The BRITE / EURAM project " The residual service life of concrete structures" : summary of subtask freeze-thaw

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The BRITE/EURAM Project

”The residual service life of concrete structures”

Summary of subtask freeze-thaw

Göran Fagerlund

Report TVBM-7100 Lund, 1995

Preface

This report is a part of the summarizing Final Technical Report of the BRITE/EURAM Project BREU-CT92-0591"The Residual Service Life of Concrete Structures". The report covers all activites that have been performed within the destruction type "Frost damage".

Two other destruction types have also been covered in the project:

* Reinforcement corrosion
* Alkali-silica reaction

Summaries of the activities on these destruction types will also be included in the Final Technical Report.

Besides the Final Technical Report there will also be produced a Manual for assessment of the structural stability of damaged concrete structures.

Six partners have participated in the BRITE/EURAM project:

1: British Cement Association, BCA (The Coordinator)
2: Instituto Eduardo Torroja, Spain
3: Geocisa, Spain
4: The Swedish Cement and Concrete Research Institute, CBI, Sweden
5: Cementa AB, Sweden
6: Division of Building Materials, Lund University, Sweden

I like to thank Dr. George Somerville from BCA, who led this very complex project with extraordinarily great skill and patience. I also want to thank all the other participants who made it a great pleasure for me to take part in the project; Carmen Andrade and Maria Alonso from Instituto Eduardo Torroja in Madrid, Don Hobbs, Paul Chana and Mike Webster from BCA and Imperial College in London, Jésus Rodriguez and Luis Ortega from Geocisa in Madrid, Kyösti Tuutti from Cementa (now at Skanska) in Danderyd, and Karin Pettersson and Björn Lagerblad from CBI in Stockholm.

Lund, December 6, 1995

Göran Fagerlund

Contents

BRITE/EURAM project BREU-CT92-0591
"The Residual Service Life of Concrete Structures".

Final Technical Report on subtask Freeze-Thaw

General comments

This part of the Final Technical Report is a summary of all deliverables within subtask freeze-thaw; see the list in the footnote below [1]. References are made in the text to the relevant deliverables where more detailed discussions can be found. Only Tasks 1, 2, 3, and 4 are treated. Tasks 5 and 6 are dealt with in the Manual produced within the project.

In some cases, frost attack cannot be treated as an isolated phenomenon since it migh affect other types of destruction. This synergism has been treated in some papers, that are not official Deliverables[2] . A summary is presented below in the APPENDIX.

Under Task 1 "Aggressivity of environment" a presentation is also made of a more general method of calculating the internal moisture level and moisture variation inside the concrete. This is of value not only for freeze-thaw, but for all types of destruction.

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[1]) Task 1:
  Deliverable 12.1: *Calculation of the moisture-time fields in concrete*
  Deliverable 24.2: *Influence of environmental factors on the frost resistance of concrete*

Task 2:
  Deliverable 30.1: *Assessment of the current state of the structure*

Task 3:
  Deliverable 28.1: *Effect of frost damage on the bond between reinforcement and corrosion*

Task 4:
  Deliverable 12.5: *The long-time water absorption in the air-pore structure of concrete*
  Deliverable 18.6: *The critical spacing factor*
  Deliverable 30.4:
    Part 1: *Freeze-thaw resistance of concrete. A survey of destruction mechanisms, technological factors, test methods*
    Part 2. *Interrelation between the service life and the air content of concrete exposed to freeze/thaw*
    Part 3. *Estimation of the future deterioration caused by frost action*

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[2])G Fagerlund: *Synergism: Effect of ASR on frost resistance. Some imaginable effects.* The Brite/Euram project, Interim Report IR7, Div. of Building Materials, Lund Institute of Technology, 1994.

G. Fagerlund, G. Somerville, K. Tuutti: *The residual service life of concrete exposed to the combined effect of frost attack and reinforcement corrosion.* Int. Conf. "Concrete across Borders", Odense, June 22-25, 1994.

Task 1
Aggressivity of environment
(Deliverables 12.1 and 24.2)

1 Important climatic factors general for all types of deterioration

All types of deterioration are, to a greater or smaller degree, determined by the following three main *outer* environmental (climatic) parameters:

1: *The moisture characteristics* expressed in terms of average values and variations in the relative humidity (RH), and in terms of the duration of contact between the surface of the concrete and liquid water. These factors determine the moisture level and variation over the concrete cross section.

2: *The temperature characteristics* in terms of the average value and time dependent variation of the outer temperature. These factors determine the temperature-time field inside the concrete, and thereby the rate of destruction. They also determine the number of freeze/thaw passages inside the concrete. The minimum temperature is of great importance for the extent of frost damage.

3: *The salinity of the surrounding air and liquid water*. Normally it is the chloride concentration, that is of major interest. The outer chloride concentration determines the chloride-time field inside the concrete, and it is, therefore, fundamental for salt scaling and reinforcement corrosion. The concentration of alkaline kations influences the alkali silica reaction.

In many cases, there is an important co-variation between the three types of outer environmental conditions; the combination of high moisture levels with a high chloride level is much more troublesome, than is the combination with low moisture levels. One major reason is that the chloride penetration is much slower in a drier concrete. Similarly, high moisture levels are more troublesome the lower the minimum temperature.

The outer environmental conditions determine the inner ”environment” in the concrete. This, in turn, determines the rate of destructiuon of the concrete. Therefore, the understanding of the interrelation beteween outer and inner climatic factors is of outmost importance for the possibility of estimating deterioration and destruction rates.

2 Calculating the interrelation between outer and inner environment- generally

Moisture levels:
A calculation of the interrelation between outer and inner moisture conditions is based on the following information:

1: Variation of the outer RH
2: Duration of contacts between concrete surface and liquid water
3: Moisture transport coefficients of concrete within ”the hygroscopic range” ($<98\%$ RH)
4: Ditto within ”the capillary range” ($\approx 98\% < \text{RH} \leq 100\%$)
5: Moisture fixation in the concrete; the moisture equilibrium curves

The calculation is based on a potential equation with the moisture content, or RH, or the vapour content, or the ”suction” as the driving potential. Any potential can be used, provided the transport coefficient and the driving potential are adjusted to each other[3]. The following potential equation is used:

$$q = -\delta(\phi)\cdot d\phi/dx \qquad (1.1)$$

Where q is the flow [kg/m^2·s], $\delta(\phi)$ is the transport coefficient, which is a function of the moisture content or any other potential used [m^2/s], ϕ is the potential, and x is the coordinate in the direction of flow [m].

A computer programme for solving Eq (1.1) has been developed[4]. The major obstacle in using Eq (1.1), and the programme, lies in lacking information of the transport coefficient within the high moisture range; "the capillary range". New data are however now available[5]. They have been used in the calculations made and presented in *Deliverable 12.1*.

Eq (1) cannot be used for the case where the concrete is placed in contact with water for very long times. Then, the water ingress is determined by quite another process, viz. dissolution of air that was entrapped in air-bubbles inside coarse air-pores, and replacement of dissolved air by water. This slow process is dealt with in detail in Task 4 in *Deliverable 12.5*.

Temperature levels:
The interrelation between the outer temperature spectrum and the inner temperature-time field can be easily calculated by computer programmes, that are available commersially. Thermal material data needed for the calculation are also available. The problem of transferring outer temperature fields to inner temperature fields is, therefore, not treated in the project.

Chloride levels:
The relation between the outer and the inner chloride concentrations can be calculated by a simple potential equation, provided all information is at hand concerning diffusion coefficients, chloride binding isotherms, and outer chloride concentrations. The following potential equation can be used:

$$q = -\delta_{eff}(c,RH)\cdot dc/dx \qquad (1.2)$$

Where q is the chloride flow [mole/m^2·s], $\delta_{eff}(c,RH)$ is the effective chloride diffusion coefficient, taking into consideration non-linear chloride binding (i.e. effect of c) and inner moisture conditions (i.e. effect of RH) [m^2/s], c is the chloride concentration [mole/m^3], and x is the coordinate in the direction of flow. The material coefficient δ_{eff} is highly dependent of the chloride level, the temperature, and the moisture condition.

In its simpliest form, when the outer concentration and chloride binding are constant, and the concrete is water saturated, the solution is the well-known solution to Ficks 2:nd law[6]. In the

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[3] *Deliverable 12.1*

[4] J Arfwidsson: *Computer models for moisture flow in porous media*. Div. of Building Physics, Lund Institute of Technology, Report TVBH-7108, Lund, 1988

[5] G Hedenblad: *Moisture permeability of mature concrete, cement mortar, and cement paste*. Div. of Building Materials, Lund Institute of Technology, Report TVBM-1014, 1993.

[6] Semi-infinte medium: $c/c_o = \text{erfc}\{x/(4\cdot t\cdot\delta_{eff}]^{1/2}\}$ where t is time and c_o is the constant surface concentration. erfc is the complementary error function (erfc=1-erf)

more complicated case, the outer chloride concentration varies with time (e.g in the splash zone, or in concrete exposed to deicing salts), the chloride binding is non-linear and highly temperature-dependent, and the moisture content varies with time and location in the concrete cover. Principles for solving Eq (1.2) numerically for these variable conditions have been presented[7]. There is, however, lacking knowledge of all these variations which makes an exact solution very difficult.

Situations when a detailed analysis of the inner environment is required:
In order to make an estimate of the future deterioration and residual service life, it is, in many cases, not necessary to solve the interrelations between the outer and inner environment. The effect of the actual climate -or "the historic climate"- on a certain destruction type is obtained directly from the observed environmental impact on the structure, as obtained from direct observations of the structure. In such cases, one can simply make an extrapolation of the "historic impact curve", or destruction curve, under condition that the destruction mechanism is well-know, and under condition that the outer climate will not change in the future. This extrapolation will give the residual service life, once the minimum technical performance has been defined.

If the outer climate is likely to undergo significant changes in the future, the inner environment will of course also be changed. Then, in order to be able to make a prediction of the future destruction, one has to make calculations of the future inner environment. One important example of such changes is new methods of deicing. Chlorides might be changed for other salts, or chemical deicing is completely abandonded. This will change both frost attack and reinforcement corrosion drastically.

Another possibility is that the environment has been changed in the past. Then, the observed destruction, being an average value of two destruction rates, is not significant for the future destruction. A new extrapolation must be made, based on information of the present and future inner climate.

Another example, where the outer climate is unchanged, but the inner environment might be drastically changed, is the application of a repair material to the surface. Then, it is necessary to estimate the effect of the repair on the future inner moisture environment. It might become higher, increasing the risk of frost damage, or it might become lower, increasing the risk of carbonation. Studies have been made of the effect of repairs on the frost resistance of concrete [8]

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[7]) L-O. Nilson, L. Tang: *A numerical method for prediction of chloride penetration into concrete structures.* Manuscript presented to the NATO/RILEM Workshop on "The Modelling of Microstructure and its Potential for Studying Transport Properties and Durability", Saint Rémy les Chevreuses, July 1994. Div. of Building Materials, Chalmers University of Technology, Gothenburg, 1994.

B. Johannesson: *Mixture theory to chemical reaction and diffusion of different media in mature concrete; a hypothetical model.* Parts 1 and 2. Manuscripts presented to the RILEM seminar on "Chloride penetration", Saint Rémy les Chevreuses, October, 1995. Div. of Building Materials, Lund Institute of Technology, 1995.

[8]) E.g. the following references:

G. Fagerlund, O. Svensson: *Durability of repair systems for concrete balconies.* Swedish Cement and Concrete Research Institute, Research Fo 2:80, 1980

G. Fagerlund. O Svensson, N. Petersons: *Repair of concrete balcony slabs.* in "Advances in Concrete Slab Technology", Ed. R.K. Dhir, J.G.L Munday, Pergamon Press Oxford, 1980.

3 Effect of the inner moisture level on the destruction- generally

The internal moisture level determines almost all deterioration processes. The mechanisms by which moisture act are:

1: The moisture level, e.g. expressed in terms of RH, determines the diffusion rate of gases, such as carbon dioxide and oxygen. Thus, it determines the incubation time before onset of reinforcement corrosion induced by carbonation, and it determines the corrosion rate. Examples of the relation between RH and diffusion of oxygen is shown in Fig 1.1[9]

2: The moisture level determines the electrical resistivity, and therefore the rate of corrosion.

3: The moisture content is probably of very big importance for binding and diffusion of chloride. The moisture relations are unknown, and have not been studied in the project.

4: The moisture level is the most important factor for frost attack. Long exposure to liquid water is normally required, if frost damage should occur.

5: The moisture content is one of the most important parameters determining the amount of ASR-expansion. Moisture levels around 100% are normally required for severe internal attack. Somewhat lower RH-levels can be sufficient for so-called pop-outs[10]

4 Calculating the interrelation between outer and inner moisture levels *(Deliverable 12.1)*

The calculation is made with Eq (1.1) using the following information:

1: Moisture transport data in the hygroscopic range, determined from steady-state measurements of moisture flow across mature concrete specimens of different concrete quality (w/c-ratio) and thickness[11] . Examples are seen in Fig 1.2.

2: Moisture transport data in the capillary range determined from non-steady-state water absorption experiments, using thin slices sucking water uni-directionally. The moisture transport coefficient for 100%, δ_{100}, is determined using Eq (1.1) for the entire moisture range $0 \leq RH \leq 100\%$, and choosing a value of δ_{100}, that makes the calculated total water absorption curve fit the real curve in the best possible manner. In this way, Eq(1.1) can be used for the entire moisture range, which makes possible for the first time, consideration to the effect of longer or shorter periods of exposure to liquid water. An example of calculated and measured water absorption curves, and the variable transport coefficient coming out of the experiment, are shown in Fig 1.2 and 1.3. The distinction between the capillary range (liquid water range) and the hygroscopic range (the diffusion range) is quite clear; liquid transport is almost 20 times as rapid as diffusion at high RH-levels.

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[9]) K Tuutti: *Corrosion of steel in concrete*. Swedish Cement and Concrete Research Institute. Report Fo 4:82, 1982.

[10]) L-O. Nilsson, O. Peterson: *Alkali-silica Reactions in Scania, Sweden - a Moisture Problem causing Pop-outs in Concrete Floors*. Div. of Building Materials, Lund Institute of Tecnlology, Report TVBM-3014, Lund 1983.

[11]) G Hedenblad: *Moisture permeability of mature concrete, cement mortar, and cement paste*. Div. of Building Materials, Lund Institute of Technology, Report TVBM-1014, 1993.

3: Moisture equilibrium curves determined experimentally for adsorption as well as for desorption. The curves used are shown in Fig 1.5[12] .

These data, introduced into the computer programme, can be used for calculating time-moisture fields in concrete. Consideration can be taken to any variation in the outer RH-level, and to shorter or longer exposures to rain. The calculation is made using the so-called *flow potential* (the Kirchoff potential). A computer programme was developped for the purpose[13].

The flow potential is defined:

$$\psi(\phi) = \int_{\phi_{ref}}^{\phi} \delta(\phi) \cdot d\phi \qquad (1.4)$$

Where $\psi(\phi)$ is the flow potential [kg/m·s] with ϕ as the driving potential, $\delta(\phi)$ is the transport coefficient [m^2/s], and ϕ expresses the moisture level [in this case the moisture content in mass, kg/m^3]. The flow potential is defined as the surface between the curve describing the moisture transport coefficient between a reference moisture level, ϕ_{ref}, and the actual moisture level, ϕ; see Fig 1.6. The flow is obtained by dividinging the flow potential by the thickness of the material between the two moisture contents ϕ_{ref} and ϕ. Thus, the flow potential is another way of expressing Eq (1) which simplifies the numerical calculations considerably.

Many calculated examples for concrete with the w/c-ratio 0,40, 0,50 and 0,60 are shown in *Deliverable 12.1*.

Not exposed to liquid water:
Examples of two concretes (w/c, 0,40 and 0,60) exposed to a certain step-wise variation in RH between 60% and 95% (2 weeks at 60% and 2 weeks at 95%) are shown in Fig 1.7. The following important observations are made:

1: The concrete does not "feel" changes in the outer environment to a bigger depth than about 2 cm for w/c 0,40, and 3 cm for w/c 0,60. On the other hand, the outer part of the concrete cover is subjected to big variations in the moisture level. This is, of course, of very big importance for the service life with regard to reinforcement corrosion.

2: The internal equlibrium RH-level on a few centimeters depth, or more, from the surface is very high (90 à 92%) compared with the average level of the outer RH, which is only 77%. This is an effect of the non-linear equilibrium moisture curves -Fig 1.5. The RH-level is high enough to cause a significant corrosion rate in concrete in which corrosion has been initiated. Besides, the RH-level is maintained on a high level, also during periods when the outer climate is dry.

Exposed to liquid water (e.g rain):
Examples of two concretes (w/c, 0,40 and 0,60) exposed to a certain step-wise variation in outer moisture -1 month in 80% RH and 6 hours of rain- are shown in Fig 1.8. The following important observations are made:

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[12]) L-O. Nilsson: *Hygroscopic moisture in concrete. Drying measurements and related properties*. Div. of Building Materials, Lund Institute of Technology. Report TVBM-1003, Lund 1980.

[13]) J. Arfwidsson: *Computer programme for calculating moisture variations in concrete, caused by variations in outer moisture*. Div. of Building Physics, Lund Institute of Technology, Lund 1993.

1: For the low w/c-ratio, the internal equilibrium moisture level is only 87%. For the high w/c-ratio, it is 96%. This is an effect of the much more rapid absorption of moisture in a concrete of lower quality. The effect will be that corrosion problems are much more severe in concretes with the higher w/c-ratios.

2: The rain is only felt into a rather small depth from the surface when the concrete has a low w/c-ratio; about 1 cm. For the concrete with the higher w/c-ratio, it is felt into a depth of 2 cm.

When there are other variations in the outer climate, the difference between a concrete with low and high w/c-ratio is even more pronounced; see *Deliverable 12.1.*

5 Environmental effects on frost damage (*Deliverable 24.2*)

5.1 Different types of frost attack

There are two types of frost attack to consider; Fig 1.9.

Type 1: Internal damage caused by freezing of water inside the concrete in such parts where the degree of water saturation exceeds the critical level.

Type 2: Salt scaling of the surface caused by freezing of weak salt solutions that stay in contact with the surface; e.g deicing salts, or sea water.

Freezing of very wet concrete seldom leads to scaling, while freezing of salt solutions at the surface seldom leads to internal attack.

The destruction mechanism and rate are treated under Task 4 below. The most important conclusion of the analysis is that *critical spacing factors* exist between air-filled voids. The spacing factor is a measure of the average distance between air-pores. Its value depends on its definition. Normally, the so-called Powers spacing factor is used according to which all air-pores are assumed to be of equal size and placed in a cubic array; Fig ?[14] .

5.2 Effect of the inner moisture level

The inner moisture content is preferably expressed in terms of degree of saturation which is defined:

$$S = w_e/P \quad (0 \leq S \leq 1) \tag{1.5}$$

Where, w_e is the amount of evaporable water [m^3/m^3], and P is the total porosity [m^3/m^3].

Another possibility is to express the moisture content in terms of the effective degree of saturation, S_{eff}, where consideration is only taken to the freezable water:

$$S_{eff} = w_f/(w_f+a) \quad (0 \leq S_{eff} \leq 1) \tag{1.6}$$

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[14] T.C. Powers: *The air requirement of frost resistant concrete.* Proc. Highway Research Board, No 29, 1949.

Where, w_f is the freezable water [m^3/m^3], and a is the air-filled pore volume [m^3/m^3].

The relation between the two degrees of saturation is:

$$S = S_{eff}(1-k_\theta)+k_\theta \tag{1.7}$$

Where k_θ is the relation between non-freezable water and total porosity; $k_\theta=(w_e-w_f)/P$.

Normal values of k_θ are 0,4 to 0,6; i.e. S_{eff}=0,80 corresponds to S=0,88 to 0,92. Therefore, the parameter S is considerably bigger than the parameter S_{eff}.

Since critical spacing factors evidently exist, it can be proven theoretically and experimentally that there also exist maximum allowable degrees of saturation. The value is only a function of the size of the critical spacing factor, and the shape of the air-filled pore system.

Experimental determination of the critical degree of saturation:
Examples of experimental determinations of S_{CR} are shown in Fig 1.10. The values are 0,90 for the non-airentrained concrete, and 0,80 for the airentrained concrete. These values can be compared with the degrees of saturation, that correspond to complete water-filling of all pores except the air-pores. These values are 0,87 and 0,61 respectively. This means that only a small absorption in air-pores can be allowed in a non-airentrained concrete. For the airentrained concrete, a considerable water absorption in the initial air-pore system must take place before frost damage can occur. Considering the very slow water absorption in air-pores[15] this means that the potential service life with regard to frost attack is considerably higher in the airentrained concrete.

Theoretical determination of the critical degree of saturation:
A theoretical determination of the critical degree of saturation is made by assuming that frost damage occurs when the water absorption inside the initial air-pore system is so big, that the residual spacing factor between pores, that are still air-filled, is equal to the critical spacing factor. The assumption made is that a small air-pore is always water-filled before a coaser air pore. The starting point is the size distribution of the air-pore system (pores bigger than about 10 μm).

After a certain water-filling of the air-pore system the following values are valid for the residual air-pore volume, a_r [m^3], the residual envelope area of air-filled pores, A_r [m^2], the residual specific area of air-filled pores, α_r [$m^2/m^3=m^{-1}$] and the residual spacing factor between air-filled pores, L_r [m] (1 m^3 of concrete is considered):

$$a_r = \int_{r_{min}}^{r_{max}} f(r)\cdot(4\pi/3)\cdot r^3\cdot dr \tag{1.8}$$

$$A_r = \int_{r_{min}}^{r_{max}} f(r)\cdot 4\pi\cdot r^2\cdot dr \tag{1.9}$$

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[15]) See *Deliverable 12.5*

$$\alpha_r = A_r/a_r \qquad (1.10)$$

$$L_r = (3/\alpha_r)\cdot\{1.4(V_p/a_r+1]^{1/3}-1\} \qquad (1.11)$$

Where, r_{max} is the radius of the biggest air-pore, r_{min} is the radius of the biggest *water-filled* air-pore, f(r) is the frequency function of all air-pores in 1 m^3 of concrete, and V_p is the amount of cement paste exclusive of air-pores $[m^3/m^3]$.

The critical degree of saturation of the air-pore system itself is:

$$S_{a,CR} = 1-a_{CR}/a_o \qquad (1.12)$$

Where, a_{CR} is the residual air-pore volume exactly when the critical spacing factor is reached $[m^3/m^3]$, a_o is the total initial air-pore volume before any water-filling $[m^3/m^3]$.

The relation between the degree of saturation of the air-pore system and the degree of saturation of the entire concrete is:

$$S_{CR} = 1-a_{CR}/P = \{a_o(1-S_{a,CR})\}/P \qquad (1.13)$$

Application of this calculation method to two air-pore systems described by the frequency function $f(r) = a\cdot\ln b/b^r$, where a and b are constants, and r the pore radius, is shown in Fig 1.11. The lower value of b (1,02) corresponds to a coarse-porous air-pore system. The higher value of b (1,03) correspondes to a more fine-porous system.

For both concretes, the critical spacing factor L_{CR} is supposed to be 0,40 mm, the total porosity is 16%, and the total air-pore volume is 4%. This gives the following values of the critical degrees of saturation:

Coarse-porous:

$S_{a;CR} = 0{,}18$
$S_{CR} = 0{,}80$

Fine-porous:

$S_{a,CR} = 0{,}44$
$S_{CR} = 0{,}86$

The critical degree of saturation, that can be determined, either experimentally, or theoretically, is a *fracture value* that can be compared with the fracture load in static design. Frost damage will only occur when the really occurring moisture content, *the moisture load*, transgresses the critical value. The risk of frost damage is imminent if the water absorption in practice can be assumed to be higher than the critical.

5 3 Water in aggregate and in defects (cracks , interfaces etc)

Porous aggregate particles, with a size distribution, that is such, that the particle is likely to become water-filled during practical conditions, will easily damage the concrete when they freeze. A relation is derived for the relation between the degree of saturation of a coarse porous particlse, the volume fraction of such particles, and the stress they create in the mortar:

$$\sigma_t = \frac{w_{f,a}[0{,}09-(1-S_{a,eff})/S_{a,eff}]\cdot E_m}{3[(1-\nu_m)-\nu_m\cdot 2(1-g_a)/(2\cdot g_a+1)]} \qquad (1.14)$$

Where, σ_t is the tangential stress in the mortar phase [Pa], $w_{f,a}$ is the freezable water in the aggregate [m^3/m^3], $S_{a,eff}$ is the effective degree of saturation of the aggregate, E_m is the E-modulus of the cement mortar, ν_m is the Poisson ratio of the cement mortar, and g_a is the volume fraction of the cement mortar [m^3/m^3].

This equation shows that only a very small amount of pores can be tolerated in an aggregate, that will become easily saturated in practice.

A similar analysis of the effect of cracks and interfaces between aggregate and cement paste, indicates, that it is only under certain circumstances (very wide water-filled cracks and interfaces) that such defects can cause frost damage[16].

5.4 Effect of outer moisture conditions

The outer environment influences the risk of frost damage mainly through its effect on the inner moisture condition. Thus, a more moist environment gives a bigger risk that the critical moisture content shall be reached and exceeded. The interrelation between the outer and inner environment is treated in *Deliverable 12.5* belonging to Task 4, described below.

Every outdoor concrete will be exposed to short periods of very moist condition. This means, that all pores except the air-pores will become saturated from time to time. This frequently occurring water content corresponds to the following minimum degree of saturation:

$$S_{min} = 1 - a_o/P \qquad (1.14)$$

Where, a_o is the total air content [m^3/m^3], and P is the total porosity [m^3/m^3].

A concrete for which the critical degree of saturation is lower than S_{min} will be very vulnerable to frost damage.

During more moist conditions, there will also be a certain water absorption in the air-pore system. It is shown in *Deliverable 12.5* that this moisture condition can often be described by an equation of the following type, which is valid for a concrete that is constantly stored in water of constant temperature:

$$S = S_{min} + B\cdot t^C \qquad (1.15)$$

Where, the coefficients B and C are functions of the permeability of the concrete, and of the shape of the air-pore system.

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[16]) See *Deliverable 30.4 Part 1*.

The three coefficients S_{min}, B and C can be easily determined by an experiment during which thin concrete slices are sucking water from a water tray[17].

By such experiments, supported by a determination of the critical dehgree of saturation, it is possible to define a potential service life t_p which can be used as a measure of the risk of frost damage:

$$t_p = \{(S_{CR}-S_{min})/B\}^{1/C} \quad (1.16)$$

This equation expresses the situation where the real water absorption, expressed in terms of Eq (1.15), equals the critical degree of saturation.

5.5 *Effect of the number of freeze-thaw cycles*

No moisture exchange with the environment during freeze/thaw:
In freeze-thaw, where the concrete is sealed from moisture gain or loss, the number of freeze-thaw cycles is only of importance for degrees of saturation above the critical. An example is shown in Fig 1.12[18]. The S_{CR}-value serves as a "hinge" for the damage degree of specimens, which are more than critically saturated. On the basis of this figure, the following relation can be derived:

$$D=0 \quad \text{for } S\leq S_{CR} \quad (1.17a)$$
$$D=K_N(S-S_{CR}) \quad \text{for } S>S_{CR} \quad (1.17b)$$

Where, D is the "damage degree" and K_N is a coefficient of fatigue. The data in Fig 1.12, and data from other tests, indicate that K_N can be formulated:

$$K_N=A\cdot N/(B+N) \quad (1.18)$$

Where A and B are constants, and N is the number of freeze-thaw cycles. This equation is shown in Fig 1.13, together with experimental data from Fig 1.12. According to his figure, A=1,18 and B=4,1.

Moisture exchange with the environment during freeze/thaw:
When the concrete is allowed to take up, or lose, water during freeze-thaw, anything might happen, depending on the type of moisture exchange occurring during freeze-thaw. If the concrete gradually loses water, the rate, by which of frost damage increases, will gradually reduce; if the concrete gains water, the frost damage will continue with constant rate, or the rate will gradually increase.

In Fig 1.14 examples of the evolution of frost damage in an unisolated very moist freeze/thaw test is shown[19]. It takes some cycles to obtain the first frost damage. Then, the increase in damage is more or less linear. The following interpretation of the figure can be made:

$$N\leq N_{CR}: \quad S\leq S_{CR} \quad D=0 \quad (1.19a)$$
$$N>N_{CR}: \quad S>S_{CR} \quad D\approx K\cdot N \quad (1.19b)$$

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[17] The test method is described in:
G. Fagerlund: *The critical degree of saturation method of assessing the freeze/thaw resistance of concrete.* Materials and Structures, Vol 10, No 58, 1977.

[18] L Rombén: The Swedish Cement and Concrete Research Institute. Report of Activities, 1972/73.

[19] S-G Bergström: *Freezing tests with cement mortar.* Swedish Cement and Concrete Research Institute. Betong Nr 2, 1955.

Thus, it takes a certaion time, or a certain number of freeze-thaw cycles, N_{CR}, to fill the concrete to the level of the critical degree of saturation. When this has happened, each new cycle increases, both the degree of saturation, and the degree of damage.

The effect of water transfer during freeze-thaw is further considered in connection with methods of extrapolating the future destruction rate; see Task 4 below.

5.6 The rate of freezing

The freezing rate is of minor importance for the critical moisture content. This can be shown both theoretically (*Deliverable 24.2*) and experimentally. Examples from a cooperative international test is shown in Fig 1.15[20]. There is a slight decrease in the critical moisture content as the freezing rate goes up, but the effect is marginal. It must be remebered, however, that the variation in the rate of temperature lowering in nature is very low. The maximum rate of temperature lowering is about 3°C/h, and a normal value is about 1,5 °C/h.

It must be considered, however, that also a small increase in the critical degree of saturation might cause a rather big reduction in the potential service life as defined by Eq (1.16). One theoretical numerical calculation is performed in *Deliverable 24.2*. The freezing rate is supposed to be increased by a factor 2. Then, theoretically the critical degree of saturation is reduced from 0,860 to 0,813, and the potential service life is reduced from 122 days to 63 days. According to this example, *the potential service life is almost inversely proportional to the freezing rate*.

In open freeze-thaw, where the concrete can absorb or lose water, the characteristics of the freeze-thaw procedure will be of great importance, also the freezing rate. This is easlily understood by looking at Fig 1.10 and 1.14. If the freeze-thaw procedure is such, that water is gradualy absorbed during the cycle, the duration of the cycle will be very important. Therefore, it might even be, that a slow freezing is much more harmful than a more rapid, simply because the concrete has greater possibility to absorb water during the freeze-thaw cycle. This was shown in *Deliverable 24.2* by investigating the results of a cooperative test of the previously used four ASTM-methods for freeze-thaw testing concrete[21]. It was found, that the test, which gave the biggest possibilities for the concrete to absorb water was the most harmful, irrespectively of the rate of freezing. This finding can, of course, be explained by the fact, that critical water contents exist, and that they are almost independent of outer conditions, such as freezing rate or number of cycles. The only important factor in a freeze-thaw cycle is the time it allows the specimen to reach the critical level. This time has little to do with the freezing rate. Other characteristics of the freeze-thaw cycle are much more important.

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[20]) G Fagerlund: *The international cooperative test of the critical degree of saturation method of assessing the freeze/thaw resistance of concrete*. Materials and Structures Vol No. 10, No. 58, 1977.

[21]) Anonymous: *Report on cooperative freezing-and thawing tests of concrete*. Highway Research Board, Special Report 47, Washington DC, 1959.

5.7 The minimum freezing temperature

Ordinary freeze-thaw without salt:
The minimum freezing temperature is of very big importance. The main reason is that the minimum temperature determines the amount of ice formed. Examples of freezable water are shown in Fig 1.16[22] . The effect is more pronounced for concrete that is never-dried than for pre-dried and re-saturated concrete.

As more and more ice is formed, the permeability of the pore system is reduced. Therefore, the internal pressure caused by further ice formation can be very high, despite the fact, that the rate of ice formation might be low. In *Deliverable 24.2* an attempt is made to estimate the effect of previous ice formation on the pressure excerted by the new ice formed at low temperature. So far, the theory is not supported by experimental data, however.

Salt scaling:
Studies were made of the effect of the minimum temperature on the salt scaling. The results are presented in Fig 1.17-1.19. Three different minimum temperatures were used; -7°C, -14°C, -22°C. The scaling as function of temperature can be described by the following simplified equation:

$$S = \text{Const}\cdot|\theta_{min}|^2 \qquad (1.20)$$

This means that a minimum temperature of -5°C only causes about 6% of the scaling, that is caused at -20°C.

5.8 Effect of salt

The presence of salt in connection with freeze-thaw is of extraordinarily big importance. Normally, salts are only present a the surface, in the form of deicing salts or sea water. In special cases, the internal stucture of the concrete is "contaminated" by salt, while the outer surface is free from salt. The effect is different in the two cases. In the first case, salt scaling occurs. This seems to be maximum at a certain "pessimum" salt concentration. In the other case, only internal damage appears. Also in this case, there are some indications, however, of the existence of a certain pessimum salt concentration.

The mechnism behind salt scaling is not properly understood. Work financed from other sources is now going on at Lund Institute of Technology.

Salt at the surface. Salt scaling
Experiments were made with concrete specimens (w/c-ratio 0,40, about 4% entrained air) that had been stored for a very long time in, either pure water (0% salt), or 3% NaCl-solution, or 6% NaCl-solution. Then, they were freeze-thaw tested in, either pure water, or 3%, or 6% NaCl-solution. Thus, 9 different combinations of outer and inner salt concentration were used. The minimum freezing temperature was -7°C, -14°C, and -22°C. Two freezing rates were used; "rapid" and "slow". The freeze-thaw cycles are shown in Fig 1.20.

The results of the scaling tests are shown in Fig 1.17-1.19[23] . It was found that 3% salt solution was, by far, the most dangerous concentration. Pure water, and 6% salt solution, in most cases gave small scaling.

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[22]) J. Vuorinen: *On determination of effective degree of saturation of concrete.* Concrete and Soil Laboratory. Imatran Voima OY, Manuscript, Oulo, Finland, 1973.

[23]) S Lindmark: *Influence of testing conditions on salt scaling resistance of concrete.* Div. of Building Materials, Lund Institute of Technology, Manuscript October 1995.

The observations made are strengthened by other tests, that have been performed in Norway, USA and Germany during the last years.

Salt inside the pore structure. Internal damage:
Some tests were made with airentrained cement mortar specimens, that had been stored in salt solution (NaCl) for a very long time. The ingress of salt solution was aided by exposing the specimens to vacuum of a certain residual pressure. Thus, very high degrees of saturation were obtained.

The salt concentrations were 0%, 2,5%, 5%, 10%. The specimens were sealed in plastic, and were exposed to freezing down to -30°C. The length change was measured. The dilation from the ordinary thermal contraction is a measure of the damage caused.

Some results for one type of mortar are shown in Fig 1.21. Three degrees of saturation were used. The 2,5% solution always caused the biggest expansion. The same results were obtained for all mortars and for all degrees of saturation.

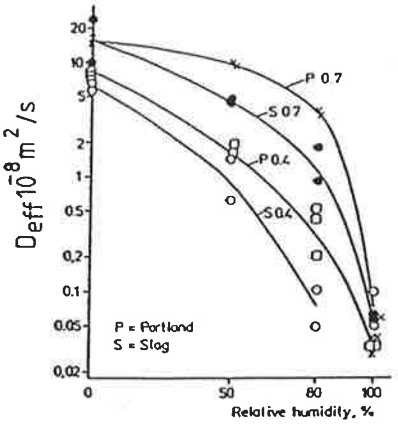

Fig 1.1: Example of the effect of RH on the diffusivity of oxygen through portland cement concrete (P), and slag cement concrete (S). *Tuutti, 1982.*

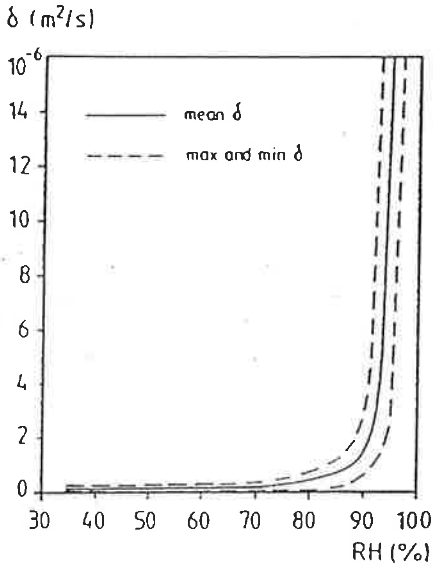

Fig 1.2: Example of experimental determinations of the moisture transport coefficient in the hygroscopic range. Moisture content as the driving potential. (w/c-ratio 0,80.) *Hedenblad, 1993.*

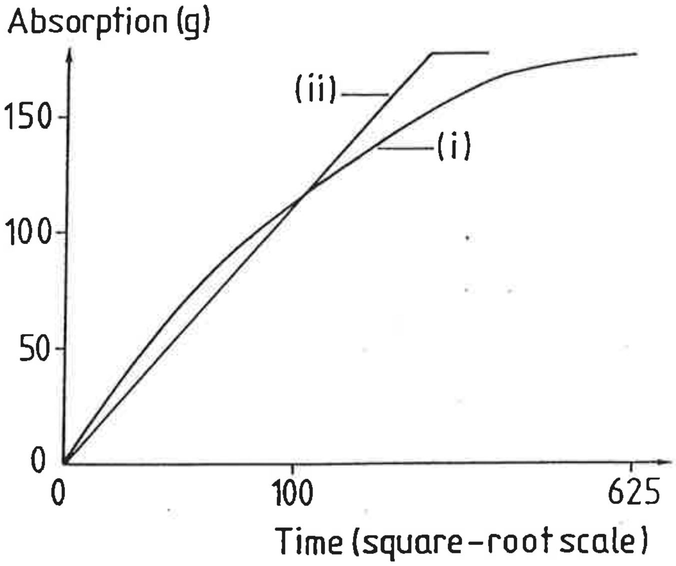

Fig 1.3: Result of a capillary absorption test together with a calculation based on; (i) measured moisture transport coefficients in the "hygroscopic" range; (ii) estimated moisture transport coefficients in the "capillary" range. *Deliverable 12.1.*

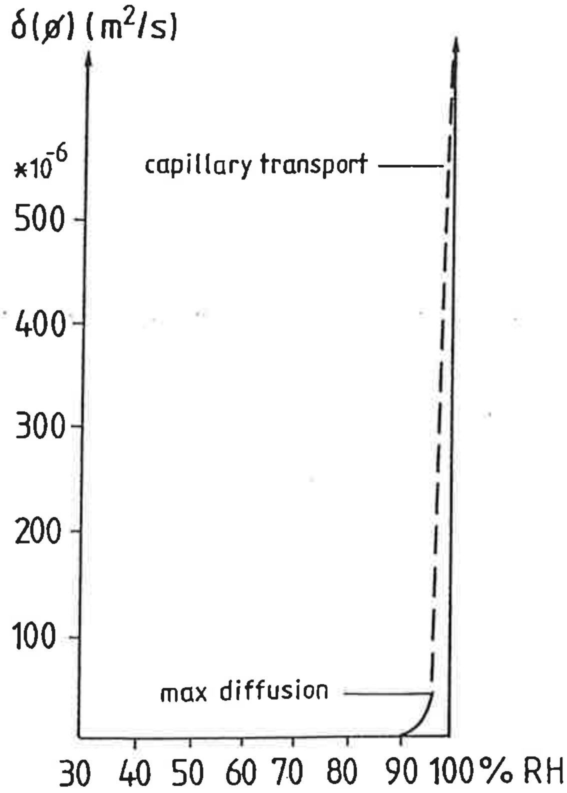

Fig 1.4: The moisture transport coefficient over the entire RH-range 30% to 100%. The data for capillary transport are taken from the experiment shown in Fig 1.3. (Concrete with w/c-ratio 0,60). *Deliverable 12.1.*

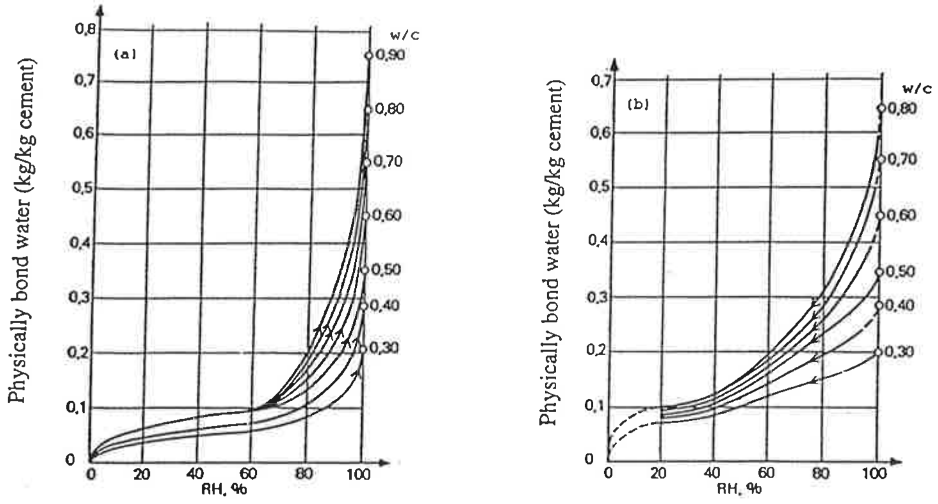

Fig 1.5: Moisture equilibrium curves for concrete. The curves are used in calculation of the relation between the outer and inner moisture conditions. *Nilsson, 1980.*

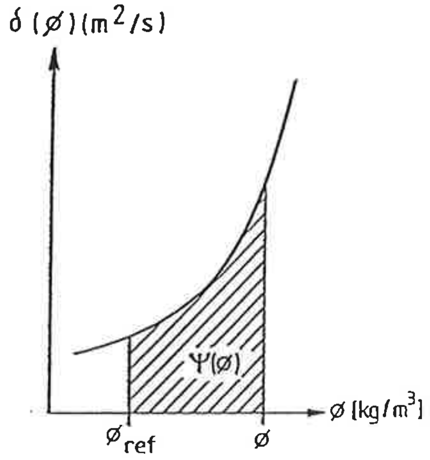

Fig 1.6: Definition of the flow potential described by Eq (1.4). *Arfwidsson, 1993.*

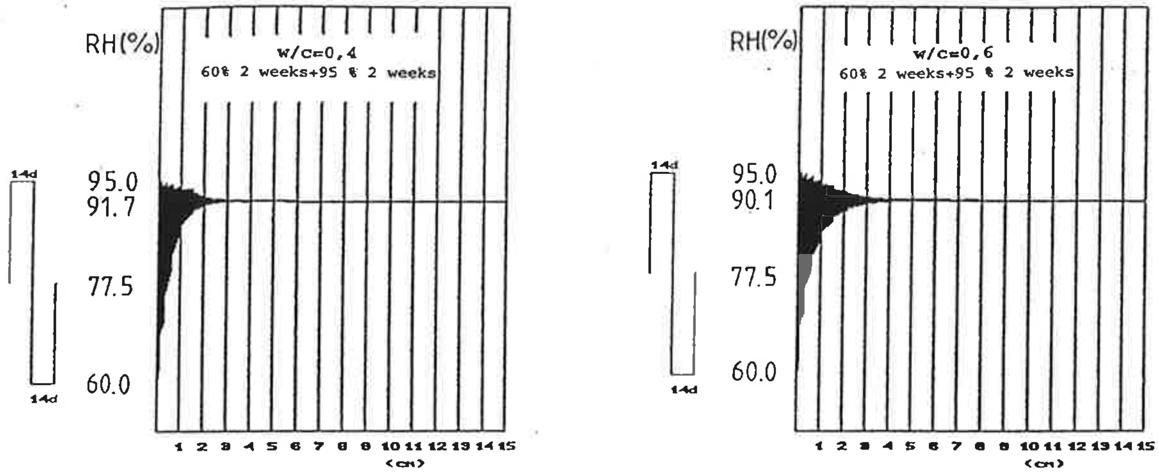

Fig 1.7: Examples of calculated moisture variations in the surface part of concrete exposed to an RH-variation between 60% and 95%. Left: w/c=0,40. Right: w/c-ratio 0.60. *Deliverable 12.1.*

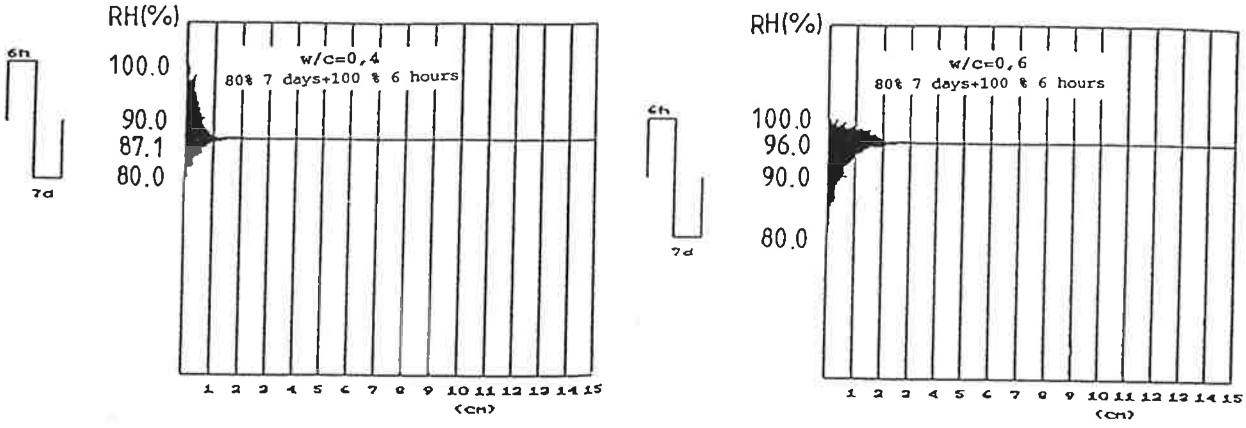

Fig 1.8: Examples of calculated moisture variations in the surface part of concrete exposed to short periods of rain, mixed with longer periods of exposure to 80% RH. Left: w/c=0,40. Right: w/c-ratio 0.60. *Deliverable 12.1.*

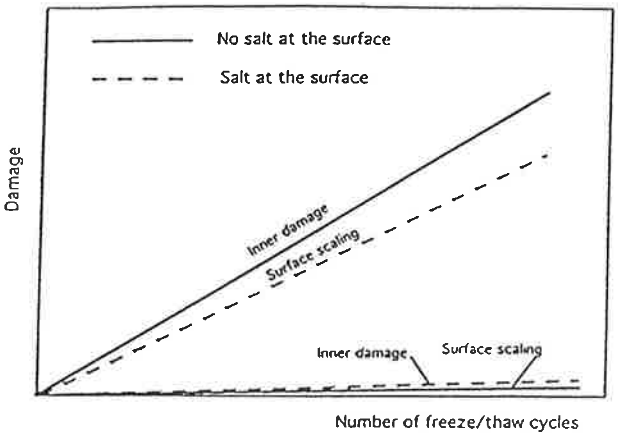

Fig 1.9: The two main types of frost damage; principles. *Deliverable 24.2.*

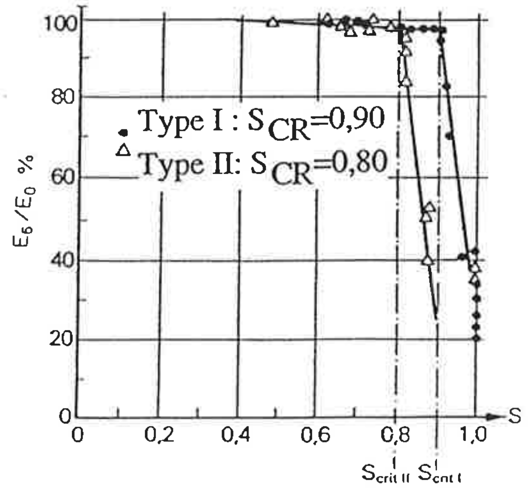

Fig 1.10: Example of a determination of the critical degree of saturation of two concretes; Type I, without airentrainment; Type II, airentrained. *Deliverable 24.2.*

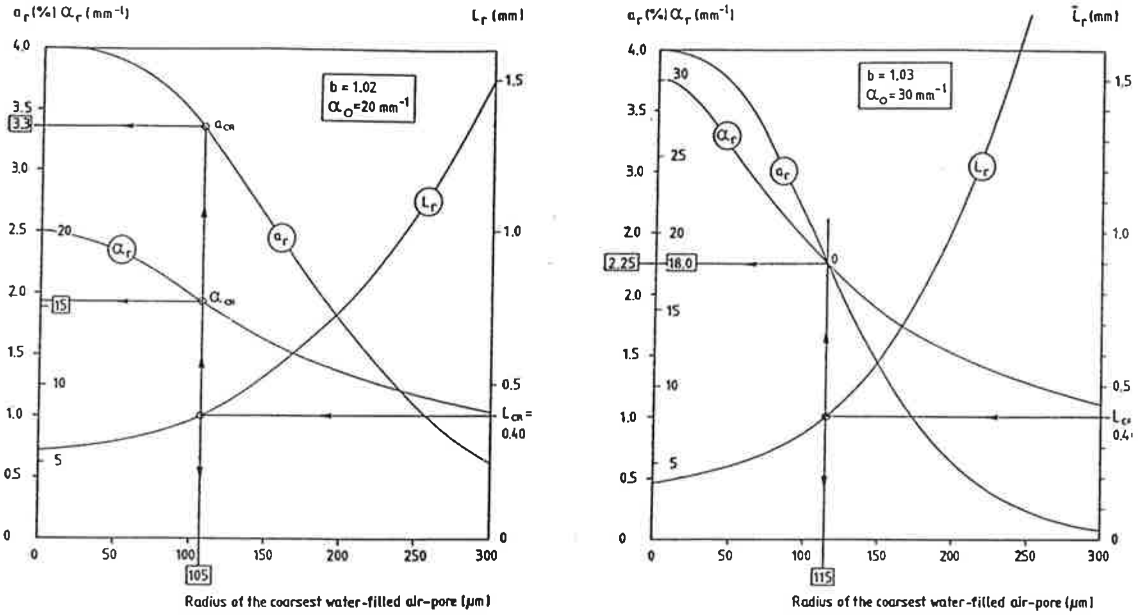

Fig 1.11: Example of the calculation of the critical degree of saturation.
Left: Coarse-porous air-pore system. Right: Fine-porous air-pore system.
Deliverable 24.2.

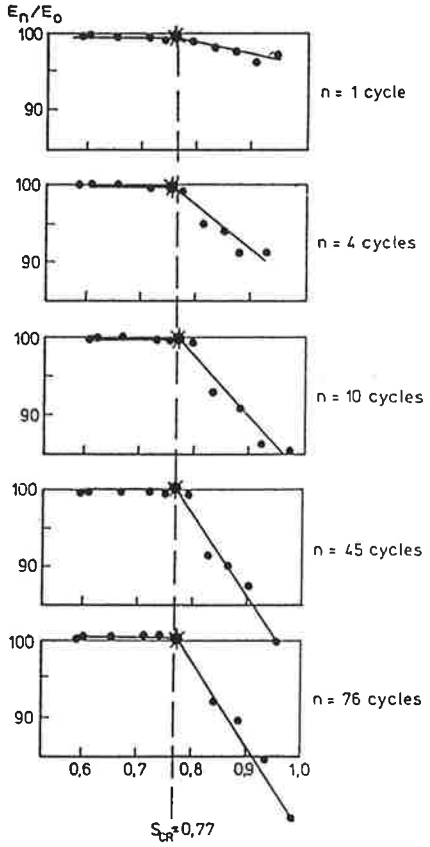

Fig 1.12: Example of the effect of the number of freeze-thaw cycles on the dynamic E-modulus. Cement mortar, w/c-ratio 0,40. *Rombén, 1973.*

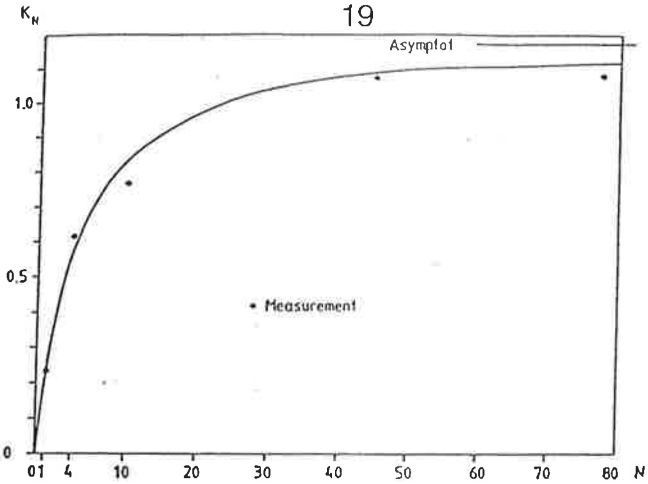

Fig 1.13: The coefficient K_N defining the the degree of damage as function of the number of freeze-thaw cycles. Constant degree of saturation ($S>S_{CR}$). *Deliverable 24.2.*

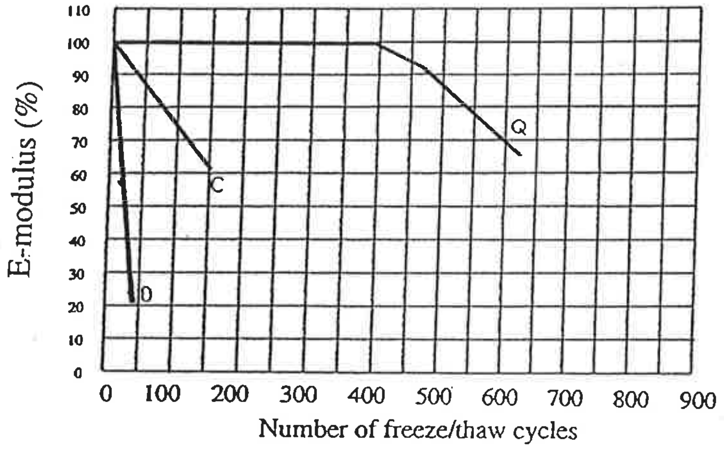

Fig 1.14: Example of the evolution of frost damage in a test where the specimen is free to absorb water during the test. Three different cement mortars. *Bergström, 1955.*

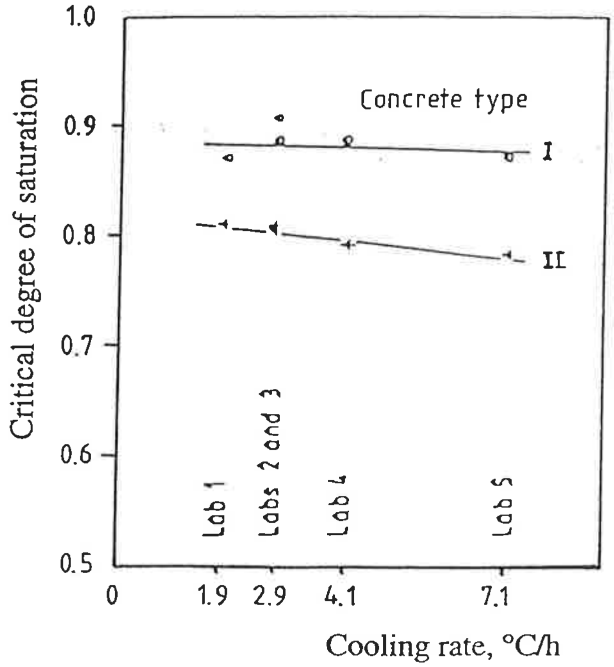

Fig 1.15:The effect of the freezing rate on the critical degree of saturation of two types of concrete. *Fagerlund, 1977.*

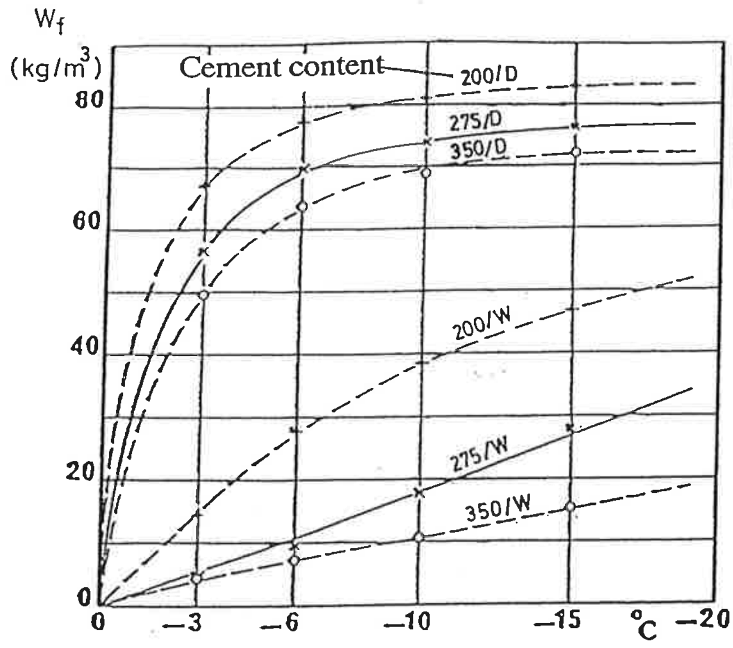

Fig 1.16: Relation between the freezing temperature and the amount of water forzen (W: never-dried specimens. D: pre-dried and re-wetted specimens). *Vuorinen, 1973.*

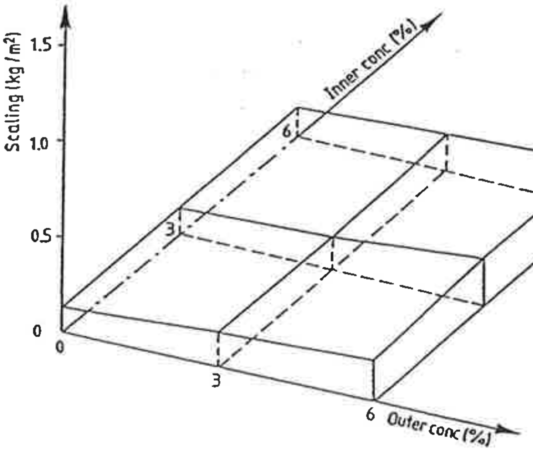

Fig 1.17: Salt scaling after 56 cycles as function of the outer and inner salt concentrations. Minimum freezing temperature -7°C. *Lindmark, 1995.*

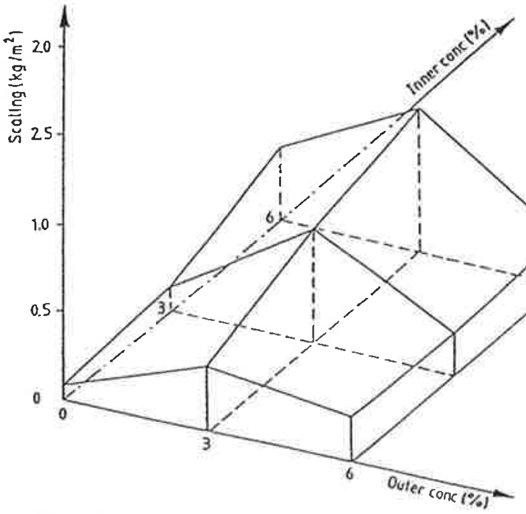

Fig 1.18: Salt scaling after 56 cycles as function of the outer and inner salt concentrations. Minimum freezing temperature -14°C. *Lindmark, 1995.*

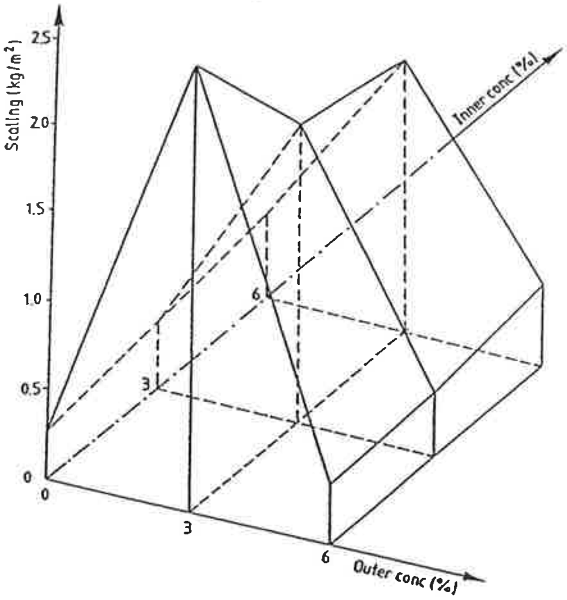

Fig 1.19: Salt scaling after 56 cycles as function of the outer and inner salt concentrations. Minimum freezing temperature -22°C. *Lindmark, 1995.*

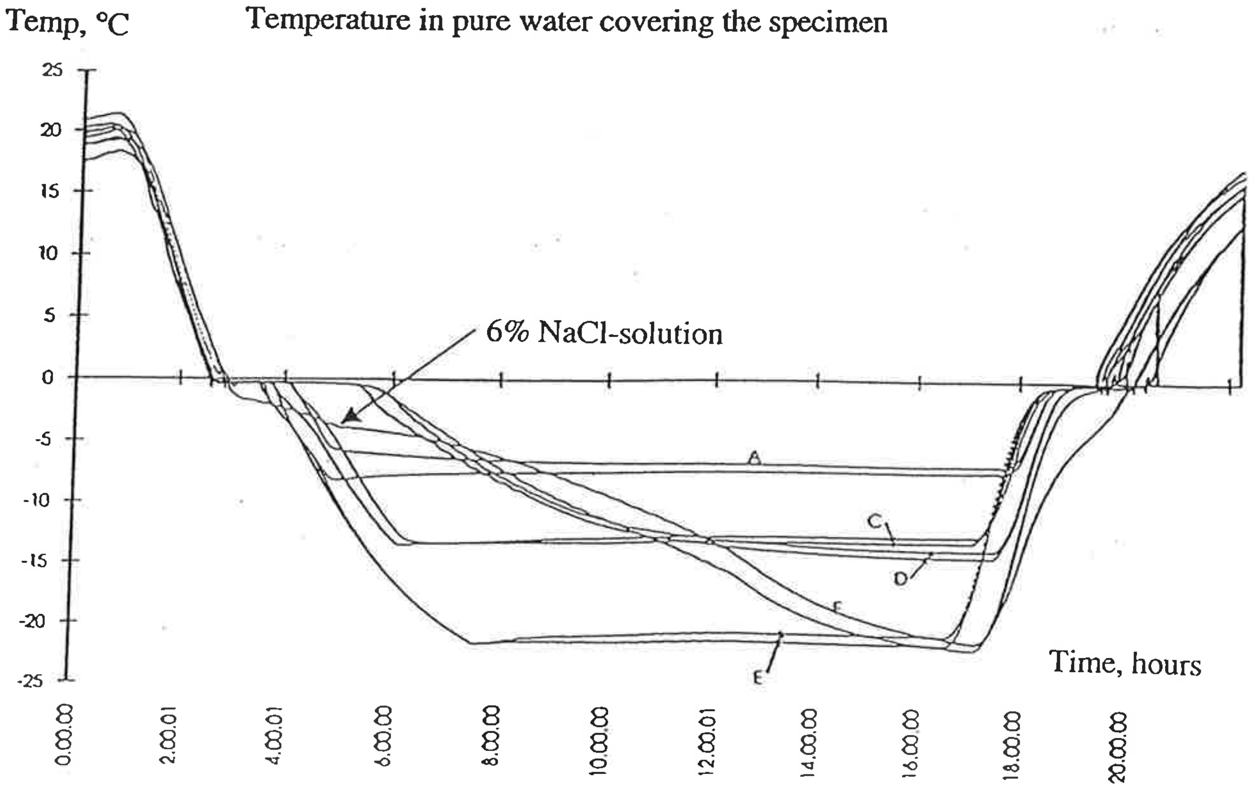

Fig 1.20: The freeze-thaw cycles used in the salt scaling tests in Fig 1.17-1.19. *Lindmark, 1995.*

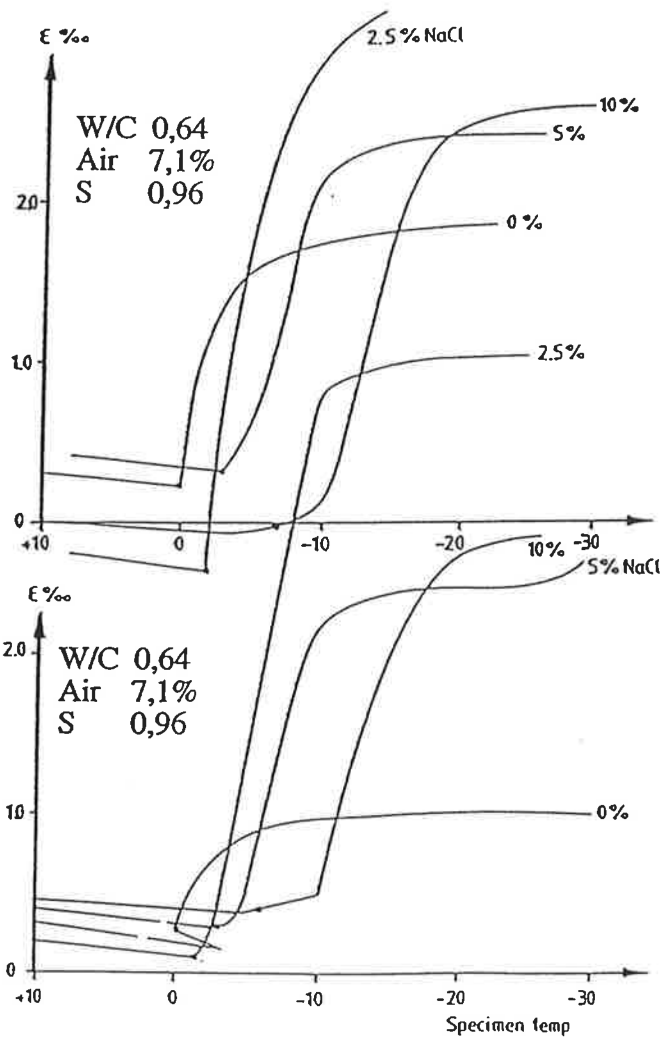

Fig 1.21: Length change-temperature curves of airentrained cement mortars containing different concentrations of NaCl-solution in the pore system, and having two different degrees of saturation. *Deliverable 24.2.*

Task 2
Assessment of current state of materials in the member
(Deliverable 30.1)

Only test methods that are directly related to frost damage, and which give valuable input to an assessment of the future destruction due to frost, are treated in *Deliverable 30.1*.

General methods for estimating the strength level of the concrete, etc. are not treated.

In the following, the different tests are merely listed. Reference is made to *Deliverable 30.1* in which the test methods are described in somewhat greater detail.

1 In-situ investigation of the structure

1.1 The mechanical properties of the concrete [24]

Determination of the actual strength and stiffness of the damaged structure, using traditional test methods.

1.2 The visual appearance-scaling and cracking[25]

Visual observations of scaling depth and profile. Examination of cores and thin sections for analysis of the cause of cracking (frost, ASR, secondary cement reaction)

1.3 The air-pore structure[26]

Traditional methods for analyzing the total air-pore volume, the specific surface, and the air-pore size distribution. "Manual" and automatic methods based on image analysis.

1.4 The effective air-pore volume[27]

A short-term water absorption test is used.

1.5 The water absorption characteristics[28]

Short-term and long-term water absorption tests are used.

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[24]) Used for analyses of the actual structural stability.

[25]) Used for determination of the cause of damage, and the extent of scaling.

[26]) Used for a determination of the general "quality of the air-pore structure and for estimation of the critical moisture content.

[27]) Used for a determination of the amount of pores, that stay air-filled also during very moist conditions.

[28]) The short-term test gives information of the rate of water absorption; i.e. the risk of frost damage of concrete exposed to liquid water only for short periods. The long-term absorption, combined with the critical degree of saturation , can be used for an estimation of the potential service life of concrete exposed to to liquid water for long periods.

1.6 The potential frost resistance[29]

Two types of test can be used:

1: Traditional freeze-thaw tests giving qualitative information.

2: The critical degree of saturation method giving quantitative information of the potential service life

1.7 The potential salt scaling reistance[30]

A traditional test method is used[31] . The minimum temperature can be adjusted to the actual environmental conditions.

1.8 The free moisture content[32]

Weighing of cores before and after complete drying. Determination of the actual degree of saturation.

1.9 The relative humidity[33]

Different test methods are described in *Deliverable 30.1*. The test can either be performed in-situ, in drilled holes, or in the laboratory, on specimens taken out from the structure at different depths. Calibration of the gauges is essential. The effect of temperature at the site, and during the test, must be considered. Calibration diagram for the temperature effect is furnished in the Deliverable.

2 Characterization of the micro- and macro climate

Some recommendations for the analysis are given.

3 Estimation of the initial and expected quality, had no deterioration taken place

Some recommendations for the analysis are given.

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(29) Gives information of the general frost resistance level. The quantitative method can also be used for a prediction of the future deterioration rate.

(30) The method gives information of the general level of salt scaling resistance. It must be used for estimating the effect of previous, or future, changes in the "salt load"; e.g. changes in the use of deicing salts.

(31) e.g. The Swedish so-called *"Borås Method"*, Swedish Standard SS 13 72 44, or the proposed RILEM method *"Draft recommendation for test method for the freeze-thaw resistance of concrete. Tests with water (CF), or with sodium chloride solution (CDF)"*. Materials and Structures, Vol 28, 1995.

(32) The actual degree of saturation, compared with the critical degree of saturation, gives a very good indication of the risk of frost damage. By comparing S_{ACT} with the degree of absorption, which is reached during a short-term absorption test, further important information of the risk of frost damage is gained

(33) Information of the relative humidity, and its variations, is fundamental for estimating the risk of reinforcement corrosion and ASR. It is of minor importance for frost attack.

Task 3
Structural performance
(Deliverable 28.1)

1 Introduction

Frost damage might affect the bond between reinforcement and concrete in two ways:

1: *Internal frost damage* causes a reduction in the tensile strength, the compressive strength, and the stiffness of the concrete matrix. This will, of course, negatively influence the bond to the reinfocement bar, and the anchoring capacity.

2: Salt scaling of the surface will reduce the effective concrete cover, and will, therefore, directly affect the anchoring capacity. A double effect is obtained when the concrete itself is also negatively affected by frost.

The effect of frost damage on bond is probably to a great extent depending on the amount of stirrups.

In the actual study, presented in *Deliverable 28.1*, only the effect of internal frost damage on bond strength was investigated. The effect of salt scaling can be more easily handled by traditional calculation methods.

No attempt was made to relate the findings, as regards loss in bond, to the structural stability of a given structure. That part of the problem is treated in Task 5 and 6 (and in the Manual).

2 Test principles

Concrete cylinders, with a centrical, or an excentrical longitudinal bar, were produced. The concrete quality, and the dimension, and other characteristics of the bar, were varied. Dummy specimens were produced, for studies of the effect of frost on the E-modulus and the tensile and compressive strength. The E-modulus was calculated from the speed of sound in the longitudinal direction of the specimen.

Frost damage was achieved by freezing and thawing specimens that had been adjusted to very high moisture contents before the test was performed. For each type of combination of concrete and bar, four different moisture contents were used, which means that four different degrees of frost damage were induced.

The adjustment of the water content was made by exposing room-dried specimens to vacuum of different residual pressures; 760 torr (no vacuum), 50 torr, 20 torr, and 2 torr. Then, water was let into the vaccum chamber, so that the specmens were completely immersed. The specimens were stored in water during 9 days before freezing in order that the degree of saturation aimed at should be reached.The equipment for vacuum treatment is shown in Fig 3.1.

Water storage without vacuum gave no frost damage. Vacuum treated specimens normally obtained severe frost damage. Treatment at 50 torr normally gave less damage than treatment at 20 torr or 2 torr. The four damage degrees are named:

Damage degree 0:	i.e. no vacuum treatment, almost no frost damage
Damage degree 50:	vacuum treatment at 50 torr
Damage degree 20:	Ditto at 20 torr
Damage degree 2:	Ditto at 2 torr

After some days of water storage, the specimems were sealed in plastic, and frozen-and-thawed 7 times during a period of 14 days. The freeze-thaw cycle is shown in Fig 3.2. Strength tests were made before and after freeze-thaw. Bond tests were made directly after the last thawing.

The test rig for the bond test is shown schematically in Fig 3.3. Load was applied by constant rate of deformation. The slip of the bar was measured at its lower end.

3 Variables

w/c-ratio:
The following w/c-ratios were used: 0,80, 0,65, 0,50. Most tests were made with w/c=0,65.

Reinforcement (quality 400 MPa):
The following variables were used:

Type	Diameter (mm)	Bond length (mm)	Remark
ribbed	12	48 and 72	
ribbed	25	100 and 130	Most used combination
smooth	12	48 and 72	
smooth	25	100 and 130	

Normally, the bar was placed centrically. In one series, an excentrical bar was used.
In one series, stirrups were used. Their diameter was 4 mm, and their spacing 30 mm.

A drawing of the specimens is shown in Fig 3.4.

Three specimens were tested for each combination of concrete, steel and damage degree. Thus, 123 specimens in total were tested for bond, and 264 specimens for strength.

4 Results

Fracture was of three types; see Fig 3.5.

Type 1: Multi-crack, which is a brittle failure
Type 2: Single-crack, which is a quasi-brittle failure
Type 3: Crush or friction failure, which is a ductile failure

The stronger the concrete, and the more undamaged, the more brittle the failure. When the concrete was severely frost damaged, the failure became more ductile; type 2 or 3.

For smooth bars, and for concrete with stirrups, failure was always of type 3.

A comprehensive account of all data is given in *Deliverable 28.1*. So far, the data have not been subjected to any more detailed theoretical analysis. In the actual summarizing report, only some results are shown, and a few comments made.

No stirrups. Centrical bar:
Typical E-modulus/bond, compressive strength/bond and tensile strength/bond relations are shown in Fig 3.6-3.8. Normally, the reduction in bond due to a certain frost damage is more pronounced than the reduction in stiffness or strength.

All data for the bond strength of centrical ribbed 25 mm bar with bond length 100 mm are collected in Fig 3.9-3.11. The data are plotted versus the measured E-modulus and strengths. Fairly linear relations are obtained. This means, that the bond strength can be estimated from a measurement of the residual E-modulus, the residual compressive strength, or the residual tensile strength. The regression coefficients are fairly high.

This also means, that the effect of frost damage is of less absolute importance in a concrete with higher strength, than in an ordinary concrete. This is shown in Fig 3.12 where the relation between bond strength, w/c-ratio and damage degree is plotted. A concrete with the w/c-ratio 0,50, and the damage degree 2 has the same residual bond strength as an unharmed concrete with the w/c-ratio 0.80.

No stirrups,. Excentrical bar:
The bond strength was very much reduced when the bar was placed excentrically. The reduction was about the same irrespectively of the damage degree. Examples are shown in Fig 3.13.

Stirrups. Excentrical bar:
Stirrups had a very positive effect on the residual bond strength, which can be seen in Fig 3.14. Even a concrete with damage degree 2 has a somewhat higher residual bond strength than an unharmed concrete without stirrups.

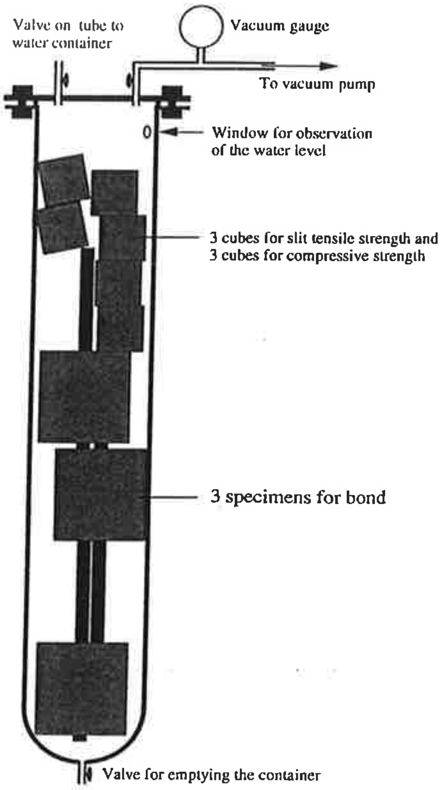

Fig 3.1: Vacuum chamber used for adjusting the specimens to different initial moisture contents. *Deliverable 30.1.*

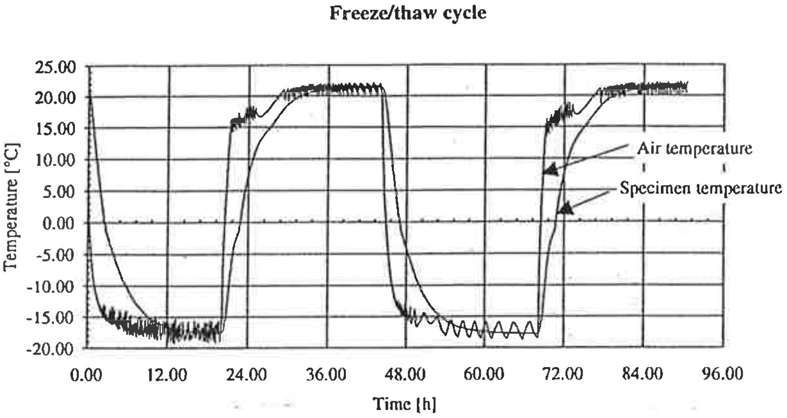

Fig 3.2: Freeze/thaw cycle measured 1 cm from the reinforcement bar inside the concrete, and in the air. *Deliverable 30.1.*

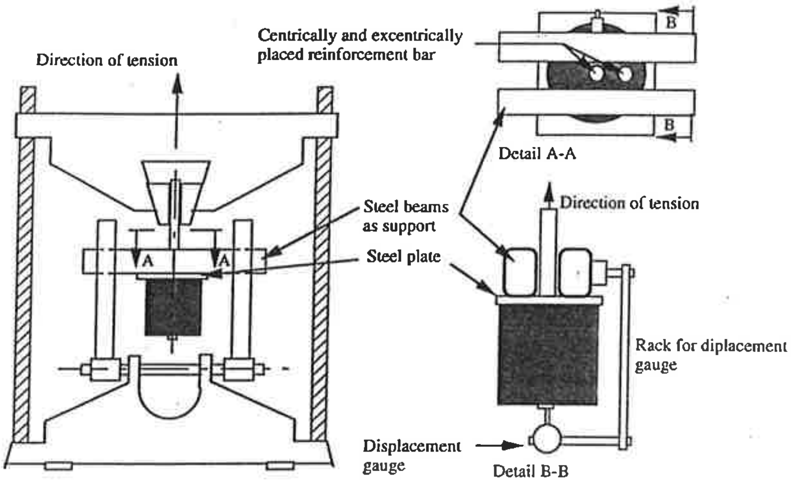

Fig 3.3: Set-up for the bond test in a 400kN hydraulic press. *Deliverable 30.1.*

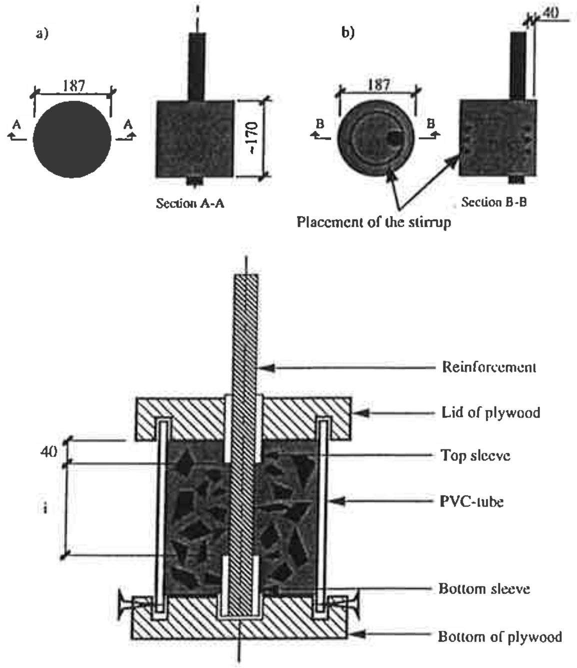

Fig 3.4 : Upper: Specimens used for the bond tests. Centrical reinforcement. Excentrical reinforcement and 3 stirrups. Lower: Mould for casting specimens used for bond tests. *Deliverable 30.1.*

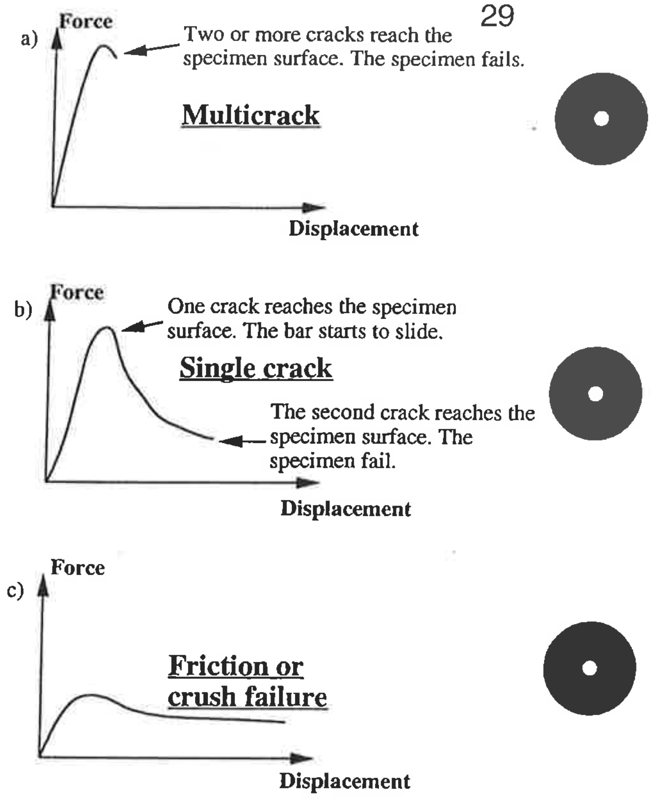

Fig 3.5: Three major types of bond fracture. *Deliverable 30.1.*

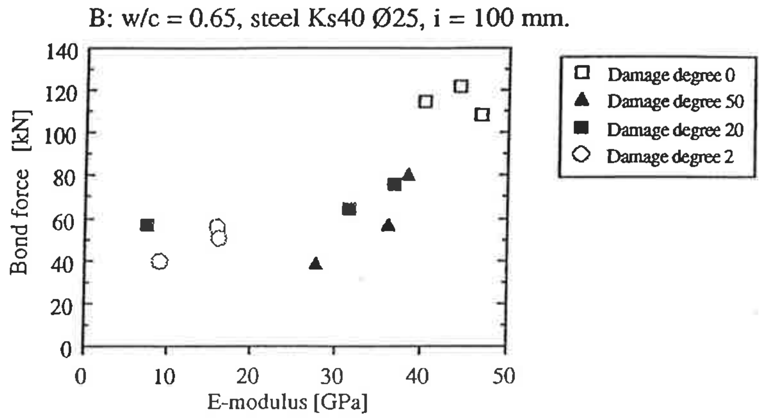

Fig 3.6: Example of the relation between the residual bond strength and the residual E-modulus. *Deliverable 30.1.*

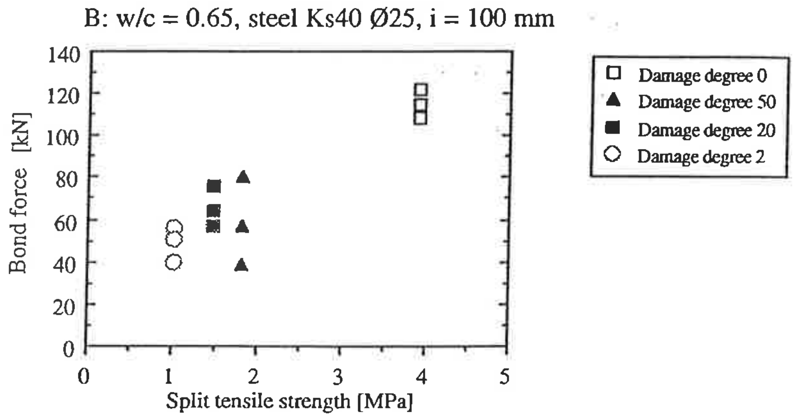

Fig 3.7: Example of the relation between the residual bond strength and the residual split tensile strength. *Deliverable 30.1.*

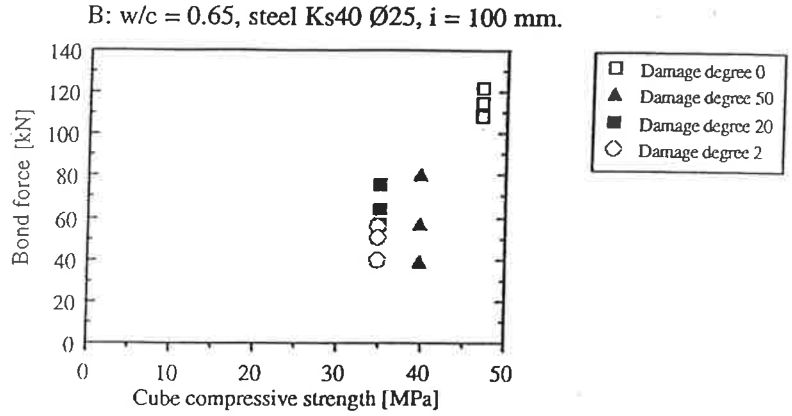

Fig 3.8: Example of the relation between the residual bond strength and the residual compressive strength. *Deliverable 30.1.*

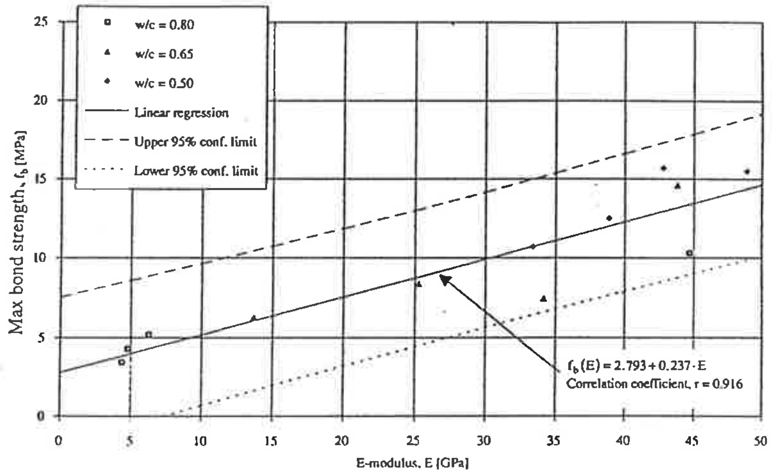

Fig 3.9: Relation between the residual E-modulus and the residual bond strength.Three different w/c-ratios, and four different damage degrees. Mean value of all results. (Φ25 mm, bond length 100 mm). *Deliverable 30.1.*

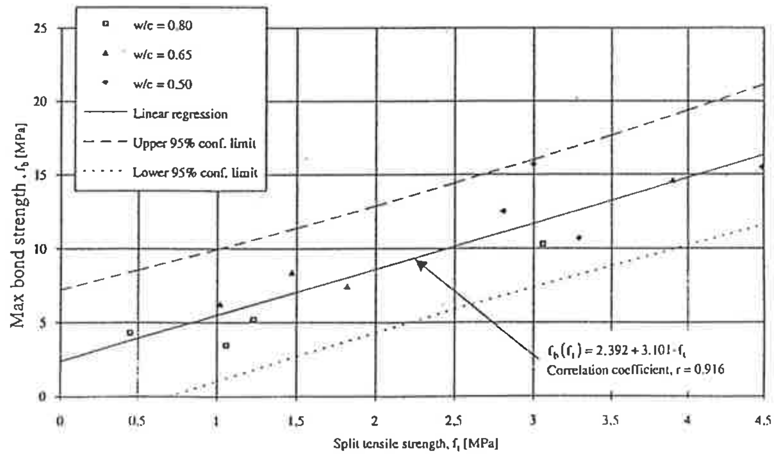

Fig 3.10: Relation between the residual split tensile strength and the residual bond strength. Three different w/c-ratios, and four different damage degrees. Mean values of all results (Φ25 mm, bond length 100 mm). *Deliverable 30.1.*

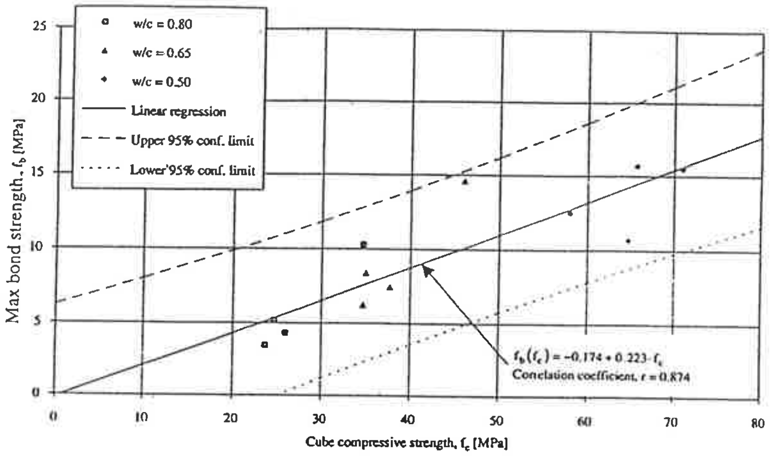

Fig 3.11: Relation between the residual compressive strength and the residual bond strength. Three different w/c-ratios, and four different damage degrees. Mean values of all results (Φ25 mm, bond length 100 mm). *Deliverable 30.1.*

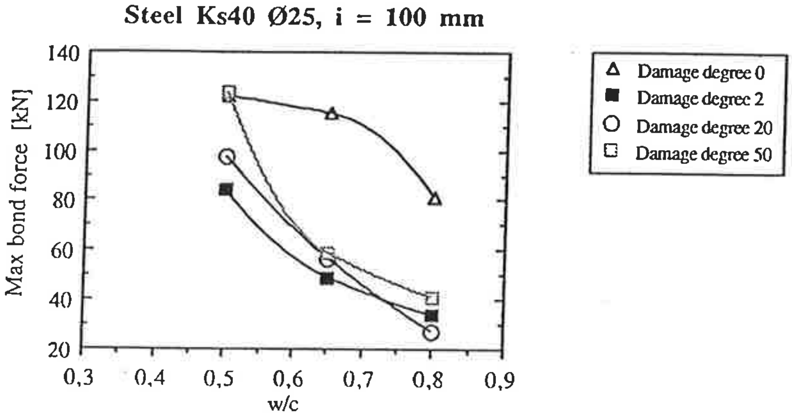

Fig 3.12: Effect of the w/c-ratio and damage degree on the residual bond strength. *Deliverable 30.1.*

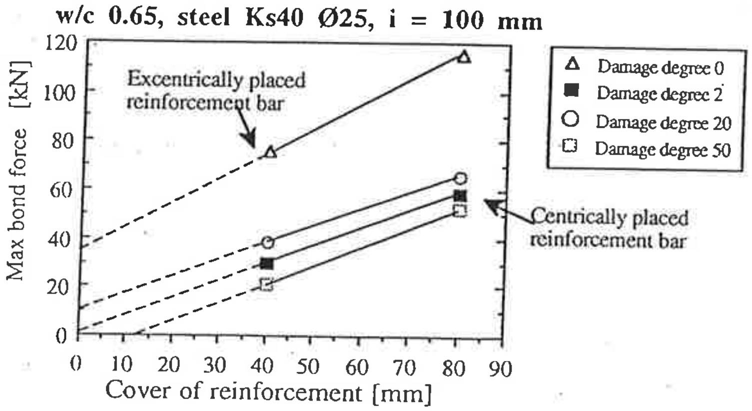

Fig 3.13: Influence of the concrete cover on the bond strength. *Deliverable 30.1.*

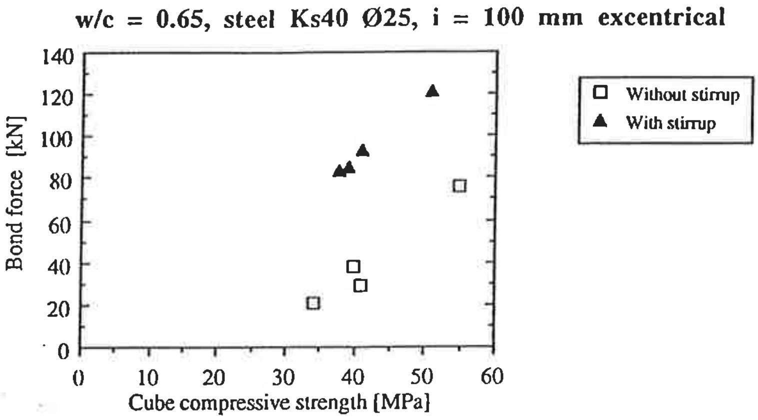

Fig 3.14: Influence of stirrups on the bond strength. Four different damage degrees. *Deliverable 30.1.*

Task 4
Deterioration rate. The future deterioration
(Deliverables 12.5, 18.6, 30.4 Parts 1 2 and 3)

1. Introduction

Under this task, four different subtasks have been dealt with:

1: Discussion of frost destruction mechanisms; mainly *Deliverable 18.6*.

2: Survey of the influence of different technological factors on the frost resistance; *Deliverable 30.4 Part 1 and Part 2*.

3: Derivation of a method for predicting the future service life with regard to frost attack; *Deliverables 12.5, and 30.4 Part 3*.

The information in the five Deliverables is comprehensive. Therefore, only a short account of the contents can be made in the present summarizing report. For a more complete information of the different ideas put forward, refer is made to the five Deliverables.

4.2 Destruction mechanisms. The critical spacing factor (*Deliverable 18.6*)

In *Deliverable 18.6*, a review is made of the different destruction mechanisms. Four mechanisms are identified:

1: Freezing in a closed container
2: The hydraulic pressure
3: The miocroscopic ice lens segregation
4: The macroscopic ice lens segregation

Mechanism 1 is only valid for special cases, such as for freezing of water in cracks, or for freezing of water in isolated porous aggregate particles, or for freezing of a high strength concrete.

Mechanism 4 is only valid for concrete of very low quality, exposed to large amounts of liquid water, such as porous concrete in a water reservoir, or in a dam.

Thus, it is only mechanisms 2 and 3, that are of practical significance. Mechanism 2 is more pronounced for normal concrete, and mechanism 3 more pronounced for high quality concrete with low w/c-ratio.

It is proven theoretically, that both mechanisms 2 and 3 lead to the existence of a so-called critical size, that can be defined as the size of the biggest piece of a completely water saturated concrete that can stand frost in a completely water saturated condition, without obtaining frost damage. The value of the critical size depends on its geometry. The traditional way of defining a size, or a distance inside a concrete, is by using the so-called *Powers spacing factor*. This is the longest distance water has to be expelled from the feezing site to the nearest air-filled pore. In defining the Powers spacing factor one assumes that all air-filled pores are of equal size and placed in a cubic array; Fig 4.1[34].

The Powers spacing factor depends on the amount of air-filled pores, on the specfic surface of such pores, and of the amount of cement paste. The following geometrical relation can be derived:

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[34] T C Powers: *The air requirement of frost resistant concrete*. Proc. Highway Research Board, No 29, 1949.

$$L=\{1{,}4[V_p/a+1]^{1/3}-1\}3/\alpha \qquad (4.1)$$

Where, L is the spacing factor [m], V_p is the fraction of cement paste exclusive of air-pores [m^3/m^3], a is the volume of air-filled pores [m^3/m^3], and α is the specific surface of the air-filled pore system [m^{-1}].

The critical spacing factor is a fundamental property in frost resistance. Above all, it can be used for a determination of the air content, that is required if a certain potential service life shall be reached; see *Deliverable 30.4 Part 2*.

Ordinary freezing in pure water. Inner damage:
The critical size can be determined experimentally by freezing completely water saturated pieces of cement paste, and observing the size of the fragments created. Some results of such measurements are presented in *Deliverable 18.6*. Some of the results are also presented in Fig 4.2. There, the size is expressed in terms of a critical thickness. It can be translated to a critical Powers spacing factor. A mean value for ordinary w/c-ratios, and for pre-dried and resaturated cement paste, was found to be:

$$L_{CR}\approx 0.40 \text{ mm}$$

Another possibility, that was utilized in the present work, is to calculate the critical spacing factor from an experimental determination of the critical degree of saturation. Besides, the air-pore distribution has to be known. The calculation method is exactly the same as that used for calculating the critical degree of saturation from knowledge of the critical spacing factor; See Eqs (1.8)-(1.11) above.

Some results, based on freeze-thaw tests of different types of concrete, are shown in Table 4.1. The corresponding air-pore distributions are shown in Fig 4.3.

Table 4.1: Determination of the critical degree of saturation, and the critical Powers spacing factor. Freezing in pure water. Internal damage.

Cement type	Nominal air content (%)	S_{CR}	L_{CR} (mm)
A	4.5	0.88	0.37
	6	0.85	0.27
			0.32
B	4.5	0.89	0.28
	6	0.88	0.33
			0.31
C	4.5	0.88	0.35
	6	0.89	0.33
			0.33
D	4.5	0.92	0.42
	6	0.91	0.35
			0.39
	Mean value		0.33
	Standard deviation		0.05

Also other calculations based on the same type of information indicate, that the critical value is about 0.35 to 0.40 mm[35]

Salt scaling:
The critical spacing factor for salt scaling is less well-known. It is not possible to use the same technique as for inner damage, since critical degrees of salt scaling cannot be easily determined. Therefore, one has to resort to direct observations of the size of fragments caused when a saturated cement paste is frozen in salt solution. There are only one a few measurements made. The results are shown in Fig 4.2. Translation of the critical thickness to a critical Powers spacing factor gives the value:

$$L_{CR} \approx 0,54 \text{ mm}$$

3 Effect of technological factors on the frost resistance (*Deliverable 30.4 Part 1*)

In *Deliverable 30.4, Part* 1, a comprehensive account is given on the interrelations between different technological factors, and the frost resistance. This information can be used in conjunction with an assessment of the potential frost resistance of a certain concrete. Different methods of freeze-testing a concrete are described as well.

There is no need to go into detail in this summarizing report. Only some head titles from the Deliverable are listed:

* Freezing of water in the concrete
* Effect of the cement type
* Effect of mineral admixtures
* Effect of airentraining admixtures
* Effect of water reducing admixtures
* Effect of mixing
* Effect of curing
* Test methods for frost attack
* Test methods for salt scaling
* Recommendations for a quality control system

4. Effect of the air content on the service life (*Deliverable 30.4 Part 2*)

The air content is the most important single parameter for frost resistance. The required air content depends on the wetness of the environment, on the critical spacing factor, and on the size distribution of the air-pores. The latter also determines the rate by which the air-pore system becomes inactivated due to water-filling.

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[35]) The critical spacing factor, as it is defined and determined here, shall not be confused with the critical spacing factor as it is defined traditionally; i.e. where all air-pores are included in defining the spacing, also the water-filled pores. This traditional spacing factor, which can be called a fictitious spacing factor, has no physical significance, although it is very often used in concrete technology. The difference between the two definitions is treated in detail in *Deliverable 18.6*. Many measurements of the critical *fictitious* spacing factor have been made. Numerous data are presented in *Deliverable 18.6*. There it is shown, that the critical fictitious spacing factor is about 0,25 mm for ordinary freezing in pure water (inner damage), and about 0,16 to 0,20 mm for salt scaling. The exact value depends very much on the freeze-thaw characteristics. The critical true spacing factor, on the other hand, is more constant, and independent of the outer climatic conditions. This distinction between a true spacing factor, and a fictitious, is very important in analyses of frost resistance and service life.

In *Deliverable 30.4 Part 2*, a theory is put forward for how the proper air content shall be selected. It is shown,that the required air content is lower the finer the air-pore system, provided the water absorption times are comparably short. For longer periods of exposure to free water, it is more favourable to have a more coarse-porous air-pore system. The reason is that fine air-bubbles are lost more rapidly due to water absorption, than are coarse air-bubbles. Thus, the favour of having a fine-porous air-pore system, giving a short spacing factor when the concrete is rather dry, is turned into an inconvenience when the concrete is very moist.

In Fig 4.4, an example is shown of a calculation of the required air in a cement paste for a given time of water exposure. The figure is valid for two different air diffusion permeabilities of the paste, and for a paste with a certain constant size distribution of air voids, and a certain constant critical spacing factor.

5. Water-filling of the air-pore system (*Deliverable 12.5*)

The service life of the concrete depends on the time it takes to reach the critical degree of saturation, which is a material constant. Therefore, for a service life prediction, it is of vital importance to find out how the air-pore system is inactivated when the concrete is stored in water. A comprehensive analysis of this inactivation process is performed in *Deliverable 12.5*.

The starting point of the analysis is the mutual transfer of air from a smaller air-bubble to an adjacent bigger air-bubble. The driving force is the difference in the internal over-pressure in the two bubbles; the smaller bubble having the biggest pressure. It is shown theoretically, that this process often leads to a reduction in the total air-bubble volume. Therefore, there will be a suction of water into the internal part of concrete, also in a case where there is no net diffusion of entrapped air to the surface of the specimen. Such water uptake, occurring simulaneously over large portions of the concrete volume, is also observed in water absorption experiments. Many examples are shown in *Deliverable 12.5*.

It is also shown theoretically, that the rate of diffusion in terms of mass of air is approximately independent of the size of the air-bubbles. The following equation is approximately valid:

$$q = \delta \cdot [1+1/\varepsilon] \cdot 4\pi \cdot s \cdot 2\sigma \qquad (4.2)$$

Where δ is the transport coefficient (diffusivity) of dissolved air [m^2/s], ε is the assumed constant ratio between the distance between the two pores, and the radius of the smallest bubble, s is the solubility of air in water [$\approx 2.5 \cdot 10^{-7}$ $kg/m^3 \cdot Pa$], and σ is the surface tension water-air [≈ 0.074 N/m]. Then, assuming $\varepsilon=5$ the following rate determining equation is obtained:

$$q = 5.6 \cdot 10^{-7} \cdot \delta \ \ kg/s \qquad (4.3)$$

Then, the time to water-fill an air-bubble with the radius r becomes:

$$t_{req} = (4 \cdot \pi/3) \cdot r^3 \cdot \rho_o/(5.6 \cdot 10^{-7} \cdot \delta) \qquad (4.4)$$

Where ρ_o is the density of air [$\approx 1.25\ kg/m^3$].

In a real material, there are numerous air-bubbles. Therefore, there are numerous inter-pore diffusions taking place simultaneously. In *Deliverable 12.5* , two types of air-pore systems are treated:

1: A pore system described by the *exponential* type of pore size frequency function:

$$f(r) = \ln b / b^{r} \tag{4.5}$$

2: A pore system described by the *power* type of pore size frequency function:

$$f(r)=1/r^{b} - 1/(r_{max})^{b} \tag{4.6}$$

Where b is a constant, and r_{max} is the radius of the biggest air pore.

Two models for the water absorption process have been treated:

Model 1: All pores start to absorb water immediately, and with the same rate
Model 2: A bigger pore does not start to absorb water until the next smaller pore is completely water-filled

The two models lead to different absorption-time curves. Model 2 is the most reasonable model from a thermodynamical point of view.

The following approximative time functions for the degree of saturation of the air-pore system, S_a which is defined by Eq (1.12), are derived for Model 2, and for he diffusivity $\delta=10^{-12}$ m^2/s:

Exponential function: $S_a \approx 2 \cdot 10^{-8} \cdot \alpha_o^{2,4} \cdot t^{0.5}$ (4.6)

Power function: $S_a \approx 3.5 \cdot 10^{-10} \cdot \alpha_o^{4.5} \cdot t^{C}$ (4.7)

Where α_o is the specific area of the entire air-pore system including water-filled pores, and C is a coefficient, that is to a certain extent a function of α_o. Normal values of C is 0,25 to 0.35 for a fine-porous and a coarse-porous system respectively. A plot of Eq (4.6) and (4.7) is made in Fig 4.5

Thus, a general expression for the long-term water absorption in the air-pore structure is:

$$S_a=B \cdot t^{C} \tag{4.8}$$

Where C varies between about 0,25 and 0,60.

S_a can be replaced by S if the initial water absorption in the finest pores is also considered. Then, the equation for the time process of water absorption in the concrete is:

$$S=A+B \cdot t^{C} \tag{4.9}$$

The coefficient A corresponds to the very rapid filling of all pores in the concrete except the isolated air-pores. A can, therefore, be replaced by the minimum water absorption that is occurring also after a very short exposure to water:

$$A \approx S_{min} \tag{4.10}$$

S_{min} can be easily obtained as the nick-point in a water uptake/square-root of time diagramme. A definition of

S_{min} is made in Eq (1.14).

The theoretical expressions above were tested against measurements of the long-term water absorption in concretes with known air-pore size distributions. The agreement between theory and experiment was very good. This comparison is made in *Deliverable 12.5*.

5 Prediction of the future frost destruction (*Deliverable 30.4 Part 3*)

Salt scaling:
Salt scaling is a surface erosion, increasing gradually with time. Each freeze-thaw cycle of a certain severeness gives its contribution to the scaling. As shown by Eq (1.20) above, it is only freezings to rather low temperatures that cause severe scaling. Therefore, the increase in scaling can be different from one year to the other. Seen over a longer period, the number of severe freeze-thaw cycles will, however, be about constant. It is quite clear, however, that scaling is most pronounced in a climate with many freezings down to very low temperatures. Thus, the Swedish climate is much more severe than, for example, the British or the Spanish.

Principally, and seen over a longer period, there are three types of scaling; see Fig 4.6:

* Retarded
* Linear
* Accelerated

Mesurements presented in *Deliverable 30.4 Part 1* indicate that scaling is only accelerated when the scaling per cycle exceeds about 0.02 kg/m^2 when the freezing temperature is -20°C. This is a scaling, that it so high, that it is hardly probable that a concrete having a higher scaling will survive more than a few years before it must be repaired or replaced.

Therefore, in practice, it is only retarded and linear scaling that is of interest for prediction of the future rate of destruction.

A: Prediction based on direct observation of the structure
When the concrete is homogeneous, the scaling will be more or less linear. There are some jumps in the scaling curve, as shown principally in Fig 4.7 , but the general trend is that the scaling will proceed with about the same rate, as it did before inspection. Besides, the climate will, as an average, be about the same as it was in the past, provided there is no change in the use of deicing salts. Then, the future scaling, Δs, can be predicted by the following equation; see Fig 4.8:

$$\Delta s \approx \frac{s_o}{t_o} \cdot \Delta t \qquad (4.11)$$

Where s_o is the scling that observed at the time of inspection, t_o.

If the concrete is inhomogeneous; e.g. if the surface layer is separated, or if the air content at the surface layer was reduced due to air losses during compaction, or if there is defective curing of the surface, there will in all cases be a retarded scaling. This means that Eq (4.11) will give a too pessimistic value of the future scaling. This is illustrated by Fig 4.9.

B: Prediction based on salt scaling tests
In some cases, it is difficult to define the extent of scaling from an inspection of the structure. It might also be, that salt was only used during a certain period in the past, but not during its entire previous history. It might also be, that deicing salts will not be used in the future, or

that it has not been used previously, but will be used in the future.

For all these cases, it is necessary to perform a direct salt scaling test. Methods are described in *Deliverable 30.4. Part 1*.

The temperature in the test shall be adjusted to the temperature to which the structure is exposed in nature. Thus, it shall be lower in Sweden than, for example, in the U.K. or in Spain.

In most cases, the test will lead to a scaling, that can be expressed:

$$\Delta Q = K \cdot N \qquad (4.12)$$

Where ΔQ is the total scaling [kg/m^2] after N cycles. K is a constant, that is determined by the test.

In most cases, the future scaling can be assumed to be linear (conservative assumption). Then, the future scaling, Δs, after additional time Δt years, assuming 10 ”dangerous” freezings each year, will be:

$$\Delta s = K \cdot 10 \cdot \Delta t / 2000 \qquad (4.13)$$

Where, 2000 [kg/m^3] is an assumed average density of the scaled material.

Internal frost damage:
The future evolution of internal damage will depend on the exposure of the concrete to free water. Three different exposure types can be distinguished:

Exposure type 1: Concrete not permanently exposed to water. Unchanged inner moisture content

Exposure type 2: Concrete not permanently exposed to water. Gradual increase in the inner moisture content

Exposure type 3: Concrete permanently exposed to water

The three types of exposure will lead to different types of the evolution of damage. The physical background, and the derivation of the equations for prediction, are not shown here. The basis behind the prediction equations is the following:

1: The observed effect of the inner moisture level on the degree of damage, as function of the number of freeze-thaw cycles. See Eq (1.17) and Fig 1.12 and Fig 1.13.

2: The theoretical and experimental analyses of the long-term absorption in the air-pore structure, as described by an equation of type Eq (4.8).

Exposure type 1:
This will lead to unchanged conditions in the future; i.e. the concrete is fixed in its present state of frost damage. The prediction equation, therefore, is:

$$D_t=D_o \tag{4.14a}$$

$$F_t=F_o \tag{4.14b}$$

Where D is damage, and F is strength; e.g. bond strength. The indici t and o indicate time t after inspection, and the time of inspection respectively.

This is a ”time-independent damage function”.

Exposure type 2:
Then, the degree of saturation will increase in proportion to the square-root of the number of freeze-thaw cycles. This will lead to a prediction equation of the following shape:

$$D_t=\kappa \cdot t^{1/2} \tag{4.15}$$

Where κ is a constant, that depends on the number of freeze-thaw cycles per year, and on the coefficients A and B in the fatigue function described by Eq (1.18).

This means that the additional damage, ΔD, after the additional time Δt counted from the time of inspection, will be:

$$\Delta D=\{[\Delta t/t_o+1]^{1/2}-1\}\cdot \kappa \cdot t_o^{1/2} \tag{4.16}$$

Where t_o is the age of the concrete at the inspection.

This is the ”square-root damage function”.

Exposure type 3:
Then, in the worst case, the degree of saturation will increase in direct proportion to the square-root of time. The prediction equation becomes:

$$D=\lambda \cdot t \tag{4.17}$$

Where λ is a constant, that depends on the number of freeze-thaw cycles per year, and on the coefficients A and B in the fatigue function described by Eq (1.18).

This means, that the additional damage, ΔD, after the additional time Δt, counted from the time of inspection, will be:

$$\Delta D=\lambda \cdot \Delta t \tag{4.18}$$

This is the ”linear damage function”.

The three types of damage functions are illustrated in Fig 4.10.

Prediction based on testing:
In a case where the concrete is no yet frost damaged, but might be so in the future, a prediction can be based on direct observations of the critical degree of saturation, and the water absorption characteristics. The residual service life, until frost damage occurs, can be estimated by the following expression; cf. Eq (1.16):

$$t_{residual}=\{(S_{CR}-A)/B\}^{1/C}-t_o \qquad (4.19)$$

Where, S_{CR} is determined by freeze-thaw experiments, and the coefficients A, B, and C are determined by a long-term absorption experiment; cf. Eq (4.9).

This expression might give a somewhat pessimistic estimate of the future service life. Another possibility is to base the prediction on the really observed moisture content. This information can be used for a more realistic equation for predicting the moisture absorption and service life. The following equation is used for calculating the rate determining coefficient B in the water absorption process:

$$B_{real}=(S_{o,real}-A)/t_o^{C} \qquad (4.20)$$

Where, $S_{o,real}$ is the observed degree of saturation at the time, t_o, at inspection. A is determined as the nick-point in a short-term water absorption test. A reasonable value of C is 0,5.

Then, the residual service life before frost attack starts can be estimated by:

$$t_{residual}=\{(S_{CR}-A)/B_{real}\}^{1/C} \qquad (4.21)$$

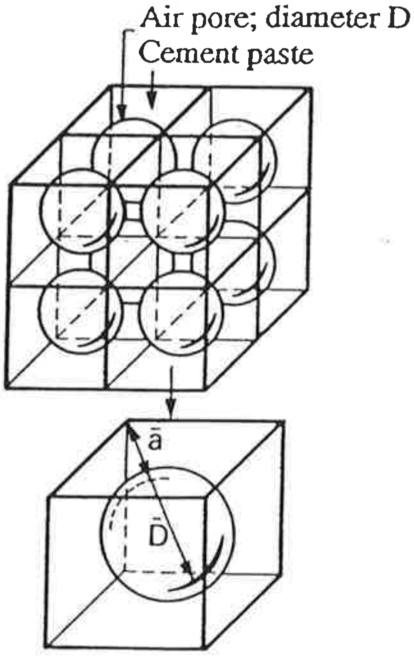

Fig 4.1: Model defining the Powers spacing factor. *Powers, 1949.*

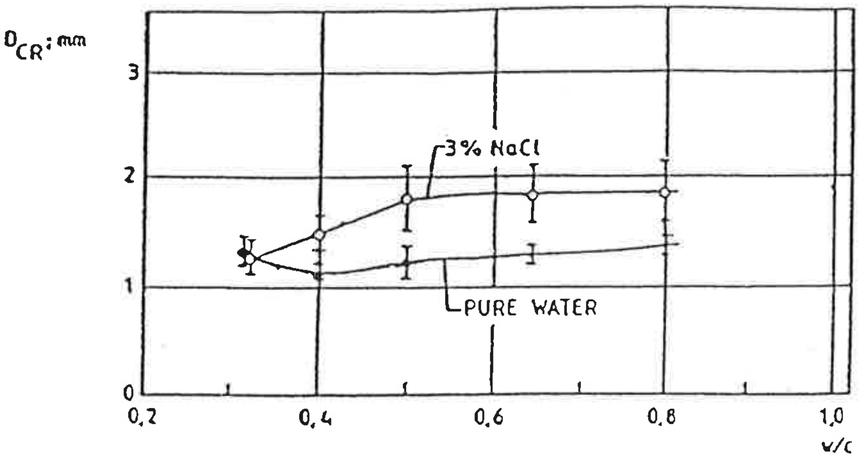

Fig 4.2: Results of a direct determination of the critical thickness through measurements of the size of fragments created at freezing completely saturated cement pastes in pure water or in 3% NaCl-solution. Pre-dried and re-saturated pastes. *Deliverable 18.6.*

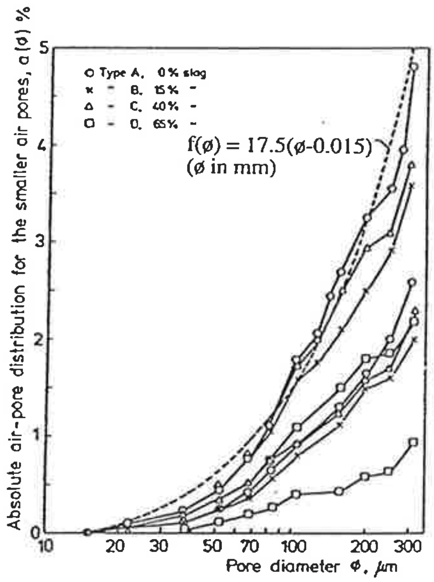

Fig 4.3: Size distribution of air-pores for the concretes in Table 4.1. The curves are determined by the Lord/Willis method using automatic image analysis. *Deliverable 18.6.*

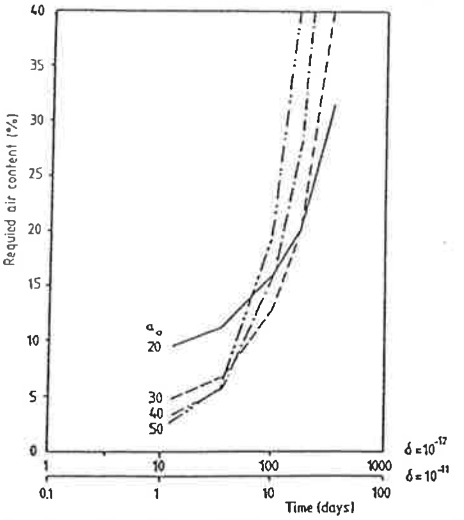

Fig 4.4: The required air content as function of the water absorption time. Exponential type of size distribution of air-pores. Two different diffusivities of dissolved air (α_0 is the specific surface of the entire air-pore system). *Deliverable 18.6.*

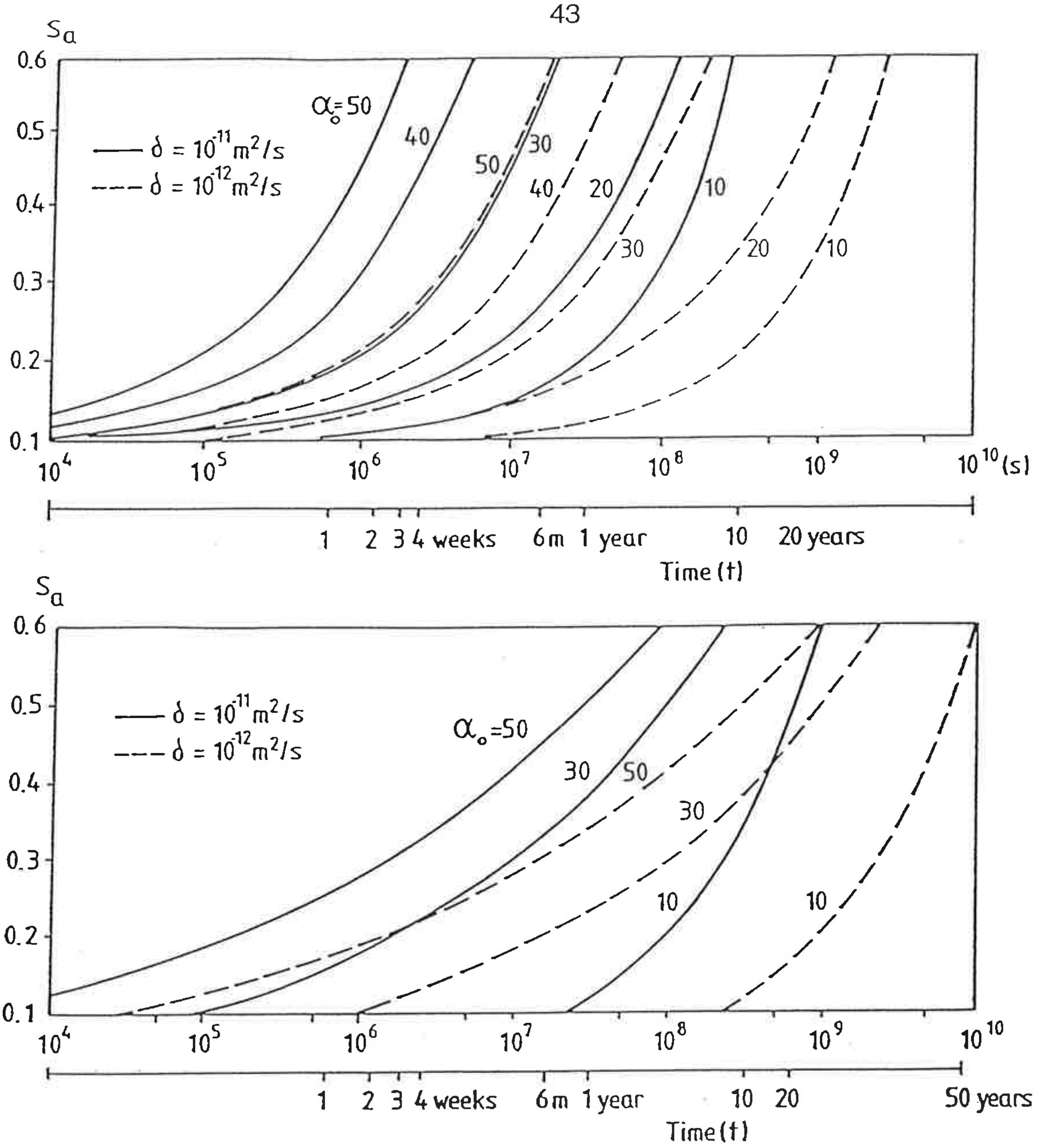

Fig 4.5: The degree of saturation as function of the water absorption time. Absorption according to Model 2. *Deliverable 12.5.*
Upper part: Exponential type of distribution. Lower part: Power type of distribution.

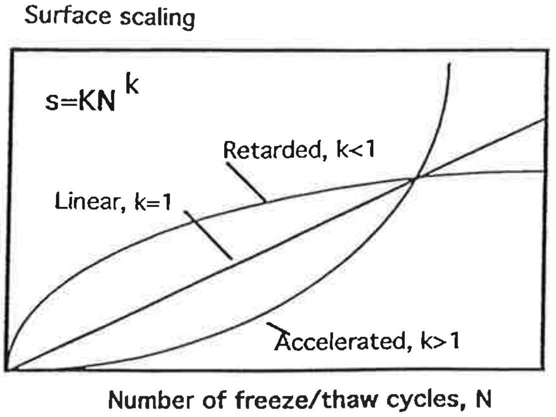

Fig 4.6: Linear, retarded and accelerated scaling in a salt scaling test; principles. *Deliverable 30.4, Part 3.*

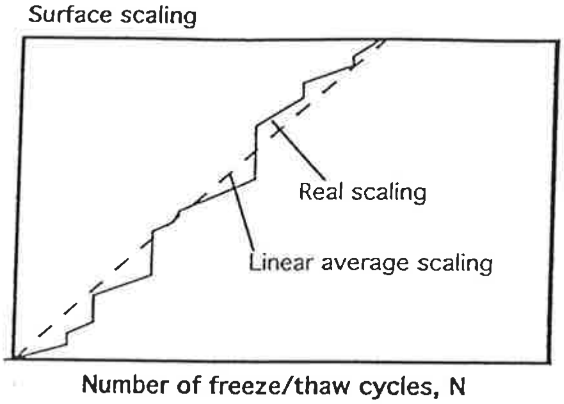

Fig 4.7: "Jumps" in a "linear" salt scaling curve. The jumps are caused by the loosening of sand grains and stones. *Deliverable 30.4, Part 3.*

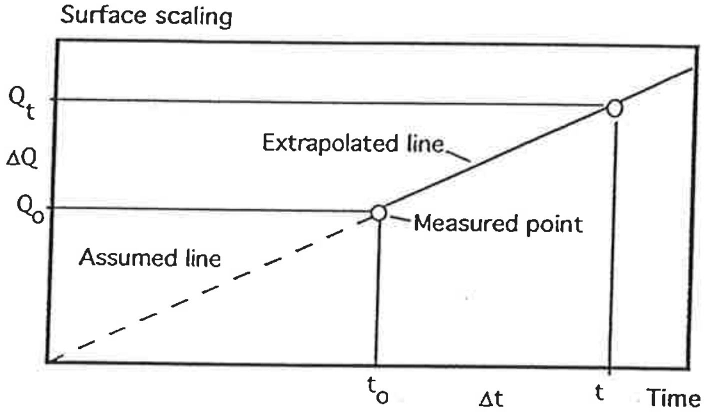

Fig 4.8: Extrapolation of the observed scaling. *Deliverable 30.4, Part 3.*

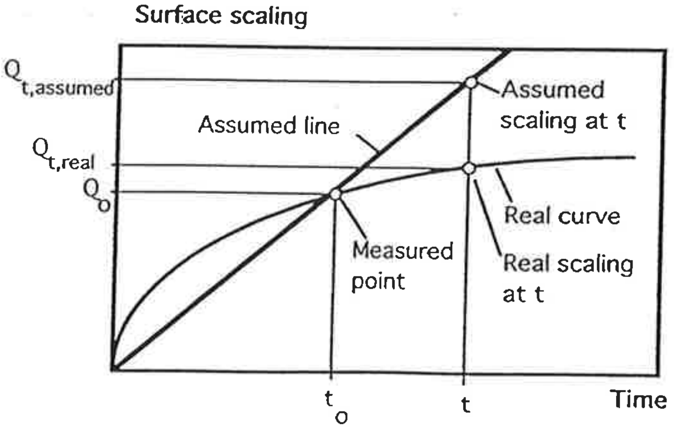

Fig 4.9: Linear extrapolation of retarded salt scaling. The retardation being caused by inhomogeneities in the surface part of the concrete. *Deliverable 30.4, Part 3.*

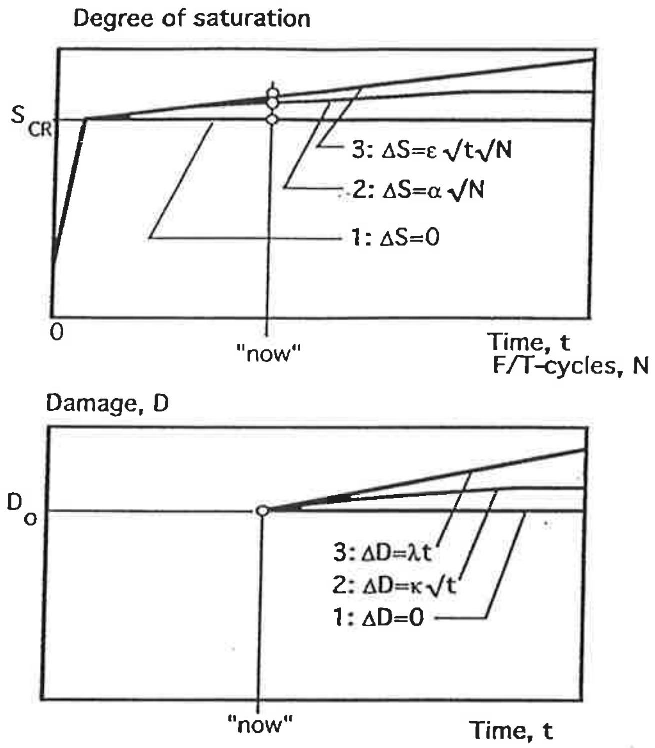

Fig 4.10: Three different possibilities of extrapolating the damage curve. Each curve is valid for a certain type of moisture exposure. *Deliverable 30.4, Part 3.*

APPENDIX

Some ideas on synergism between frost attack and other destruction types

1. Introduction

Three destruction mechanisms are treated in the project:

1: Reinforcement corrosion
2: Frost attack
3: Alkali-silica reaction (ASR)

Therefore,there are at least 6 imaginable types of synergy between indivdual types of destruction, provided only two mechanisms at a time are considered. A qualitative analysis indicates that there is in fact only 2 types of synergy that are of significant importance:

1: Effect of salt scaling on reinforcement corrosion
2: Effect of ASR on frost attack

Some work has been done on these two types of synergy. A short account will be given

2. Effect of salt scaling on reinforcement corrosion

A gradual surface erosion caused by frost/salt action will be of great significance for the service life with regard to corrosion. This synergy has been dealt with theoretically in a conference paper paper[36].

A concrete with a linear surface erosion described by the following equation is considered:

$$x_e = C_e \cdot t \tag{A.1}$$

Where, x_e is the erosion depth [m], C_e is the "erosion coefficient" [m/s], and t is time [s].

The penetration of the threshold chloride concentration, or the carbonation front, is described by the normal square-root relationship when there is no erosion:

$$x_c = C_c \cdot t^{1/2} \tag{A.2}$$

Where, x_c is the depth of the treshold concentration, or carbonation front [m], C_c is a rate-determining coefficient [$m \cdot s^{-1/2}$]. This equation can also be written:

$$x \cdot (dx/dt) = C_c^2/2 \tag{A.3}$$

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[36]) G. Fagerlund, G. Somerville, K. Tuutti: *The residual service life of concrete exposed to the combined effect of frost attack and reinforcement corrosion.* Int. Conf. "Concrete across Borders", Odense, June 22-25, 1994.

When the two processes are combined, the differential equation instead becomes:

$$(x-C_e\cdot t)\cdot(dx/dt)=C_c^2/2 \qquad (A.4)$$

This equation can easily be solved numerically. The solution shows, that the time to start of corrosion might be very much reduced when there is salt scaling at the surface. One example is shown in Fig A.1.

The following coefficients were used in the example. The values are not unrealistic:

$C_e=5\cdot 10^{-4}$ m/year
$C_c=3\cdot 10^{-3}$ m·year$^{-1/2}$

The figure shows that the service life is considerably reduced when salt scaling takes place. The effect can definitely not be neglected.

3. Effect of ASR on frost damage

Imaginable effects are identified and treated briefly in an internal report[37] .

The following negative factors were identified:

1: ASR "opens" the concrete to increased inflow of water to air-pores and defects. Concrete with porous aggregate, or porous interfaces (e.g. due to water separation), ought to be more vulnerable.

2: ASR creates cracks, that can becomewater-filled and cause frost damage. Non-airentrained concrete ought to be more vulnerable.

3: Reaction products from ASR fill the air-pores, inactivating these.

4: Gel produced from ASR contains large amounts of water that might be freezable.

The following positive factor was identified:

1: The gel caused by ASR densifies the cement paste matrix, leading to a reduced ingress of water in the cement paste structure. It might be, that this positive factor in certain cases compensate for the negative effect of cracks.

A study was made at the Swedish Cement and Concrete Research Institute on the effect of ASR on frost resistance. Specimens with varying degree of ASR were exposed to a traditional freeze-thaw test during which the specimens were allowed to absorb water. The negative effect of ASR on frost resistance was clearly demonstrated[38]

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[37]) G Fagerlund: *Synergism: Effect of ASR on frost resistance. Some imaginable effects.* The Brite/Euram project, Interim Report IR7, Div. of Building Materials, Lund Institute of Technology, 1994.

[38]) J. Trägård, B. Lagerblad: *Influence of ASR expansion on the frost resistance of concrete*. The Brite/Euram project, Internal Report, Swedish Cement and Concrete Research Institute, 1994.

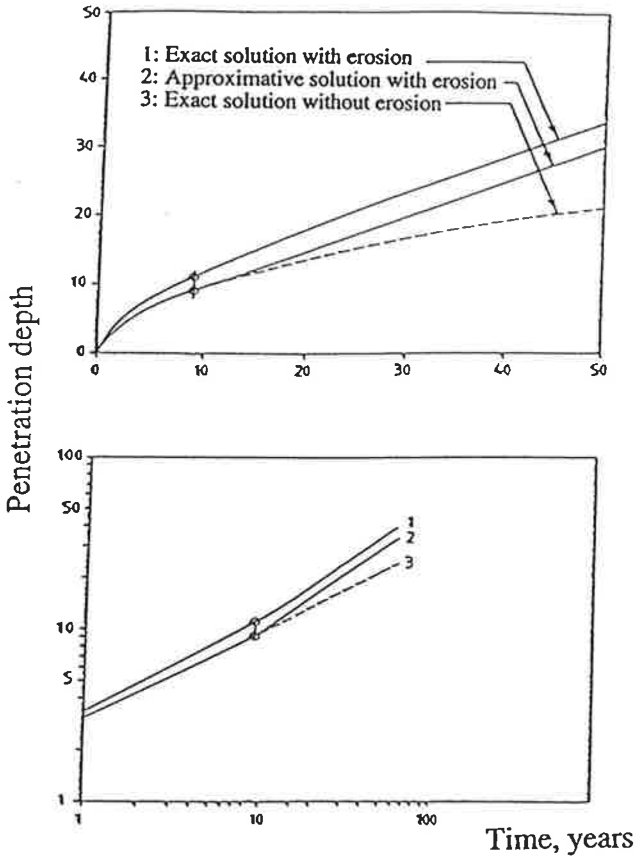

Fig A.1:Example of the penetration-time curve of the threshold chloride concentration, with and without salt scaling. Approximative and exact solutions to the differential equation valid for penetration with combined scaling.
Fagerlund, Somerville, Tuutti, 1994.