Frost Deterioration Process and Interaction with Carbonation and Chloride Penetration - Analysis and Modelling of Test Results

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Summary

This report represents the research done within the Task 5 - Service life models with interaction, of the DuraInt Project.

The overall objective of this research was to analyse the frost test results obtained from both the field and laboratory and to study the effects of frost attack on other types of degradation in concrete structures, namely carbonation and chloride penetration.

As it was desired to apply these findings to practical service life design, a special effort was made to determine "interaction factors" for service life models based on the "factor approach". By the interaction factors the accelerating effect of frost attack on other types of degradation is taken into account. The report contains several tables of interaction factors which could serve as a basis for further study with the goal of application to the service life design of structures.

In addition, the interaction of different degradation mechanisms is demonstrated using computer simulation. An example of non-frost resistant concrete in an edge beam was studied. By computer simulation, the influence of both internal frost attack and frost scaling on the process of carbonation and chloride penetration could be illustrated.

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Preface

This publication was made as a report of the project "DuraInt" which began in 2008 with the objective of evaluating the effect of interacted deterioration parameters on the service life of concrete structures in cold environments. The project DuraInt included both field and laboratory tests together with theoretical and computational efforts for the development of practical service life models. Especially the interaction of degradation mechanisms was studied. The DuraInt project was funded by TEKES (Finnish Funding Agency for Technology and Innovation) together with the Finnish Transport Agency (FTA) and companies. Much international cooperation in the form of visits, training and seminars was included in the project program.

Participants of the steering group in the DuraInt project were the following:
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Abstract

This report represents the research done within the Task 5 - Service life models with interaction, of the DuraInt Project.

The overall objective of this research was to analyse the frost test results obtained from both the field and laboratory and to study the effects of frost attack on other types of degradation in concrete structures, namely carbonation and chloride penetration.

As it was desired to apply these findings to practical service life design, a special effort was made to determine “interaction factors” for service life models based on the “factor approach”. By the interaction factors the accelerating effect of frost attack on other types of degradation is taken into account. The report contains several tables of interaction factors which could serve as a basis for further study with the goal of application to the service life design of structures.

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Contents

1 Introduction

Using field tests for monitoring the long term performance of materials and structures is necessary for developing reliable degradation models. The degradation models are needed in all lifetime engineering, such as service life design, LCC and LCA analyses, risk analyses and life cycle management systems. Using measurement data on real degradation it is possible to improve degradation models which again can be used in the design and analysis processes.

The interaction between degradation mechanisms is an obvious consequence of concrete being exposed to the environment. It is therefore natural to assume that internal frost attack affects frost scaling, carbonation and chloride penetration. In addition, frost scaling interacts with both carbonation and chloride penetration, and vice-versa.

This report represents the research done within the Task 5 - Service life models with interaction, of the DuraInt Project. The overall objective of this research was to analyse the frost test results obtained from both the field and laboratory and to study the effects of frost attack on other types of degradation in concrete structures, namely carbonation and chloride penetration. Figure 1 presents the strategy adopted for modelling the interaction between degradation mechanisms.

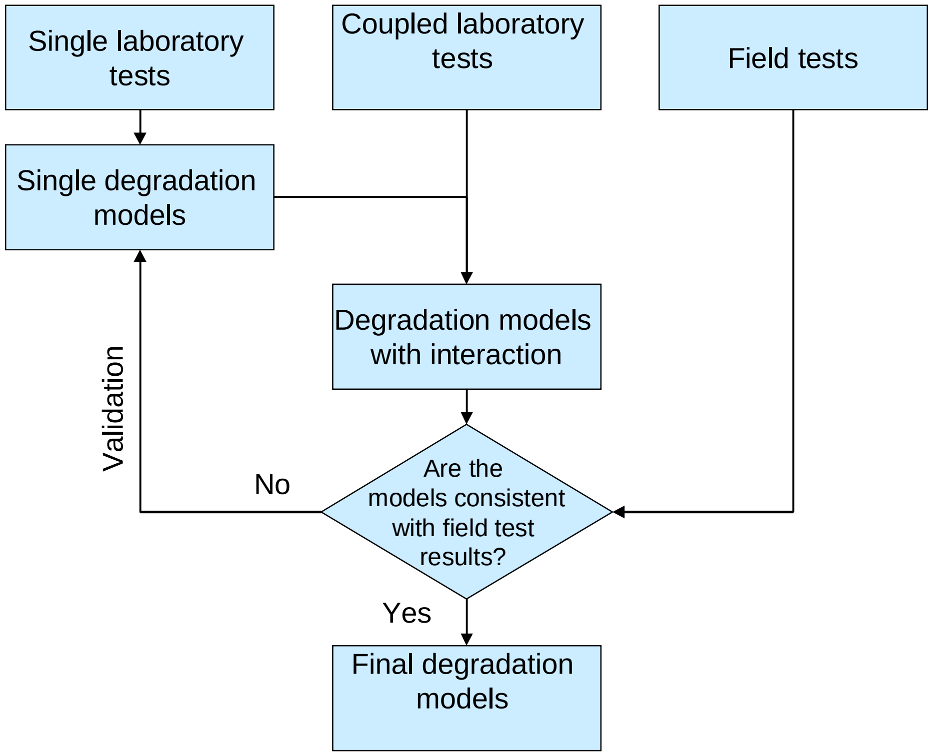

Figure 1. DuraInt strategy for creating models for interaction between degradation mechanisms.

As a result of the interaction between the degradation mechanisms, there is a need to model the combined effect and assess the real rate of degradation. This constitutes the main objective of this report. A theoretical study on this subject was presented by E. Vesikari in 2009 [1].

An additional objective was to develop practical service life models based on the "factor approach". The objective was to update the service life models of the Finnish national codes for concrete structures and supplement them with "interaction factors" related to different degradation modes. In this report several tables of interaction factors have been presented.

Another aim of the project was to apply computer simulation to visualize the interaction of degradation processes. For this purpose the simulation software developed at VTT in the 1990's was applied after updating it with the newly developed degradation models [2, 3]. In the simulation software, degradation models are applied in a differentiated form which allows taking into account the interaction between degradation modes at every time step.

2 Frost scaling

In this chapter two models for the estimation of concrete scaling due to frost attack are proposed. The models parameters are defined for a specific data set based on *in situ* exposure measurements. In addition, a model is also proposed for estimating the scaling of concrete slabs in laboratory tests. Finally, a schematic approach to the Service Life Design (SLD) of concrete structures subjected to frost scaling is presented. The approach is based on the Factor Method used in the Finnish Concrete Code.

2.1 Model function

The preliminary analyses of the field test results show that the scaling of concrete was found to be approximately linear with time, proportional to a power of water-cement ratio and inversely proportional to a power of air content. The scaling rate was also found to be strongly dependent on the cement type. Accordingly, the chosen form of the degradation function was the following.

$$\Delta V = k_{cem} \cdot B \cdot \frac{(w/c)^{n_1}}{a^{n_2}} \cdot t \tag{1}$$

where:

ΔV	decrease of the volume of the concrete specimen, %;
t	time, years;
w/c	water/cement ratio;
a	air content, %;
B	regression coefficient;
n_1	exponent of water/cement ratio;
n_2	exponent of air content, and
k_{cem}	cement factor.

From Eq. (1), it can be seen that the scaling of concrete is considered to be linear with time, and that the degradation starts when $t = 0$.

The dimensions of the test specimens were 150 mm x 150 mm x 75 mm. If it is considered that the volume change of the specimens is due to the scaling from the top surface only, then Eq. (1) can be transformed into Eq. (2) which shows the corresponding scaling depth of the top surface.

$$d = 0.75 \cdot B \cdot k_{cem} \cdot \frac{(w/c)^{n_1}}{a^{n_2}} \cdot t \tag{2}$$

where:

d	depth of scaling, mm, and
k_{exp}	coefficient of exposure.

For laboratory tests, the measure for surface scaling is the mass of disintegrated concrete during the test and expressed as kg/m^2. For the loss of mass the following equation, analogous to Eq. (1), is proposed:

$$\Delta m = k_{cem} \cdot E \cdot \frac{(w/c)^{n_3}}{a^{n_4}} \tag{3}$$

where:

Δm	weight of material scaled from the surface of the specimen, kg/m^2;
E	regression coefficient;
n_3	exponent of water/cement ratio, and
n_4	exponent of air content.

Considering an average volumetric mass of concrete of 2300 kg/m^3, the corresponding depth of scaling (d, in mm) can be expressed as:

$$d = k_{cem} \cdot \frac{E}{2.3} \cdot \frac{(w/c)^{n_3}}{a^{n_4}} \tag{4}$$

In many Finnish codes and standards the P-factor is used to evaluate the frost resistance of concrete. The P-factor, developed based mainly on laboratory tests, is assumed to be proportional to frost resistance and service life of a structure and inversely proportional to the rate of frost scaling. A detail account on the calculation of the P-factor can be found in the *Siltabetonien P-lukumenettely* (P-factor method for bridge concrete, in Finnish) report [4].

The P-factor is determined based on the composition of concrete as follows:

$$P = \frac{46 \times k_{cur} \times k_B}{\frac{10 \times (WAS)^{1,20}}{\sqrt{a}} - 1} \tag{5}$$

where:

k_{cur}	curing factor;
t_{cur}	curing time, days;
k_B	binding agent factor;
a	air content, and
WAS	reduced (water + air)/binder ratio.

As the P factor is assumed to be inversely proportional to rate of scaling, the following model function is proposed for the analysis:

$$\Delta V = \frac{K}{P} \cdot t \tag{6}$$

where K is a regression coefficient.

The regression coefficients, exponents and cement factors presented in the proposed models are determined by regression analysis of the model equation to *in situ* and laboratory data. The Method of Least Squares is applied, using MS Excel's Solver Function with a nonlinear Generalized Reduced Gradient algorithm. All the variables were initialized with the value 1.0. The parameter precision was fixed at 10^{-5} and the maximum number of iterations is limited to 10000.

2.2 Data for analyses

The concrete specimens of the DuraInt project that are subject to *in situ* exposure are still very young [5]. At the time of writing of this report, the concrete specimens have only been exposed to three winters, and therefore, are not yet sufficiently degraded to be modelled. In fact, this constitutes one of the major difficulties in any attempt to model frost degradation since the accepted levels of damage are low and time is required for adequate quality concrete to show signs of degradation.

2.2.1 *In situ* data - BTB field tests

To evaluate the proposed models the results of another research project, *BTB - Beständighet Tösaltad Betong* (Durability of concrete to deicing salts, in Swedish) [6, 7], where used. The BTB project's test field was established in 1996 by the side of motorway Rv40, near the city of Borås, in southern Sweden. From this project, the results of the *BTB400 additional series* were used, with up to 12 years of exposure to the XF4 environmental exposure class according to the SFS-EN 206-1:2001 [8], i.e., high moisture conditions, low temperatures and high amount of de-icing salts). The data of the *BTB400 additional series* was used due to the rather poor performance of the concretes of the *BTB main series* as a result of problems with the air entrainment.

The BTB additional series was composed of 66 concrete mixes with water-cement ratios ranging from 0.3 to 0.5. Concrete was produced without air, or with a target air content of 4.5%. The 11 binders used in the mixes are:

- Swedish Anläggning cement - CEM I 42,5 N BV/SR/LA
- Swedish Slite standard cement - CEM I 52,5 R
- Swedish Anläggning cement + 5% silica fume
- Swedish Portland-limestone filler cement - CEM II/A-LL 42,5 R
- Finnish standard cement (Yleissementti) - CEM II/A-M (S-LL) 42,5 N
- Swedish Anläggning cement + 30% blast furnace slag
- Dutch blast furnace slag cement - CEM III/B
- Finnish Rapid cement - CEM II/A-LL 42,5 R
- Swedish Anläggning (Modified) - reference unknown
- Finnish Sulphate Resisting cement (SR) - CEM I 42,5 N-SR
- Finnish Mega - CEM I 42,5 R.

Detailed test results can be found in the reports of the BTB project [6, 7]. Of the tests performed on concrete specimens, those that are of interest for the frost scaling modelling are:

- Frost-salt test according to SS 137244 [9];
- Air pore analysis complying with ASTM C 457 [10];
- Ultrasonic pulse transit time according to CEN /TR 15177 [11].

In Appendix 1 the basic data set used for the regression analysis is presented.

2.2.2 Laboratory data - DuraInt and BTB

In the case of laboratory testing, in addition to the testing of the concrete mentioned in 2.2.1, DuraInt project laboratory test results were also used [12].

The frost scaling test was performed according to the standard CEN/TS 12390-9 (slab test) [13].

2.3 Modelling the frost scaling of field specimens

The regression analysis was performed considering that the k_{cem} parameter for Anläggning cement = 1.0. Given that product of $k_{cem} \cdot B$ is constant for a single regression analysis, by normalising the k_{cem} parameters to the Anläggning cement it is considered that the influence of the cement and the B parameter can be separated. In the following, only a summary of the results of the regression analysis are presented.

2.3.1 Analysis of BTB additional series *in situ* data

The BTB additional series was assumed to represent better the performance of test concretes as the quality of air-entrainment was guaranteed in this series. Practically no internal frost damage was observed in this series. For this reason all the specimens were accepted for the frost scaling analysis.

In Tables 1 & 2 the results of the regression analysis are presented. The average values and coefficient of variation (CoV), calculated between 2 - 12 years, for parameters k_{cem}, B, n_1 and n_2 are given. In Eq. (7) the proposed model, after regression analysis of the data, is presented:

$$\Delta V = k_{cem} \cdot 3.15 \cdot \frac{(w/c)^{4.56}}{a^{0.61}} \cdot t \tag{7}$$

Table 1. Average values and coefficients of variation for k_{cem} parameter.

k_{cem}	Average	CoV
Anläggning	1.000	-
Slite standard	1.771	24.97
Anläggning + 5% SF	1.588	10.00
Portland-limestone filler	2.240	26.71
Finnish Standard	2.806	17.58
Anläggning + 30% GGBS	2.879	12.56
Blast furnace slag	11.339	20.16
Finnish sulphate resisting	1.902	16.57
Anläggning modified	1.466	13.61
Finnish Rapid †	1.565	18.13
Finnish Mega †	2.609	28.07

† based on one composition

Table 2. Average values and coefficients of variation for model parameter B, n_1 and n_2.

Parameter	Average	CoV
B	3.145	18.87
n_1	4.559	11.23
n_2	0.610	8.00

In Figure 2 the regression values over time for the parameters B, n_1 and n_2 determined for the BTB additional series is presented. The variation in time of the three parameters seems to tend to a constant value. An initial variation is expected in the beginning years as concrete, exposed to the extremities of the environment, has characteristic performance of swelling and contraction prior to active scaling.

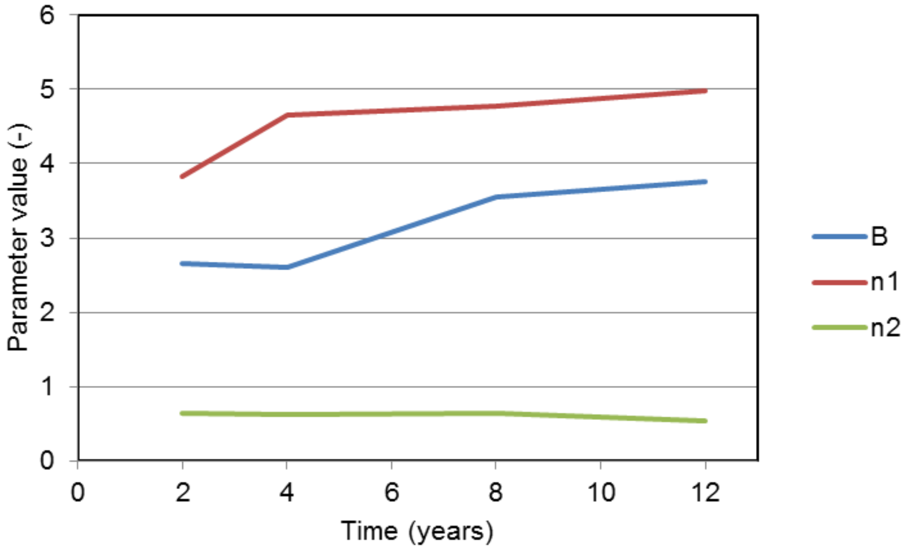

Figure 2. Variation of the parameters B, n_1 and n_2 over time.

In Table 3 the mean error for the 12 year performance of concrete scaling and the estimated value is presented. In addition, the concrete scaling range for 12 year (minimum and maximum scaling value) and the estimated range is also presented.

The average error in the estimation of concrete scaling is quite large. However, this value is misleading as it represents the average of concretes with varying degrees of scaling. For concrete with very little scaling, i.e. 0.2 kg/m^2, and an estimated value of 0.37 kg/m^2, the corresponding error is 85%. This appears to be a significant difference but in fact the depths of scaling in question are very small, less than 0.1 mm and 0.2 mm, respectively for 12 years exposure. The larger errors are in general associated with concretes that have very small scaling.

Table 3. Average error in estimating concrete scaling and in situ and estimated concrete scaling range for 12 years.

Cement type	Average error (%)	Concrete scaling range (min. - max. value, kg/m^2)	
		in situ	estimate
Anläggning	44.8	0.35 - 8.41	0.11 - 7.31
Slite standard	108.7	0.10 - 0.69	0.19 - 1.16
Anläggning + 5% SF	60.6	0.28 - 0.87	0.16 - 1.24
Portland-limestone filler	118.3	0.14 - 1.13	0.24 - 1.62
Finnish Standard	105.6	0.23 - 1.16	0.29 - 2.10
Anläggning + 30% GGBS	47.0	0.38 - 1.82	0.30 - 2.08
Blast furnace slag	124.5	0.46 - 8.52	1.25 - 12.08
Finnish sulphate resisting	59.9	0.20 - 1.83	0.37 - 2.19
Anläggning Modified	53.1	0.27 - 1.65	0.15 - 1.60
Finnish Rapid †	20.3	0.54	0.65
Finnish Mega †	56.7	0.65	1.01

† based on one composition

An overall analysis of the data and the results has identified certain aspects that influence the modelling of concrete scaling:

- Representativeness of the data - the data used for the modelling (BTB addition series) is based on concretes with low *w/b* ratio (in general less than 0.45) which is considered to be a high performance concrete. This does not represent the concrete used in practice where *w/b* can be as high as 0.60. In addition, only two levels of air entrainment where used - no air and 4.5%. Both these factors limit the sensitivity of the analysis to these parameters;
- Given that the data comes from high performance concrete, the effect of no air entrainment on concrete scaling is not always observed. From a modelling point of view this increase the variation associated with the calculated parameters;
- Model does not take into account the possible initial period when no scaling is actively occurring, and model simplification based on linear scaling from t = 0 is not representative of the behaviour of most concretes;
- Despite having 12 year data available, given that the concrete is high performance the scaling is, in general, in the initial phase;

As an example, the results of the modelling of frost scaling for concrete produced with Finnish sulphate resisting cement are presented in Figures 3 & 4. In Figure 3 the concretes with a target air content of 4.5% are presented, whereas in Figure 4 the concretes with no air entrainment are presented.

From Figures 3 & 4 it can be seen that the model's outcome fitting to the scaling results depends mainly on the quality of the concrete. While for the concrete with no air entrainment the model outcome is far from the actual measured scaling (except for the high performance concrete with w/b ratio = 0.3), for the concrete with air entrainment the fit is relatively close.

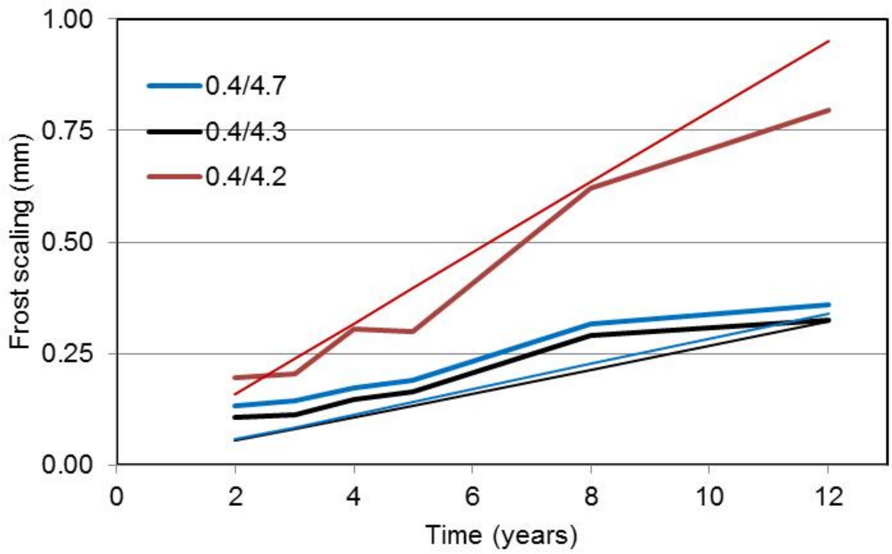

Figure 3. Comparison of model fit to in situ data for Finnish sulphate resisting concretes with w/b ratio of 0.40 - 0.50 and target value for air entrainment of 4.5% (measured values shown).

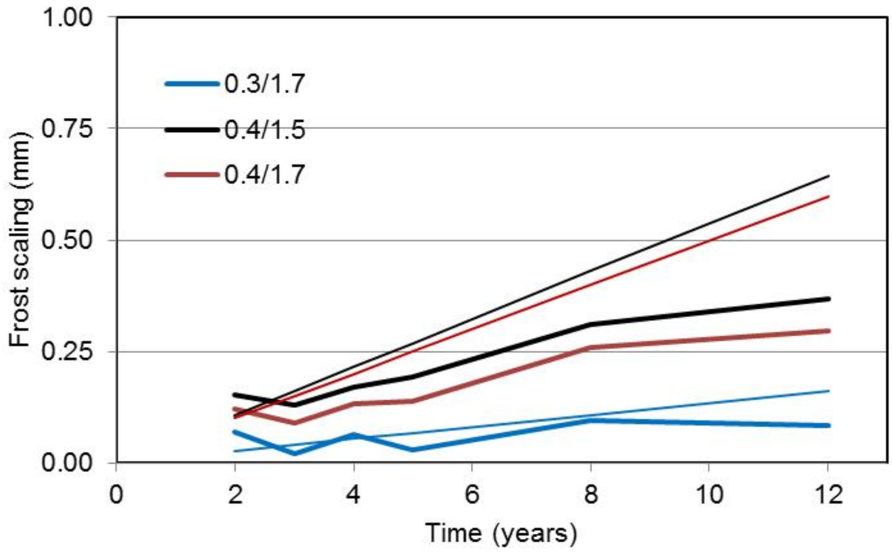

Figure 4. Comparison of model fit to in situ data for Finnish sulphate resisting concretes with w/b ratio of 0.30 - 0.40 and no air entrainment (measured values shown).

2.3.2 Model parametric study

A parametric study was performed to understand the influence of the individual parameters on the model outcome. For this exercise, a reference concrete based on the results of the regression analysis (see Table 2) was considered. In Table 4 the limits for the variation of the individual model parameters is presented.

The calculation is performed for a service life of 50 years. The results of the parametric study are presented in Figures 5 - 10.

Table 4. Values considered for the parametric study - reference and limiting values.

Parameters	w/b	a	n_1	n_2	B	k_{cem}
Reference	0.50	4.00	4.56	0.61	3.15	1.00
Minimum	0.30	2.00	2.50	0.30	2.00	0.50
Maximum	0.70	6.00	6.50	0.90	4.00	3.00

For the purpose of analyzing Figures 5 - 10, the volume of scaling (vertical axis) of 14% corresponds approximately to 10.5 mm of depth of scaling or to 24 kg/m^2 of scaled concrete.

Caution is required when commenting the results of a parametric study. For this specific case, the accentuations of trends depend on the values chosen for the parameters.

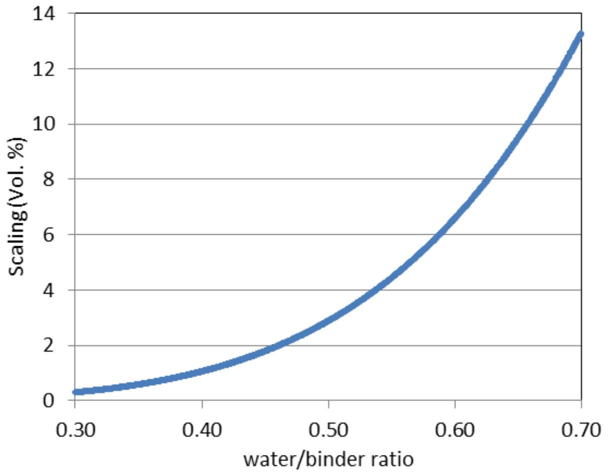

Figure 5. Parametric study - influence of the water/binder ratio.

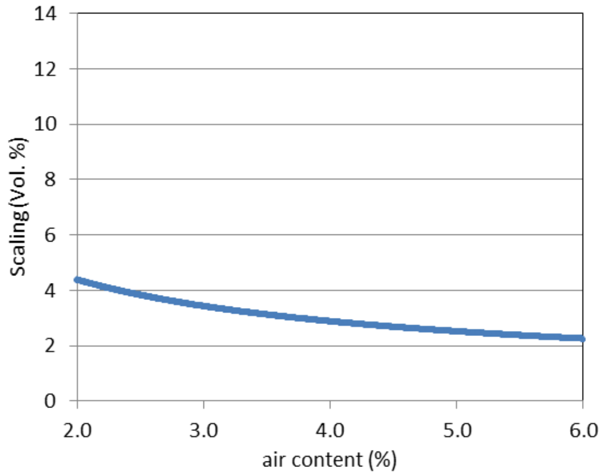

Figure 6. Parametric study - influence of the air content.

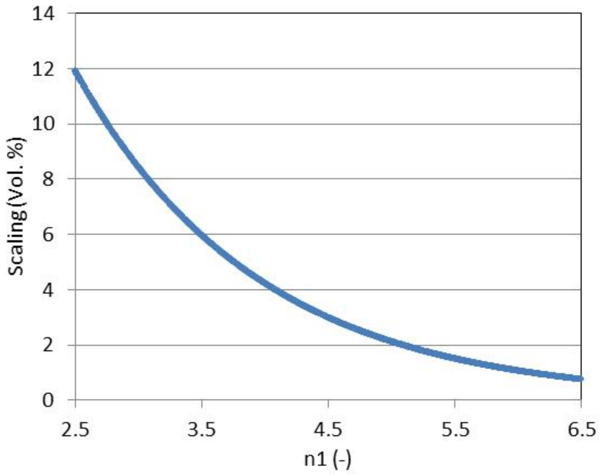

Figure 7. Parametric study - influence of the n_1 parameter.

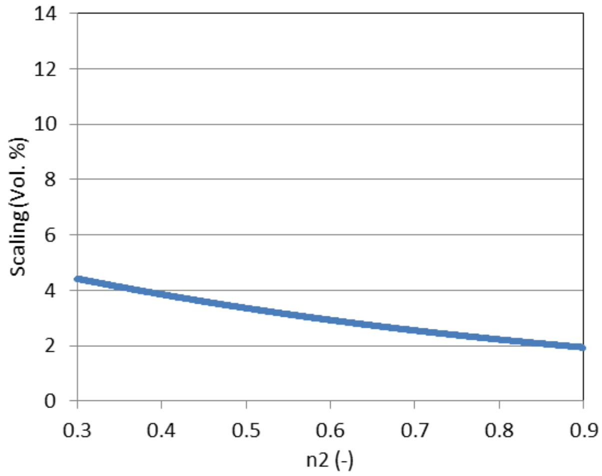

Figure 8. Parametric study - influence of the n_2 parameter.

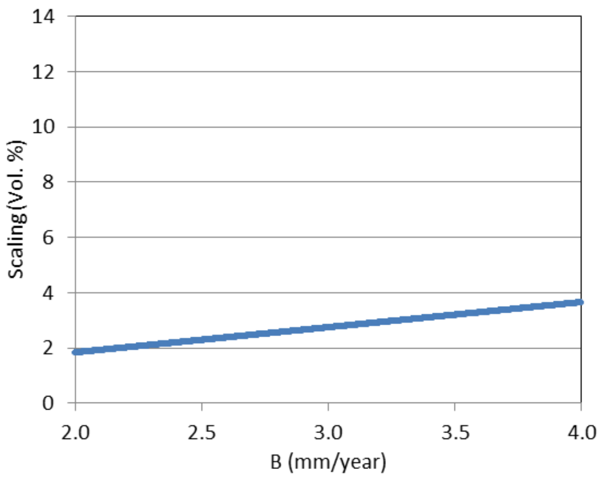

Figure 9. Parametric study - influence of the B parameter.

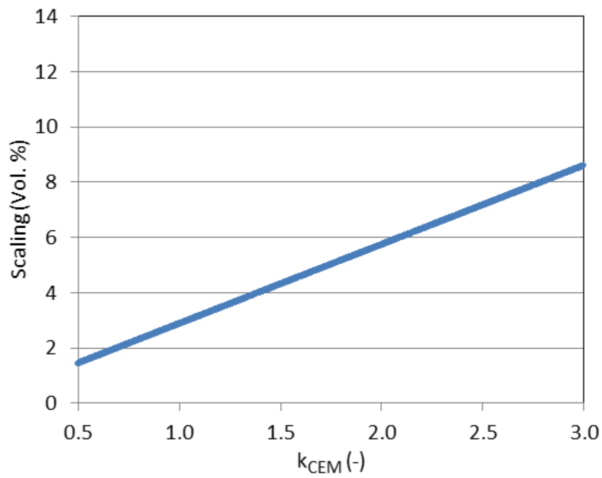

Figure 10. Parametric study -influence of the k_{cem} parameter.

A direct comparison of the results for the two material parameters *w/b* ratio and air content show that the influence of the first is much greater than the second. Both *w/b* ratio and air content double the amount of scaling for variation intervals considered valid for normal concrete, i.e., 0.45 - 0.6 and 2.5 - 5.5, respectively. This tendency is also reflected in the exponents for parameters w/b ratio and air content. For the *B* and k_{cem} parameters, the relationship is linear, with higher slope for the latter.

2.3.3 Analysis of BTB additional series *in situ* data with P factor method

From the regression analysis performed, the average value of factor K for this data series was determined to be 3.40 with a CoV of 25%. In Appendix 1 the basic data set used for the regression analysis is presented. Figure 11 presents the variation with time of the K factor.

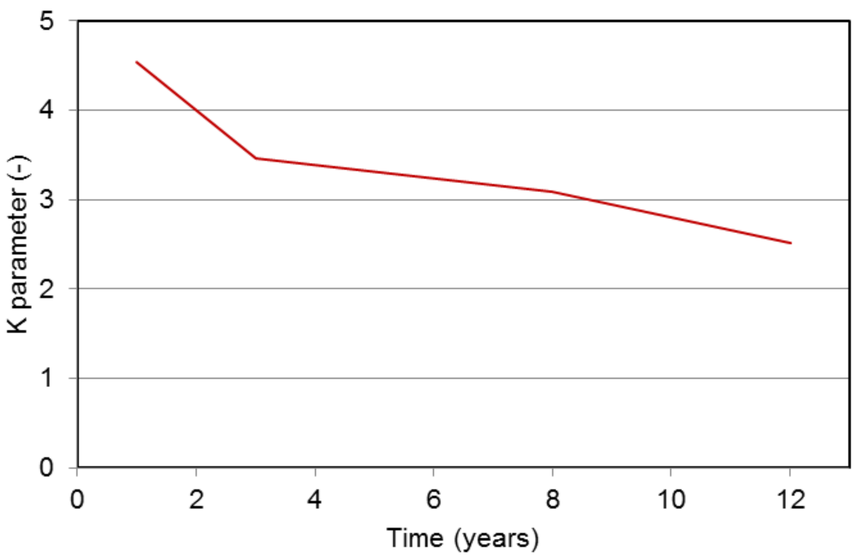

Figure 11. Variation with time of the K factor.

In Table 5 the mean error for the 12 year performance of concrete scaling and the estimated value is presented. In addition, the concrete scaling range for 12 year (minimum and maximum scaling value) and the estimated range are also presented.

Table 5. Average error in estimating concrete scaling and in situ and estimated concrete scaling range for 12 years with the P factor.

Cement type	Average error (%)	Concrete scaling range (min. - max. value, kg/m^2)	
		in situ	estimated
Anläggning	83.8	0.33 - 7.08	0.39 - 5.02
Slite standard	304.0	0.09 - 0.69	0.39 - 1.92
Anläggning + 5% SF	123.5	0.26 - 0.81	0.27 - 2.21
Portland-limestone filler	231.5	0.13 - 1.05	0.41 - 2.14
Finnish Standard	138.6	0.22 - 1.01	0.44 - 2.44
Anläggning + 30% GGBS	77.0	0.36 - 1.70	0.58 - 2.87
Blast furnace slag	68.1	0.43 - 7.95	1.11 - 7.32
Finnish sulphate resisting	136.0	0.18 - 1.71	1.04 - 2.32
Anläggning Modified	151.5	0.25 - 1.54	0.38 - 2.44
Finnish Rapid †	147.9	0.50	1.29
Finnish Mega †	81.9	0.61	1.17

† based on one composition

Similar considerations as those presented for Table 3, in 2.3.1, apply for the analysis of the results presented in Table 5.

As an example, the results of the modelling of frost scaling for concrete produced with Finnish sulphate resisting cement are presented in Figures 12 & 13. In Figure 12 the concretes with a target air content of 4.5% are presented, whereas in Figure 13 the concretes with no air entrainment are presented.

From Figures 12 & 13 it can be seen that for no concrete does the model outcome fit the scaling results accurately.

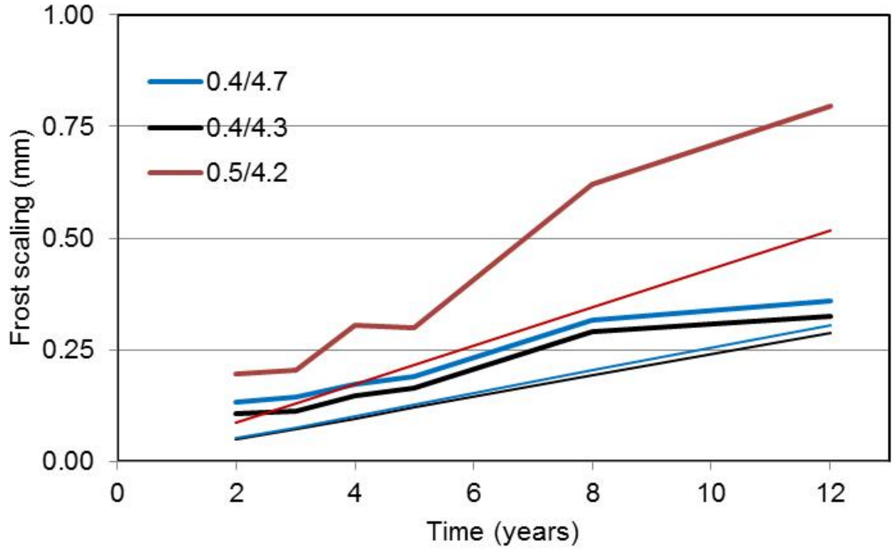

Figure 12. Comparison of P-factor model fit to in situ data for Finnish sulphate resisting concrete with w/b ratio of 0.40 - 0.50 and target value for air entrainment of 4.5% (measured values shown).

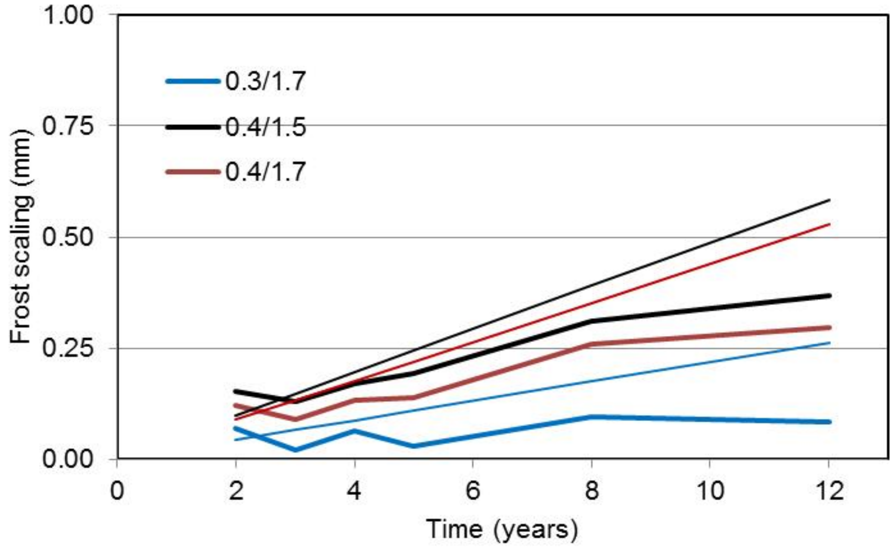

Figure 13. Comparison of P-factor model fit to in situ data for Finnish sulphate resisting concrete with w/b ratio of 0.30 - 0.40 and no air entrainment (measured values shown).

2.4 Frost scaling of laboratory specimens

Frost scaling tests were done in the laboratory for all test concretes of both BTB and DuraInt projects. The laboratory scaling tests were done according to the standard CEN/TS 12390-9 (slab test) [13].

The DuraInt laboratory tests for frost scaling were done using two kinds of preconditioning of the test specimens: standard preconditioning (Case 1), and a 1 year aging exposed to atmospheric CO_2 with a relative humidity of 65% (Case 2). In Case 1, specimens where stored 7 days at RH 65%. In Case 2, to be able to differentiate the effects of carbonation and drying, specimens were tested in two ways: a) with the carbonated surface, and, b) only the dried surface (the carbonated layer was removed by sawing).

2.4.1 Analysis of laboratory data

Using the model equation presented in Eq. (3) and the regression analysis procedure described in 2.1, values of E, n_3 and n_4 were determined. In Table 66 these are presented as well as the average error in estimating the test result.

Table 6. Values for model parameter E, n_3 and n_4, and average error in estimating the test result.

Parameter	BTB	Case 1	Case 2 - CO_2	Case 2 - RH
E	39.694	36.371	43.387	1.132
n_3	1.888	0.000	3.415	0.000
n_4	2.156	2.466	0.708	0.699
error (%)	377.1	99.9	41.4	66.1

A comparison of the results for the laboratory cured specimens with the tests performed on *in situ* exposed concrete with time reveals no significant relationship. However, in the Case 2a (1 year carbonation) the exponent of the water/binder ratio and the exponent of the air content seem to be of the same order of magnitude as in the field tests. Accordingly, the chosen form of the degradation function was the following.

3 Service life models for Frost Scaling

According to the Finnish codes the general formula for the predicted service life of concrete structures is the following [16]:

$$t_L = A \cdot B \cdot C \cdot D \cdot F \cdot G \cdot t_{Lr} \tag{8}$$

where

t_L predicted service life, years;
t_{Lr} reference service life, years;
A - G factors.

A constant value of 50 years has been applied for the reference service life when the required safety for exceeding the design service life is 95%. If the required safety is 90% the corresponding reference service life is 61 years. These values comply with the assumption that the average service life is 145 years, the service life is log-normally distributed and the coefficient of variation of service life is 0.6. The lifetime safety factor for 95% safety (average service life divided by the 95% percentile service life) is 2.9 (See Figure 14).

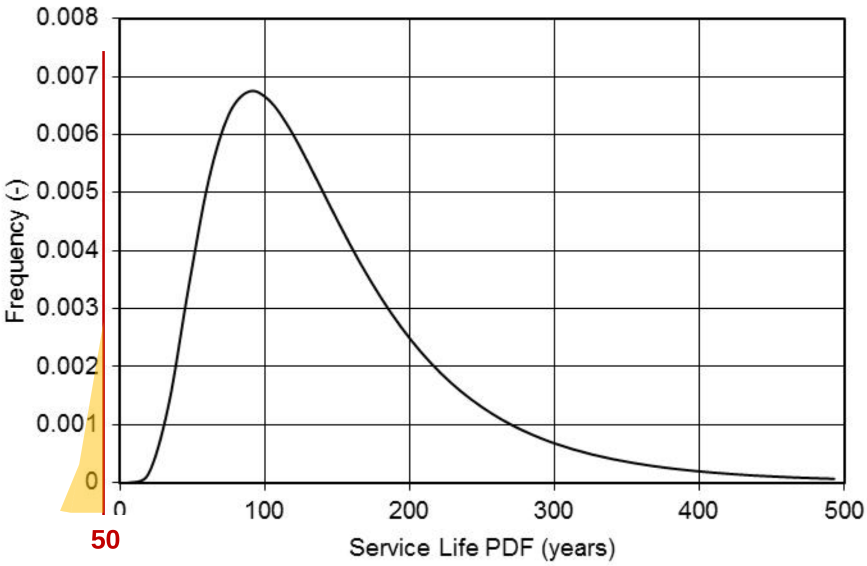

Figure 14. Lognormal PDF for the design service life.

By the factors A - G different aspects related to service life are considered. In general the aspects represented by the factors are the following:

- A - Materials, porosity
- B - Structure, details of design
- C - Workmanship
- D - Interior climate
- E - Environmental exposure
- F - In-service stresses
- G - Level of maintenance.

Sometimes two or more factors may be reserved for the same aspect. For example there may be two material factors (A_1, A_2), two structural factors (B_1, B_2) and four environmental factors (E_1, E_2, E_3, E_4). Usually these factors are just multiplied with each other to obtain the final factor.

The service life is predicted for all the applicable degradation mechanisms, such as frost attack and carbonation initiated corrosion. The shortest of the predicted service lives is the determinative one.

The task for service life modelling is to find out the data and equations behind the factors. For the designers, the factors are normally presented in tables with due design parameters.

3.1 Model based on w/c, air content and cement factor

Usually service life models can be derived from the degradation models if the maximum allowable degradation is known.

Considering that the maximum allowable depth of frost scaling is d_{max} the service life could be derived from Eq. (3).

$$t_{L;avg} = \frac{d_{\max} \cdot a^{n_2}}{0.75 \cdot B \cdot \left(\frac{w}{c}\right)^{n_1} \cdot k_{\exp} \cdot k_{cem}} \quad (9)$$

where

$t_{L:avg}$ average service life, years

Considering the service life safety factor γ_L and the corresponding reference service life t_{Lref}, the following equation can be derived for the predicted service life:

$$t_L = \frac{d_{\max} \cdot a^{n_2}}{\gamma_L \cdot 0.75 \cdot B \cdot \left(\frac{w}{c}\right)^{n_1} \cdot t_{Lr}} \cdot \frac{1}{k_{cem}} \cdot \frac{1}{k_{\exp}} \cdot t_{Lr} \quad (10)$$

This could be presented in the factor form as:

$$t_L = A_1 \cdot A_2 \cdot E \cdot t_{Lr} \quad (11)$$

where

$$A_1 = \frac{d_{\max} \cdot a^{n_2}}{\gamma_L \cdot 0.75 \cdot B \cdot \left(\frac{w}{c}\right)^{n_1} \cdot t_{Lr}} \quad (12)$$

$$A_2 = \frac{1}{k_{cem}}$$

$$E = \frac{1}{k_{\exp}}$$

As an example, the values of factor A_1 are given in Table 7 as a function of air content and water cement ratio. The values of the constant parameters considered are the following (ref. Table 2.):

d_{max} = 15 mm
γ_L = 2.9
t_{Lr} = 50 years
B = 3.2 mm/year
n_1 = 4.6
n_2 = 0.6.

Table 7. Factors A1.

Air %	w/c								
	0.3	0.35	0.4	0.45	0.5	0.55	0.6	0.65	0.7
1.5	10.00	6.88	3.72	2.16	1.33	0.86	0.58	0.40	0.28
2	10.00	8.17	4.42	2.57	1.58	1.02	0.68	0.47	0.34
2.5	10.00	9.34	5.06	2.94	1.81	1.17	0.78	0.54	0.39
3	10.00	10.00	5.64	3.28	2.02	1.30	0.87	0.60	0.43
3.5	10.00	10.00	6.19	3.60	2.22	1.43	0.96	0.66	0.47
4	10.00	10.00	6.70	3.90	2.40	1.55	1.04	0.72	0.51
4.5	10.00	10.00	7.19	4.18	2.58	1.66	1.11	0.77	0.55
5	10.00	10.00	7.66	4.46	2.75	1.77	1.19	0.82	0.58
5.5	10.00	10.00	8.11	4.72	2.91	1.88	1.26	0.87	0.62
6	10.00	10.00	8.55	4.97	3.06	1.98	1.32	0.92	0.65
6.5	10.00	10.00	8.97	5.22	3.21	2.07	1.39	0.96	0.68
7	10.00	10.00	9.38	5.45	3.36	2.17	1.45	1.01	0.71
7.5	10.00	10.00	9.77	5.69	3.50	2.26	1.51	1.05	0.74
8	10.00	10.00	10.00	5.91	3.64	2.35	1.57	1.09	0.77
8.5	10.00	10.00	10.00	6.13	3.77	2.43	1.63	1.13	0.80
9	10.00	10.00	10.00	6.34	3.91	2.52	1.69	1.17	0.83

The values of A_2, presented in Table 8, are determined as the inverse values of the k_{cem} in Table 1.

Table 8. Factors A_2.

Cement	A2
Anläggning	1.00
Slite	0.56
Anl+Silica5%	0.63
Portl limestone	0.45
Finnish Std	0.36
Anlägg+slag30%	0.35
Slag cement	0.09
Finnish SR	0.53
AnläggningM	0.68
Finnish Rapid	0.64
Finnish Mega	0.38

3.1.1 Example of calculations

For a calculation example, the characteristics of a typical concrete are used: a water/binder ratio of 0.5, an air content of 4.2% and the cement type used is considered to be a Finnish SR cement.

From Table 7 we can read A1 is 2.48, and from Table 8 A2 is 0.53. The environmental parameter E = 1.0 for XF4 class. With the values of the model parameters defined, the service life can be predicted as:

$$t_L = 2.48 \cdot 0.53 \cdot 1 \cdot 50 = 65 \, years$$

3.2 Model based on the P factor

The P factor based model for frost scaling as expressed for the thickness of scaled off concrete is the following

$$d = 0.75 \cdot K \cdot k_{\exp} \cdot \frac{1}{P} \cdot t \tag{13}$$

where d is the thickness of scaled off concrete, mm.

The corresponding model of the design service life would be:

$$t_L = \frac{d_{\max} \cdot P}{\gamma_L \cdot 0.75 \cdot K \cdot t_{Lr}} \cdot \frac{1}{k_{\exp}} \cdot t_{Lr} \tag{14}$$

where:
- d_{max} = 15 mm
- γ_L = 2.9
- t_{Lr} = 50 years
- K = 3.4 [mm].

By replacing the values in the equation, the following is obtained for the service life.

$$t_L = 0.041 \cdot P \cdot \frac{1}{k_{\exp}} \cdot t_{Lr} \tag{15}$$

This can be modified to the factor approach as follows:

$$t_L = A \cdot E \cdot t_{Lr} \tag{16}$$

where

$$A = 0.041 \cdot P \tag{17}$$

$$E = \frac{1}{k_{\exp}}$$

3.2.1 Example of calculations

The same concrete as in Chapter 3.3.1 is evaluated (water/binder ratio 0.5, air content 4.2%, Finnish SR cement). The corresponding P-factor is 34.

Using Equation (15) it is obtained (k_{exp} = 1):

$$t_L = 0.041 \cdot 34 \cdot 1 \cdot 50 = 69 \, years$$

4 Effect of frost attack on carbonation

In this chapter a model for the effect of internal frost attack and frost scaling on concrete carbonation is proposed. In addition, the interaction factors for service life design are presented.

The model proposed is based on the laboratory testing of concrete specimens and the relationship between the performance indicators for each degradation mechanism. Data from the *in situ* specimens of the DuraInt project cannot be used given the brief time of exposure.

4.1 Theory

4.1.1 Differentiation of carbonation determination

The depth of carbonation can normally be evaluated using the square root of time formula [1]:

$$x_{ca} = k_{ca}\sqrt{t} \tag{20}$$

where:

x_{ca} depth of carbonation, mm;
t age of concrete, years, and
k_{ca} coefficient of carbonation, mm/a$^{0.5}$.

The increase of carbonation in an increment of time can be presented as:

$$\Delta x_{ca} = \frac{k_{ca}^2}{2 \cdot x_{ca}} \cdot \Delta t \tag{21}$$

where Δx_{ca} is the increase of carbonation depth during the time step t, t+Δt, mm.

The total carbonation depth can be determined by totalling the incremental depths of carbonation as follows:

$$\begin{aligned} x_{ca}(t) &= \sum \Delta x_{ca} \\ \Delta x_{ca}(t, t+\Delta t) &= \frac{k_{ca}^2}{2} \cdot \frac{1}{x_{ca}(t)} \cdot \Delta t \end{aligned} \tag{22}$$

The increase of the carbonation is thus proportional to the square of the carbonation coefficient and inversely proportional to the carbonation depth itself.

4.1.2 Effect of internal frost attack on carbonation

In the case of concrete exposed to internal frost attack, the carbonation coefficient increases with the increasing internal damage in concrete. Even in that case Eq. (22) can be assumed to apply. The total carbonation depth is determined as the sum of incremental carbonation depths which are determined from Eq. (23). The carbonation coefficient $k_{ca;IntFr}$ increases with time with increasing frost deterioration in concrete [1]:

$$x_{ca;IntFr} = \sum \Delta x_{ca;IntFr} \tag{23}$$

$$\Delta x_{ca;IntFr}(t, t+\Delta t) = \frac{k^2_{ca;IntFr}(t)}{2} \cdot \frac{1}{x_{ca;IntFr}(t)} \cdot \Delta t$$

where:

$k_{ca;IntFr}$ carbonation coefficient of concrete which is attacked by internal frost damage (dependent on time), mm/a$^{0.5}$, and

$x_{ca;IntFr}$ depth of carbonation in internally damaged concrete, mm.

The data used for determining the relationship between the performance indicator for carbonation and internal frost attack is reported in [12]. It is based on concretes with varying w/b ratios (0.5 - 0.6, for CEM II/A-M(S-LL) 42,5 N, and 0.65 for CEM I 42,5 N SR with slag).

In Figure 15 a summary of carbonation test results with frost attacked concrete specimens is presented [12]. According to the test results the depth of carbonation increases with increased internal frost damage as presented in the figure.

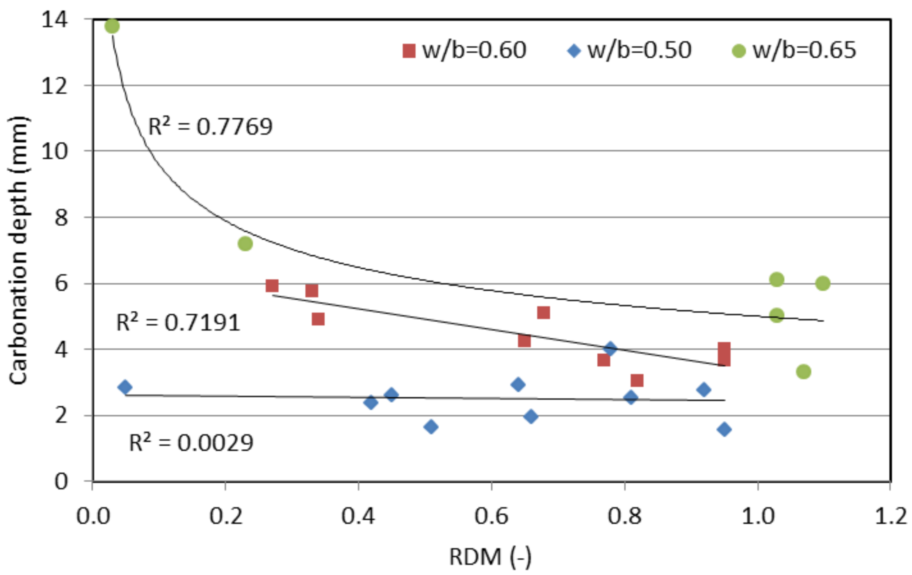

Figure 15. The effect of frost deterioration measured by relative dynamic modulus on the depth of carbonation.

The carbonation coefficient of concrete is directly proportional to the depth of carbonation. The following empirical relationship is suggested for the frost-interacted carbonation coefficient and the original carbonation coefficient, as an average. Based on the data presented in Figure 16 the coefficients in Eq. (24) are determined.

$$\frac{k_{ca;IntFr}}{k_{ca0}} = 1 + 0.64 \cdot \left(1 - \frac{RDM}{100}\right)^{1.32} \tag{24}$$

where

k_{ca0} carbonation coefficient of intact concrete, mm/√year;

$k_{ca;IntFr}$ carbonation coefficient of frost interacted concrete, mm/√year, and
RDM relative dynamic modulus of concrete, %.

The relationship presented in Eq. (24) is not representative of all cement types. Additional data is required to be able to model the relationship by cement type including the influence of w/b ratio.

4.1.3 Effect of frost scaling on carbonation

The rate of carbonation is inversely proportional to the thickness of the already carbonated concrete. When the frost scaling reduces the thickness of the already carbonated concrete, Eq. (21) is changed so that the depth of scaling is reduced from the thickness of the already carbonated concrete as presented in Eq. (25). As a result of the frost scaling, the rate of carbonation increases.

$$x_{ca;FrSc} = \sum \Delta x_{ca;FrSc}$$
$$\Delta x_{ca;FrSc}(t, t+\Delta t) = \frac{k_{ca}^2}{2} \cdot \frac{1}{x_{ca;FrSc}(t) - x_{FrSc}(t)} \cdot \Delta t \tag{25}$$

where
$x_{ca;FrSc}(t)$ is the depth of carbonation as influenced by frost scaling (measured from the initial surface of the structure), mm,
$x_{FrSc(t)}$ the depth of frost scaling, mm.

4.2 Determination of interaction factors for service life evaluation

4.2.1 Interaction factors for the effect of internal frost attack on the initiation time of corrosion based on carbonation

Assuming that the internal damage of concrete increases linearly with time between the start of the service life until the end of service life and assuming that the limit state of service life with respect to internal frost attack is 2/3 of the original relative dynamic modulus, the following equation can be written:

$$RDM(t) = 100 - 33.3 \cdot \frac{t}{t_L} \tag{26}$$

where
$t_{L;IntFr}$ predicted service life of the structure with respect to internal frost attack, and
t time, years.

By inserting Eq. (26) to Eq. (23) the following relationship is obtained between the original carbonation coefficient and the frost interacted carbonation coefficient with time.

$$k_{ca;IntFr} = k_{ca0} \cdot \left(1 + 0.21 \cdot \left(\frac{t}{t_{L;IntFr}}\right)^{1.32}\right) \tag{27}$$

.

Considering further that the predicted initiation time of corrosion with respect to chloride penetration without the effect of frost attack is $t_{0;ca}$ the coefficient of carbonation can be written as follows:

$$k_{ca0} = \frac{C}{\sqrt{t_{0;ca}}} \tag{28}$$

where

$t_{0;ca}$ original predicted initiation time of corrosion with respect to chloride penetration, and

C concrete cover, mm.

Table 9. Interaction coefficients for the effect of internal frost attack on the initiation time of corrosion with regard to carbonation ($I_{ca;IntFr}$).

$t_{0;ca}$	$t_{L;intFr}$									
	20	40	60	80	100	120	140	160	180	200
10	0,90	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
20	0,85	0,90	0,95	0,95	0,95	0,95	0,95	0,95	0,95	1,00
30	0,80	0,90	0,93	0,93	0,97	0,97	0,97	0,97	0,97	0,97
40	0,73	0,85	0,90	0,93	0,95	0,95	0,95	0,95	0,98	0,98
50	0,71	0,84	0,90	0,92	0,94	0,96	0,96	0,98	0,98	0,98
60	0,66	0,81	0,86	0,90	0,93	0,95	0,95	0,97	0,97	0,97
70	0,62	0,78	0,84	0,88	0,91	0,93	0,94	0,96	0,96	0,96
80	0,59	0,75	0,82	0,87	0,90	0,91	0,92	0,94	0,95	0,95
90	0,57	0,72	0,80	0,85	0,88	0,90	0,92	0,93	0,94	0,94
100	0,55	0,70	0,79	0,84	0,87	0,89	0,91	0,92	0,93	0,94
110	0,52	0,68	0,76	0,82	0,85	0,88	0,90	0,91	0,93	0,94
120	0,50	0,66	0,75	0,80	0,84	0,87	0,88	0,90	0,92	0,92
130	0,48	0,64	0,73	0,79	0,83	0,85	0,88	0,89	0,91	0,91
140	0,47	0,63	0,71	0,77	0,81	0,84	0,86	0,88	0,90	0,91
150	0,45	0,61	0,70	0,76	0,80	0,83	0,85	0,87	0,89	0,90
160	0,44	0,59	0,69	0,75	0,79	0,82	0,85	0,87	0,88	0,89
170	0,43	0,58	0,67	0,73	0,78	0,81	0,83	0,86	0,87	0,89
180	0,41	0,56	0,66	0,72	0,77	0,80	0,83	0,85	0,87	0,88
190	0,40	0,56	0,65	0,71	0,76	0,79	0,82	0,84	0,86	0,87
200	0,39	0,54	0,64	0,70	0,75	0,78	0,81	0,83	0,85	0,86

The interaction coefficients for the initiation time of corrosion calculated using Eq. (13), Eq. (15) and Eq. (18) are presented in Table 9. The interaction coefficients are tabulated with the original initiation time of corrosion, $t_{0;ca}$, and the service life with respect to internal frost attack, $t_{L;IntFr}$, as they are assumed to be determined first in the process of service life evaluation. Concrete cover is not a relevant parameter in this case. The corrected value of the initiation time of corrosion is determined as follows:

$$t_{0;ca;IntFr} = I_{ca;IntFr} \cdot t_{0;ca} \tag{19}$$

where

$t_{0;ca;IntFr}$ corrected value of the initiation time of corrosion when interacted by internal frost attack (carbonation initiation), years, and,

$I_{ca;IntFr}$ interaction factor for the effect of internal frost attack on the initiation time of corrosion (carbonation initiation).

4.2.2 Interaction factors for the effect frost scaling on the initiation time of corrosion based on carbonation

Usually the depth of frost scaling can be considered linear with time unless internal frost action takes place (as it is assumed in this case). So the depth of frost scaling is determined as follows:

$$x_{FrSc} = k_{FrSc} \cdot t \qquad (30)$$

where k_{FrSc} is the coefficient of frost scaling, mm/year.

Considering that the limit depth of scaling at the end of the service life of the structure is 15 mm then the coefficient k_{FrSc} for frost scaling can be written as:

$$k_{FrSc} = \frac{15mm}{t_{L;FrSc}} \qquad (31)$$

Now the algorithm for determination of the interaction coefficient for frost scaling on the initiation time of corrosion is available (Eq. (25), Eqs. (30 - 31)). In the service life prediction the initiation time of corrosion without the effect of frost action and the service life based on the scaling of concrete are assumed to be first determined. Then the interaction factor for the effect of frost scaling on the initiation time of corrosion is obtained from the Tables 10 and 11 depending on the original thickness of concrete cover. The updated initiation time of corrosion is determined as follows:

$$t_{0;ca;FrSc} = I_{ca;FrSc} \cdot t_{0;ca} \qquad (22)$$

where

$t_{0;ca;FrSc}$ updated value of the initiation time of corrosion based on carbonation, years, and,

$I_{ca;FrSc}$ interaction factor for the effect of frost scaling on the initiation time of carbonation initiated corrosion.

The interaction factors for frost scaling on the initiation time of carbonation induced corrosion depend on the thickness of concrete cover. In Table 10 the interaction factors are presented for the concrete cover of 25 mm. In Table 11 the interaction factors are presented for the concrete cover of 50 mm. The intermediate values can be interpolated.

Caution is required when interpreting the results are they are based on a limited number of cement types, and the outcome of the model has not been validated with in situ exposure data.

Table 10. Interaction factors for the effect of frost scaling on the initiation time of carbonation induced corrosion when the depth of concrete cover is 25 mm.

t_0	$t_{L;FrSc}$ 20	40	60	80	100	120	140	160	180	200
10	0.80	0.90	0.90	0.90	1.00	1.00	1.00	1.00	1.00	1.00
20	0.70	0.85	0.85	0.90	0.90	0.95	0.95	0.95	0.95	0.95
30	0.60	0.77	0.83	0.87	0.90	0.90	0.90	0.93	0.93	0.93
40	0.53	0.70	0.78	0.83	0.85	0.88	0.90	0.90	0.90	0.93
50	0.47	0.65	0.76	0.80	0.84	0.86	0.88	0.90	0.92	0.92
60	0.42	0.61	0.71	0.76	0.81	0.83	0.86	0.88	0.88	0.90
70	0.38	0.57	0.67	0.74	0.78	0.81	0.84	0.86	0.87	0.88
80	0.34	0.53	0.63	0.71	0.75	0.78	0.81	0.84	0.85	0.86
90	0.31	0.49	0.61	0.67	0.73	0.76	0.80	0.82	0.83	0.85
100	0.28	0.46	0.58	0.65	0.71	0.75	0.78	0.80	0.82	0.83
110	0.27	0.44	0.55	0.62	0.68	0.72	0.75	0.78	0.80	0.82
120	0.24	0.41	0.52	0.61	0.66	0.71	0.74	0.76	0.78	0.81
130	0.23	0.39	0.50	0.58	0.64	0.68	0.72	0.74	0.77	0.79
140	0.22	0.37	0.48	0.56	0.62	0.66	0.70	0.73	0.76	0.78
150	0.20	0.35	0.46	0.54	0.60	0.65	0.68	0.72	0.74	0.76
160	0.19	0.33	0.44	0.52	0.58	0.63	0.67	0.70	0.72	0.75
170	0.18	0.32	0.43	0.50	0.57	0.62	0.66	0.69	0.71	0.73
180	0.17	0.31	0.41	0.49	0.55	0.60	0.64	0.67	0.70	0.72
190	0.16	0.30	0.40	0.48	0.53	0.59	0.62	0.66	0.69	0.71
200	0.16	0.28	0.38	0.46	0.52	0.57	0.61	0.65	0.67	0.70

Table 11. Interaction factors for the effect of frost scaling on the initiation time of carbonation induced corrosion when the depth of concrete cover is 50 mm.

t_0	$t_{L;FrSc}$ 20	40	60	80	100	120	140	160	180	200
10	0.90	0.90	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	0.85	0.90	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
30	0.77	0.87	0.90	0.93	0.93	0.93	0.93	0.97	0.97	0.97
40	0.70	0.83	0.88	0.90	0.93	0.93	0.93	0.95	0.95	0.95
50	0.65	0.80	0.86	0.90	0.92	0.94	0.94	0.96	0.96	0.96
60	0.61	0.76	0.83	0.88	0.90	0.92	0.93	0.93	0.95	0.95
70	0.57	0.74	0.81	0.86	0.88	0.90	0.91	0.93	0.93	0.94
80	0.53	0.71	0.78	0.84	0.86	0.89	0.90	0.91	0.92	0.94
90	0.49	0.67	0.76	0.82	0.85	0.88	0.89	0.90	0.91	0.92
100	0.46	0.65	0.75	0.80	0.83	0.86	0.88	0.89	0.90	0.91
110	0.44	0.62	0.72	0.78	0.82	0.84	0.86	0.88	0.89	0.90
120	0.41	0.61	0.71	0.76	0.81	0.83	0.85	0.87	0.88	0.89
130	0.39	0.58	0.68	0.74	0.79	0.82	0.84	0.86	0.88	0.88
140	0.37	0.56	0.66	0.73	0.78	0.81	0.83	0.85	0.86	0.88
150	0.35	0.54	0.65	0.72	0.76	0.79	0.82	0.84	0.85	0.87
160	0.33	0.52	0.63	0.70	0.75	0.78	0.81	0.83	0.85	0.86
170	0.32	0.50	0.62	0.69	0.73	0.77	0.80	0.82	0.84	0.85
180	0.31	0.49	0.60	0.67	0.72	0.76	0.79	0.81	0.83	0.84
190	0.30	0.48	0.59	0.66	0.71	0.75	0.78	0.80	0.82	0.84
200	0.28	0.46	0.57	0.65	0.70	0.74	0.77	0.79	0.81	0.83

5 Effect of frost attack on chloride penetration

In this chapter a model for the effect of internal frost attack and frost scaling on chloride penetration into concrete is proposed. In addition, the interaction factors for service life design are presented.

The model proposed is based on the laboratory testing of concrete specimens and the relationship between the performance indicators for each degradation mechanism. Data from the *in situ* specimens of the DuraInt project cannot be used given the brief time of exposure.

5.1 Theory

5.1.1 Differentiation of the determination of chloride penetration

In the following the analogy with carbonation is applied when treating the problem of chloride penetration [1]. Similar to the depth of carbonation, also the depth of critical chloride content can be roughly estimated using the square root of time rule:

$$x_{cl} = k_{cl}\sqrt{t} \tag{33}$$

where:

- x_{cl} depth of critical chloride content, mm
- t age of concrete, years, and
- k_{cl} coefficient of chloride penetration, mm/a$^{0.5}$.

Based on the analogy with carbonation, the total depth of the critical chloride content can be determined by totalling the incremental depths of chloride penetration as follows:

$$x_{cl}(t) = \sum \Delta x_{cl} \tag{34}$$

$$\Delta x_{cl}(t, t + \Delta t) = \frac{k_{cl}^2}{2} \cdot \frac{1}{x_{cl}(t)} \cdot \Delta t$$

The proposed model does not take into account the time dependency of the diffusion coefficient, and it is assumed that the surface concentration of chloride on the exposed surface is constant.

5.1.2 Effect of internal frost attack on chloride penetration

The depth of the critical chloride content in concrete exposed to frost attack can be determined from Eq. (35):

$$x_{cl;IntFr} = \sum \Delta x_{cl;IntFr} \qquad (35)$$

$$\Delta x_{cl;IntFr}(t, t+\Delta t) = \frac{k^2_{cl;IntFr}(t)}{2} \cdot \frac{1}{x_{cl;IntFr}(t)} \cdot \Delta t$$

where:

$k_{cl;IntFr}$ coefficient of chloride penetration in concrete exposed to frost action (dependent on time), and

x_{cl} depth of critical chloride content, mm.

The effect of internal frost attack (as measured by RDM) on the chloride migration coefficient of concrete is presented in Figure 16. The tests are reported in [8].

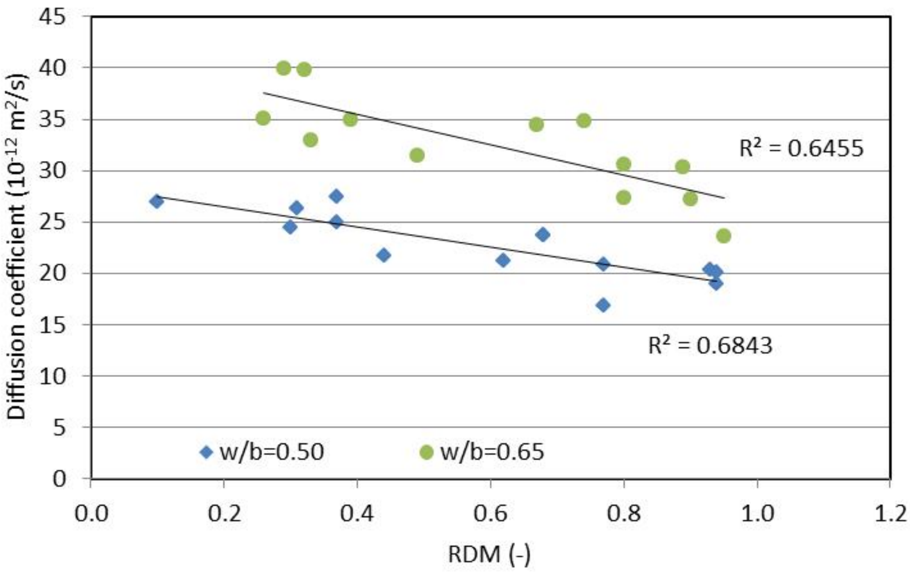

Figure 16. The effect of internal frost attack (measured by RDM) on the chloride migration coefficient of concrete (non-steady state migration).

The relation between the coefficient of chloride penetration and the diffusion coefficient is presented in the following equation [1]:

$$k_{cl} = 2\sqrt{3D} \cdot \left(1 - \sqrt{\frac{c_{crit}}{c_s}}\right) \qquad (36)$$

where:

k_{cl} coefficient of chloride penetration, mm/√year and

t time, year

D the diffusion coefficient of concrete with respect to chloride ions, mm^2/year

c_{crit} critical chloride content, %,

c_S chloride content at the surface of concrete, %.

The following relationship is suggested for the coefficient of chloride penetration in internally damaged concrete (RDM < 100%) and in intact concrete (RDM = 100%). Based on the data presented in Figure 7 the coefficients in Eq. (27) are determined.

$$\frac{k_{cl;IntFr}}{k_{cl0}} = 1 + 0.30 \cdot \left(1 - \frac{RDM}{100}\right)^{0.93} \tag{37}$$

where:

k_{cl0} is coefficient of chloride penetration of intact concrete, mm/√year and
RDM relative dynamic modulus of the damaged concrete, %.

5.1.3 Effect of frost scaling on chloride penetration

The analogy with the case of carbonation can still be used for evaluating the effects of frost scaling on the rate of chloride penetration. The depth of critical chloride content can be determined as follows:

$$x_{cl;FrSc} = \sum \Delta x_{cl;FrSc} \tag{38}$$

$$\Delta x_{cl;FrSc}(t, t + \Delta t) = \frac{k_{cl}^2}{2} \cdot \frac{1}{x_{cl;FrSc}(t) - x_{FrSc}(t)} \cdot \Delta t$$

where:

$x_{cl;FrSc}(t)$ depth of chloride penetration at time t, mm,
$x_{FrSc}(t)$ depth of frost scaling at time t, mm.

5.2 Determination of interaction factors for service life evaluation

5.2.1 Interaction factors for the effect of internal frost attack on the initiation time of corrosion based on chloride penetration

By inserting Eq. 16 to Eq. 28 the following relationship was obtained:

$$k_{cl;IntFr} = k_{cl0} \cdot \left(1 + 0.11 \cdot \left(\frac{t}{t_{L;IntFr}}\right)^{0.93}\right) \tag{39}$$

Considering that the predicted initiation time of corrosion without the effect of frost attack is $t_{0;cl}$ the coefficient of carbonation and the depth of carbonation (of the unaffected concrete) can be determined as follows:

$$k_{cl0} = \frac{C}{\sqrt{t_{0;cl}}} \tag{40}$$

where:

$t_{0;cl}$ original predicted initiation time of corrosion with respect to chloride penetration, and,
C concrete cover, mm.

Analogically with the case of carbonation, the interaction coefficients can now be determined as a function of $t_{0;cl}$ and $t_{L;IntFr}$. The results are presented in Table 12.

Table 12. Interaction coefficients for initiation time of corrosion as affected by internal frost attack.

t_0	$t_{L;intFr}$ 20	40	60	80	100	120	140	160	180	200
10	0.90	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	0.90	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
30	0.86	0.90	0.93	0.93	0.96	0.96	0.96	0.96	0.96	0.96
40	0.82	0.90	0.92	0.92	0.95	0.95	0.95	0.95	0.95	0.97
50	0.81	0.89	0.91	0.93	0.95	0.95	0.95	0.97	0.97	0.97
60	0.79	0.88	0.91	0.93	0.94	0.94	0.96	0.96	0.96	0.96
70	0.76	0.85	0.89	0.91	0.92	0.94	0.95	0.95	0.95	0.97
80	0.74	0.84	0.88	0.91	0.92	0.93	0.94	0.94	0.94	0.96
90	0.73	0.83	0.87	0.89	0.91	0.93	0.93	0.94	0.95	0.95
100	0.71	0.81	0.85	0.88	0.90	0.91	0.92	0.93	0.93	0.94
110	0.69	0.79	0.85	0.88	0.89	0.91	0.92	0.93	0.93	0.94
120	0.68	0.78	0.84	0.87	0.89	0.90	0.91	0.92	0.93	0.94
130	0.66	0.77	0.82	0.86	0.88	0.89	0.91	0.92	0.93	0.93
140	0.65	0.76	0.82	0.85	0.87	0.89	0.90	0.91	0.92	0.92
150	0.64	0.75	0.81	0.84	0.87	0.88	0.89	0.91	0.91	0.92
160	0.62	0.74	0.80	0.84	0.86	0.88	0.89	0.90	0.91	0.91
170	0.62	0.73	0.79	0.83	0.85	0.87	0.88	0.89	0.91	0.91
180	0.60	0.72	0.78	0.82	0.84	0.87	0.88	0.89	0.90	0.91
190	0.59	0.71	0.78	0.82	0.84	0.86	0.87	0.89	0.89	0.91
200	0.59	0.71	0.77	0.81	0.83	0.85	0.87	0.88	0.89	0.90

The initiation time of corrosion can be updated using the values presented in Table 12 as follows:

$$t_{0;cl;IntFr} = I_{cl;IntFr} \cdot t_{0;cl} \tag{41}$$

where:

$t_{0;cl;IntFr}$ updated value of the initiation time of corrosion based on chloride penetration and interacted by internal frost attack, years, and

$I_{cl;IntFr}$ interaction factor for the effect of internal frost attack on the initiation time of chloride initiated corrosion.

5.2.2 Interaction factors for the effect frost scaling on the initiation time of corrosion based on chloride penetration

Based on the analogy with carbonation the depth of critical chloride content with the interaction of frost scaling can be determined as presented in Eq. (42).

$$\begin{aligned} x_{cl;FrSc} &= \sum \Delta x_{cl;FrSc} \\ \Delta x_{cl;FrSc}(t, t+\Delta t) &= \frac{k_{cl}^2}{2} \cdot \frac{1}{x_{cl;FrSc}(t) - x_{FrSc}(t)} \cdot \Delta t \end{aligned} \tag{42}$$

where:

$x_{cl;FrSc}(t)$ is the depth of critical chloride content as influenced by frost scaling (measured from the initial surface of the structure), mm,

$x_{FrSc}(t)$ the depth of frost scaling, mm.

For evaluating the depth of frost scaling with time Eq. (30 - 31) can be assumed to be valid.

The depth of frost scaling can be determined using Eq. (38) and Eq. (42). The interaction coefficients can be calculated from Eq. (41) taking into account that the chloride penetration coefficient of the unaffected concrete can be determined using Eq. (33).

$$k_{cl} = \frac{C}{\sqrt{t_{0;cl}}} \tag{43}$$

where k_{cl} is the coefficient of chloride penetration mm/year$^{0.5}$.

When comparing the algorithms for calculating the interaction coefficients for the initiation times of corrosion for carbonation and chloride induced corrosion the uniformity is clear. So, the interaction coefficients are the same for both cases. The updated initiation time can be determined from Eq. (43) and the values for interaction coefficients can be taken from Tables 10 and 11 (depending on the thickness of the concrete cover).

$$t_{0;cl;FrSc} = I_{cl;FrSc} \cdot t_{0;cl} \tag{44}$$

where:

$t_{0;cl;FrSc}$ updated value of the initiation time of corrosion based on chloride penetration, years, and,

$I_{cl;FrSc}$ interaction factor for the effect of frost scaling on the initiation time of chloride initiated corrosion.

Caution is required when interpreting the results as they are based on a limited number of cement types, and the outcome of the model has not been validated with in situ exposure data.

6 Computer simulation

6.1 General

A computer simulation software developed at VTT in the 1990s was used to illustrate the interaction of degradation mechanisms [2, 3]. The simulation program was updated with the developed degradation models for frost scaling, internal frost attack, carbonation and chloride penetration.

The concrete structure, which in this case was a concrete edge beam, is assumed to be exposed to normal climatic stresses (including variation in temperature and relative humidity of the air, solar radiation, wind and rain). Both daily and seasonal changes are taken into account in the weather models which are based on long-term statistical data on the Finnish climate collected by the Finnish Meteorological Institute. Optional geographical sites (defining the weather) are: Helsinki Airport, Jyväskylä Airport and Sodankylä Observatory.

Calculation methods of the thermal and moisture mechanics are used to determine the variations in temperature and moisture content inside the beam. The surfaces of the structure may be exposed to all weathering stresses or partly protected from stresses such as rain and solar radiation. Use of coatings is also possible, though they were not used in this research.

Degradation of the beam is emulated using mathematical models of degradation in concrete and steel reinforcement. The model used for simulating the frost attack is based on the theory of the critical degree of saturation (internal frost attack is assumed to occur if freezing takes place while the critical degree of saturation is exceeded). The rate of surface scaling is assumed to be proportional to the freezing times of the concrete surface. The amount of de-icing salts spread on the road affects the scaling rate. The model used to evaluate corrosion of the steel reinforcement includes an initiation period and a propagation period. Initiation of corrosion is assumed to take place when either carbonation or the critical chloride content reaches the depth of reinforcement. Once initiated, the rate of corrosion depends on the temperature and moisture content of the concrete. Possible interaction of the degradation modes can also be introduced to the simulation processes.

Many design parameters of concrete are available. The computer program is able to design the concrete mix based on some basic parameters such as the nominal strength, air content and cement type. The critical degree of water saturation is determined by the program for evaluation of the internal frost attack.

The edge beam was modelled for the simulation as presented in Figure 17. The simulation software is 1-dimensional and thus only a 1-dimensional grid could be used in the calculations. The dotted rings in the figure refer to temperature and moisture measurements in the field. The theoretical calculation of temperature and moisture content was validated by these measurements [14].

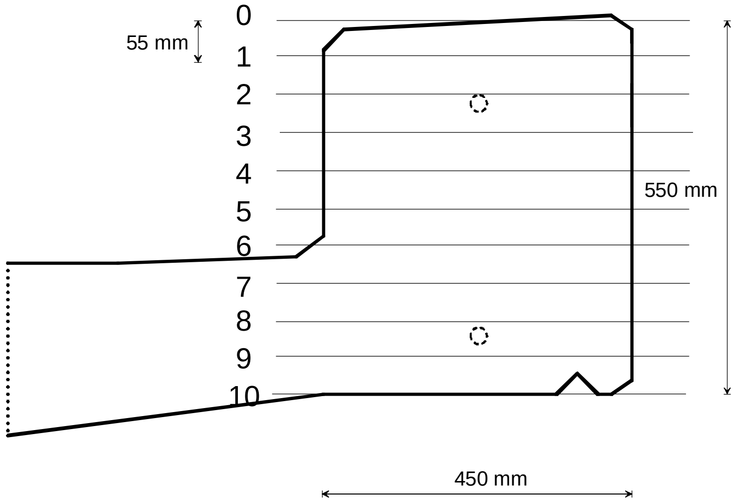

Figure 17. 1-dimensional nodal network for the edge beam.

The increment of time employed in the step-by-step calculation process was 1 hour and the calculation covers a period of 150 years unless the limit state of degradation is reached earlier.

6.2 Results of the computer simulation

In the examples below the parameters of the concrete and exposure were chosen as follows:

- Nominal cube strength of concrete 30 MN/m^2
- Air content of concrete 2 %
- Concrete cover 30 mm
- Helsinki Airport weather
- Exposed to rain and splash.

The calculation was done (a) without chlorides (only carbonation) and (b) with chlorides (chloride penetration is determinative for the initiation of corrosion).

The process of degradation is shown in the following figures as related to the limit state defined for each degradation mode. The limit states were the following:

- RDM 66.7% for internal frost attack
- 15 mm depth for frost scaling
- Depth of concrete cover (in this case 30 mm) for carbonation
- Depth of concrete cover (in this case 30 mm) for chloride penetration

Case 1: Without chlorides

Degradation curves for the upper surface of the beam without chlorides (only carbonation) are presented in Figure 18. Both internal frost attack and surface scaling affect the carbonation curve considerably. At the beginning of the service life frost scaling forces the rate of carbonation to equal to the rate of scaling. Later the increasing internal cracking makes the carbonation rate accelerate. The service life in this case is about 50 years. The determinative degradation mode is internal frost damage.

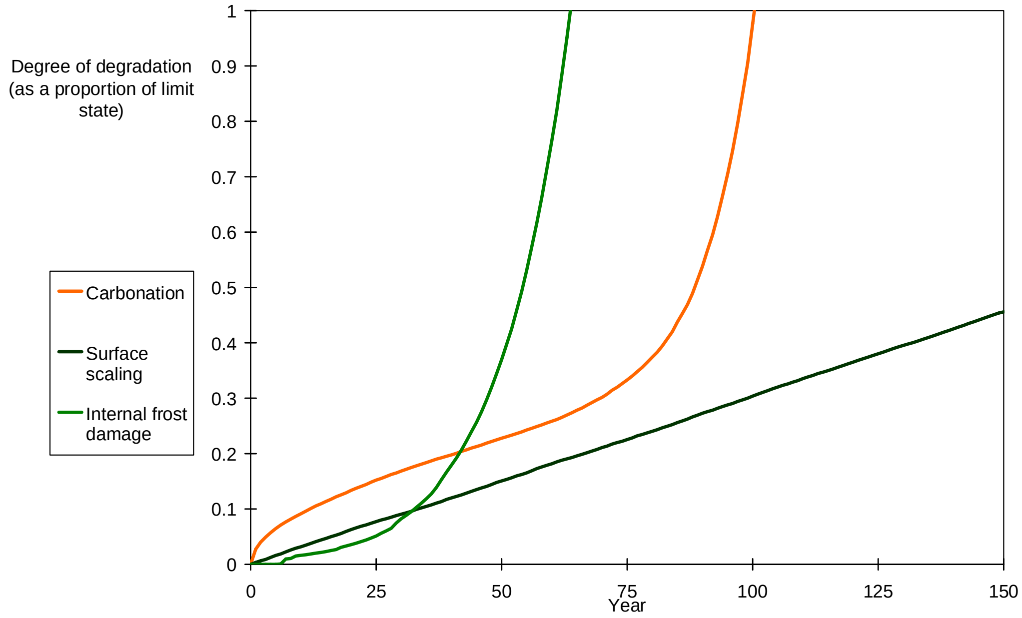

Figure 18. Degradation curves showing the influence of internal frost damage and surface scaling on carbonation at the top surface of the beam.

Figure 19 shows the degradation curves for the bottom surface of the beam. Frost damage is very small within 150 years. The carbonation rate retards as it typically does in undamaged concrete. None of the curves reaches the limit state in 150 years.

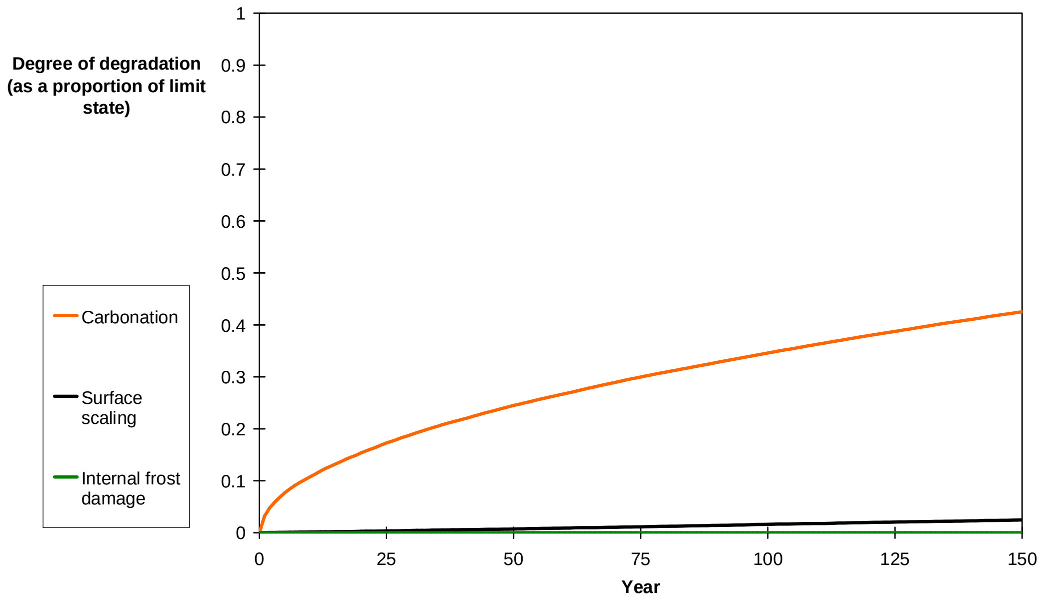

Figure 19. Degradation curves at the bottom surface of the beam.

<u>Case 2: With chlorides</u>

In this simulation process concrete is assumed to be exposed to chlorides. Thus the frost scaling is rapid (Figure 20). Frost scaling causes also the chloride penetration to accelerate. The effect of the internal frost attack on the chloride penetration rate is also clear. The service life in this case is only about 25 years. Surface scaling is the determinative degradation mechanism.

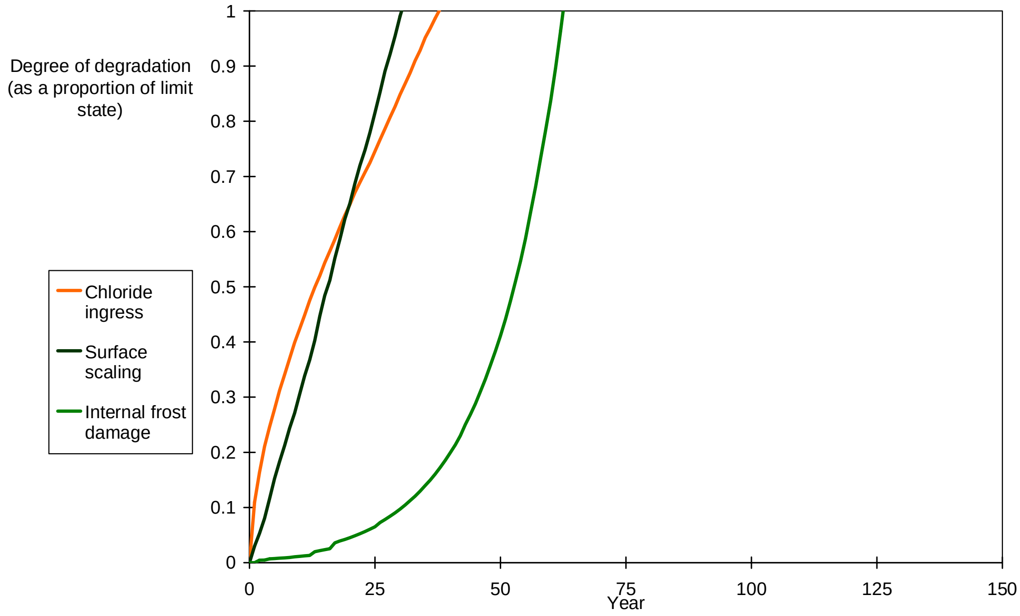

Figure 20. Degradation curves showing the interaction of internal frost attack and surface scaling on chloride penetration at the top surface of the beam.

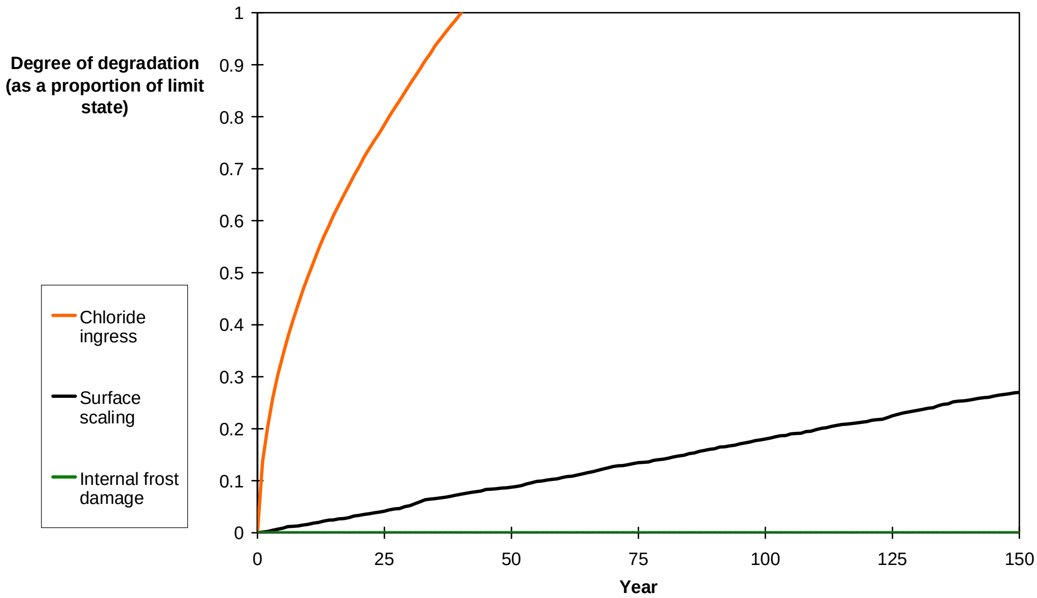

Figure 21. Degradation curves showing the interaction of frost scaling on chloride penetration at the bottom surface of the beam.

The corresponding degradation curves at the lower edge of the beam are seen in Figure 21. The influence of frost attack on the chloride penetration curve is much smaller in this case. The initiation time of corrosion is about 35 years.

The above examples show typical interactions between frost attack and the chemical degradation mechanisms. Both the internal frost attack and the frost scaling may greatly accelerate the rate of carbonation and chloride penetration as shown in Figures 18 and 19. Sometimes internal frost attack is the determinative degradation mechanism for service life (Figure 18). In other cases frost scaling or chloride penetration may be the determinative mechanism (Figure 19).

Considering carbonation and chloride penetration the effect of other degradation mechanisms could be even greater if all interactions would be taken into account. The above simulation process did not take into account the effect of internal frost attack on the rate of scaling which may be considerable when the internal frost cracking is high. Carbonation and chloride penetration may also have mutual interactions which have not been taken into account in the above simulation processes.

7 Conclusions

The overall objective of this research was to analyse the frost test results obtained from both field and laboratory and to study the effects of frost attack on other types of degradation in concrete structures, such as carbonation and chloride penetration.

As it was desired to apply these findings to practical service life design, a special effort was made to determine “interaction factors” for service life models based on the “factor approach”. By the interaction factors the accelerating effect of frost attack on other types of degradation, such as carbonation and chloride penetration, can be taken into account.

The field test measurement results from BTB test field in years 1996 - 2010 were used for analysing the phenomena of frost scaling and internal frost attack since the test results of the DuraInt project had very little exposure time (3 years). The proposed empirical models take into account the principal concrete parameters such as the water/binder ratio; the air content and the cement type were created.

According to field test results the significance of water/cement ratio is very high and it seems even to increase with age. The significance of air content is also noticeable but it seems to decrease with time. The significance of binder type is also very high at the beginning but it tends to reduce with time. Of all the binders studied, the slag cement clearly under-performed for frost scaling resistance.

The model outcome only partially fit the results of *in situ* concrete exposure. It was found that even 12 year exposure period can be considered short given the high quality of the concrete used. In addition, assumptions made during result analysis and modelling influence the outcome. These are mainly that concrete scaling occurred only on one surface, and that scaling degradation with time was considered to be linear and starting at $t = 0$.

The results of laboratory tests for frost scaling were also analysed. When comparing them with field test results it was obvious that the results of the standard laboratory test did not correlate well with the field test results. However, the results of the tests made with preconditioned specimens seemed to correlate fairly well. The preconditioning lasted 1 year in air with 65% RH. During this time the concrete surfaces were carbonated.

The effect of internal frost attack on the rate of carbonation and on the rate of chloride penetration is obvious. This phenomenon was studied using “coupled” tests first causing concrete to crack internally by freezing tests and then starting an accelerated carbonation test or chloride penetration test. A calculation model for the rate of carbonation and for the penetration of chlorides with increasing internal damage in concrete was developed. Interaction factors for the effect of internal damage on the initiation time of corrosion based on carbonation or chloride penetration were developed.

The effect of frost scaling on the rate of carbonation/chloride penetration was also studied. Interaction factors for the effect of frost scaling on the initiation time of corrosion based on carbonation or chloride penetration were derived. These interaction coefficients are applicable to both carbonation and chloride penetration but they are dependent on the thickness of concrete cover.

One important aspect to be remembered is that the effect of frost scaling on the rate of carbonation/chloride penetration was not verified with in situ data. This remains as one requirement to be fulfilled in the future.

Lastly the interaction of different degradation mechanisms was demonstrated using computer simulation. An edge beam made of non-frost resistant concrete was studied. By computer simulation the influence of both internal frost attack and frost scaling on the process of carbonation and chloride penetration could be illustrated.

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Appendix 1 - Data set for Frost scaling

Ref.	Cement Type	w/b (-)	Air content (%)		*In situ* exposed scaling (% volume) /Time (years)						Lab. Scaling
			Design	Real	2	3	4	5	8	12	kg/m^2
401	Anläggning	0.30	4.5	4.6	0.290	0.090	0.105	0.216	0.174	0.354	0.049
402	Anläggning	0.35	4.5	4.5	0.373	0.251	0.283	0.340	0.493	0.835	0.034
403	Anläggning	0.40	4.5	4.5	0.367	0.330	0.331	0.449	0.716	1.247	0.022
403B	Anläggning	0.40	4.5	4.8	0.154	0.237	0.198	0.342	0.564	0.583	0.040
404	Anläggning	0.50	4.5	4.2	0.288	0.440	0.409	0.649	1.256	1.784	0.026
405	Anläggning	0.75	4.5	4.2	1.118	1.462	1.842	2.209	4.821	8.415	0.110
406	Anläggning	0.30	2	1.4	0.361	0.219	0.152	0.224	0.297	0.417	1.564
407	Anläggning	0.35	2	1.0	0.304	0.220	0.252	0.246	0.483	0.604	6.727
408	Anläggning	0.40	2	1.1	0.404	0.356	0.388	0.469	1.095	1.327	8.109
408B	Anläggning	0.40	2	1.0	0.297	0.199	0.274	0.280	0.999	1.147	8.596
409	Anläggning	0.50	2	0.8	0.687	0.726	0.907	1.027	1.810	2.372	15.656
411	Slite standard	0.30	4.5	4.6	0.316	0.239	0.250	0.173	0.388	0.097	0.055
412	Slite standard	0.35	4.5	4.4	0.195	0.154	0.066	0.134	0.397	0.319	0.032
413	Slite standard	0.40	4.5	4.6	0.240	0.218	0.271	0.229	0.604	0.688	0.039
414	Slite standard	0.30	2	1.7	0.266	0.175	0.171	0.125	0.260	-	0.149
415	Slite standard	0.35	2	1.8	0.224	0.162	0.167	0.068	0.297	0.224	0.735
416	Slite standard	0.40	2	2.0	0.269	0.173	0.238	0.290	0.611	0.469	4.590
417	Anläggning + 5% SF	0.30	4.5	5.0	0.210	0.151	0.181	0.236	0.528	0.568	0.081
417B	Anläggning + 5% SF	0.30	4.5	4.2	0.126	0.092	0.071	0.262	0.340	0.279	0.133
418	Anläggning + 5% SF	0.35	4.5	4.2	0.148	0.092	0.107	0.097	0.409	0.419	0.050
419	Anläggning + 5% SF	0.40	4.5	4.4	0.288	0.308	0.283	0.403	0.723	0.873	0.027
420	Anläggning + 5% SF	0.30	2	1.6	0.159	0.087	0.133	0.185	0.369	0.354	0.790
421	Anläggning + 5% SF	0.35	2	1.2	0.185	0.137	0.121	0.238	0.428	0.417	1.953
422	Anläggning + 5% SF	0.40	2	1.5	0.267	0.267	0.242	0.339	0.588	0.604	3.359
423	Portland-limestone filler	0.30	4.5	4.4	0.326	0.161	0.104	0.135	0.311	0.140	0.054
424	Portland-limestone filler	0.35	4.5	4.8	0.236	0.056	0.062	0.323	0.431	0.386	0.039
425	Portland-limestone filler	0.40	4.5	4.2	0.630	0.487	0.540	0.560	1.104	1.126	0.040
426	Portland-limestone filler	0.30	2	2.1	0.369	0.156	0.119	0.229	0.354	0.140	0.077
427	Portland-limestone filler	0.35	2	2.0	0.276	0.104	0.114	0.255	0.412	0.307	1.873
428	Portland-limestone filler	0.40	2	1.7	0.329	0.185	0.228	0.369	0.676	0.654	5.437
429	Finnish Standard	0.30	4.5	4.8	0.319	0.141	0.157	0.355	0.450	0.380	0.072
430	Finnish Standard	0.35	4.5	4.3	0.279	0.150	0.217	0.217	0.538	0.518	0.075
431	Finnish Standard	0.40	4.5	4.8	0.421	0.362	0.462	0.559	0.942	1.037	0.076
432	Finnish Standard	0.30	2	2.0	0.279	0.129	0.155	0.154	0.331	0.233	1.015
433	Finnish Standard	0.35	2	1.5	0.248	0.140	0.181	0.150	0.430	0.383	7.784
434	Finnish Standard	0.40	2	1.6	0.564	0.479	0.547	0.595	1.014	1.087	10.193
461	Finnish Standard	0.40	4.5	4.0	0.369	0.363	0.467	0.510	0.937	1.161	0.177
435	Anläggning + 30% GGBS	0.30	4.5	4.8	0.264	0.183	0.157	0.188	0.345	0.504	0.132
436	Anläggning + 30% GGBS	0.35	4.5	4.7	0.295	0.316	0.328	0.396	0.737	1.104	0.083
437	Anläggning + 30% GGBS	0.40	4.5	4.4	0.409	0.386	0.519	0.698	1.413	1.816	0.037
438	Anläggning + 30% GGBS	0.30	2	2.2	0.188	0.097	0.102	0.102	0.685	0.383	0.175
439	Anläggning + 30% GGBS	0.35	2	1.7	0.305	0.266	0.321	0.414	0.854	0.987	0.517
440	Anläggning + 30% GGBS	0.40	2	1.7	0.273	0.273	0.338	0.417	0.823	1.118	1.546
441	Blast furnace slag	0.30	4.5	4.3	0.297	0.279	0.312	0.295	0.512	0.462	0.431
442	Blast furnace slag	0.35	4.5	4.8	0.509	0.519	0.612	0.695	0.978	1.125	0.718
443	Blast furnace slag	0.40	4.5	4.6	1.209	1.396	1.596	1.711	2.634	3.203	0.778
444	Blast furnace slag	0.30	2	2.2	0.373	0.386	0.471	0.484	0.688	0.725	0.212
445	Blast furnace slag	0.35	2	1.5	0.838	1.050	1.154	1.103	1.765	1.923	0.512
446	Blast furnace slag	0.40	2	0.9	3.343	3.826	4.093	4.071	7.866	8.518	0.781
451	Finnish sulphate resisting	0.40	4.5	4.7	0.302	0.333	0.397	0.438	0.726	0.828	0.033
451B	Finnish sulphate resisting	0.40	4.5	4.3	0.247	0.262	0.335	0.377	0.669	0.747	0.041
452	Finnish sulphate resisting	0.50	4.5	4.2	0.450	0.471	0.700	0.687	1.428	1.829	0.110
453	Finnish sulphate resisting	0.30	2	1.7	0.159	0.048	0.143	0.068	0.217	0.195	1.416
454	Finnish sulphate resisting	0.40	2	1.5	0.348	0.298	0.390	0.442	0.716	0.844	6.437
454B	Finnish sulphate resisting	0.40	2	1.7	0.279	0.204	0.307	0.316	0.597	0.683	5.474
464	Finnish sulphate resisting	0.40	4.5	4.9	0.155	0.176	0.273	0.314	0.647	0.932	0.043

Appendix 1 - Data set for Frost scaling (continued)

Ref.	Cement Type	w/b (-)	Air content (%)		*In situ* exposed scaling (% volume) /Time (years)						Lab. Scaling
			Design	Real	2	3	4	5	8	12	kg/m^2
455	Anläggning modified	0.30	4.5	4.8	0.202	0.106	0.173	0.162	0.329	0.414	0.212
456	Anläggning modified	0.40	4.5	4.7	0.286	0.265	0.374	0.420	0.735	0.918	0.040
456B	Anläggning modified	0.40	4.5	4.7	0.126	0.121	0.195	0.200	0.559	0.680	0.044
457	Anläggning modified	0.50	4.5	4.6	0.283	0.329	0.461	0.523	1.114	1.646	0.029
458	Anläggning modified	0.30	2	1.2	0.100	0.058	0.141	0.063	0.274	0.267	0.563
459	Anläggning modified	0.40	2	1.4	0.245	0.235	0.317	0.250	0.554	0.640	3.480
459B	Anläggning modified	0.40	2	1.4	0.212	0.165	0.260	0.244	0.561	0.683	6.321
462	Finnish Rapid	0.40	4.5	4.2	0.131	0.126	0.190	0.210	0.452	0.542	0.162
463	Finnish Mega	0.40	4.5	4.7	0.271	0.248	0.340	0.333	0.547	0.647	0.171