see Fig. 3. Similar with the “principle of effective stress” in soil mechanics, the total stress (σ_{ij}) equals to the effective stress on the concrete skeleton (σ^*_{ij}) plus the internal pore stress (s_{ij} and p). The dynamic equilibrium of concrete and liquid is considered, besides, after cracking happens, the permeability will also be affected.

4. SIMULATION CASES AND DISCUSSIONS

4.1 Structural simulation and effect of reinforcing bars

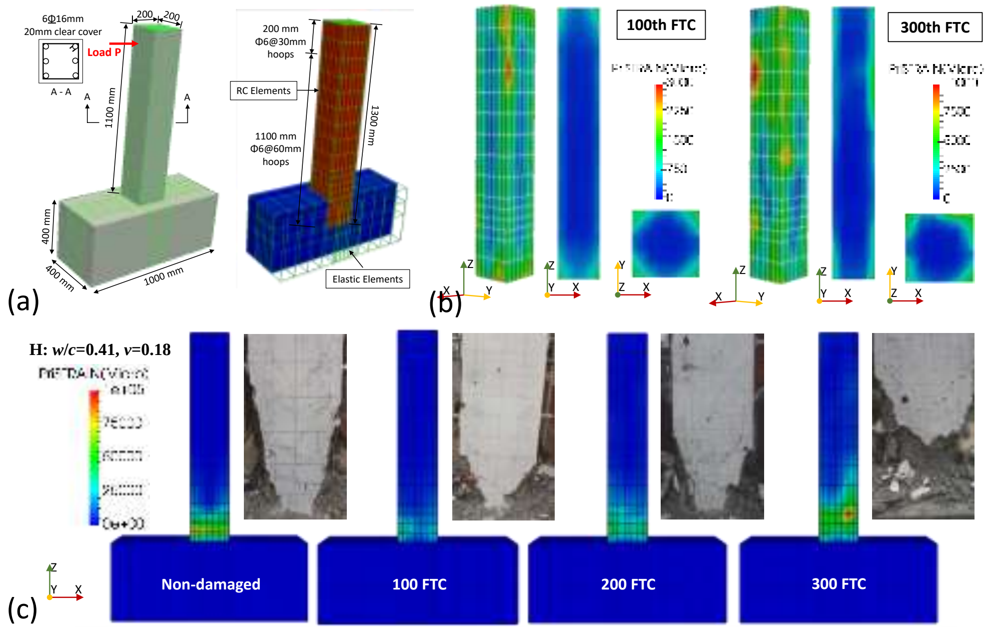

Figure 4 – (a) RC column referring [3] (b) Frost damage profile (c) Failure pattern after loading

As shown in Fig. 4, the frost damage process can be simulated with the RC columns. The temperature non-uniformity causes the uneven frost damage as the early stage, while the reinforcement is the main reason for the later frost damage cumulation by controlling the crack width and water uptake from outside. Thus, a big gap in damage level can be seen where the rebar frame is located. The restoring force characteristics of RC columns with different numbers of FTC are simulated and compared with previously reported experiments. The simulated failure modes of RC columns indicate consistency with the experimental observations.

The impact of arranging reinforcement, especially lateral tie or hoops, is discussed, see Fig. 5. The lateral reinforcement ratio by volume may not affect the non-damaged loading capacity if the mode of failure is flexure. However, it is crucial in view of the damage control under freeze/thaw cycles. Reinforcing bars are capable of increasing in the local tensile capacity of plain concrete to confine the expanded ice. Furthermore, reinforcing bars profitably suppress intake of additional water, which may inflate in the preceding frost events, by controlling the crack opening. Small quantity of reinforcement may effectively reduce the frost structural damage significantly, while this effect is not proportional to the reinforcement ratio by volume but with a decreasing speed associated with expected remaining life of structural concrete.

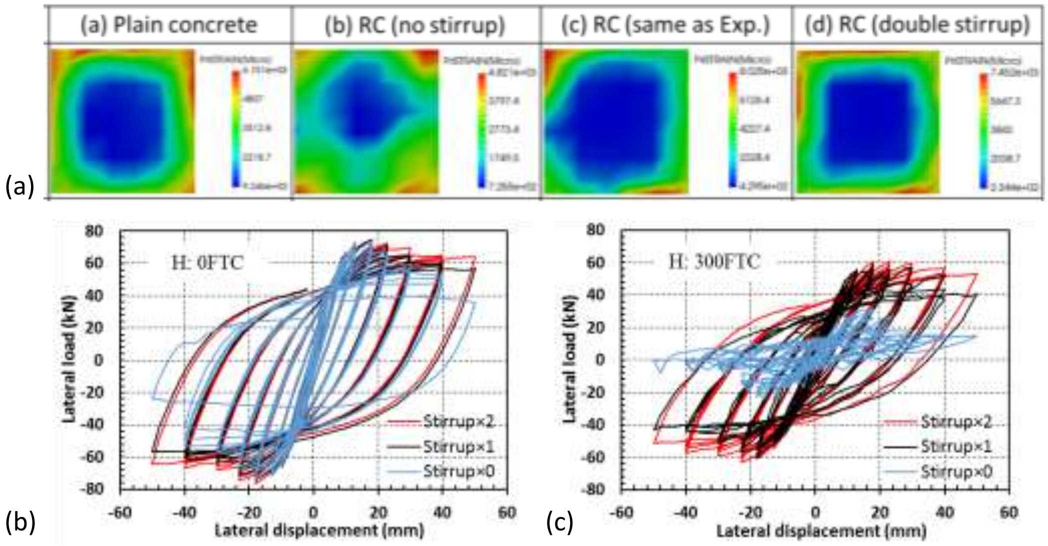

Figure 5 – (a) Frost damage at cross-section (b) Non-damaged column with different stirrups (c) 300FTC damaged column with different stirrups

5. CONCLUSIONS

A multi-scale platform for simulating the frost damage process and the mechanical impact is proposed for RC structures. It is proved that by precisely capturing the natural frost deteriortation process, the residual mechanical behaviors can be predicted nicely. With the aid of this numerical platform, the damage characters affected by reinforcing bars can also be clarified.

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Concretes with high volume fly-ash or slag; On Frost Scaling, Air Void Characteristics and Pre-Conditioning

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ABSTRACT
Results for concretes with high volume fly-ash and slag content have been collected from three independent studies. The frost scaling results from accelerated freeze-thaw tests show poor relation to the air void characteristics in the sense of specific surface area and spacing factor. Both the fly-ash and slag concretes suffer from prolonged pre-conditioning prior to freeze-thaw exposure.

Key words: Concrete, pore structure characteristics, pre-conditioning, frost scaling

1. INTRODUCTION

Results from three independent studies conducted over some years (2011-2016) [1],[2],[3] are presented. Both "normal" fly-ash cement (CEM II) and "high volume" fly-ash (FA) binders were investigated, as well as slag cements (CEM III). The relation between air void characteristics and frost scaling in laboratory tests are presented. The effect of prolonged pre-conditioning prior to start of frost testing is also discussed.

2. CONCRETE MIX DESIGN AND EXPERIMENTAL PROGRAM

Table 1 shows the concrete binder mix proportions in the three investigations. It is notable that all FA, also for contents above 35%, have been included in the calculation of w/b even though 35% is the upper limit in NS-EN 206/NA. The concretes in investigation [1] were ready-mixed and cast in-situ. The concretes used local 0-32 mm aggregate. Samples were cored from walls and sent to the laboratory. In investigation [2] and [3] the concretes were mixed in the laboratory. The concretes used 0-16 mm aggregate from Årdal, and specimens were cast in moulds.
Air void characterisation were measured according to EN 480-11:2006 [4], where air content, specific surface area and spacing factor are determined. In [1] (drilled cores) the concretes were

analysed on arrival to the laboratory; concrete maturity was 3 months (91 days). In [2] and [3] the concretes were analysed after 28 days standard curing.

Compressive strength was measured after 28 and 91 days (in [1] only 91 days), EN 12390.

Freeze-thaw testing was performed according to CEN/TS 12390-9:2006 (slab test) [5]. The concretes in [1] was only tested according to the standard procedure (1 week pre-condition), while in [2] and [3] two pre-condition regimes were given before start frost exposure (56 cycles):

1. Standard procedure: 1 week pre-conditioning in 65% RH, and start frost exposure at 31 d age.
2. 14 weeks pre-conditioning in 65% RH, and start frost exposure at 17±1 weeks age. This regime is in NS-EN 206/NA required for concrete binders that contain more than 35% slag, but in this case the concretes were also given this pre-conditioning irrespectively of binder composition.

Table 1 – Overview of concrete binder mix proportions in the three investigations (%-values are by weight of total binder)

Investi-gation	Cement type	Total FA or slag content	Silica fume content	Water/binder ratio[1]	Effective water-binder ratio[2]
[1]	CEM II/A-V	19, 32 and 48% FA	5%	0.40	0.39, 0.43 and 0.46
[2]	CEM II/A-V CEM III	17, 35 and 50% FA 48 and 75% slag	4%	0.35-0.44	0.39
[3]	CEM II/A-V	17, 30 and 40% FA	4%	0.37-0.42	0.39 (0.44 for 40% FA)

[1] all FA is included in the calculation, k_{FA}=1.0, [2] all FA is included in the calculation, k_{FA}=0,7 for extra added FA

3. RESULTS

Results from the air void characterisation, compressive strength tests and scaling after 56 freeze-thaw cycles are shown in Table 2. The effective w/b-ratio from the mix design is also given. Figure 1 shows simply the strength results - though the concretes are not directly comparable (different aggregate in [1] and w/b-ratio as indicated). There is a tendency of reduced strength with increasing FA-content, not surprisingly. In Figure 2 and 3 the two concretes containing slag are indicated with grey dots, while black ones are with FA. In Figure 2 the freeze-thaw scaling results are plotted against w/b-ratio and FA/slag-content, respectively. Most interesting is perhaps the plot against FA/slag, as both the FA- and slag concretes suffer from long pre-conditioning. Figure 3 shows the frost scaling results vs. specific area and spacing factor. The results reveal that the air void characteristics are generally superb, as found by the method EN 480-11. Traditionally, for CEM I-based concretes, a specific surface higher than around 25 mm^{-1} and a spacing factor less than around 0.25 mm have been regarded well suited for securing frost resistant concrete. The air void characteristics of these concretes score far better than this, but the relation to the scaling results is very poor. This is the case both for the standard test and for 14 weeks pre-conditioning. In NA of NS-EN 206 a scaling ≤ 0.50 kg/m^2 is criteria for frost durable concrete.

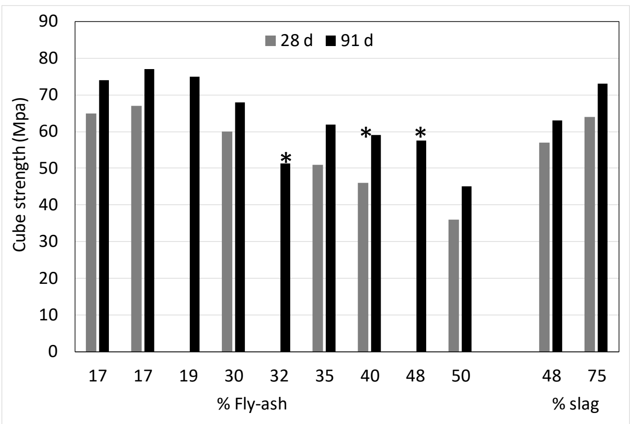

*Figure 1 – Comp. strength vs. FA/slag content. The concretes marked with * have higher w/b-ratio than 0.39, see Table 2.*

Table 2 – Effective w/b-ratio, air void characteristics, strength and freeze-thaw scaling

Refs.	Total FA or slag content	Air content[2] (%)	Specific surface area (mm^{-1})	Spacing factor (mm)	Cube strength (MPa) 28d (91d)	Scaling after 56 cycles (kg/m^2) Std. test (14 w cond.)[1]
[1]	19% FA	4.1	39	0,15	- (75[4])	0.19 (-)
	32% FA	4.3 / 4.4	23 / 25	0.24 / 0.22	- (51[4])	1.52 (-)
	48% FA[3]	1.8 / 5.5	70 / 47	0.12 / 0.10	- (58[4])	3.66 (-)
[2][5]	17% FA	4.1 / 3.9	38 / 33	0.14 / 0.17	65 (74)	0.08 (0.19)
	35% FA	4.5 / 5.0	53 / 46	0.09 / 0.11	51 (62)	0.09 (0.76)
	50% FA	4.7 / 6.0	63 / 45	0.08 / 0.10	36 (45)	1.32 (1.99)
	48% slag	5.2 / 6.2	38 / 37	0.12 / 0.12	57 (63)	0.20 (0.47)
	75% slag	7.3 / 5.1	38 / 34	0.10 / 0.15	64 (73)	0.63 (1.59)
[3][5]	17% FA	3.3	34	0.19	67 (77)	0.31 (0.73)
	30% FA	3.1	53	0.12	60 (68)	0.34 (1.28)
	40% FA	5.2	40	0.13	46 (59)	2.02 (2.96)

[1] (14 w cond.) means 14 weeks conditioning in 65% RH. [2] All results are average of two samples except 32% and 48% FA in [1] where separate results were reported. [3] In the test report it was noted that the samples contained many very small pores that were not spherical and not necessarily traditional air pores. It was also noted that the reported values were perhaps "to good". [4] Cylinder strength from drilled cores – converted to cube strength with factor 0.8. [5] Each number (X / Y) for [2] and [3] is average of two samples, and each result is from different concrete batches.

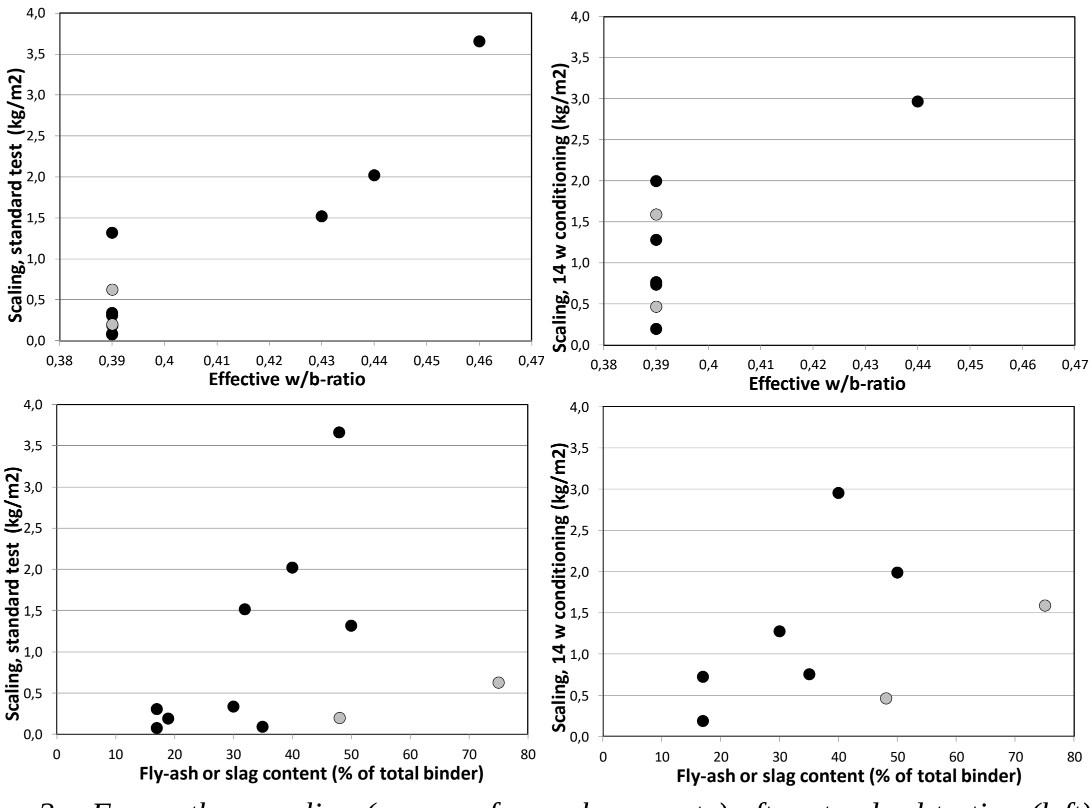

Figure 2 – Freeze-thaw scaling (average for each concrete) after standard testing (left) and prolonged conditioning (right) vs. effective w/b-ratio (top) and fly-ash/slag-content (bottom).

4. CONCLUSIONS

The air void characterization for the tested concretes with high volume fly-ash and slag contents show poor relation to the frost scaling resistance from accelerated freeze-thaw tests. All concretes suffered from 14 weeks pre-conditioning in terms of increased frost scaling. The results are to a large degree in accordance with findings in the literature, represented by [6] and [7], although the literature study have been limited.

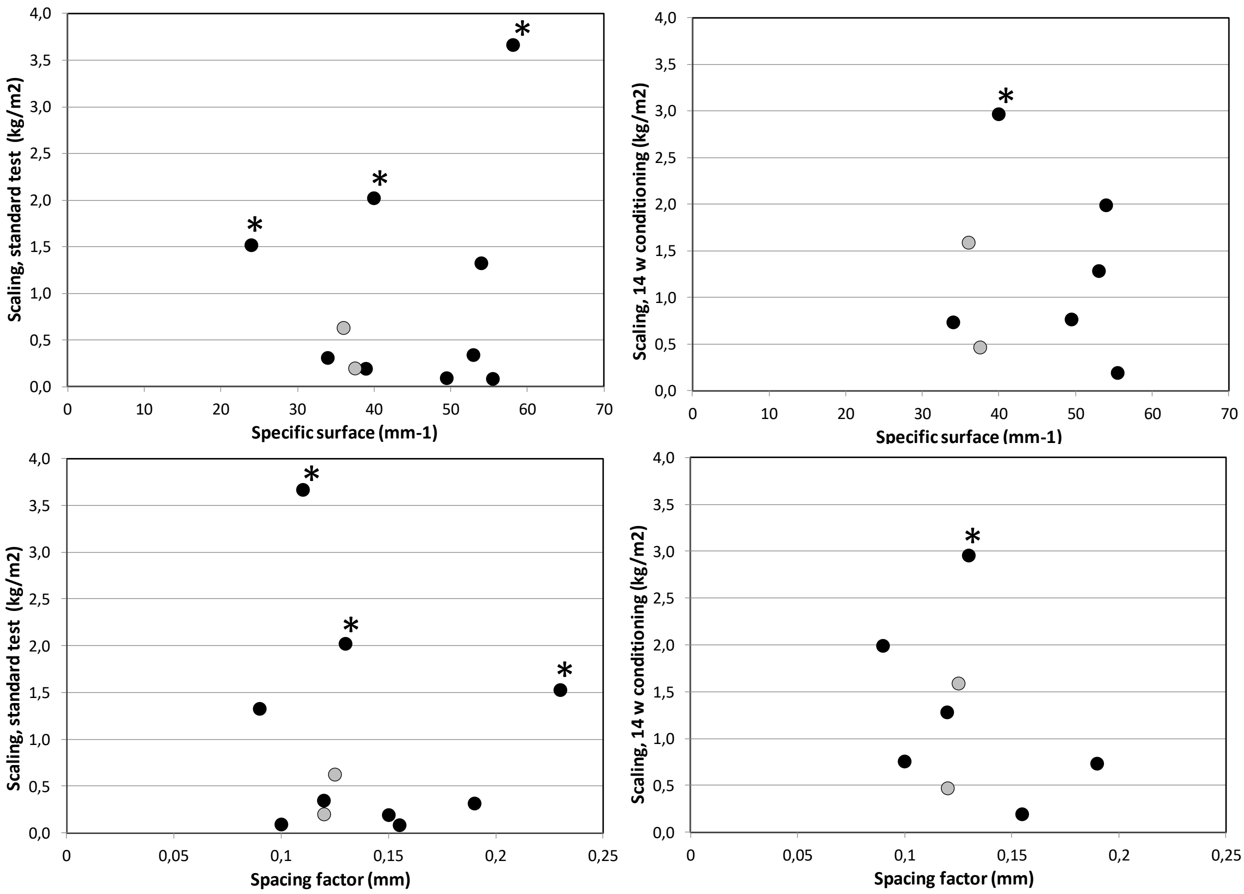

*Figure 3 – Freeze-thaw scaling after standard testing (left) and prolonged conditioning (right) vs. specific surface area (top) and spacing factor (bottom). The concretes with w/b higher than 0.39 is marked with *.*

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Frost testing of HP/HVFA concrete for severe offshore conditions

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ABSTRACT
In the rim of DaCS (Durable advanced Concrete Solutions) project, financed by Norwegian research council and the industry partners, one of the work packages is associated with production and documentation of frost durable low-carbon concrete suitable, in addition, for offshore and arctic exposures conditions. A number of freeze-thaw experiments with high-volume fly ash concrete were carried out. This paper presents some preliminary results made to proceed in the understanding of : 1.how internal and superficial frost damage occur in ASTM C666 procedure A, rapid freeze-thaw testing in fresh water (by far the most common way of frost testing concrete), 2. how this is related to water uptake during curing and during subsequent freeze-thaw, and 3. how the air entrainment contributes to frost resistance in this test. The results show that the water uptake during curing, presumably due to self-desiccation and air void filling, is much lower than the accelerated uptake due to the wet freeze/thaw. Furthermore, air voids in FA-concrete seem to be filled less than inwet curing. A surprisingly high scaling was observed for FA-concrete in these fresh water tests even without any internal damage and cracking and scaling were accelerated as both occurred.
Key words: Frost resistance, Fly ash, Air voids, Freshwater, Scaling, Cracking

1. INTRODUCTION

Concrete has proven to be the only material used in large offshore structures in subarctic exposure that could withstand severe freeze-thaw and ice loads and maintain longevity, and recently the developers of White Rose oil field on Canadian Grand Banks opted for concrete for a 120m tall semi-submersible gravity-based structure [http://westwhiteroseproject.ca/]. Increasing demand for low-carbon binders leads to an increasing need for technology on how to produce frost-resistant low-carbon concrete for these kinds of severe conditions. Fly ash concrete was found capable to withstand severe freeze-thaw cycles with moderate Fly ash dosage and when the concrete is properly air-entrained and cured [1]. However, for high fly-ash replacement and little curing frost durability can be a problem [2].
Rapid freeze-thaw testing in fresh water ASTM C666 procedure A [3] has been observed to cause surface scaling (no deicing salt) and internal damage. The last we can relate to the accelerated water uptake during freeze-thaw [4], which was found to continue during cycling

and to correlate to internal cracking [5]. The specimens of ASTM C666 are cured in lime water continuously until the start of freeze-thaw and, hence there is an initial water uptake that is caused by self-desiccation and possibly some air-void filling. This paper presents new findings on the water uptake due to self-dessication and freeze/thaw and its relation to frost damage.

2. EXPERIMENTS AND MATERIALS

All concrete mixes were exposed to ASTM C666 Proc A testing with simultaneous measurements of RDM (Relative Dynamic E-Modulus), scaling and weight increase due to water absorption. The compensation of loss of evaporable water in the scaled mass was done by multiplying dried scaled mass with the evaporable water content of concrete measured by the PF-testing [6] and corrected for the increased paste fraction at the surface.
Table 1 shows some key mix details, air-void system, water uptake in curing and compressive strengths. The mix code refers to w/b-ratio, FA/b-ratio and the air-entrainment, i.e. 0.40-35 A means w/b=0.40, FA/b = 0.35 and air-entrained, while 0.40-35 0 is a code for non-air-entrained mix. All mixes contain 4% of silica fume (SF) of binder, the slump variation was 190 - 220 mm. The w/b = 0.293 mixes were made to represent w/c = 0.45 for zero pozzolan hydration, i.e. opposed to a reference mix 0.45-0 A.
As we see, most entrained air-void systems were very effective in terms of both air-void spacing [4] and Pore Protection Factors [6]. The absorption during curing is lower for the fly ash concrete compared to the previous measurements on OPC and OPC+SF concrete [7]. The absorption during curing, similar strength and the air content of 0.293-35 A and 0.45-0 A indicate that Fly Ash (FA) did not contribute to hydration the first 14-28 days (Analyses of hydration with other methods are underway). Furthermore, the levels of absorption in Fly Ash concrete are low compared to previous experiences with this simple method on OPC and OPC+SF specimens.
Non-air-entrained mixes were produced to see whether frost-durable low-w/b FA-concrete could be made without all the difficulties associated with air-entrainment. There is evidence of concrete without air being frost durable, however, it had low w/b, at least 10% SF and no FA [8].

Table 1. Properties of fresh and hardened concrete

Mix	Paste volume [L]	Air content, fresh[1] [%]	Air content, hardened[2] [%]	Spacing factor[2] [mm]	PF [%]	Absorption during curing [vol% paste] ASTM cylinder	Cubes[3] 100 mm			Compressive strength [MPa]		
						14d[4]	28d	91d	1y	28d	91d	1y
0.40-35 A	265	5,6	4,7	0,24	30,2		6,8	7,4	8,5	59,3	71,6	81,2
0.40-35 0	263	1,2	2,8	0,79	16,0		6,1	7,2	7,8	73,2	90,2	101,9
0.45-35 A	266	5,8	4,1	0,18	32,5		5,9	7,0		50,4	63,3	
0.45-35 0	263	1,3	2,0	0,68	17,0		6,5	7,0		67	81,9	
0.293-35 A	266	5,9	5,9	0,20	36,1	5,3	5,6	6,2		81,5	93	
0.293-35 0	262	2,0	1,8	0,63	17,6	5,1	6,0	6,8		98,8	114,9	
0.45-0 A	270	5,1	6,2	0,30	31,7	6,2	6,9	8,3		77,2		

[1] Density method
[2] Acc. to Fonseca et al. and ASTM C457, scanning - 3200ppi
[3] Measured on cubes 100 x 100 x 100 mm^3
[4] Beginning of ASTM C666 testing, measured on cores 300 mm D~95-100mm

3. RESULTS AND DISCUSSION

Figure 1 shows internal damage and surface scaling during freeze-thaw cycling. From the results, it is clear that a proper air-void system with $L \leq 0.24$ mm protects the fly ash concrete whereas the OPC mix 0.45-0 A with $L = 0.30$ mm suffers internal damage.
We could clearly see that the drop in RDM is accompanied by a sudden increase in the scaling rate. From that case, for "bad concrete", internal cracking and scaling could possibly be interrelated. For stronger concretes with low water-to-binder ratio (0,293-35) or well-cured fly ash concretes (0.40-35 water-cured for 1 year) the cracking and scaling results do not complement one another. Looking at the dashed lines for non-air-entrained concretes, we can see that the resistance to freeze-thaw is improved with increased compressive strength of concrete (also valid for the air-entrained concretes), which is obtained either by reducing w/b-ratio (0.293-35 0) or by prolonging water-curing time (0.40-35 0 1y). However, none of the mixes without AEA had even passed 150 cycles with RDM being above 80%.
The relatively high scaling is surprising since the test is done in fresh water and the scaling levels are quite high after 300 cycles for the worst specimens.
Despite high PF-value for 0.45-0 A, concrete is not frost resistant.

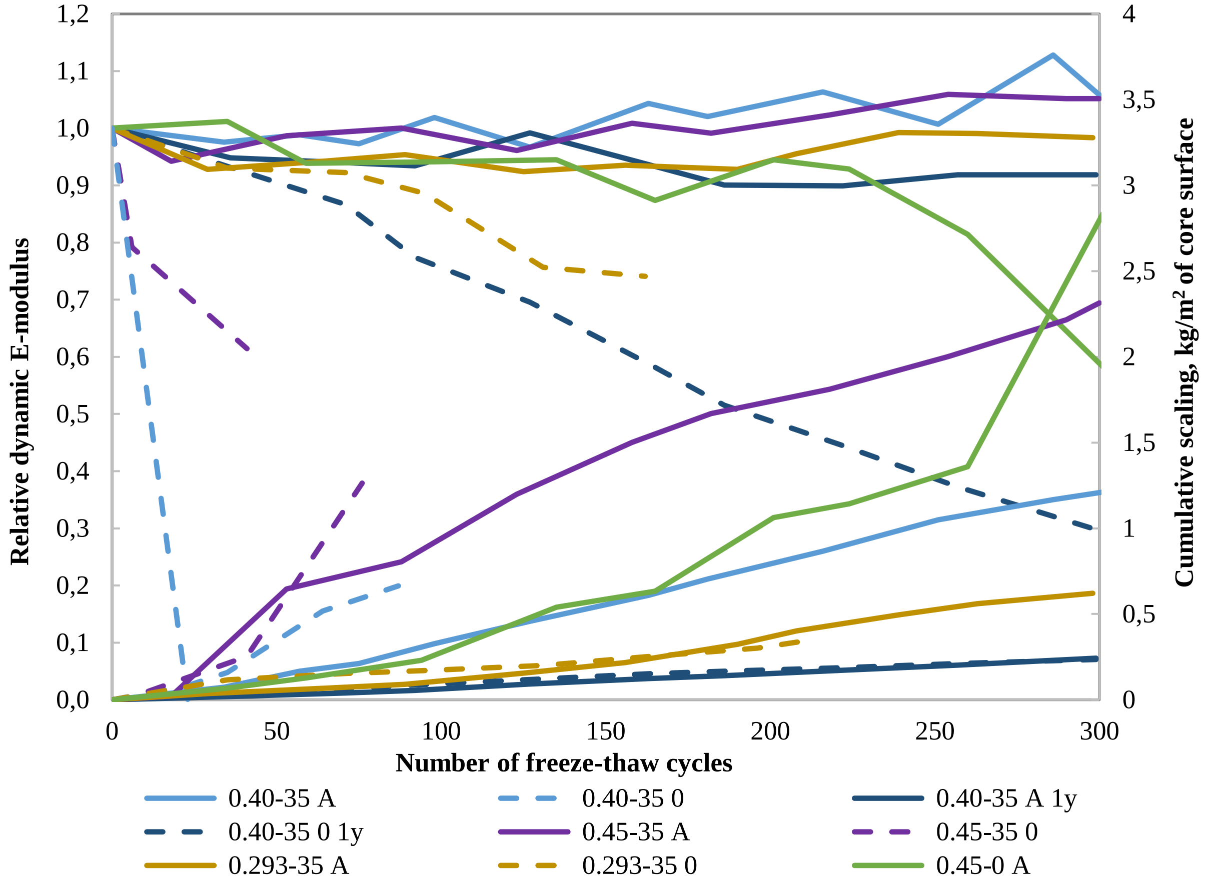

Figure 1. Relative dynamic E-modulus and cumulative surface scaling in freshwater freeze-thaw test ASTM C666 procedure A. Average values for 2 cores

Figure 2 shows the cumulative water uptake during the freeze-thaw test, compared to absorption during curing in limewater. The uptake by freeze/thaw is much larger than that due to self-desiccation. Water uptake for a reference concrete without FA resonates with the drop in RDM shown above. On the contrary, the non-air-entrained mix 0.293-35 0 failed at 164 cycles (pulse was not sent through), but it did not affect water uptake and scaling curves for measurements for 50 more cycles. Mixes with FA of the same w/(C+SF)-ratio as in reference concrete show lower water uptake and lower absorption during curing, probably due to known lower chemical shrinkage of the FA concretes. It is interesting to note that there is a negligible difference in uptake values between non- and air-entrained mix (as in the surface scaling), meaning that, unlike SF, FA does not cause increased air void filling in water curing.

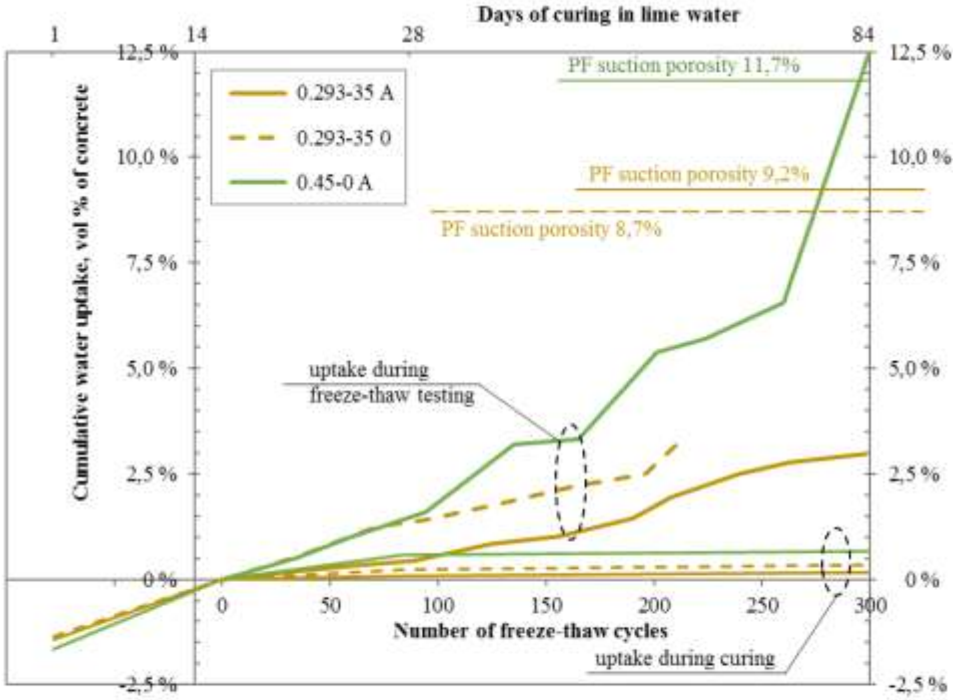

Figure 2. Cumulative water uptake during freeze-thaw testing in ASTM C666 procedure A and water absorption during curing (for selected mixes). Average values for 2 cores.

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Frost-resistance of concrete with supplementary cementitious materials – experiences from testing and field exposure

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ABSTRACT
Results from two Swedish projects studying the frost resistance of concrete with supplementary cementitious materials (SCM) carried out in the recent years are presented here. One is along-term study of exposure at three different field sites, two saline environments, a highway site and a marine site, and one outdoor site without salt. Results after 19 winter seasons are now available and presented. The other project was aimed at validating the slab method (reference method in CEN/TS 12390-9) for determining the salt-frost resistance of concrete. This method has been used in Sweden for many years and has proved successful in predicting the salt-frost resistance of Portland cement concrete. However, doubts have been raised whether the same can be said when used on concrete SCM. Both projects have been sponsored by the Swedish Transport Administration.
The results from the projects show that both slag and fly-ash diminish the salt-frost resistance of concrete. Carbonation of the surface also increases the scaling of concretes with air entraining agents, especially for slag concretes. The exposure giving the highest scaling values, is the road environment.

1. RESULTS FROM 19 YEARS FIELD EXPOSURE AND CORRELATION WITH INITIAL SALT-FROST TEST RESULTS

Specimens from more than 100 concrete mixes have been exposed on three different field sites which represent three different exposure conditions. Two of them are categorized as exposure class XF4 according to EN 206 [2]; a highway field site close to Borås where de-icing salts are used and a marine field site at the Swedish west coast. The third is a field site without salt exposure at RISE in Borås (the SP site) corresponding to exposure class XF3. The water-to-binder ratio of the mixes were 0,30; 0,35; 0,40; 0,50 and 0,75 and 9 binder combinations were tested (see Table 1).
The target air-content was also varied; 4,5 %, 3 % and only natural air.

Table 1 – Binder types/combinations investigated

Binder type/combination		Comments
1	CEM I 42.5N MH/SR/LA	Swedish low alkali, sulfate-resistant cement (Anl)
2	CEM I + 5 % silica by binder weight	Swedish low alkali cement and silica fume in the form of slurry
3	CEM I 52.5N	Swedish Standard Portland cement (Slite)
4	CEM II/A-LL	Swedish cement with 10-15% limestone filler
5	CEM II/A-S	Finnish cement with 15-18% slag
6	CEM I + 30 % slag by binder weight	Swedish low alkali cement and ground blast furnace slag added in the mixer
7	CEM III/B	Dutch slag cement, ~70 % slag
8	FRHPC	Finnish rapid hardening cement
9	FSRPC	Finnish sulfate-resistant cement

The project and all the results are described in detail in [1]. Here only results for mixes with 4,5 % air will be presented. Figures 1 to 3 show the volume change of the concrete mixes after 19 winter seasons at the highway site, the marine site and the SP site for the mixes with 4,5 % air. The volume change represents the external frost damage.

As can be seen from these figures the highway site gives the most severe exposure. It is also clear that high amounts of slag have a negative influence on the salt-frost resistance. If the w/b ratio is kept below 0,40 all binder combinations except the one with 70 % slag performs well after 19 winters even at the highway site. With 70 % slag the w/b ratio should be below 0,35.

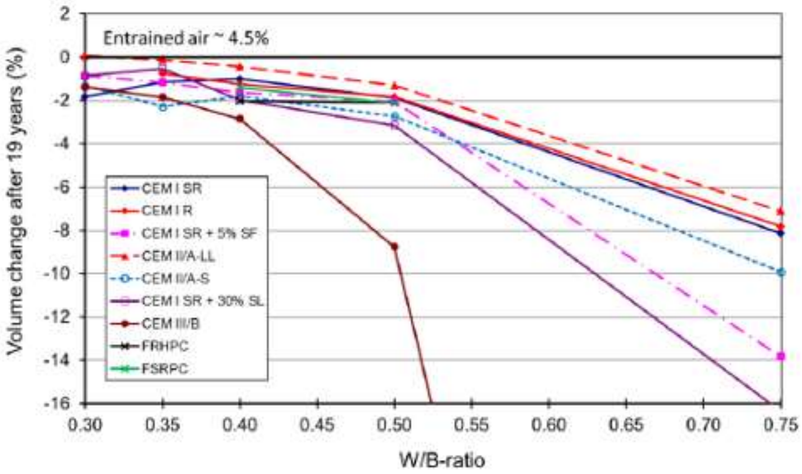

Figure 1 – Volume change after 19 winter seasons at the highway site.

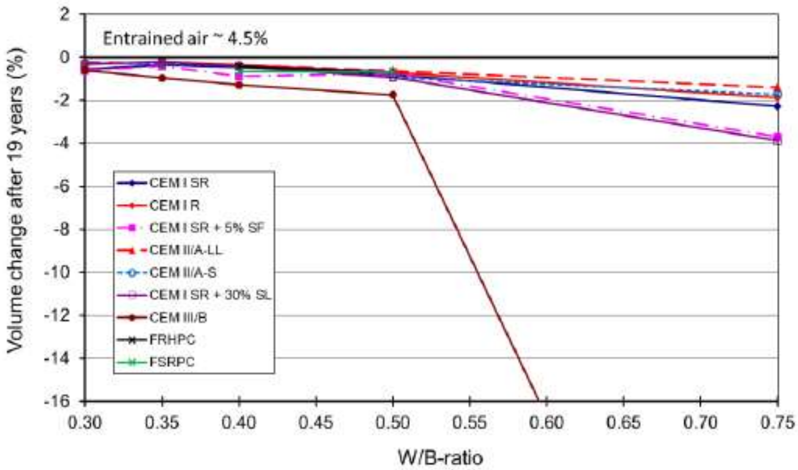

Figure 2 – Volume change after 19 winter seasons at the marine site.

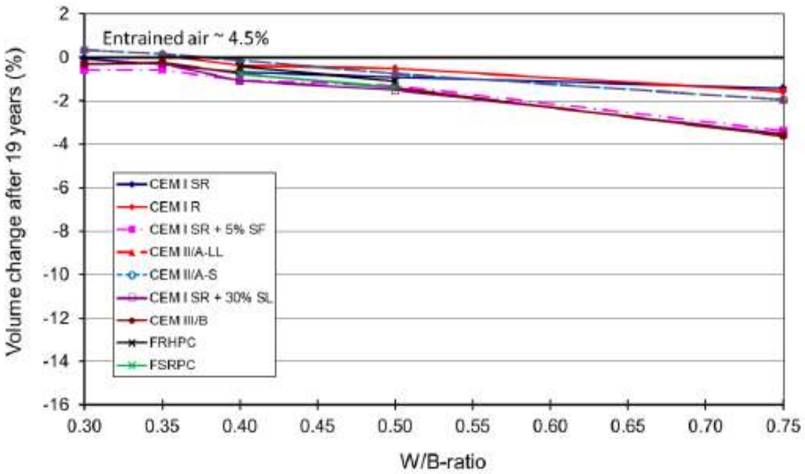

Figure 3 – Volume change after 19 winter seasons at the SP-site.

At the marine site all mixes with w/b not over 0,50 perform well, with regard to external frost damage. In Figure 4 the relation between the result from the initial slab test and the performance after 19 winter seasons at the highway site are presented. Here all the mixes of the project are included, also those without entrained air. A dangerous situation is if the mix passes the slab test but do not perform well in field, i.e. a result in the lower right quadrant. Some of the slag mixes are very close to this situation, i.e. the slab test may overestimate the salt-frost resistance.

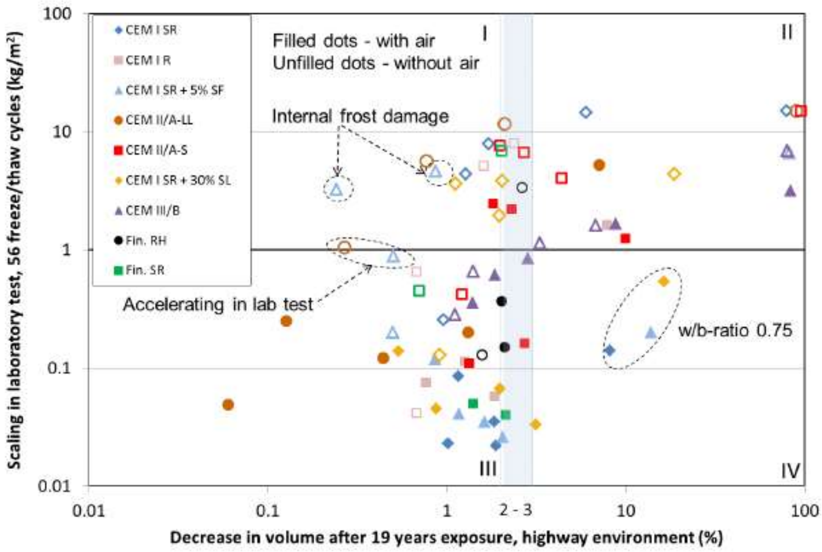

Figure 4 – Scaling results from slab test in relation to weight loss after 19 winter seasons.

2. LABORATORY TESTS OF THE SALT-FROST RESISTANCE OF CONCRETES WITH SLAG AND FLY ASH

In the second project the slab test, with five modifications of it, were used on mixes with 0 - 65% slag and 0 - 35 % fly-ash. A w/b ratio =0,45 was used (requirement for XF4 in Sweden). The modifications of the test method consisted of adding one week exposure to 1 % CO_2, since it has been noticed that carbonation influences the salt-frost resistance, and prolonging exposure to 65 % RH before the frost test starts. Putting forward the start of the cutting of the specimens from 21 days age until 91 days age was also investigated. A detailed description of the project can be found in [3].

The great influence of carbonation on the scaling after 56 frost-cycles for the different mixes can be seen in Figure 5, especially for the slag mixes. At levels below 20 % the influence is

marginal. (A and R are two different cements, number followed by S represents amount of slag, and followed by F amount of fly ash. 21 and 91 are age in days when specimens are sawed.) Delaying the start of exposure to frost cycling diminishes the scaling for all mixes, but especially when 35 % of fly ash is used. The red lines correspond to the limits for acceptable frost resistance (=1 kg/m2) and good frost (=0,5 kg/m2) resistance according to the Swedish classification [4].
Prolonging the curing in 65 % RH before exposure to frost-cycling also increases the scaling as can be seen in figure 6. (1 w=1 week = standard curing, 3 weeks & 11 weeks). As with carbonation the influence increases with increasing amount of slag and fly ash. Thus, the salt frost resistance of concretes with slag and fly ash is more sensitive to the degree of humidity and drying at early age than ordinary Portland cement concrete is.

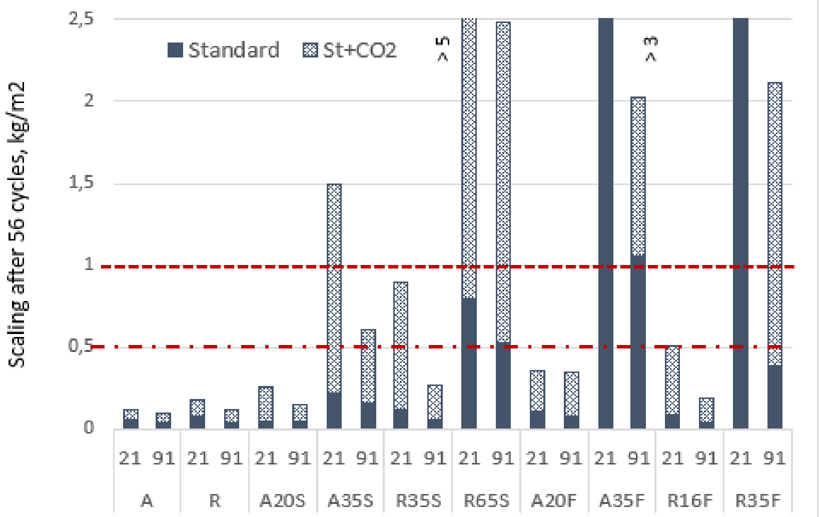

Figure 5 – Scaling after 56 cycles with (St+CO2) and without (St) previous carbonation, for specimens sawed at 21 days and 91 days age.

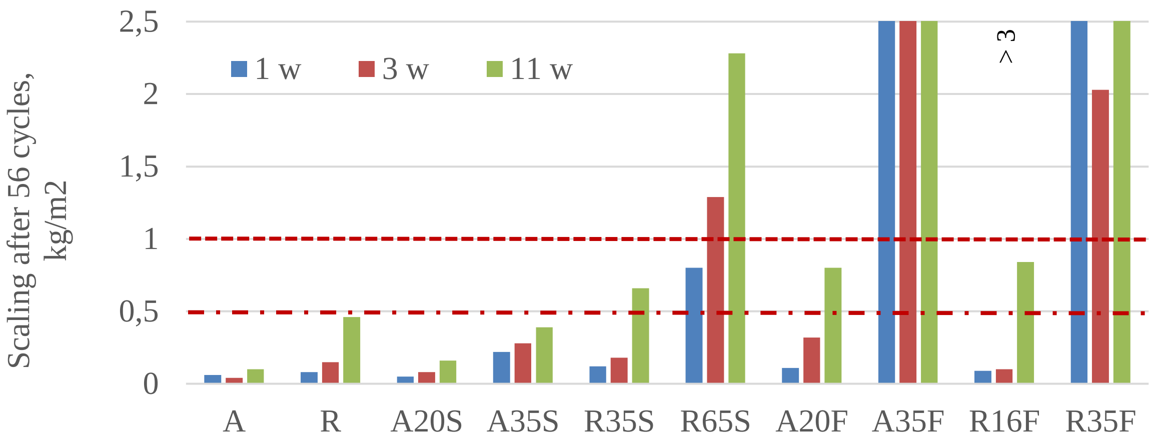

Figure 6 – Scaling after 56 cycles after 1, 3 and 11 weeks preconditioning in 65 % RH

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Assessing freeze-thaw performance of concrete - Some considerations

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ABSTRACT
Due to the variability of environmental loading, concrete is seldom subject to the effect of a single deterioration mechanism. The possible synergetic effect of two or more deterioration mechanisms, acting simultaneously or intermittently, affects the deterioration rate resulting in service life predictions remarkably different from expected.
In Finland, the most common deterioration mechanisms affecting concrete are freeze-thaw and reinforcement corrosion due to both concrete carbonation and the ingress of chlorides. This paper provides a brief glimpse of research conducted at VTT Technical Research Centre of Finland Ltd., on the effect of combined deterioration mechanisms on concrete durability performance. The scope of the research has been broad, i.e., for example evaluating how cracks resulting from freeze-thaw deterioration influence chloride ingress; how freeze-thaw cycles without cracking assist chloride transport in the concrete; or how carbonation changes the surface properties affecting freeze-thaw scaling. Results are based on both accelerated laboratory testing and in-situ exposure testing from field stations.
The research results show that a holistic approach to concrete durability performance should be taken into account when estimating the service life of concrete structures.

Key words: Concrete, frost resistance, testing, performance, freeze-thaw, deterioration

1. INTRODUCTION

Concrete exposed to extremely harsh winters such as those experienced in Nordic countries is expected to perform in such difficult conditions despite the unique combinations of degradations mechanisms that can occurs, such as freeze-thaw damage, chloride ingress, carbonation, among others. Research has recently focused on coupling degradation mechanisms to realistically reflect *in situ* performance conditions. For instance, evaluating how cracks resulting from freeze-thaw influence chloride ingress, or how carbonation changes the surface properties and thereby influencing frost-salt scaling and chloride penetration. In this paper, the results of research projects at VTT based on assessing coupling deterioration mechanisms are presented. Durability performance has been assessed both by accelerated laboratory testing and from *in situ* exposure results and from field stations.
This research provides the background for understanding of service life performance, and supports a holistic approach towards ageing management of infrastructure.

2. VTT RESEARCH ON FREEZE-THAW PERFORMANCE

2.1 Assessing coupled deterioration mechanisms including frost attack, carbonation and chloride penetration (DURAINT)

DURAINT was a research project studying coupled deterioration mechanisms including frost attack, carbonation and chloride penetration. Research was based on an extensive laboratory-testing regime, in parallel to the exposure of several concrete specimens at field stations. Laboratory testing was designed to investigate the coupled interaction of deterioration mechanisms [1]. This included evaluation of freeze-thaw internal damage and surface scaling, chloride ingress and carbonation. The deterioration mechanism were also cross-studied, for example, both the effect of freeze-thaw on carbonation and the effect of carbonation on freeze-thaws, were studied [2]. Details of the mixture designs as well as fresh and hardened properties are presented in other reports [3].

In general, standardized laboratory testing methods were used. Freeze-thaw with and without salt was performed according to the slab test methods to assess internal damage and surface scaling. Natural carbonation was tested at room exposure (RH 65%, 20°C), while accelerated carbonation was performed at 1% or 4% CO_2 concentration (RH 60%, 20°C). For complete details of the study, please consult following references [3, 4].

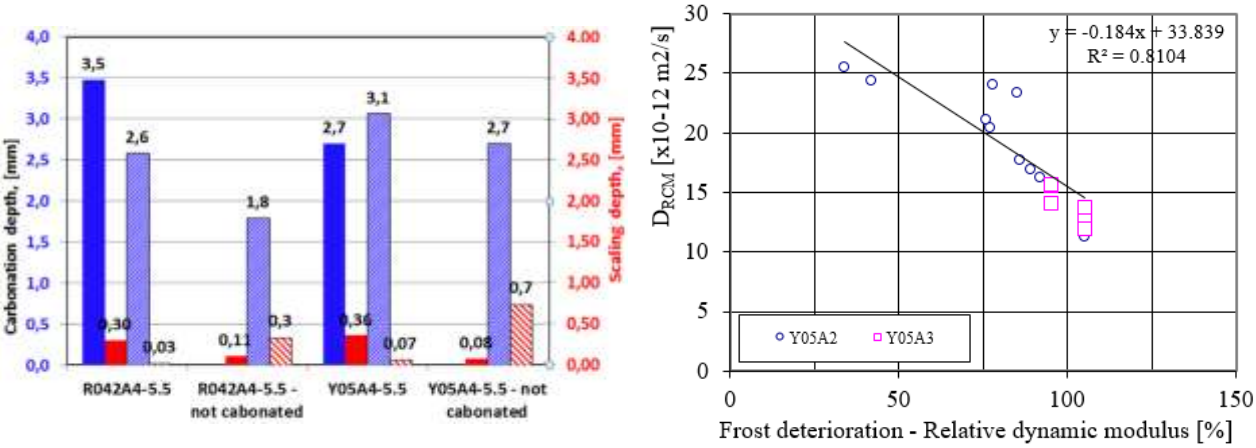

Figure 1 – Example of results from DURAINT [6]. (a) Standard freeze-thaw scaling test, and two modified methods, with aged and carbonated/non-carbonated surface. Surface carbonation at RH 65 % (1.3 years) considerably increased the freeze-thaw salt scaling degree. (b) Relationship between the effect of freeze-thaw internal cracking (measured by RDM) and chloride migration coefficient (D_{RCM}).

The results of this project show the synergetic effect on concrete deterioration of coupled deterioration and quantitatively support that a realistic ageing should be adopted for predicting deterioration performance.

2.2 Concrete Service Life Assessment: modelling frost attack degradation in the presence of chlorides (CSLA)

CSLA Project aimed at investigating the effect of selected freeze-thaw exposures on the chloride ingress in concrete [5]. Two concrete were studied with a water/binder ratio of 0.42 and 0.55 (referred to as B42 and B55, respectively). CEM I 42,5 N-SR3 was used to minimise chloride binding. Four different freeze-thaw exposure cycles (varying minimum negative temperature and length of time at which the minimum temperature was held) were chosen to promote varying freeze-thaw behaviour of the brine solution in the pore structure of the concrete [6]. The first three exposure cycles, where the minimum temperature reached is studied, follow closely the reference test procedure curve for freeze-thaw scaling, varying only in the minimum temperature reached: -20°C (reference), -10°C and -5°C. Each of these has 24 hour cycle duration. The fourth exposure follows the reference exposure (-20 °C), except that the lowest temperature is maintained for 60 hours, so that the total length of one exposure cycle is 84 hours. The total number of freeze-thaw cycles the concrete samples were subject to were 112 cycles for the 24-hour exposure, and 41 cycles for the 84-hour exposure. As a reference for measuring the chloride ingress due to diffusion, additional exposures were conducted with full immersion at constant +5 °C, and +20 °C. At regular intervals during exposure cycles, the following measurements were conducted: scaled material mass, mass variation (water uptake – scaled material) and fundamental frequency. Chloride profiles were determined on specimens removed from the freeze-thaw cambers.

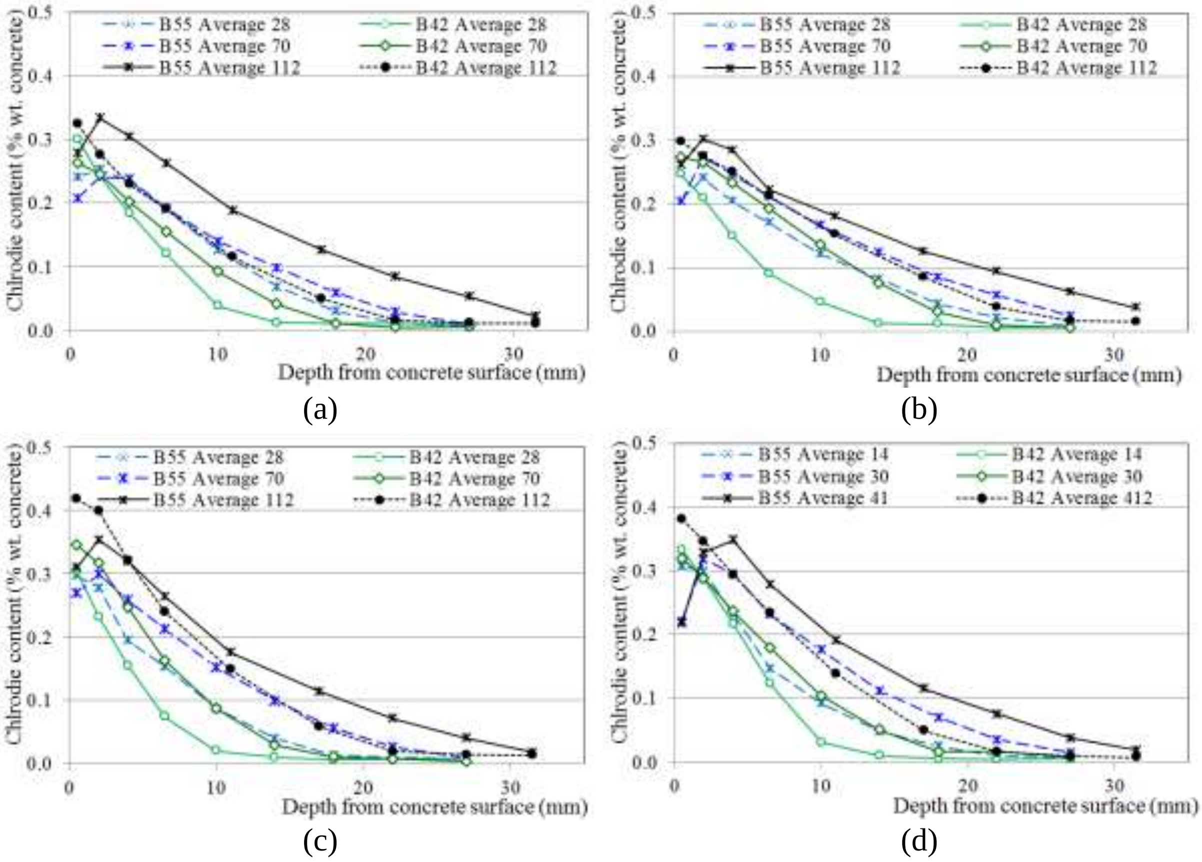

Figure 2 – Chloride profiles measured in water after 28, 70 and 112 days of exposure cycles for both B55 and B42 concretes, at (a) constant +5 ºC, (b) constant +20 ºC, (c) -20 ºC with 24h cycle, and (d) -20 ºC with 84h cycle [7].

The results of this project show that chlorides can continue to ingress into concrete during the long freezing periods. Measured chloride profiles for ponding and freeze-thaw exposure were similar. Ingress during freezing can be attributed to the micro ice-lens pumping at the surface, and the temperature gradient induced ice growth movement in the pore structure of the concrete that is aligned with the direction of chloride ingress due to diffusion.

3. QUANTIFYING RELEVANT CONCRETE CHARACTERSITCS

Much research effort into freeze-thaw performance is focused on quantifying the consequences of freeze-thaw loading on concrete performance as measured by the amount of damage occurring. Many different variables are studied (additions, admixtures, compositions, etc.). The quality of the concrete is assessed based on the outcome of accelerated tests where the amount of damage is measured (internal cracking and/or surface scaling). Mix design fulfils the requirements for water/binder ratio and strength class provided in standards and performance is related to the quality of the entrained air (specific surface and spacing factor). Performance assessment focuses on the outcome, the resistance to damage.
Possible due to the current (incomplete) mechanistic understanding of the deterioration process, of the obvious parameters that are expected to control the damage process, only the quality of the air entrainment is systematically measured. Other parameters such as the characteristics of the porosity network that relate to moisture transport and the concrete strength (tensile) which relate to the stresses induced by ice growth are rarely quantified and specifically studied.
Current research focuses on assessing freeze-thaw performance as a function of mix design. However, mix design should be the means to ensure that the concrete characteristics needed to obtain adequate freeze-thaw performance are reached.
Research that defines what the governing concrete characteristics are is needed. Research should identify what are these main characteristics and how they interact with each other when assessing freeze-thaw performance.

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Monitoring of the freeze-thaw attack on concrete

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ABSTRACT
The transferability of lab-test results to practical conditions is an important aspect. In a short time the test should give information on the durability of concrete for the whole service life of the structure. To enable that, the testing conditions need to be well chosen. The paper presents monitoring results showing the water transport and freezing and thawing of water inside the concrete during the CIF-test. A comparison to monitoring data of structures is possible.

Key words: Concrete, freeze-thaw attack, monitoring, resistivity.

1. INTRODUCTION

Hydraulic structures are solid structures that place special requirements on concrete properties. A low heat of hydration of the concrete is necessary to minimize restraint. This results in certain limitations concerning cement properties and content. To assure a sufficient freeze-thaw resistance in exposure class XF3, the concrete of hydraulic structures of the Federal Waterways and Shipping Administration (WSV) has to undergo CIF testing according to the BAW Code of Practice "Frost Resistance Tests for Concrete" [1]. One question which has been discussed ever since the CIF test became mandatory in 2004 is the transferability of test results to the performance of the exposed concrete during operation of the structure. For this reason, a monitoring system was installed on several structures [2] to contribute to new findings concerning temperature exposure and the degree of water saturation of concrete under different conditions. Taking the example of a lock the data was evaluated to receive an impression of the intensity of freeze-thaw attack and to compare that to the actual condition of the structure [3].

2. MEASURING SYSTEM

A non-destructive determination of the degree of saturation of concrete is only possible using indirect measurement methods. A continuous, depth-dependent measurement of the resistivity was transferred to the degree of saturation by means of a calibration in the laboratory [2]. The resistivity measurement was conducted using a multiring electrode (MRE). The MRE is a sensor consisting of several rings of stainless steel, each with a thickness of 2.5 mm, with an insulating plastic ring between two steel rings. It enables AC resistance measurements of the concrete between two adjacent steel rings in eight steps at a frequency of 10.8 Hz and at a depth of 7 to 42 mm from the concrete surface. The measuring depth can be increased to 87 mm using two MREs and a distance piece [4]. A multitemperature probe (MTP) is installed near the MRE in order to monitor temperature exposure. The MTP is equipped with eight PT 1000 sensors to facilitate temperature measurements at eight different distances from the concrete surface. Fig. 1 shows a typical arrangement for one measuring point and the installation in a structure.

Figure 1 – MRE and MTP (left), Sensor installation in a structure (right)

3. MONITORING OF A FREEZE-THAW ATTACK

3.1 General

If the degree of water saturation reaches a critical level inside the concrete a freeze-thaw damage can occur if the concrete temperature is low enough that the water in the pore structure can freeze [5]. The freezing temperature of water in the pore structure depends on the pore size [6, 7]. At temperatures below 0 °C there will be water in the liquid and frozen state inside the pore system of concrete. Concerning the transferability of results of laboratory tests to practical application one important aspect is the comparison of the conditions in lab-tests and structures.

3.2 Effect of temperature on the resistivity

The Arrhenius equation (Equation 1) is used to compensate the influence of temperature on the resistivity of concrete [8]. This equation enables to transfer the resistivity measured at different temperatures at structures to a temperature of 20 °C used for the calibration test in the lab. The constant b depends on concrete properties and the degree of saturation [8].

$$\rho_{el} = \rho_{el,0} \cdot e^{b\cdot\left(\frac{1}{T}-\frac{1}{T_0}\right)} \qquad (1)$$

ρ_{el} Resistivity at temperature T in Ωm
$\rho_{el,0}$ Resistivity at temperature T_0 in Ωm
T, T_0 Absolute temperature in K
b Constant in K

When aiming at a determination of the degree of saturation at real structures with varying temperatures by means of resistivity measurements a good compensation of temperatures effects is required. Otherwise changes of the resistivity may be caused either by temperature or by changing degrees of saturation. Equation 1 can compensate temperature effects when the water in the pore structure is in a liquid phase. When water freezes the transport of ions is affected as ice hardly contributes to the transport of ions as its resistivity is very high [11]. Equation 1 is not able to compensate for that [2, 3]. As a result the resistivity increases dramatically in a very short time which is much shorter than observed when wet concrete is drying. These observations were attributed to freezing of water in the pore structure [2, 3]. Similar observation were made by [9, 10] for cement pastes. This observation is very helpful as it enables to monitor freeze-thaw attack in laboratory tests as well as in real structures. Even without a complex calibration of the resistivity to the degree of saturation it is detectable if water freezes inside the concrete. If the degree of saturation is below a concrete specific level ice formation does not occur [2, 3, 11]. This would imply that the micro-ice-lens pump cannot be activated. These correlations are very helpful for the interpretation of resistivity monitoring data.

3.3 Application of the temperature influence on resistivity

In order to describe the effect of freezing water in the pore structure on the resistivity a factor $F_G(T)$ (equation 2) was introduced [2] which describes the ratio of resistivity after compensating the temperature influence in the frozen state of the pore water to the resistivity in a liquid state.

$$F_G(T)=R_g(T) / R_f \qquad (2)$$

R_f Temperature-compensated resistivity (liquid pore water) before freezing
$R_g(T)$ Temperature-compensated resistivity (frozen pore water) at a temperature T

The hypothesis is that the factor $F_G(T)$ indicates the amount of freezable water at a certain temperature. This needs to be investigated. If the hypothesis turns out to be true the temperature dependency of the resistivity might enable to get information on the pore structure of the concrete by means of the aforementioned correlation between the pore size and the freezing temperature of water in the pores.
To further investigate this, two blocks with non air-entrained concrete with blast-furnace slag cement and fly ash were casted (Fig. 2). MRE and MTP were installed. Data was monitored under practical conditions and after coring and specimen preparation the monitoring was continued during the CIF-test to compare both exposure situations. As an example Fig. 3 shows results of core samples which were tested at a high age of 6.5 years in the CIF-Test. The data of the resistivity measurements are compensated for temperature effects according to equation 1.


Figure 2 – Concrete blocks (left) with sensors (right)

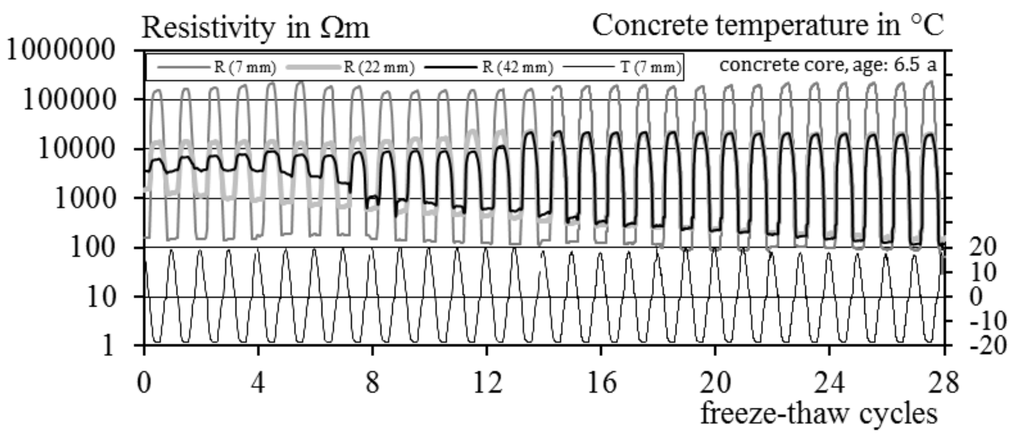

Figure 3 – Temperature compensated Resistivity at different distances to the concrete surface

Close to the surface (7 mm) a resistivity of about 120 Ωm is observed at temperatures higher than about 0 °C. Inside the specimen (42 mm) the decrease of resistivity at temperatures higher than 0 °C at the beginning of the test (about 3500 Ωm) and after 28 freeze-thaw cycles (about

120 Ωm) is a result of the frost induced water ingress into the specimen. At the end of the test comparable resistivity values are measured as close to the surface. This hints at comparable degrees of saturation inside the specimen caused by the micro-ice-lens pump according to Setzer [13]. At temperatures below about 0 °C the resistivity rises up to about 150000 Ωm at a temperature of about – 20 °C, resulting in a factor F_G(-20°C)=1500 which remains more or less constant close to the surface (7 mm) during the complete test. At a distance to the surface of 42 mm the factor F_G(-20°C) rises from about 2 at the beginning of the test up to about 160 after 28 freeze-thaw cycles. It is assumed that this is caused by the frost induced rising degree of saturation. More freezable water is available resulting in a rising factor F_G(T).

Correlations of resistivity data to internal damage and scaling including further results of lab-specimen and cores are presently being analysed and verified with observations of structures.

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