CONTECVET : A validated Users Manual for assessing the residual service life of concrete structures

Fagerlund, Göran; et al.

2001

Lund Institute of Technology
Lund University
Division of Building Materials

EC Innovation project IN30902I

CONTECVET
A validated Users Manual for assessing the residual service life of concrete structures

Manual for assessing concrete structures affected by frost

Prepared by
Div. of Building Materials, Lund Institute of Technology

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Preface

This manual is one of three manuals produced within the European project CONTECVET in which the Department of Building Materials, Lund Institute of Technology took part. Other participants in the project came from UK, Spain, and Sweden.

The three manuals are:

Manual for assessing concrete structures affected by frost (The present manual)
Main author and editor: Göran Fagerlund. Department of Building Materials, Lund Institute of Technology.
Author of Chapter1: George Somerville. British Cement Association
Author of Appendix I,J,K: Joakim Jeppson. SkanskaAB

Manual for assessing concrete structures affected by ASR.
Prepared by British Cement Association

Manual for assessing corrosion-affected concrete structures
Prepared by the Spanish company GEOCISA and Eduardo Torroja Institute, Madrid

Besides a fourth report was prepared on the effect of leaching on the structural stability of concrete structures:

Leaching of concrete. The leaching process. Extrapolation of deterioration. Effect on the structural stability
Prepared by the Department of Building Materials, Lund Institute of Technology and published at the Department as Report TVBM-3091, 2000)

The three main reports can be obtained from partners in the project. The fourth report can be obtained from the Department of Building Materials, Lund Institute of Technology.

Lund, May 2001
Göran Fagerlund

EC Innovation Programme
IN30902I

CONTECVET
A validated users manual for assessing the residual service life of concrete structures

Manual for assessing concrete structures affected by frost

BCA British Cement Association (UK)
GEOCISA Geotecnia y Cimientos S.A. (ES)
CBI Swedish Cement and Concrete Research Institute (SW)
IETcc Institute Eduardo Torroja of Construction Science (ES)
DGAV Dirección General de Arquitectura y Vivienda. Generalitat Valenciana (ES)
IBERDROLA (ES)
ENRESA (ES)
TRL Transport Research Laboratory (UK)
NCP National Car Parks Ltd (UK)
VUAB Vattenfall Utveckling AB (SW)
Banverket (SW)
SNRA Swedish National Road Administration (SW)
Lund Institute of Technology (SW)
Skanska Teknik AB (SW)

This manual has been produced by the Swedish partners within the Innovation project CONTECVET (IN30902I). The chaper "General Introduction" is produced by the British partner BCA. The lead in drafting this report was taken by Lund Institute of Technology. The project has been co-ordinated at European level by BCA and at national level by GEOCISA in Spain and CBI in Sweden.

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Contents **Page**

PREFACE

This Manual relates to the assessment of concrete structures, where the primary cause of deterioration is frost . Other Manuals in the series cover alkali-silica reaction (ASR) and reinforcement corrosion. All are related and each Manual has a common Introduction, setting out the overall approach and general principals, prior to giving procedures relevant to the particular deterioration mechanism. Background material is contained in a series of Annexes.

The Manuals are intended to be operational and to fit in with existing formalized inspection procedures. The focus is on a structural assessment and, therefore, much material has been excluded, which is well documented elsewhere (eg on test methods, routine inspection and modelling of deterioration mechanisms); nevertheless, references are given where information on these issues can be found.

The basis for structural assessment is the modification of design procedures, given in EN 1992-1, to allow for the effects of deterioration. Every effort has been made to keep the procedures as general as possible. However, for particular types of structure, or combinations of actions, it may be necessary to develop these general procedures in more detail; this may be done by deriving a series of complementary documents, ideally at a European level, but possibly at a local level – in adding to methodologies which may already exist.

1. GENERAL INTRODUCTION

1.1. Scope

This Manual is written for use be experienced and expert personnel, and gives recommendations on the practice, principles and performance requirements for the inspection and assessment of deteriorating reinforced concrete structures and elements. The scope is limited to buildings, bridges and dams in reinforced concrete, although subject to amendment and further elaboration in complementary documents, the basic approach may also apply to other types of structure, or forms of concrete construction.

The primary cause of deterioration considered in this Manual is frost; companion Manuals cover alkali-silica reaction (ASR) and reinforcement corrosion. It is recognised that other causes of damage or deterioration can occur, singly or in combination with these primary causes; in such cases, further elaboration of the approach in this Manual may be necessary.

1.2 Purpose of Assessment

Before beginning an assessment, it is important to be clear on the objectives. In general, assessment may be concerned with a proposed change in use, or in loading conditions. This Manual relates only to assessment where deterioration is involved, as defined in Section 1.1. In that context, assessment is an aid to decision making, as part of the asset management process.

A prime concern to the owner will be safety. However, he will also be concerned with maintaining the function of the structure, during its expected life time, at minimum total cost, ie with the development of an optimum management strategy. This means that deterioration, as such, is secondary to the effect that it can have on the strength, stiffness and serviceability of the structure.

In setting objectives for an assessment, it is important to remember that the owner will want advice on possible future actions. Some of these are shown typically in Table 1, where it may be seen that the results from the assessment are not the only factor involved.

Table 1: Possible future actions after assessment and factors involved.

ACTIONS

1.	Do nothing; inspect again in x years		
2.	No action now, but monitor		
3.	Routine maintenance; cosmetics; some patch repairs	]	evaluate cost/
4.	Remedial action: specialist repairs and/or protection	]	benefit in whole
5.	Partially replace, or upgrade, or strengthen	]	life costing
6.	Demolish and rebuild	]	terms

TIMESCALE	FACTORS
Now	Results from assessment
1-5 years	Future change in function
5-10 years	Future change in standards
10-25 years	Type and nature of structure
Longer term	Risk and consequences of failure

Recognition of this is important in ensuring that the right information is obtained, to permit confident management decisions to be made.

1.3 Overall approach

The recommendations in this Manual are based on the principle of progressive screening. This means that the investigation should be taken no further than is necessary to reach a decision, ie to decide on which action, given in Table 1, is appropriate, with an acceptable level of confidence.

In the sections which follow, two primary stages are foreseen:

1) Preliminary Assessment
2) Detailed Assessment

In general, preliminary assessment is a qualitative approach to determine whether or not a further, more rigorous, evaluation is necessary (the Detailed Assessment). However, in some cases, it can be self-contained when associated with simple analysis and calculations; in this Manual, this is then called the Simplified Method.

A schematic outline of progressive assessment procedures is given in Table 2. Table 2 indicated the type of input necessary both for Preliminary and Detailed Assessment.

Table 2: Schematic outline of progressive assessment procedures

Assessment Phase	Conclusion: Based on	Conclusion: Result	Conclusion: Reason	Recommendations
Preliminary	Records Survey data Site Measurement Cores Crack pattern & widths Simple analyses	Adequate	Sufficient residual service life and load -carrying capacity.	Monitor
		Borderline	Insufficient data; or residual service life and load-carrying capacity marginally less than that required.	Detailed assessment
		Inadequate	Insufficient residual service life and load-carrying capacity.	Modify adequacy criteria, and reassess. Consider alternative remedial actions. Detailed assessment.
Detailed	As preliminary plus: Monitoring Laboratory tests More sophisticated analyses (i.e. more INSIGHT, Figure 1)	Adequate	Sufficient capacity for required loading (by calculation or load test).	Monitor
		Borderline	Insufficient data; or residual service life and load-carrying capacity marginally less than that required.	Load test to classify as adequate or inadequate. Consider future management and maintenance.
		Inadequate	Insufficient residual service life and load-carrying capacity (by calculation or load test).	Options are: Modify adequacy criteria and/or evaluate actual loading, and reassess. Consider possible actions in Table 2.

In this Manual, the decision-making process, at the end of the Preliminary phase, is based on a Simplified Index of Structural Damage (SISD rating).

The necessary input in Table 2 is targeted mainly at establishing the extent of the damage due to deterioration - and, of course, with identifying the primary cause of that damage. This involved a mix of ‘Overview’ and ‘Insight’ as illustrated in Figure 1. An overview will always be necessary; how much insight is required will depend on the nature and scale of the symptoms of deterioration. The further an Assessment has to proceed, the more insight is necessary.

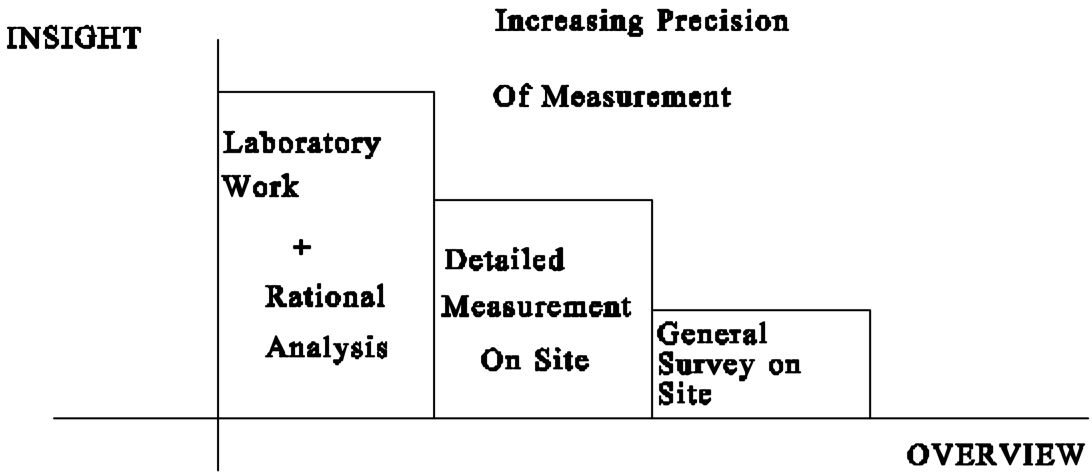

Figure 1: The essential balance between Insight and Overview in assessment

Table 2 also indicated that some analysis and calculations are always necessary, since the prime objective is structural assessment. Again this need can be treated as progressive, with the following options:

1) Simple (elastic) analysis, with full (design) partial factors.
2) More refined analysis, with better structural idealisations.
3) As for 1 and 2, but with assessment-specific imposed loads (usually reduced).
4) Taking account of additional safety characteristics (eg, partial redundancy; membrane action; the influence of non-structural elements).
5) Full reliability analysis (for exceptional cases).

Option 1 is usually sufficient for Preliminary Assessment. As more information becomes available from survey data, modified values for reduced sections, or changed mechanical properties, can be fed in, to better represent the structural capacity of the deteriorated structure.

1.4 Strategy and principles

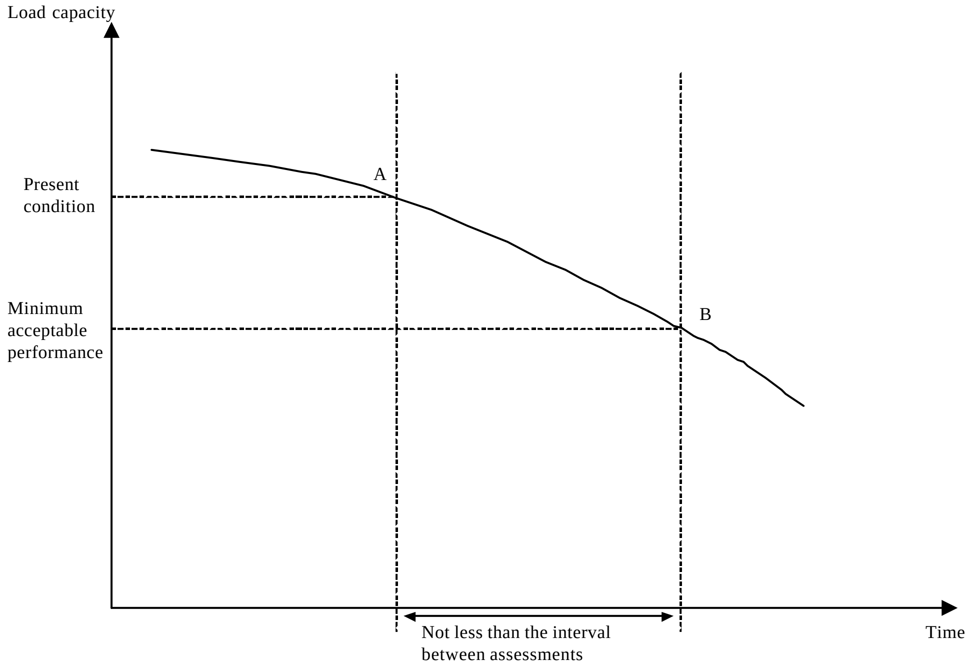

Figure 2: Assessment strategy

The assessment strategy is outlined in Figure 2. Present condition is assessed at point A. The prediction of future state should not then reach the defined minimum acceptable performance, before either the next assessment point or some remedial action is taken. The vertical axis in Figure 2 is expressed in terms of load (structural) capacity, and the Figure shows a reduction between points A and B. Prior to reaching that stage, it will be necessary to assess the extent of the deterioration at point A, and how that extends towards point B. This is stressed, since the shape of the curve A-B may be different for deterioration, compared with structural capacity.

From Figure 2, some general principles can be established for assessment, as follows:

1) The sequence of events is:
   a) quantify the effects of deterioration
   b) identify the prime causes
   c) predict future deterioration
   d) assess structural implications
   e) establish values for minimum acceptable performance
   f) take decisions on future action

2) Events a) to c), in principle 1 above, are central to the Preliminary Assessment stage and effectively involve damage classification. In addition, it will be necessary to 'understand' the structure – physically, plus its design basis and structure sensitivity - mainly via existing records.

3) In assessing structural implications, the following stages may be necessary:

   a) The effect of the deterioration on how the structure as a whole actually carries the imposed loads. This is the 'analysis of structure' phase, in determining maximum values for key load effects. Any loss of stiffness will be especially important in this evaluation.

   b) The effect of the deterioration on the resistance of sections and elements, for all critical action effects – since any particular level of deterioration may affect each of these differently.

   c) A review of structural sensitivity, including the possibility of failure mechanisms, caused uniquely by the deterioration.

4) Figure 2 will require discussion with the owner at an early stage of Detailed Assessment, for two reasons:

   a) to establish an agreed level for minimum acceptable performance, taking account of any statutory requirements, and in the light of future operational requirements for the structure.

   b) to agree future inspection, monitoring, or assessment procedures and intervals.

   In short, to establish criteria for point B.

1.5 Procedures

A flow diagram is shown schematically in Figure 3, based on Sections 1.3 and 1.4, showing how to start and to proceed as far as the Preliminary Assessment stage (SISD rating). Figure 3 is general and may have to be adapted for individual cases; guidance on how to develop Figure 3 is given later in this Manual.

The key to these procedures is to focus on both the deterioration and the structure, even from an early stage and to minimise the amount of investigative work required early on. Consistent with that, there are three important stages, as shown in Figure 3.

1) Desk top study
2) Preliminary assessment
3) Detailed investigation (if considered necessary)

Calculations are recommended at all stages, as an aid to decision-making. In theory, an assessment might be stopped after the desk study, if conservative analyses indicate a considerable margin of safety and the rate of deterioration is low, in relation to the inspection intervals. A decision at this point will also depend on how much detailed information is available from records about the structure and on the availability of data from previous inspections and/or testing.

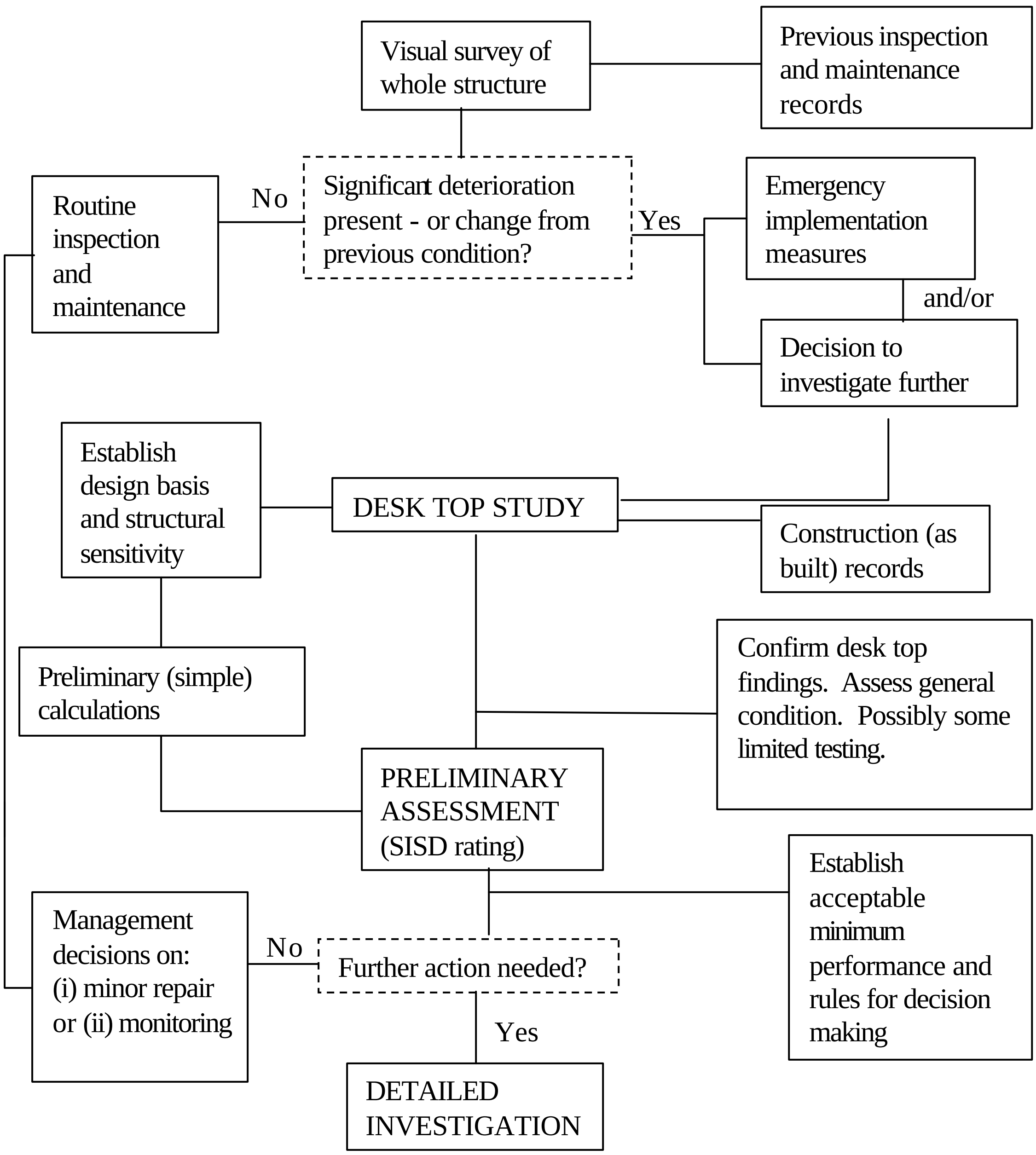

Figure 3: Flow diagram for progressive screening

More commonly, the first decision stage is at Preliminary Assessment. This is a qualitative approach to risk assessment, based on damage classification methods, with some effort made to predict future rates of deterioration. The prime purpose of the SISD rating is to prioritise actions when families of similar structures are involved and, in particular, to decide whether or not a full Detailed Assessment is required (see Section 1.7). However, if accompanied by some simple calculations, particularly on the residual resistance to key action effects, then it may be self-contained; in this Manual, this is designated the Simplified Method.

1.6 Synergetic effects

1.6.1 General

The format of the Manuals, listed in the Preface, is based on the assumption that preliminary investigations will identify the dominant deterioration mechanism and that all subsequent procedures follow from that (see Sections 1.3 and 1.5). However, the effects which this primary mechanism can produce may be exacerbated by defects due to other causes. Some examples are given in Section 1.6.2 below.

Moreover, two or more deterioration mechanisms may act simultaneously, and the combined effects may be more severe and require consideration. Some examples are given in Section 1.6.3 below.

Mainly, this is a question of diagnosis, in identifying primary cause from the observed effects and hence is assessing the current structural significance of the deterioration and, especially, the rate of its reduction in the future. For the examples in Section 1.6.3, more direct assessment of the combined effects may be required.

1.6.2 Defects which may influence the effects of a primary deterioration mechanism

Some examples are given in Table 3, in two separate categories. Category 1 defects tend to reduce the outer surface of the concrete, either physically or in terms of quality. Leaching can also increase the rate of carbonation or chloride penetration. Most concrete structures are subject to cracking at some stage in their lives. The examples listed in category 2 may:

- occur in different timescales
- be permanent or transitory
- be dormant (even healed) or live

Table 3: Some examples of defects and actions which may affect a primary deterioration mechanism

Category 1 *Actions or defects affecting the concrete*	*Category 2* *Non-structural and structural cracking*	
Weathering	Plastic settlement	
Abrasion	Plastic shrinkage	
Leaching	Early age thermal effects	
Honeycombing	Long-term shrinkage	
Pop-outs	Creep	
	Ambient temperature	-movement and restraint - internal temperature gradients
	Design loads	
	Settlement	
	Restraints	- determinacy - non structural elements

1.6.3 Deterioration mechanisms acting in combination

Mostly one mechanism will dominate, but, in some cases, the effects of others may require consideration in combination. Some examples are given in Table 4. There is little experimental verification of these possible synergetic effects, but logic would suggest that they be considered, should early diagnosis indicate the significant presence of more than one mechanism.

Table 4: Some examples of synergy, due to deterioration mechanisms acting simultaneously

Combination of mechanisms	*Possible effects*
Surface sealing due to frost and corrosion	This may lead to a gradual reduction of the cover to the reinforcement and, hence, increases the likelihood of corrosion.
Alkali-silica reaction, and either frost action or corrosion	The expansive action of ASR may lead to wide cracks which can fill with water, and which, if frozen, may cause internal mechanical damage. This same action may also permit easier access to the reinforcement of water containing chlorides, causing more severe corrosion. On the other hand, gel caused by ASR may fill pores, thus densifying the cement matrix.
Leaching and frost action	The influx of water may increase the moisture uptake and, hence, reduce the internal frost resistance.
Leaching and corrosion	The leaching of lime fromthe concrete cover increases the rate of carbonation and the diffusivity of chlorides and reduces the critical threshold level.

1.7 Detailed investigation

1.7.1 General

If a Detailed Investigation is considered to be necessary, then the prime concern is with quantifying structural capacity i.e. in assessing the effect of the deterioration on strength, stiffness, stability and serviceability. This means having enough information available to:

1. fully understand the form and action of the structure;
2. interpret the effects of deterioration in structurally significant terms.

Deterioration can affect structural behaviour in a number of ways:

1. loss of section e.g. concrete spalling, corrosion of reinforcement (general or pitting)

2. reduction in mechanical properties e.g. in the strength of materials, or the stiffness and ductility of elements

3. excessive deformation (local or overall), thus inducing alternative distributions of load, or modes of failure, or rupture of critical sections.

In assessing the influence of these factors on structural capacity, it is important to note:

1. any particular level of deterioration (e.g. loss of rebar section due to corrosion) may influence bending, shear, bond, or other action effect, differently. It follows that each action effect (global and local) should be considered individually.

2. the influence of structural sensitivity on actual load capacity (e.g. the degree of redundancy; the influence of reinforcement detailing, etc.).

With regard to individual deterioration mechanisms, it should be noted that both ASR and frost action only affect the concrete directly - in terms of reduced cross-section, stiffness and reduced mechanical properties. In these cases, Detailed Investigation involved the derivation of modified (reduced) values for these properties, to be used in conventional design models for structural analysis, section strength and serviceability – where the concrete is deemed to make a contribution (see Section 1.3).

For corrosion, the situation is more complex. While the principles in the previous paragraph equally apply, it may also be necessary to check the validity of the design models in an assessment situation.

1.7.2 Minimum acceptable technical performance

The prime concern is with safety, either for the structure overall, or locally for individual elements, connections and sections. This is a matter for decision by individual authorities and owners, in deciding what is acceptable, relative to what was originally provided and to current acceptable standards – bearing in mind, the consequences of failure (see Section 1.8).
However, other performance criteria have to be considered, mainly under serviceability conditions. These would include:

1. a limit on cracking, due to the risk of serious local spalling, likely to be hazardous to life or property (a safety issue in some cases);

2. a limit on deflection, or other deformation, which might impair the function of the structure;

3. a limit on crack width, because of aesthetic or serviceability reasons;

4. a limit on expansion due to ASR, in the presence of restraints, in already highly-stressed sections;

5. consideration of synergetic effects, e.g. the influence of scaling due to frost action on an increase in corrosion rate.

The key point being made is that engineering judgement is essential, in interpreting the scientific data from investigations and testing regimes, in order to take sensible management decisions on what is critical and on when action is necessary.

1.8 Safety levels, risk, confidence levels, etc.

An owner may want a full reliability assessment taking account of variability and uncertainty in a general way, while recognising the stochastic nature of the many factors involved – in the deterioration processes at least, if not always in their effects on structural capacity. He may also wish to directly compare the assessed capacity with that provided in the original design by the use of traditional limit state design (semi-probabilistic, using partial safety factors).

Either way, there will be decisions to take on what is acceptable. Assuming that the same overall reliability is the norm, and taking the partial safety factor approach for purposes of illustration, then there is a case for lower values for the safety factors compared with design. The reasons for this are given in Table 5, which shows that, in general, more reliable information is available in assessment, compared with that in design.

Table 5: Design v Assessment: Significant Differences

Item	*Design*	*Assessment*
Material properties	Assumed	Measured
Dead loads	Calculated	Accurately determined
Live loads	Assumed	Assessed
Analysis	Code based	More rigorous alternatives
Load effects	Bending, shear compression, cracking dominate	Anchorage, bond & detailing may be more important
Environment	Assumed classification	Definition of macro-and micro-climates
Reliability	Code values for safety factors	Small factors for the same reliability

This might justify lower values for partial safety factors. On the loading side of the design (or assessment) condition:

$$S_d \text{ £ } R_d$$

this could be associated with progressively more rigorous analytical methods (see Section 1.3) when establishing safety criteria for point B in Figure 2. On the resistance side, reductions are again possible; however, this will depend on the action effect under consideration, since the basis for some design models is empirical, eg shear, and may not translate directly to the assessment situation.

It is not possible to make recommendations for reduced partial factors, which are generally acceptable. Each case has to be considered individually and there may be minimum statutory requirements for particular types of structure in individual countries, However, the principles behind Table 5 are valid and such an approach has been developed to some extent in some countries (eg, the Highways Agency assessment standards from bridges in the UK).

1.9 Asset Management

As stated in Section 1.2, structural assessment is an aid to decision-making as part of the asset management process and, as such, is an addendum to existing inspection, maintenance and management procedures. Table 1 indicates the type of management decision which may have to be made and Table 2 shows how assessment could progress as an aid in that direction.

So far nothing has been said on how to choose the most effective remedial action and, indeed, that is beyond the scope of the Manual. However, assessment and choice of remedial action are inter-related. Not only must the repair option be effective and compatible with the structural system, but also its expected life may influence the future inspection and assessment strategy.

This point is illustrated in Figure 4, which has been developed from Figure 2, with points A and B having the same meaning. If a decision is taken at point C to take remedial action and the choice is between repair options 1 and 2, restoring load capacity to level i) and ii) respectively, then the shorter life of option 2 would influence the interval between inspections.

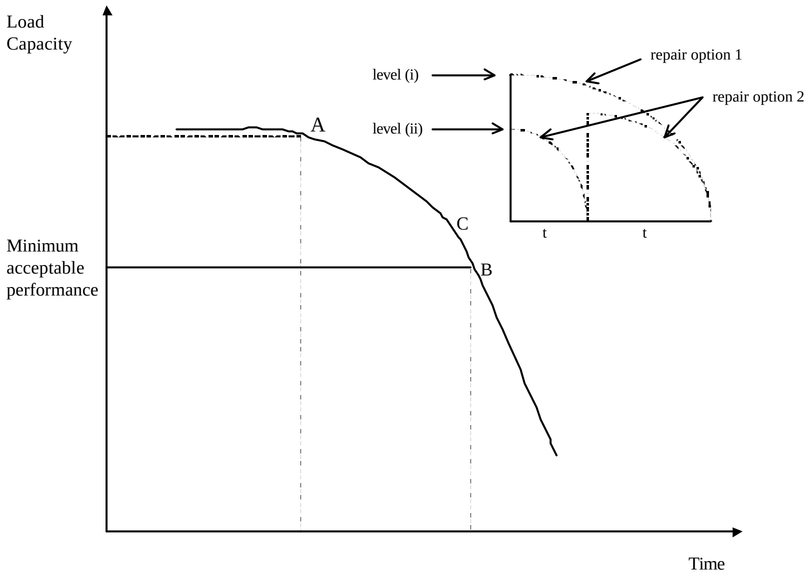

Figure 4: Schematic illustration of two different repair options

Although option 2 may be the cheapest in first cost terms, option 1 may be preferable in whole life costing terms, if the cost of disruptions, and having to repair twice in time 2t, is taken into account.

The scenario in Figure 4 has deliberately been made simplistic and the situation will rarely be this straightforward in practice. It is included here to illustrate the interaction between assessment and remedial action - and to make the important point that any assessment method - whether simple or complex - must fit within asset management systems such as this.

2. TYPES OF FROST DAMAGE

Frost damage only affects the concrete in the structure. The reinforcement is not affected directly, but will be indirectly affected by reduction in bond and reduction in the concrete cover.

There are two types of frost damage. Both are considered in the Manual.

1: *Internal damage* caused by freezing of water inside the concrete. The damage is always confined to such parts of the concrete where the water content exceeds a critical value. This is individual for each concrete, and can also be different in different parts of the same structure. The damage is shown as loss in compressive and tensile strength, loss in E-modulus, and loss in bond strength.
Internal damage will affect the moment and shear capacity of slabs and beams and the compression capacity of columns by lowering the compressive strength and bond strength. It might seriously affect the structural capacity of pre-stressed concrete by significantly lowering the E-modulus of the concrete. It also changes the moment and force distribution in the structure by changing the stiffness in parts of the structure.

2. *Surface scaling* caused by freezing of the concrete surface when it stays in contact with saline solutions of weak concentration. The initial scaling occurs in the cement paste phase while the aggregate grains are intact. Due to the gradually deeper and deeper scaling also coarser aggregate grains are lost. In serious cases a big portion of the cover can be eroded which has very big effect on the anchorage capacity of the reinforcement bars. It also affects the function of compression reinforcement and shear reinforcement

Scaling also affects the service life with regard to reinforcement corrosion by reducing the concrete cover.

3. ASSESSMENT PROCEDURE-PRINCIPLES

3.1 Assessment of a single structure

An assessment of a structure is made in the following steps (also, see Figure 3):

Step 1: Visual inspection of the structure concerning:

* Signs of damage (type, location, frequency)
* Verification that frost is the cause of damage (crack pattern, scaling)
* Microclimate (moisture load, de-icing salts, marine environment)

Step 2: Collecting information concerning:

* Age of the structure
* Design of the structure (drawings, loads, material requirements, design code)
* Material qualities built in (specifications, test results)
* Management since erection of the structure (use of de-icing agents, previous damage and/or repair)
* Previous inspections (written reports)

Step 3: Preliminary (simplified) assessment based on 1: and 2:. The procedure is:

Level 1: Qualitative preliminary assessment

* Assessment only based on visual observations of damage and estimation of consequences of failure, making use of a Structure Severity Rating scheme.

Level 2: Quantitative preliminary assessment

* Identification of critical sections within the structure
* Preliminary control of the *present* structural stability by a preliminary structural analysis of the critical sections using lower bound values of material properties of frost damaged concrete.

Step 4: Decision concerning how to deal with the structure, based on the preliminary assessment. Options are:

* No measures need to be taken at present
* A detailed assessment must be performed.
* The structure is immediately repaired, strengthened, or demolished.

Step 5: Detailed assessment of the structural stability. The procedure is:

A: Assessment of the present structural stability.

* If there is doubt about the cause of the observed damage, a petrographic analysis identifying the cause shall be made. If another damage mechanism than frost is the cause, the *Corrosion Manual* or the *ASR Manual* (or any other manual) must be used.
* Determination of relevant material data by measurements of the structure in-situ, and by laboratory measurements on drilled-out cores.
* Determination of relevant cross-section data by measurements on the structure (scaling, residual cover)
* Determination of the actual loads
* Determination of the design code to be used in the assessment
* Determination of the safety level; partial coefficients for material and load
* Re-design (control) of crucial cross-sections.

B: Assessment of the future structural stability.

* Evaluation of the moisture level (moisture class) for different parts of the structure.
* Extrapolation of material data.
* Extrapolation of scaling and cover
* Re-design (control) of crucial sections for selected future points of time.

Step 6: Assessment of synergetic effects of other destruction mechanisms, especially:

* Salt scaling, affecting reinforcement, and its effects on structural stability (a: reducing the time to start of corrosion. b: reducing the effectiveness of reinforcement)
* ASR, increasing internal frost damage.
* Leaching of lime, increasing internal frost damage, and increasing salt frost scaling.
* Leaching of lime, affecting reinforcement, and its effects on structural stability (reducing the time to start of corrosion)

Comments:

1. **Step 1** is always required. No assessment shall be made before an experienced materials investigator, and preferably also the structural engineer, has visited the structure.

2. **Parts of step 2** can be omitted if the damage is seemingly limited, and only a qualitative preliminary assessment is made.

3. **Step 5**, the detailed assessment, is only required if the preliminary assessment indicates that there is risk of an impermissibly low safety margin, and to get information about when repair is required in order to decide upon a management strategy.

4. **Step 6** is sometimes important also for a preliminary assessment.

3.2 Assessment of a population of similar structures

In a case where there is a whole population of frost damaged structures, or building elements of similar type, a detailed assessment of each individual structure is not rational or feasible. In such a case the following assessment procedure can be used:

Step 1: Simplified visual inspection of all structures aiming at identifying the most serious cases.
Step 2: Collecting the most relevant technical information from erection of the structure/element (drawings, material qualities)
Step 3: Making a qualitative preliminary assessment based on information from step 1: and 2:. On basis of this, dividing the structures/elements in ”severity classes”.
Step 4: On basis of step 3: give priority to the structures/elements that shall be the subject of detailed structural assessment, or of immediate repair, or demolition.

Examples of structures for which this strategy can be applied are:
* A population of similar (or identical) façade elements showing sign of internal frost damage (and/or reinforcement corrosion)
* A population of similar bridge decks, or bridge edge beams, with salt scaling.
* A population of similar parking decks with salt scaling and/or internal frost damage (and/or reinforcement corrosion)
* A dam with big variation in concrete quality in different parts. Selecting the most critical parts.

4. PRELIMINARY ASSESSMENT

4.1 Introduction

A preliminary assessment shall only be used for the following purposes:

1: To identify individual structures, or parts of a structure, for which a detailed structural assessment should be made for safety reasons.
2: To divide a population of structures in ”severity classes” , in order to identify structures for which a detailed assessment and/or remedial actions are most urgent.

The preliminary assessment can be made on two levels:

Level 1: *Qualitative*, only based on visual inspection, and qualitative estimation of the aggressiveness of the environment, and an analysis of the consequences of structural failure.
Level 2. *Quantitative*, based on the qualitative assessment supplemented by calculations of the structural capacity in vital parts, using lower bound data for mechanical properties.

4.2 Qualitative preliminary assessment

4.2.1 Internal frost damage

Important factors for the preliminary assessment are:

* The moisture characteristics of the environment in which the structure (element) is placed.
* The consequences of failure (on people and property).

On basis of these factors the following Simplified Index of Structural Damage Rating (SISD) scheme is established.

Table 6. Simplified Index of Structural Damage Rating (SISD) for internal frost damage
(n=negligible, M=moderate, S=severe, VS=very severe)

Environment	Consequences of failure	
	Slight	Severe
1: Moist	n	M
2: Very moist	M	S
3: Extremely moist	S	VS

The classification of environment is based on its effect on the moisture level inside the structure. The higher the moisture level, the bigger the risk of frost damage.

Table 7: Classification of consequences of failure

Consequence of failure	Definition	Examples
Slight	Structural failure of the entire structure, or parts of it, will cause no, or small, risk of damage to property and no risk of damage to people.	Pieces falling from a hydraulic structure to water or ground
Severe	Structural failure of the entire structure, or parts of it, will cause big risk of damage to people and property.	Breakage of a dam causing flooding downstream. Falling of big pieces of a façade to the street

Table 8: Classification of the environment for internal frost damage

Environment [1)]	Moisture characteristics	Examples
1: Moist	*Outer:* Periods of exposure to water followed by longer periods of drying. *Inner:* No accumulation of water over time Moisture level not above the breaking point in a water absorption test [2)]	Vertical parts of façades. Fairly rain protected parts of structures (bottom parts of slabs exposed to air)
2: Very moist	*Outer:* Long periods of exposure to water followed by periods of drying. *Inner:* A certain increase in water over time. Moisture level above the breaking point in a water absorption test[2)]	Horizontal surfaces exposed to rain and melting snow, like balcony slabs, hydraulic structures some meters above the water line.
3: Extremely moist	*Outer:* Constant exposure to water. No drying periods. *Inner:* A gradual increase in water content over time. Moisture level far above the breaking point in a water absorption test[2)]	Hydraulic structures close to the water line. Foundations in ground water above the lowest level of zero temperature. Bridge piers in fresh water.

1) Environments causing low moisture levels in the concrete are not included, since these, theoretically, cannot give internal frost damage.

2) The breaking point, and the test, are described in ANNEX G, paragraph 4.3 and paragraph 5.2

4.2.2 Salt frost scaling

Important factors for the preliminary assessment are:

* The present scaling depth
* The future use of de-icing agents or exposure to sea water
* The consequences of failure (on people and property).

On basis of these factors the following Simplified Index of Structural Damage Rating (SISD) is established. It is assumed that the cover is of normal thickness.

Table 9: Simplified Index of Structural Damage Rating (SISD) for salt frost scaling
(n=negligible, M=moderate, S=severe, VS=very severe)

Scaling % of cover 1)	Environment	Consequences of failure 5)	
		Slight	Severe
<25%	No salt	n	n
	Indirect salt spray, or sea water spray	n	M
	Direct sea water exposure	n	M
	De-icing salt exposure above -10°C	M	S
	De-icing salt exposure down to -25°C	S	VS
25 to 50% 2) 3)	No salt	M	M, S, VS 4)
	Indirect salt spray or sea water spray	M	S
	Direct sea water exposure	S	VS
	De-icing salt exposure above -10°C	S	VS
	De-icing salt exposure down to -25°C	S	VS

1) The table can only be used if the residual cover after scaling is above 20 mm. If it is lower, a quantitative preliminary assessment, or detailed assessment, shall be made.

2) If the scaling is bigger than 50% of the cover, a quantitative preliminary assessment or a detailed assessment shall be made.

3) A control of the effect of scaling on reinforcement corrosion must also be made.

4) Scaling of this magnitude is unusual in a salt-free environment. Probably, de-icing salt has been used in the past, and shall not be used in the future. Then, SSR=M. If salt was not used in the past, the big scaling indicates a very low frost resistance. Then SSR=S. If salt is not used at present, but might be used in the future SSR=VS.

5) Consequences of failure are defined in the same way as for internal frost damage.

4.3 Quantitative preliminary assessment

4.3.1 Internal frost damage

A control of the *present* structural capacity is made in selected critical sections of the structure, using traditional design procedure.

The design code, and the load to be used in the structural control, are normally defined by the Owner. The characteristic values of the compressive, tensile, and bond strength to be used, can be based on measurements of cores from the structure. Lacking these, lower bound values for frost damaged concrete can be used.

Lower bound strength values are based on experimental data in ANNEX E, and are also discussed in ANNEX H. Values are given in Table 5. Any of the three values can be used. Values in column 3 are the lowest observed residual strength of severely frost damaged concrete. They can be used for concrete having an initial compressive strength of at least 35 MPa, and an initial tensile strength of at least 3 MPa. There are few concrete structures built for outdoor exposure, with lower initial strength level.

It is not possible to give any definite value for the lower bound of the E-modulus of frost damaged concrete. Any value is possible. The lowest value observed for severely frost damaged concrete is 5GPa, but this is so low that it can hardly be used for a meaningful assessment of deformations of the structure. Calculation of deformation, therefore, belongs to a detailed assessment, and is then based on direct observations of the E-modulus.

Table 10: Lower bound values for concrete with internal frost damage.

Strength type	Relation between [1] reduced and initial strength (%)	Biggest reduction in strength (%)	Lowest strength [2] (MPa)
Compressive	$(1- 20/f_{c,o})\cdot 100$	35%	20 MPa
Split tensile	$(3 - 11/f_{t,o})\cdot 100$	70%	1 MPa
Bond strength [3] ribbed bars	$(10 - 35/f_{t,o})\cdot 100$	70%	3 MPa
Bond strength [3] plain bars	$(2.4 - 10/f_{t,o})\cdot 100$	100%	0 MPa

1) $f_{c,o}$ and $f_{t,o}$ are the initial compressive strength and split tensile strength before frost damage. The relation is limited to the values in column 2.

2) The lowest observed strength of severely frost damaged concrete with an initial compressive strength above 35 MPa, and an initial split tensile strength above 3 MPa.

3) Bond strength is the intrinsic bond strength between a bar and concrete, no consideration taken to the effect of cover and confinement by stirrups.

4.3.2 Salt frost scaling

Surface scaling is obvious. The scaling depth can be measured or estimated by observing the structure. Thereafter, a simple control can be made of its effect on the structural capacity by just reducing the cross-section and effective height of the structural member.

Depth scaling, where the whole cover, or a big portion of the cover, has been lost, might have serious effects on the anchorage capacity, and the function of shear reinforcement and compression reinforcement. This must be treated in a detailed assessment whereby design formulas in the *Corrosion Manual* can be used.

4.4 Synergetic effects

If there are other destruction mechanisms acting simultaneously with frost, the preliminary assessment procedure described above cannot be used.

If leaching is going on, at the same time as there is internal frost action, a fairly common situation in hydraulic structures, the negative effects on strength can be considerably bigger than the values given above.

It is especially important to consider the effect of surface scaling on reinforcement corrosion. If the scaling is deep, it will have very big effect on the structural stability with regard to corrosion.

In case synergetic effects exist, a detailed assessment considering synergy must be performed. Different types of interaction between destruction mechanisms are described in more detail in ANNEX F

5. DETAILED ASSESSMENT

5.1 Introduction

A detailed assessment is made when one wants more precise information of the present and future structural status and safety. It is always based on quantitative information on strength and stiffness of the damaged structure. Thus, a detailed assessment requires testing of the structure, or laboratory testing of cores taken from the structure.

The assessment is performed as a *re-design* of the structure using the actual measured material data (or lower bound data), the actual cross-section data, and the actual load data. A total re-design is often not required, only a control of critical sections in the structure. Data used in the assessment shall be taken from these sections. Therefore, the structural engineer, who is responsible for the assessment, is also responsible for identifying places in the structure from which data shall be acquired.

Any design code can be used; the one used at the original design, or the present official code. It is decided by the Owner.

The assessment is divided in two parts:

Part 1: Assessment of the *present* structural capacity and safety based on present data.
Part 2: Assessment of the *future* structural capacity and safety based on extrapolated data. This assessment can be made for one or more future points of time.

Structural assessment for different types of action in different types of structural members is discussed in more detail in ANNEX I, J and K. Below, only important principles are described.

5.2 The present structural capacity

5.2.1 Internal frost damage

Internal frost only affects the strength and stiffness of concrete. It, however, also affects the bond between reinforcement and concrete.

The effect of internal frost damage is analysed by a re-design of the structure using actual data for; (i) strength and stiffness of concrete, (ii) amount, and location of reinforcement, (iii) cross-section, (iv) concrete cover, (v) load.

The design code, and the load to be used in the structural control, are normally defined by the Owner. The same formulae are used for calculating the structural capacity of frost damaged concrete as are used for undamaged concrete.

The characteristic values of the compressive, tensile, and bond strength to be used, are based on measurements of cores from the structure. In the general case, material data varies from place to place within the same structure. Therefore, many sections of the structure must normally be analysed for structural capacity, using individual data for each section. The location of sections for measurements of strength, E-modulus, scaling, and other essential properties, are determined by an experienced investigator in collaboration with the structural engineer who is going to perform the assessment.

Methods of acquiring material data, and methods for translating observed data into characteristic values to be used in the assessment, are described in ANNEX G.

Characteristic values, and lower bound values of strength, and E-modulus to be used in the assessment are shown in Table 11. The lower bound values are so low that they can hardly be used for an assessment. Thus, normally a testing of the structure is required.

Normal design procedures are used. Extra consideration must, however, be taken to the fact that the E-modulus of damaged concrete is different in different parts of the structure. Therefore, the moment and force distribution can be different in the frost damaged structure than what was anticipated when the structure was originally designed.

Pre-stressed concrete structures can be more affected by internal frost damage than normal structures. Both the E-modulus and the tensile strength can be very much affected. This reduces the pre-stressing force and the possibility to anchor tendons.

Table 11: Characteristic values for mechanical properties of concrete

Strength type	Characteristic value	Lower bound value (MPa)
Compressive	$f_c = m-1.65\cdot\sigma$	$f_c=f_{c,o}-20$ (minimum value 20 MPa)
Split tensile	$f_t=m-1.65\cdot\sigma$	$f_t=3\cdot f_{t,o}-11$ (minimum value 1 MPa)
Bond Ribbed bars	$f_b=f_{b,o}(f_t/f_{t,o})$ or $f_b=3\cdot f_t$	$f_b=10\cdot f_{t,o} - 35$ (minimum value 3 MPa)
Bond Plain bars	$f_b=0.8\cdot f_t-1.2$	$2.4\cdot f_{t,o} - 10$ (minimum value 0 MPa)
E-modulus	$E=m-1.65\cdot\sigma$	5 GPa
Static E-modulus	$0.85\cdot E_{dyn}$	5 GPa

f_c=present characteristic compressive strength
f_t= present characteristic split tensile strength
f_b= present characteristic "intrinsic" bond strength
$f_{c,o}$=present characteristic compressive strength of undamaged concrete
$f_{t,o}$=present characteristic tensile strength of undamaged concrete
$f_{b,o}$=bond strength of undamaged concrete as defined by the design code (usually related to the tensile strength of undamaged concrete, $f_{t,o}$)
E=present E-modulus
m= mean value of results from tested specimens (or measurements in-situ).
The measured values for compressive strength are corrected for size, shape and moisture content of specimens.
σ= standard deviation of these results

5.2.2 Salt frost scaling

Salt scaling reduces the effective height of the cross-section, and the size of this. New values are inserted in a re-design of the moment, shear, and compression capacity of scaled parts of the structure.

Scaling also affects the anchorage capacity by reducing the cover. Scaling seldom affects the strength of the residual cover. Thus, bond strength is normally unaffected by scaling. The reduced anchorage capacity is calculated by inserting the new reduced bond strength, the reduced cover, and the actual amount of reinforcement in the formula for anchorage capacity given in the design code used for the assessment.

When scaling is so deep that the whole cover is lost, there is a risk of buckling of compressed reinforcement. In this case, reinforcement corrosion will be the most important factor determining the structural capacity. Therefore, formulas in the *Manual for Reinforcement Corrosion* can be used also for the assessment of this type of deep salt frost scaling.

In the normal case, scaling and residual cover will vary quite much within the same structure. Relevant values for the actual section must be used in the assessment.

5.3 The future structural capacity

5.3.1 Internal frost damage

If a structure is frost damaged, there is reason to assume that damage will continue in the future, unless the environment around the structure is radically changed (e.g. by heat insulation, drainage of water, etc). Extrapolated values of strength and stiffness are used in a re-design valid for different points of time in the future. The re-design is made according to the same principles as described above for the assessment of the present structural status.

The future development of strength and E-modulus of the concrete depends on the moisture characteristics. Visual inspection, and control of the moisture in the concrete (ANNEX G), are used for classifying the structure in one of three ”moisture classes” or environmental classes:

1: Moist
2: Very moist
3: Extremely moist

The characteristics of these classes are described in Table 3: above.

The material data used in the assessment of the present status are extrapolated in time. The extrapolation is based on two parameters:

1: The present value for undamaged concrete
2: The present value for damaged concrete (or the present amount of damage)

The equations used for extrapolation are shown in Table 12.

In parts of a structure in moisture class 3, where frost damage has not yet occurred, but might do so in the future, there are certain possibilities to estimate the residual time until damage, by making freeze-thaw and water absorption tests. The technique is described in ANNEX C and G.

Table 12: Extrapolation of material data from Table 11.

Moisture class	Extrapolation
1: Moist	$R(t)=R(o)$
2: Very moist	$R(t)=R_o-[R_o-R(o)]\cdot(1+\Delta t/t_o)^{1/2}$ or $R(t)=R_o\{1-D(o)\cdot(1+\Delta t/t_o)^{1/2}\}$
3: Extremely moist	$R(t)=R_o-[R_o-R(o)]\cdot(1+\Delta t/t_o)$ or $R(t)=R_o\{1-D(o)\cdot(1+\Delta t/t_o)\}$

R= any mechanical property (strength or E)
R(o)= present value of R in frost damaged concrete
R_o=present value of R in undamaged concrete
R(t)=extrapolated value of R at exposure time t (the age of the structure)
Δt=additional time counted from t_o (the present exposure time= the age of the structure)
D(o)=damage at time of inspection ($D(o)=[R_o-R(o)]/Ro=1-R(o)/R_o$

5.3.2 Salt frost scaling

Salt frost scaling, is assumed to be a linear process. Therefore, also the residual cover is a linear process. The equations for extrapolation are shown in Table 13

Table 13: Extrapolation of surface scaling and cover

Parameter	Extrapolation
Surface scaling	$S(t)=S(o)\cdot(1+\Delta t/t_o)$
Cover	$C(t)=C(o)-S(o)(\Delta t/t_o)$

S(t)=the scaling depth at exposure time t (the age of the structure)
S(o)=the present scaling depth
C(t)=the residual cover at exposure time t
C(o)=the present residual cover
Δt=additional time after t_o (the present exposure time=the age of the structure)

In a case where the exposure to de-icing salt will change in the future (increased, decreased, or ceased), an extrapolation according to Table 8 will be misleading. In such cases, the extrapolation can be based on a salt-frost test. The principles are described in ANNEX D and G.

5.4 Synergetic effects

If there are other destruction mechanisms acting simultaneously with frost, the assessment procedure for the *present* status described above can still be used, provided it is based on *measured* values of strength and E-modulus, and not on lower bound values.

It is not possible to use the assessment for the *future* structural status, since it is based on extrapolated material data. The extrapolation is only valid for frost damage, and not for combined destruction.

In case synergetic effects exist, a detailed assessment, considering synergy, must be performed. Different types of interaction between destruction mechanisms are described in more detail in ANNEX F

5. DECISIONS AND MEASURES FOLLOWING THE ASSESSMENT

On the basis of the assessment it will be possible to decide upon a a strategy for how to deal with the structure. There are some options (also, see paragraph 1.9):

1: The assessment shows clearly, that there is no risk at present for structural failure, and that there is no risk for structural failure within the nearest decades.

In this case, the structure can be left for the time being. An inspection plan is drawn up in which the most important parameters to be inspected are treated. Inspection methods and frequency of inspection are stated. In the inspection plans there can also be figures for the maximum allowable future damage. These figures are based on the structural assessment made.

2: The same as 1:, except for that structural failure, or other serious problems caused by frost damage, might come within rather few years.

In this case one must consider if an immediate repair/strengthening of the structure is not the best solution. It must be considered that *severe internal frost damage* is extremely difficult, or even impossible, to repair. Therefore, it is often better to make a repair early, before there is so big damage that the structure probably has to be demolished.

If the structure is only damaged by *salt scaling*, repair can wait, since it will not be more complicated to make repair at a later stage. There must be an inspection plan, however, for the follow-up of the scaling process in with criterions are given for when repair must be made. In this plan the effects of reinforcement corrosion must be considered.

3: The assessment shows that there is imminent risk of structural failure, or indirect damage caused by malfunction of the structure.
In this case there are only two options; (i) repair and/or strengthening,
(ii) demolition.

If repair is required, it must fulfil the following requirements (by ”repair” is meant the entire combination of the repair material/repair system and the old structure):

1: The repair must fully restore the load-carrying capacity and safety of the structure. In some cases, when the load has been increased, or when a new design code shall be applied, the repair must also *increase* the load-carrying capacity compared to the requirements when the structure was built.

2: The repair must stop the ongoing frost destruction (or at least slow this down to an acceptable level) so that the Owners requirements for the residual service life are fulfilled.

3: The repaired structure must be durable. This means that repair materials shall be durable themselves, and that they shall not increase the moisture condition of the old concrete.

These requirements are quite difficult to cope with. The most difficult problem is to fulfil the third requirement of durable repair. Many repair systems tend to increase the moisture content in the old structure by allowing moisture to come in, but hinder it to evaporate. This means that a concrete after repair can enter a higher moisture class (e.g. move from class ”moist” to class ”very moist”). The consequence might be that a structure, that has low or no internal frost damage at the time of repair, becomes rapidly deteriorated after repair. Another consequence is that the repair materials bonded to the old structure, e.g. a new surface replacing a scaled surface, might freeze loose due to accumulation of moisture at the interface between repair and old concrete.

A repair system must therefore be selected with care. Freeze-thaw tests of the combined old concrete and new repair material-repair system shall be performed. High quality, frost resistant cement-bound repair materials (concrete and mortar) often function well. Plastic repair materials shall normally be avoided, unless durability tests clearly show that they are ”compatible” with the old concrete.

ANNEX A
Internal frost damage

Contents **Page**

ANNEX A: Internal frost damage

Types of frost damage

There are two types of frost damage:

1: *Internal damage* caused by freezing of water inside the concrete. The damage is always confined to such parts of the concrete where the degree of water saturation exceeds the critical value. The damage causes loss in compressive and tensile strength, loss in E-modulus, and loss in bond strength; see ANNEX E.

2. *Surface scaling* caused by freezing of the concrete surface when it stays in contact with saline solutions of weak concentration. The initial scaling occurs in the cement paste phase while the aggregate grains are intact. Due to the gradually deeper and deeper scaling also coarser aggregate grains are lost. In serious cases a big portion of the cover can be eroded which has very big effect on the service life with regard to reinforcement corrosion. See ANNEX F.

Freezing of very moist concrete (also concrete containing salt water in its pores) but with no external salt water or pure water in contact with the surface seldom leads to surface scaling, but more often to internal damage.

Freezing of concrete in presence of an external salt solution at the surface seldom leads to internal damage, but often to surface scaling.

Internal frost damage is described in this ANNEX.
Salt scaling is described in ANNEX B.

1. Freezable water

The freezing-point of water in a pore is lower the smaller the pore diameter. The exact relation between the pore size and the freezing point is not fully clarified. It depends on which types of meniscus systems ice-water, ice-vapour and water-vapour that appear within the complex pore system; /1/. A reasonable assumption, also giving the maximum possible amount of freezable water, is that the ice phase remains under ordinary atmospheric pressure while the unfrozen part of the pore water is exposed to an under-pressure that is described by the Kelvin law of capillary condensation. This implies that the interfaces between ice and vapour are plane while the interfaces between water and vapour are curved in the normal manner. No curved interfaces between ice and water are assumed to appear. Besides, water that is adsorbed on the pore walls is supposed to be non-freezable. Under these assumptions the following relation between the pore diameter and the freezing temperature can be used; /2/.

$$r = -\frac{2\cdot\sigma_{lg}\cdot M}{\rho_l\cdot\Delta H}\cdot\frac{1}{\ln\{(T_o-\Delta T)/T_o\}} + 19{,}7\cdot 10^{-10}\,(1/\Delta T)^{1/3} \qquad (1)$$

Where,
- r the pore radius (m)
- σ_{lg} the surface tension between water and vapour (N/m)
- M the molecular weight of water (kg/mole)
- ρ_l the density of water (kg/m^3)
- T_o the freezing point of bulk water (273,15°K)
- ΔT the freezing point depression (°K)
- ΔH the molar heat of fusion of water (J/mole)

Where the first term on the right hand side is the radius of the liquid meniscus, and the second term is the thickness of the adsorbed unfrozen layer. In Table 1 some examples of the relation between the pore diameter and the freezing point are shown. The corresponding relative humidity (RH) according to the Kelvin equation is also listed.

Table 1: Relation between the pore diameter, the corresponding relative humidity when capillary condensation takes place in the pore, and the freezing point.

Pore diameter (Å)	RH (%)	Freezing point (°C)
450	95	-6
280	92	-10
200	88	-15
160	85	-20
115	80	-30
95	76	-40

Therefore, at normal freezing temperatures (-10°C to -20°C), neither water in the gel pores, nor water in the finest capillaries is freezable. Besides, in concrete in equilibrium with an outer RH of 95% (like an outdoor structure protected from rain) no water is freezable until a temperature of -6°C is reached in the concrete.

Some measurements of the freezable water in concrete at -15°C are shown in Figure 1. Curve A shows the results for concrete that has been stored in water all the time from the mixing to the test. Curve B shows the results for companion specimens that have been pre-dried once at +105°C, and then re-saturated be means of vacuum treatment. The total evaporable water content is also shown in the figure.

Figure 1 shows that the freezable water content is always lower than the total water content. For a never-dried concrete there is almost no freezable water at all, if the water-cement ratio is below 0,3. For a pre-dried concrete ("aged" concrete), however, the freezable water content is much higher. So for example, for a concrete with W/C=0,3 pre-dried at +105°C, the freezable water is almost as high as for a never-dried concrete with W/C=0,6. Similar, but not quite as big, effects occur even when the concrete has been exposed to much milder drying at room temperature; /3/.

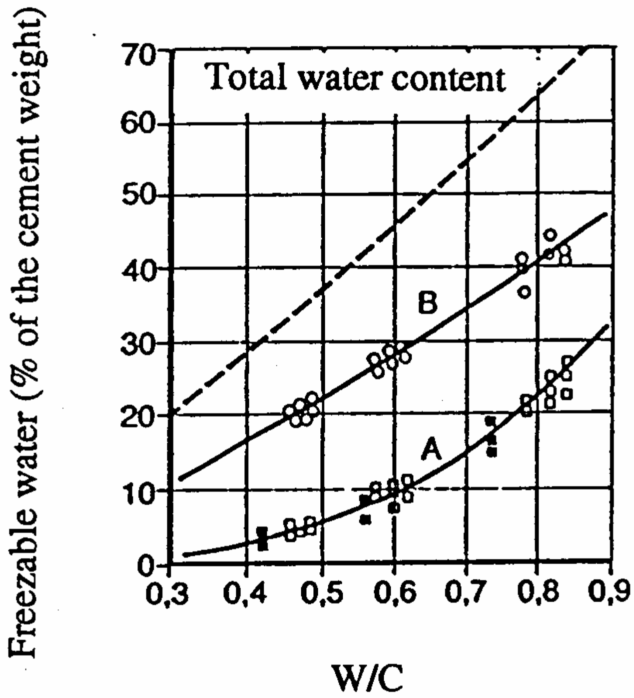

Figure 1: Freezable water of concrete at -15°C. A: Never dried specimens.
B: pre-dried and re-saturated specimens; /4/. (Non-porous aggregate)

The reason behind this increase in the freezable water caused by drying has never been fully clarified. It certainly depends on some sort of structural change brought about by drying. The most plausible explanation is that a very large fraction of the potentially freezable water in the never-dried concrete is located in isolated small capillary pores which are unreachable by the ice front, because the entrance pore leading to the isolated pore is too narrow to allow freezing. Therefore, this isolated water remains unfrozen –super-cooled- until it freezes by homogeneous nucleation at about -40°C At this temperature, big amounts of water normally freezes in concrete of all types.

Because of pre-drying, a micro-crack system develops in the cement gel surrounding the previously isolated capillaries. Thereby, ice-formation can be nucleated by ice penetrating the crack system from other, coarser pores. Therefore, ice-formation in the pre-dried specimen occurs more close to the freezing temperature determined by the actual pore size; eqn. (1).

The reasonableness of this explanation is strengthened by the fact that ice melting in never-dried specimens occurs at considerably higher temperatures than ice formation.

The big ice formation around -40°C signifies that concrete that is frost resistant at temperatures down to -30°C might become non-resistant if the temperature is lowered to -40°C or lower. This is of big importance for *concrete used in extremely cold climate* such as in storage containers for liquified gas or frozen food.

For a concrete under practical conditions, at the normal freezing temperatures in nature, one can roughly assume that all capillary pores contain freezable water, while the gel pore water is unfrozen. This implies that the maximum freezable water content is; based on formulae in /5/:

$$w_f = B(W/B-0{,}39\cdot\beta) \qquad (2)$$

Where w_f the freezable water (litres/m^3)
B the binder (cement) content (kg/m^3)
W the mixing water content (litres/m^3)
β the degree of hydration (-)

Strictly speaking, this equation is only valid for concrete with portland cement, but it can also be used for an approximate estimate of the freezable water in concrete with mineral admixtures.
The equation gives the following amount of freezable water:

w/B=0.40 (B≈400): w_f≈50 l/m^3 (Figure 1, Curve B gives: 65 l/m^3)
w/B=0.60 (B≈300): w_f≈85 l/m^3 (-”- 85 l/m^3)

The non-freezable water can also be expressed in terms of a fraction k_θ of the total evaporable water content, w_e [litres/m^3]. If it is assumed that only the gel pores and the capillary pores contain water the following relation is valid; based on formulae in /5/.

$$k_\theta = 1 - (W/B-0{,}39\cdot\beta)/(W/B-0{,}19\cdot\beta) \qquad (3)$$

For typical concrete the values in Table 2 are valid:

Table 2: The maximum amount of freezable water in typical concrete.

W/B	B (kg/m^3)	β	w_e (litres/m^3)	k_θ (eqn. (3))	w_f (litres/m^3)	w_f/B
0,75	240	0,80	144	0.27	105	0,44
0,65	275	0,80	137	0.32	93	0,34
0,55	330	0,75	134	0.37	85	0,26
0,45	400	0,70	127	0.45	70	0,18
0,30	500	0,50	103	0.51	50	0,10

The agreement between these values of freezable water and the experimental in Figure 1 is good. For the highest W/B-ratio the theoretical value is somewhat higher.

In the practical case, also some air-pores contain water. Therefore, the amount of freezable water is somewhat higher than the amount calculated by eqn. (2) or (3).

The high amount of freezable water also in dense concrete indicates that in fact all types of concrete are vulnerable to frost if they are completely saturated.

Pores in aggregate are almost always so big, that all water absorbed in them will freeze close to 0°C. This means that light-weight aggregate concrete will contain substantially more freezable water than what is given by eqn. (2) or Table 2.

All water in cracks and interfaces between aggregate and cement paste is also freezable.

2. Damage mechanisms

2.1 Damage mechanism 1; Freezing of a closed container

In the most simple case every small ”unit cell” or ”representative cell” of the concrete can be looked upon as a closed container. No water transfer is possible from the cell. The 9% volume increase when water is transformed into ice must therefore be taken care of within the cell. The size (width) of a representative cell is about 5 mm for cement paste and about 25 mm for concrete. The size is so small that the effect of temperature gradients in the cell can be neglected. All water in the cell is therefore assumed to be freezing simultaneously.

A simple model of the cell can be used for estimating the maximum pressures occurring during freezing; see Figure 2. The cell is supposed to be a hole-sphere of cement paste, which is *completely water-filled* and incompressible.

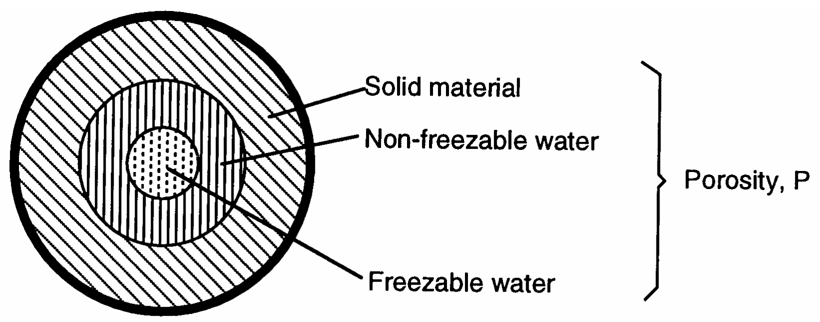

Fig 2: Illustration of damage mechanism 1; hole-sphere with impermeable wall.

All pore water is supposed to be contained in the hole, and all solid material in the wall. The pressure in the pore-water depends on the temperature. According to the phase diagram of water, the pressure in unfrozen water is increased by about 10MPa for each degree of temperature reduction. The following relation is valid for the tensile stress in the sphere wall, provided the wall is completely plasticised, and that the deformability of water and solid material is neglected.

Total fracture occurs when the tensile strength is transgressed.

$$\sigma \approx -10 \cdot \theta_f \cdot P^{2/3}/(1-P^{2/3}) \qquad (4)$$

where σ the tensile stress in the pore wall [MPa]
P the "freezable porosity" of the hole-sphere (r^3/R^3) [m^3/m^3]
θ_f the freezing temperature [°C]

The porosity of a cement paste with W/C=0,6 is about 60%. The tensile strength is not above 6 MPa. This means that the minimum possible freezing temperature is less than -0.5°C if the hole-sphere shall remain intact. Lower freezing temperatures will cause total destruction.

If a cement paste with tensile strength 6 MPa shall sustain -10°C the porosity must be below 1.5% (15 litres of freezable water.) If the temperature is -20°C the porosity must be below 0.5%. The very big pressure caused also by freezing of a small water volume is the reason why a small amount of water in a deep and narrow crack in rock can cause fracture of big rock volumes. The pressure arising at the site of ice formation is also transferred over big distance by hydrostatic action within the water-filled crack.

The calculations clearly show that a completely water saturated concrete cannot resist freezing without damage. Even a very low water content is enough to damage the concrete seriously. Therefore, the concrete must contain a certain amount of air-filled space. One can make an estimate of the maximum allowable effective degree of saturation of the concrete by neglecting the compressibility of solid material, water and ice:

$$S_{eff,CR} = 0,917 \qquad (5)$$

Where $S_{eff,CR}$ the maximum allowable effective degree of saturation defined by

$$S_{eff} = W_f/(W_f+a) \qquad (6)$$

Where W_f the volume of freezable water [m^3/m^3]
a the volume of air-filled pores [m^3/m^3]

Damage mechanism 1 can be applied to four cases:

Case1: Assessment of the absolute minimum required air content in a concrete.
The minimum air content is calculated by eqn. (5) expressed in the following way using eqn (6):

$$a_{min}=0.1 \cdot W_f \qquad (7)$$

For a concrete with w/c=0.75, with freezable water according to curve B in Figure 1, the minimum air content is 1vol-% or 10 litres/m^3.

Case 2: The effect of porous aggregate grains embedded in the concrete.

Such grains can, as a first approximation, be looked upon as closed containers from which water transfer to the cement paste is impossible. Aggregate grains having a total porosity above 0,5 to 1% and becoming water-filled to an effective degree of saturation higher than 0,92, when embedded in the paste, must according to the theoretical analysis performed, be very hazardous. This has also been found in practice; see Figure 3. Only aggregate pores smaller than about 1 µm become water-filled when the aggregate is completely embedded in the paste. Therefore, fine-porous natural aggregate, such as limestone and slate are exceedingly frost sensitive. Theoretical analyses of the stress conditions in the embedded aggregate, and in the surrounding cement paste during freezing, have been performed in /6/.

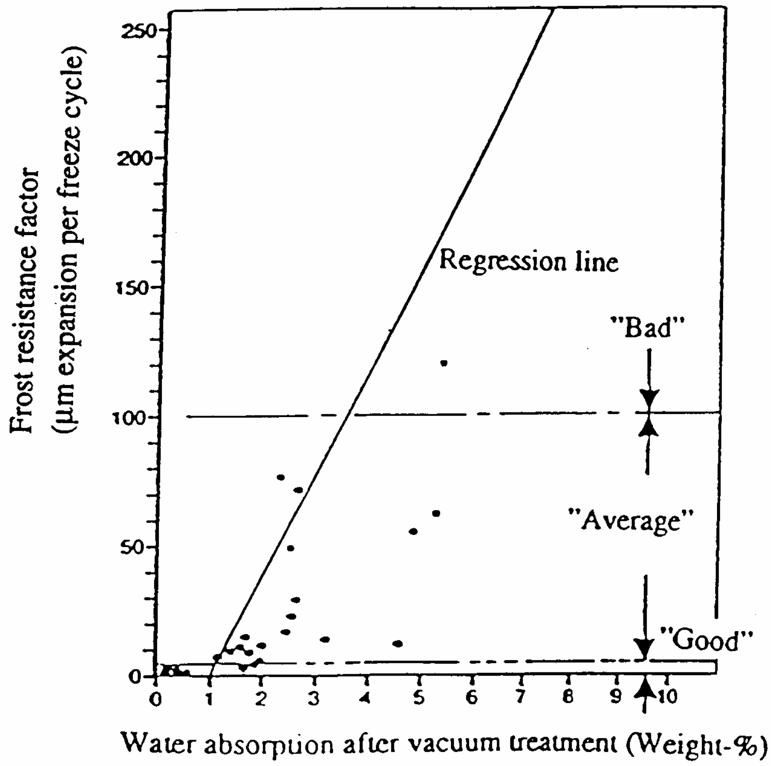

Figure 3: Expansion at freezing of concrete as function of the porosity of the aggregate; /7/.

3: High performance concrete with very low W/C-ratio.

It is sometimes claimed that one can avoid air in such a concrete due to its low amount of freezable water, or even lack of any freezable water. The calculations show, however, that also a very low amount of freezable water (about 5 litres per m^3) is sufficient to destroy the concrete if this can become fully saturated. Therefore, a certain, but often low, air content is required even in a very dense concrete.

4: Cracks and other defects.

Cracks, that are open to the surface of the concrete, can be assumed to be water-filled during very wet conditions. The stresses occurring as a consequence of freezing depend on the crack geometry, the crack frequency, and the possibility of water to be squeezed out from the crack during freezing. The worst case occurs when the crack is so deep that water cannot be squeezed out at the same time as water transfer into the cement paste matrix is impossible.

Then, very high pressures can be built up. Normally, however, the cement paste contains some air-filled pores to which the surplus water can be displaced. It is possible to make a simplified calculation of the maximum tolerable crack width using the following equation. It is based on the assumption that the air content in a cement paste slice of a certain critical thickness (D_{CR}/2) close to the crack wall shall be high enough to accommodate displaced water from half the crack volume and from the cement paste slice itself. D_{CR} is the so called critical thickness which will be described in the next paragraph. If 1 m^2 of crack surface is regarded the following relation is valid:

$$t_{max} = D_{CR}\,(a-0{,}09\cdot W_f)/0{,}09 \qquad (8)$$

Where
- t_{max} the critical crack width [m]
- W_f the freezable water in the cement paste [m^3/m^3]
- D_{CR} the critical thickness (m)
- a the air-filled pore volume in the cement paste [m^3/m^3]

For D_{CR}=1mm (see paragraph 3.2) and W_f=200 litres per m^3 of cement paste the following maximum allowable crack widths are valid:

* air content 6% (2% in concrete); t_{max}=0,5 mm
* air content 12% (4% in concrete); t_{max}=1,1 mm
* air content 18% (6% in concrete); t_{max}=1,8 mm

2.2 Damage mechanism 2; Hydraulic Pressure

It is a well-known fact that a much higher air-pore volume is needed in a concrete than that predicted by damage mechanism 1. This is explained by damage mechanism 2 according to which excess water caused by freezing cannot be accommodated at the freezing site, but has to expelled to an air-filled pore which is big enough to take care of it without causing destructive stresses. This water transfer occurs through a narrow and partly ice-filled web of capillary pores and gel pores. High pore-water pressure therefore arises and is transferred to the pore walls, which are exposed to tensile stresses. The pressure is often referred to as hydraulic pressure. The concrete is damaged when the tensile stress exceeds the tensile strength. The damage mechanism is visualised in Figure 4.

The maximum hydraulic pressure p_{max} can be calculated for the three simple geometry of completely saturated concrete bodies shown in Fig 5, /8/.

$$p_{max} = 0{,}09\cdot(dWf/dt)\cdot(1/K)\cdot f(X) \qquad (9)$$

Where
- dW_f/dt the rate of ice formation [$m^3/(m^3\cdot s)$]
- K the permeability [$m^2/(Pa\cdot s)$]
  (Defined by Darcy´s law: *flux[$m^3/(m^2.s)$] =K·gradient [Pa/m]*)

The function f(X) is a measure of the maximum distance that water has to be transferred until it reaches the periphery of the specimen. f(X) is different for different types of specimen geometry. Some examples are:

* A water saturated slice with thickness D; Figure 5a.

$$f(X) = D^2/8 \qquad (10)$$

* A water saturated sphere with diameter Φ; Figure 5b.

$$f(X) = \Phi^2/24 \qquad (11)$$

* A water saturated shell with thickness L surrounding an air pore with the specific surface area α; Figure 5c. The outer periphery is impermeable. This is the model used by Powers /17/ in his definition of the spacing factor; see paragraph 3.2 and Figure 14.

$$f(X) = [L\cdot\alpha/9+1/2]\cdot L^2 \qquad (12)$$

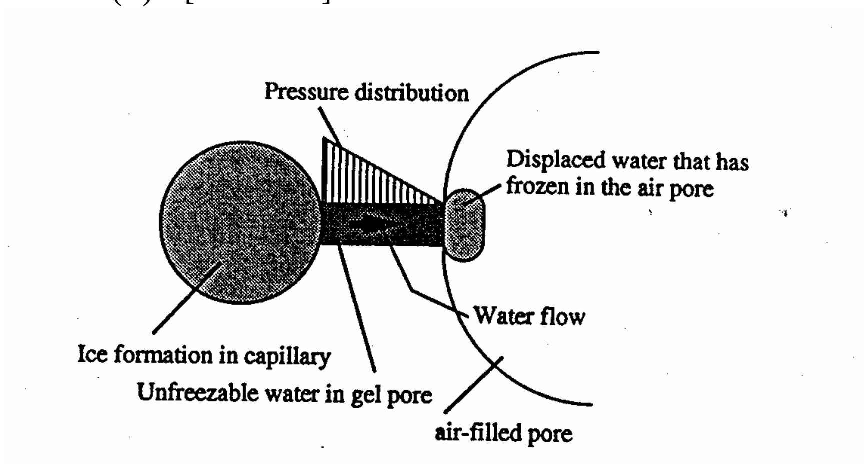

Figure 4: Illustration of damage mechanism 2.

Thus, the hydraulic pressure will increase with increasing rate of ice formation, with increasing size of the saturated volume, and with decreasing permeability. The concrete is damaged when the following condition is satisfied:

$$p_{max} = f_t \qquad (13)$$

Where f_t the tensile strength of the concrete.

Thus, there exist maximum material sizes, or critical distances, which must not be exceeded if the concrete shall be frost resistant. The following relations are valid for the three types of geometry in Figure 5:

* Water saturated slice:

$$D_{CR} = \{8\cdot f_t\cdot K/(0,09\cdot dW_f/dt)\}^{1/2} \qquad (14)$$

* Water saturated sphere:

$$\Phi_{CR} = \{24\cdot f_t\cdot K/(0,09\cdot dW_f/dt)\}^{1/2} \qquad (15)$$

* Water saturated shell, or Powers´ spacing factor:

$$L_{CR}^2\{L_{CR}\cdot\alpha/9+1/2\} = f_t\cdot K/(0{,}09\cdot dW_f/dt) \tag{16}$$

The value L_{CR} is also a function of the size of the air pore enclosed in the shell. Thus, L_{CR} is not the same true material property as D_{CR} and Φ_{CR}.

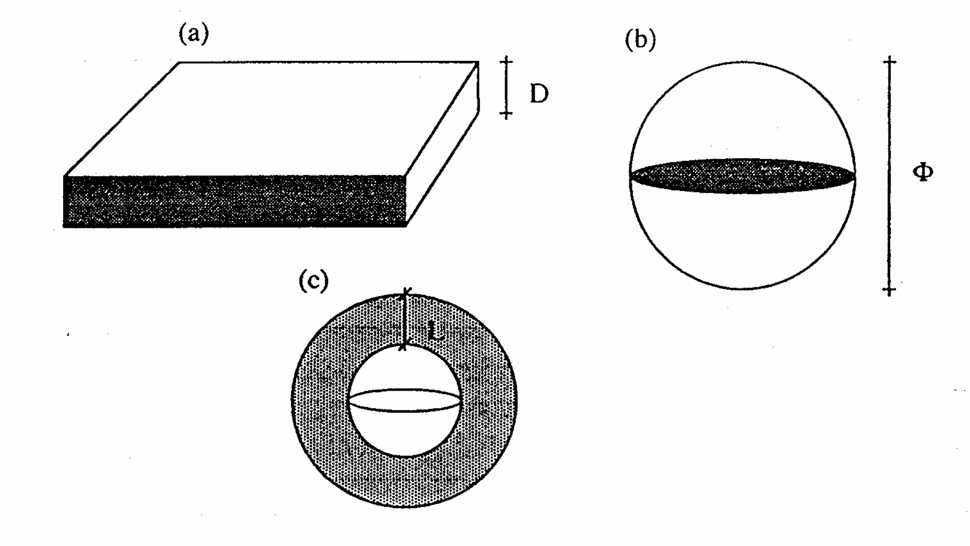

Figure 5: Different saturated concrete volumes

Equations (14)-(16) show that geometrical relations between the different critical sizes exist. The relation between the critical thickness and the critical spacing factor is.

$$D_{CR} = 2\cdot L_{CR}\{2\cdot\alpha\cdot L_{CR}/9+1\}^{1/2} \tag{17}$$

This relation is a function of the size of the spherical air-filled pore inside the shell. A typical value of α for a concrete exposed to normal conditions is 15 mm^{-1}. Then, if the critical thickness D_{CR} is **1 mm**, the critical spacing factor L_{CR} is **0,35 mm**. The critical spacing factor is therefore always smaller than the critical thickness.

The hydraulic pressure is only acting as long as new ice is formed. Therefore, it should vanish when the temperature is kept constant. The following relation is valid:

$$dW_f/dt = (dW_f/d\theta)\cdot(d\theta/dt) \tag{18}$$

Where $dW_f/d\theta$ is a material function that is only dependent of the amount of ice formed at each temperature. Hence, $dW_f/d\theta$ is a function of the pore size distribution; c.f. eqn. (1). The function $d\theta/dt$ is the rate of temperature lowering of the specimen, which is almost directly proportional to the rate of lowering of the outer temperature, and of the distance of the point in the material considered to the outer surface of the structure. Principally, according to eqn. (17), one should therefore obtain a length-change/temperature curve of the type shown in Figure 6a when the temperature is kept constant during a certain time. Due to the lack of ice formation during this period the specimen should contract. In reality however one has obtained curves of the type seen in Figure 6b according to which the concrete length is almost constant when the temperature is constant.

This does not necessarily contradict the hydraulic pressure mechanism. There might be a certain ice formation despite the fact that the temperature is constant. This can depend on a heat balance prevailing between latent heat developed due to freezing of some water, and heat loss to the environment. Besides, one might imagine that ice that was formed at higher temperatures "lock" the material structure making it impossible for the specimen to contract when hydraulic pressure disappears. This is a special case of damage mechanism 1, which is described above. A further possibility is that the material was at an earlier stage, permanently and irreversibly damaged so that it cannot contract.

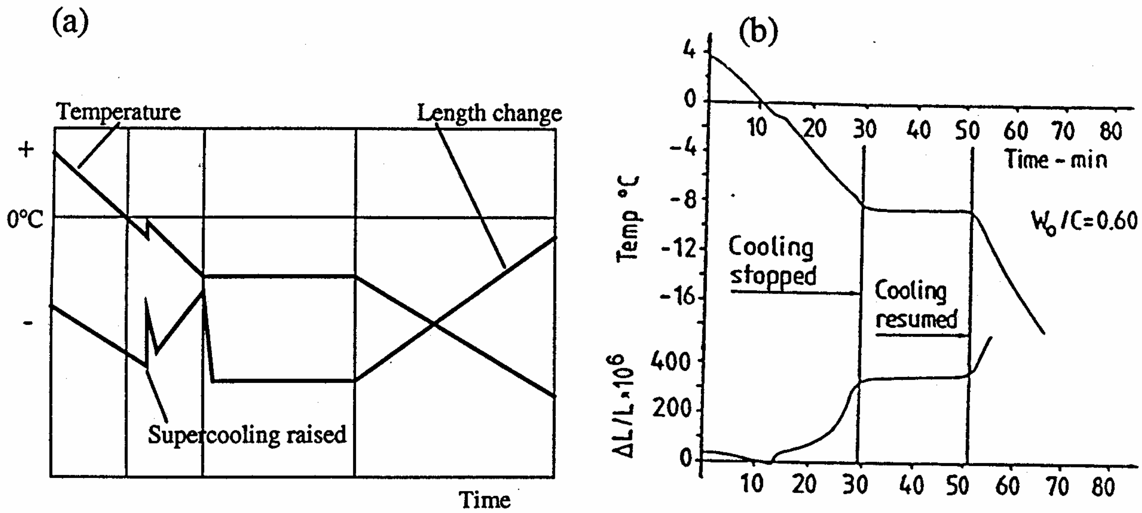

Figure 6: (a) Expected temperature-time and length-time curves at damage mechanism 2. (b) Measured temperature and length curves of a cement paste with the W/C-ratio 0,60. /9/.

According to eqn. (14)-(16), the pressure should be at its maximum when the rate of ice formation is at its maximum. This normally occurs at the beginning of the freezing process, at around 0°C. At later stages, the rate of ice formation is almost always much smaller. However, one must consider that the permeability is gradually decreased with decreased temperature due to the increased amount of ice formed inside the pore system. The reduced permeability might very well more than compensate for the reduced rate of ice formation. The fact that one often notices larger expansions of concrete at lower temperatures is therefore not necessarily a contradiction of the hydraulic pressure mechanism. This is visualised in Figure 7.

The critical size depends on the rate of ice formation, which is almost directly proportional to the rate of temperature lowering of the surrounding air or water.

Theoretically, according to eqn. (14) a doubling of the rate of temperature lowering gives a 30% reduction of the critical size. Damage mechanism 2 does therefore imply that the damage risk is increased with increasing rate of freezing.

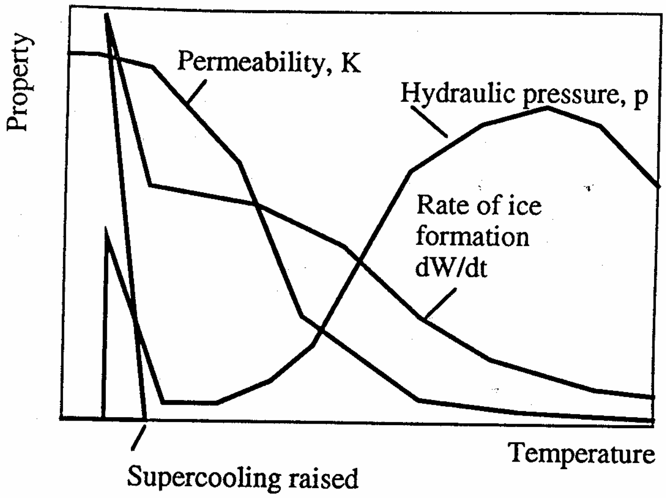

Figure 7: Hypothetical curves of the rate of ice formation, the permeability, and the hydraulic pressure.

Damage mechanism 2 is of special importance in two cases:

Case 1: The initial freezing.
Due to super-cooling, the pore water does not freeze until the concrete temperature is some degrees lower than the theoretical freezing point. This super-cooling can be 5°C or more. When freezing suddenly is initiated somewhere in the pore water, it spreads rapidly over a large concrete volume containing super-cooled water. Then, the concrete temperature rises momentarily to the real freezing temperature which is 0°C, or a few degrees lower. A large amount of ice is formed during a few seconds and there is often a rapid expansion of the specimen. This expansion normally is reversible and is due to a "pumping effect" when a large amount of water is expelled to air-filled pores during a short time. In most cases, this expansion is not large enough to destroy the concrete. The largest expansion normally occurs at a later stage at lower temperatures; see Figure 7.

Case 2: Concrete with high W/C-ratio.
In concrete with high W/C-ratio the freezable water content is higher than the non-freezable water content; see Table 2. Therefore, damage mechanism 3 described below cannot have the same significance as it might have in more fine-porous concrete. It is however not excluded that damage mechanism 2 is the dominant mechanism also for dense concrete with low W/C-ratio. The relative importance of damage mechanisms 2 and 3 has never been clarified.

2.3 Damage mechanism 3; Microscopic ice lens growth

Every concrete will, due to its fine pore structure, at the same time contain ice bodies in the coarser capillaries and in certain air pores, and unfrozen water in the finest capillaries, and in the gel pores. The lower the W/C-ratio, the larger the fraction of unfrozen water. This co-existence of water and ice makes a damage mechanism possible that is similar to the mechanism that causes frost heave in the ground. At any temperature below 0°C unfrozen water has a higher free energy content than ice. The ice bodies will therefore attract water. Therefore, a water transfer towards the freezing site occurs. The microscopic ice bodies in the capillaries will grow and thereby expose the pore walls to pressure. This means that the ice will also be exposed to pressure. The free energy of the ice body will therefore increase, at the same time as the free energy of the unfrozen water will decrease due to the drying effect caused by the water transfer. The growth of the ice body, or ”ice lens”, will not cease until the free energy of the ice is high enough to balance the free energy of the remaining unfrozen water. Before this occurs, pressures high enough to seriously damage the concrete can probably be built up. This is shown by the calculations below. The damage mechanism is illustrated in Figure 8.

It is difficult to quantify the pressure. It depends on the type of meniscus system ice-water-air that appears inside the concrete. It also depends on how big the drying effect is. The higher the content of unfrozen water in relatively coarse pores, the larger the possible water transfer, and the larger the pressures built up. For the idealised case in Figure 9a, in which an isolated ice lens surrounded by unfrozen water has access to unlimited amount of water, the following expression gives the pressure that can be exerted by the ice body.

$$p = (\Delta H/T)\cdot[\Delta\theta/(v_i - v_w)] \tag{19}$$

Where,

Where,	p	the pressure between the ice body and the pore wall [Pa]
	ΔH	the molar latent heat of fusion [$6\cdot10^6$ J/kmole]
	T	the actual temperature [$T=273{,}15-\Delta\theta$ °K]
	$\Delta\theta$	the actual freezing point depression of unfrozen water [°K]
	v_i	the molar volume of ice [$19{,}8\cdot10^{-3}$ m^3/kmole]
	v_w	the molar volume of liquid water [$18\cdot10^{-3}$ m^3/kmole]

Thus, the pressure will increase with decreasing temperature. At -20°C the pressure is 260 MPa.

Powers /10/ treats the case where the pressure from the ice body acts directly against the pore wall without a layer of unfrozen water between the solid wall and the ice body. This leads to considerably lower pressures, which can however still be big enough to cause damage.

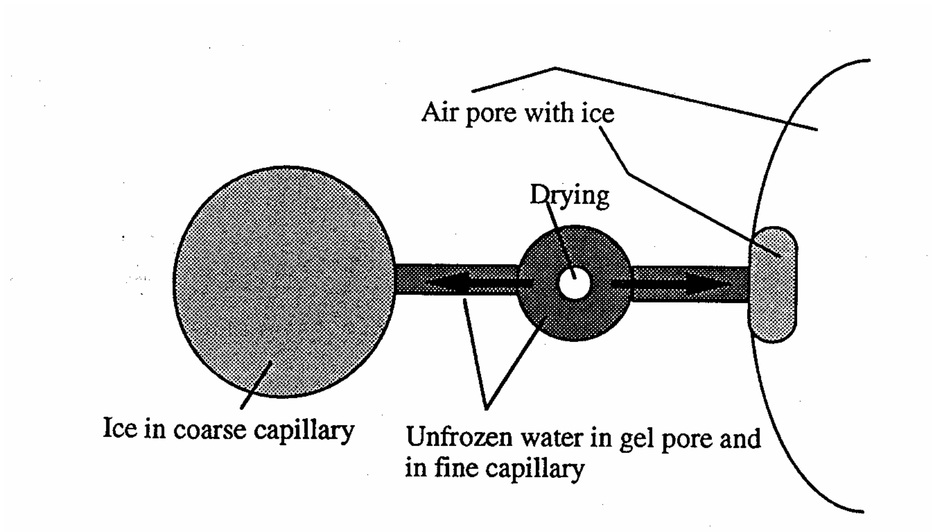

Figure 8: Illustration of damage mechanism 3.

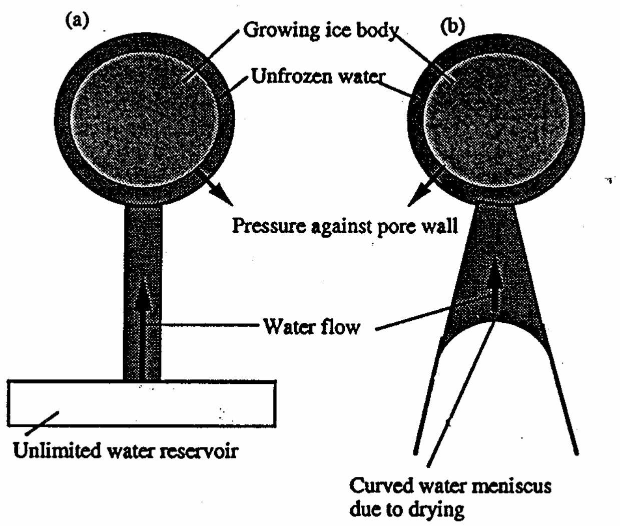

Figure 9: Model for calculating the pressure caused by microscopic ice lens growth. (a) Unlimited supply of unfrozen water. (b) Limited amount of unfrozen water; the drying effect.

Drying due to transfer of non-freezable water reduces the pressure. The following expression can be used for the model in Fig 9b.

$$p = (\Delta H/T)\cdot[\Delta\theta/(v_i - v_w)] - [v_w/(v_i - v_w)]\cdot p_d \qquad (20)$$

Where the second term on the right hand side is the reduction due to desiccation. p_d is the under-pressure in the water phase caused by drying. It depends on the curvature of the meniscus between water and air caused by drying, and can be described by the Laplace law:

$$p_d = 2\cdot\sigma/r_d \qquad (21)$$

Where σ the surface tension between air and water [$75\cdot10^{-3}$ N/m]
r_d the radius of the meniscus between air and water [m]

If the temperature is -20°C, as in the example above, and the drying has created an under-pressure which can be described by the meniscus radius 70Å corresponding to 85% RH according to the Kelvin equation, the pressure exerted by the ice body is now reduced from 260 MPa to about 45 MPa.

Pressure due to this mechanism appears as long as water transport to the ice lens is possible; i.e. as long as there is no energy balance between ice and water. Therefore, pressure can appear even when there is no lowering of the temperature. One practical example of this behaviour is seen in Figure 10.

During the very first freezing, taking place around 0°C, some water is transferred by hydraulic action to the air pores where it will freeze momentarily. Growth of these ice bodies will occur without pressure being exerted, since the pores are not completely filled by ice. Water transfer, therefore, primarily takes place towards these ice bodies. After a certain time, other ice bodies, being under pressure, will melt, and the melted water will be transferred towards the stress-free
ice in air pores. Therefore, the maximum pressure appearing in the cement paste depends on the possibility of water transfer towards air pores. The pressure will diminish when the distance between air pores is decreased. Damage mechanism 3 therefore also predicts the existence of *critical distances*, e.g critical thickness or critical spacing factor. This statement is supported by measurements; see Figure 11 showing length measurements of cement paste with different spacing factors. The larger the spacing factor, the bigger the expansion of the cement paste during freezing. At very low spacing factors, a considerable contraction takes place. This indicates that the contraction caused by drying due to water transfer dominates over the pressure caused by the growing ice lenses; the second term on the right hand side of eqn. (20) dominates over the first term.

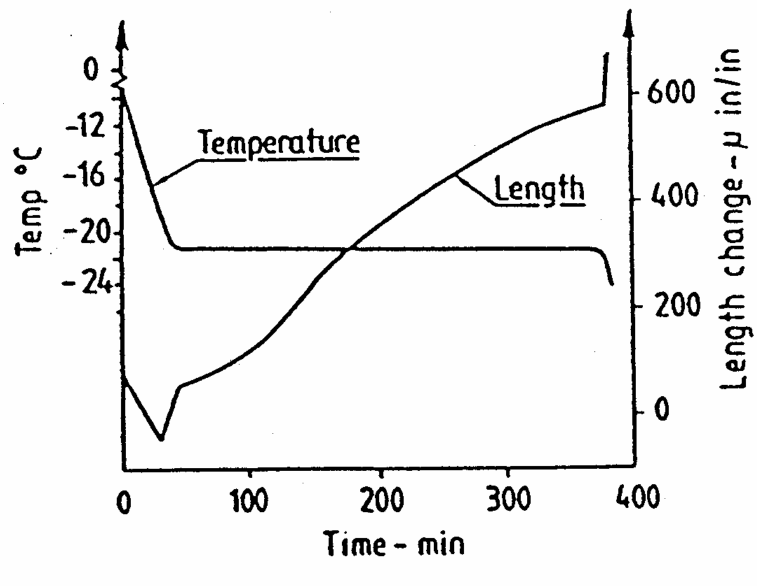

Figure 10: Measured temperature-time and length-time curves at freezing of a cement paste with the W/C-ratio 0,45; /9/.

Theoretically, damage mechanism 3 ought to be more pronounced the lower the freezing rate, and the longer the freezing period. Then, the pressure has more time to develop. This is a great difference between damage mechanisms 3 and 2. The latter is favoured by rapid freezing.

The damage mechanism has been treated by many authors. The first application to concrete was made by Powers & Helmuth in 1953; /9/.

Damage mechanism 3 is of importance in at least two cases:

Case 1: Concrete with low W/C-ratio.
Such concrete has large amount of non-freezable water; see Table 2. This means that large pressures can be built up before the drying effect limits the pressure.

Case 2: Freezing in the presence of de-icing salts or sea water.
This is treated in ANNEX B.

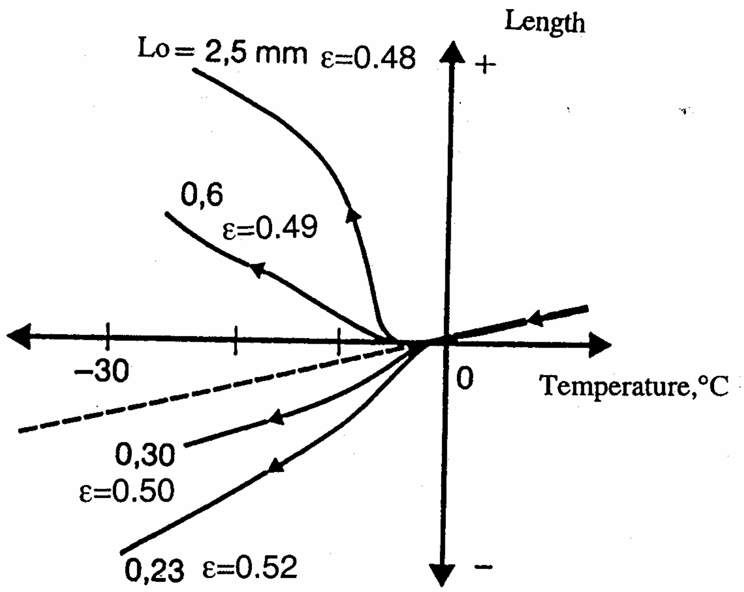

Figure 11: Effect of the Powers´ spacing factor on the length change at freezing of cement pastes with the W/C-ratio 0,60; /9/. (e, amount of evaporable water (m^3/m^3). Cooling rate 0,25°C/h)

2.4 Damage mechanism 4; Macroscopic ice lens growth

This mechanism is the same as that causing frost heave in the ground. The mechanism requires that a stable (immobile) ice formation front arises in the concrete, for example at its surface part, and that this front is continuously supplied by water from a ”reservoir” located to the unfrozen part of the concrete, or outside this. The mechanism is illustrated by Figure 12a.

The ice front, or the zero-degree front, is immovable when energy balance prevails; heat transferred to the front as a sum of the specific heat, and the heat of ice formation of water migrating to the front, must be exactly as high as the heat loss from the front to the surroundings.

Therefore, the first condition for macroscopic ice lens growth can be written:

$$dQ/dt = \{dQ/dt\}_f + \{dQ/dt\}_c \tag{22}$$

Where dQ/dt heat flow from the ice front [J/s]
$\{dQdt\}_f$ latent heat at freezing of water transferred to the ice front [J/s]
$\{dQ/dt\}_c$ heat capacity of water transferred to the ice front [J/s]

The two fluxes on the right hand side are determined by the permeability of the concrete, and by the driving force for moisture transport to the ice-front. This force is of exactly the same type as for destruction mechanism 3; i.e. energy differences between ice and unfrozen water. In mechanism 4 the driving force is strengthened, since the water is always warmer than the ice.

When the permeability is too low, the ice front can no longer be supplied with water, and the ice front will advance. Consequently, the ice lens segregation will cease. Powers /10/ postulates that the ice lens segregation should cease when the permeability of the concrete defined by eqn. (23) is smaller than $50 \cdot 10^{-12}$ [s]. The exact value of the critical permeability depends on the outer temperature conditions. When it is very cold outside the structure, the heat loss to the environment is high, and consequently the permeability must be high if the ice lens segregation shall be possible; /11/.

$$dq/dt = B \cdot dp/dx \tag{23}$$

Where dq/dt the water flux [kg/(m^2·s)]
dp/dx the pressure gradient [N/(m^2·m)]
B the coefficient of permeability [s]

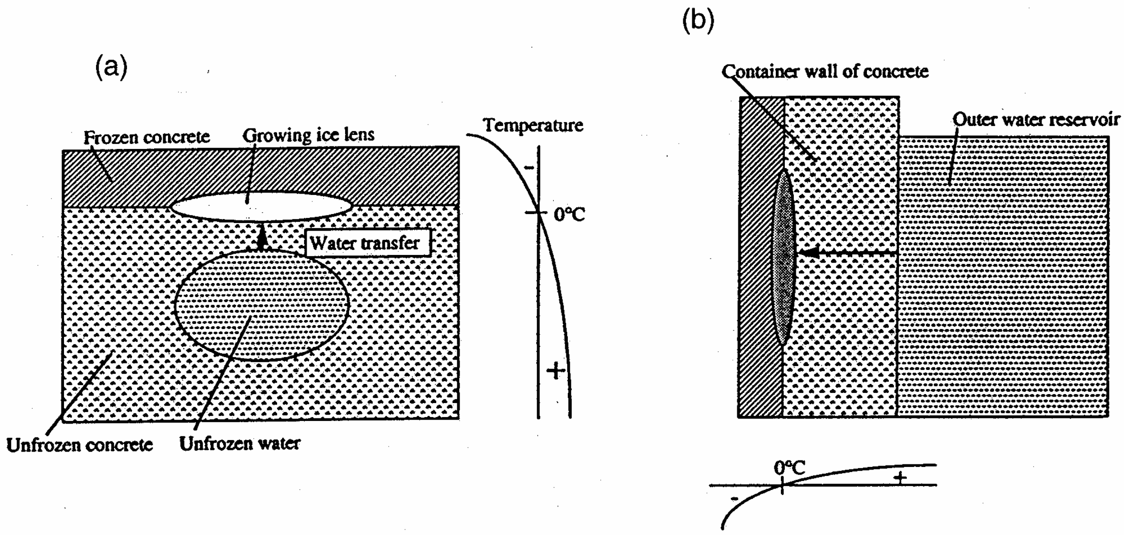

Figure 12: Illustration of damage mechanism 4.

The permeability levels just discussed are valid when water is only transported by energy differentials. Ice lens growth can occur at a much lower permeability if water is also transported by outer water pressure; Fig 12b. A normal well-cured out-door concrete is however dense enough to make ice lens growth impossible even during such conditions.
The driving force diminishes if the unfrozen part of the concrete is drying by the water transport to the ice front. The mechanism is exactly the same as that described under damage mechanism 3 above; eqn. (20).

A second condition for macroscopic ice lens growth is that the pressure in the ice lens is not so high that the lens penetrates an adjacent entrance pore, instead of growing in situ. This condition is given by the following relation; /12/:

$$p_{max} = 3,75 \cdot 10^{9}\{1-\exp(-4,54 \cdot 10^{-10}/r)\} \tag{24}$$

Where r the radius of an equivalent cylindrical entrance pore to the ice lens [m]

Damage can therefore be hindered if the concrete strength exceeds a certain value given indirectly by eqn. (24). (Similarly, frost heave in the ground can be stopped by exposing the ground to a pressure which is high enough.)

Already before the pressure given by eqn. (24) is reached, a stress relief can be obtained if the growing ice body can force the previously formed ice out of the material towards its warm face. For some coarse-porous materials such as clay brick one has observed long ”ice worms” coming out from the pores at the warm face. In such cases the material has had access to large amounts of unfrozen water during a long period.

The diameter of the coarsest pore in a concrete will probably always be smaller than
1 µm. Therefore, according to eqn. (24), a pressure of 3,4 MPa or more can arise before ice lens growth is stopped. Even before that happens the permeability criterion is normally ”activated”. A concrete with very low tensile strength and high permeability can, however, become damaged by macroscopic ice lens growth, especially if the concrete is exposed to outer water pressure like in Figure 12(b). The damage mechanism has been observed in real structures by Collins; /13/.

The theory was developed for frost heave in soil as early as during the 1930:ies. The first application to concrete was made by Powers, /10/. The damage mechanism is of special importance in two cases:

Case 1: Green concrete.
Normally, all criterions for macroscopic ice lens growth are fulfilled in a concrete that freezes a short time after casting. The ice formation occurs immediately below the surface that is cold, while the interior of the concrete, which is warmer, supplies water so that the ice lens can grow almost without restraint. The growth does not cease until the permeability is reduced below the critical value due to hydration, or until the interior of the concrete is dried so much that heat balance according to eqn. (22) can no longer be maintained. The ice lenses are often formed at the interfaces between coarse aggregate grains and the cement paste. De-lamination caused by early freezing has also been observed; /14/. If de-lamination is observed in a damaged structure, and if this was erected during winter under freezing conditions, one must suspect early freezing as the cause of damage.

Case 2: Concrete of low quality in hydraulic structures, such as dams.
In this case, the water reservoir on the warmer side provides the water required for the ice lens growth. In the worst case the concrete can be totally de-laminated.

3. The critical distance between air-filled pores

3.1 The critical flow distance

As shown in paragraph 2.1, a completely saturated concrete will be severely frost damaged by one single freeze-thaw cycle. However, a real material is never completely saturated, even during very moist conditions, but contains a number of air-filled pores that are bigger than about 10 µm; smaller pores becoming water-filled already during a very short water storage period, see paragraph 5.2. These pores save the concrete, provided their total volume is big enough. Theoretically, an air filled volume of about 1% would be big enough to accommodate the 9% increase in volume of frozen pore water; see paragraph 2.1. In reality, a bigger air volume is required, since water has to move from the freezing site to a pore, where it can be accommodated.

The two most important mechanisms for frost destruction of concrete are mechanisms 2 and 3 described above. Both predict the existence of a critical, or maximum allowable, distance between a place where ice is formed, and the nearest air-filled escape pore. Thus, both mechanisms predict the existence of a *critical flow distance,* or a critical spacing, between air-filled pores. By *critical* is meant *maximum acceptable*.

The distance between adjacent air-filled pores is not constant, but varies from pore to pore. It is however possible to derive an expression for the average air-pore spacing, taking into consideration also to the size distribution of air-filled pores. Such a formula is derived in /15/ and is shown in eqn. (25). It expresses the probability that an arbitrary point in the water-filled part of the cement paste shall be located within the distance L´ from the periphery of the nearest air-filled pore. Thus, the average total spacing between air-filled pores is 2·L´.

$$a\cdot\{1+L'\cdot\alpha+(L')^2\cdot\alpha\cdot[u]_1/[u]_2+1{,}33\cdot(L')^3\cdot\alpha\cdot[u]_o/[u]_2\}=C \qquad (25)$$

where a the total volume of air-filled pores [m^3/m^3 of cement paste (air-filled pores included in the volume of cement paste)]

$[u]_i$ the i:th statistical moment of the size distribution of air-filled pores

α the specific surface of the *air-filled pores*; i.e. the total envelope area of all air-filled pores divided by their total volume [m^{-1}]

C is a constant that is different for different probabilities that all points in the water-filled matrix shall be within the distance L´ from the nearest air-pore (be protected). For C=1 the probability is 63%; for C=2,3 the probability is 90 %, etc. In /15/ it is shown that the distance L´ for C=1 (63% probability) is equal to the so-called Philleo spacing factor; /16/.

Thus, the value L´ depends on the volume fraction of the material that is to be protected.

Eqn. (25) is a general expression giving a sort of *statistical spacing L'* between air-filled pores. The material is damaged if a critical value L_{CR}' is transgressed. Hence, the more water in the material, the bigger the distance L', and the bigger the risk that the material shall be damaged. This means that internal frost damage is to a very high degree *a moisture mechanics problem*. There is no frost damage unless the water content is so high that the critical spacing is transgressed.

The critical flow distance is visualised in Figure 13.

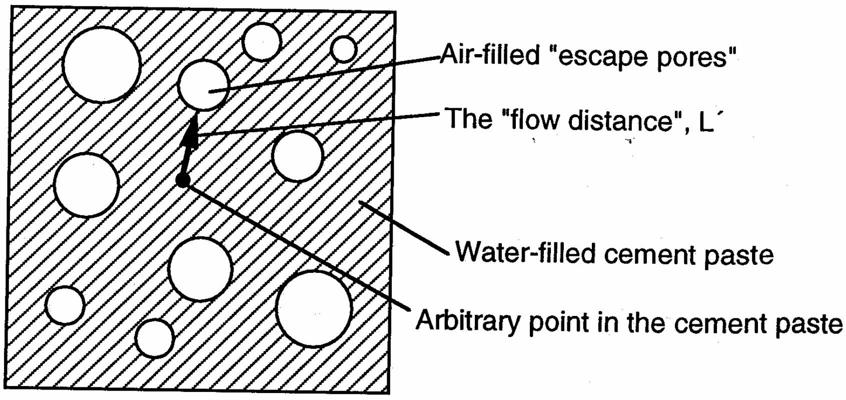

Figure 13: The critical flow distance

3.2 The critical Powers spacing factor

An approximate way of expressing the distance between air-filled pores is to assume that all air-filled pores are of equal size and placed in a loose-packed array, every air-pore being surrounded by a cubical shell consisting of water saturated cement paste. The biggest distance water has to be transported during freezing corresponds to the distance between the corner of the cube and the periphery of the air-pore. The model is shown in Figure 14. The distance is calculated by

$$L=(3/\alpha)\cdot(1{,}4(1/a)^{1/3}-1) \qquad (26)$$

Where a and α have the same meaning as in eqn. (25).

Geometrical relations exist between the critical spacing factor and other critical sizes, like the critical thickness D; /8/. Probably, the relation is to a certain extent depending on the major destruction mechanism. For the hydraulic pressure mechanism, the relation between the critical thickness of a slice, D_{CR} -eqn.(14)- and the critical spacing factor, L_{CR} -eqn. (16)- is given by eqn. (17).

Thus, the relation between L_{CR} and D_{CR} is also a function of the size of the air-filled pores, which is expressed in terms of its specific surface α. For normal concrete, D_{CR} is of the order 1 mm, and the specific surface of air-filled pores is often about 15 to 20 mm^{-1}. Thus L_{CR} is of the order 0.35 mm.

This model was first used by Powers, /17/, and is therefore often called the *Powers´ spacing factor*. Powers himself, however, included all so-called air-pores in calculating the spacing factor. He also included air-pores that were water-filled during natural conditions. This means that the critical Powers spacing factor, as defined by Powers, is considerably lower than the "true" critical spacing factor in which only air-filled pores are included in the values of α and a. As shown in paragraph 7.2 it is of the order 0.22 to 0.25 mm.

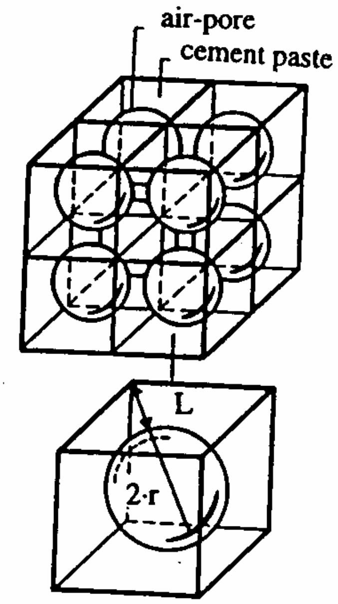

Figure 14: Definition of the Powers spacing factor

4. The critical water content

4.1 Principles and method of determination of the critical water content

When concrete is exposed to free water, this is absorbed; first in the gel pores, capillary pores, and the finest aggregate pores, and when these are completely filled, also in the air-pores, coarser aggregate pores and "defect pores" (interfaces and cracks etc). The process of water absorption in air-pores and other coarse pores is described in paragraph 5.

Due to the gradual absorption in the air-pores, the average spacing between these gradually increases. Thus, the flow distance as described by eqn. (25), or the Powers spacing factor as described by eqn. (26), gradually increases. At a certain water content, the actual spacing equals the critical. This water content is the *critical water content*. The effect of water content on the residual spacing between air-filled pores and the occurrence of the critical moisture content is visualised in Figure 15.

Therefore, for each concrete there exists an individual maximum allowable, critical, water content. This can be defined in terms of a moisture content w, moisture ratio U, or a degree of saturation S:

$$w=W_e/V \quad [kg/m^3] \tag{27}$$

$$U=W_e/Q_d \quad [kg/kg] \tag{28}$$

$$S=V_w/P \quad [-] \tag{29}$$

where W_e the weight of all pore water [kg]

V the total volume of concrete[m^3]

Q_d the dry weight (+105°C) [kg]

V_w the volume of all evaporable water in the concrete (in the cement paste, in partly water-filled air-pores, in aggregate pores, in interfaces, etc [m^3]

P the total volume of all pores in the concrete [m^3].

The critical water content, w_{CR}, U_{CR} or S_{CR}, can be determined experimentally by a freeze-thaw test of sealed specimens which are adjusted to individual water contents before the test, and which are then exposed to a low number of freeze-thaw cycles; /18/. A mechanical property, like strength or E-modulus, is determined before and after the test. Another possibility is to measure the length change caused by e few freeze-thaw cycles. An example of an experimental determination of S_{CR} of concrete is shown in Figure 16. Other examples are shown in Figure 17.

The value of the critical water content is defined as the breaking point in a diagram where damage is plotted against the water content prevailing during the freeze-thaw test. As a measure of ”damage”, strength loss, permanent expansion, or loss in (dynamic) E-modulus can be used. The last possibility is the most rational one because it is sensitive to all types of damage, it is rapid, and it does not destroy the specimen making it possible to use this for new freeze-thaw exposure.

The critical moisture content can also be calculated theoretically; /19/.

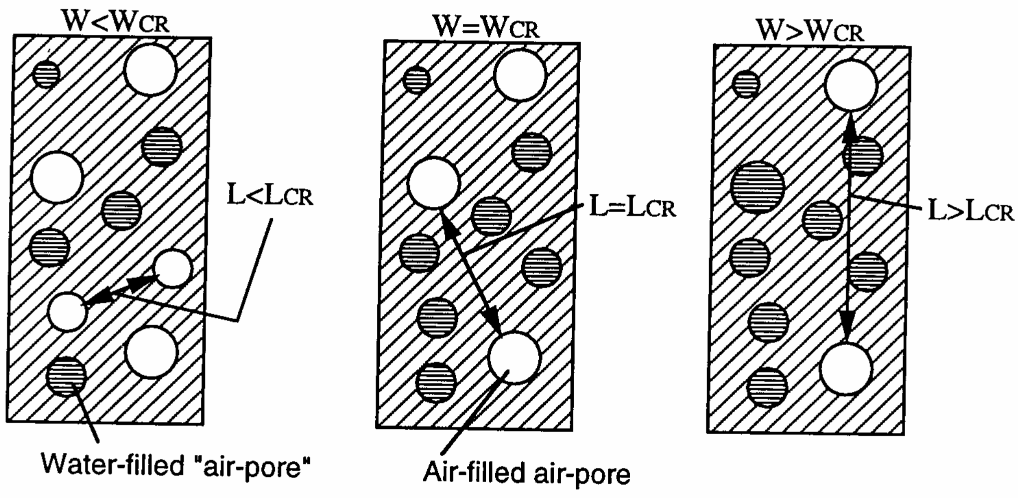

Figure 15: A gradually increased water content gives a gradually increased air-pore spacing. At a critical water content the critical air-pore spacing is reached.

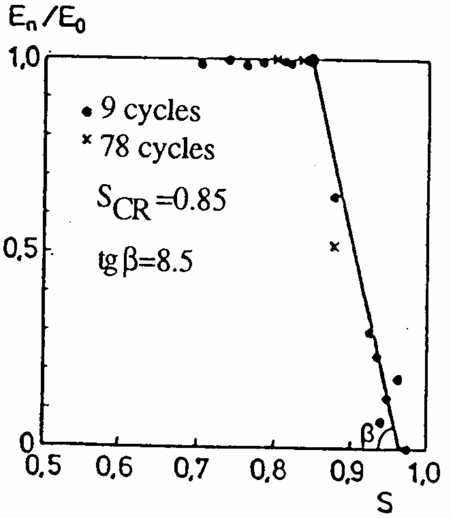

Figure 16: The critical degree of saturation of a concrete with w/c-ratio 0.45 and air content 6%; /25/.

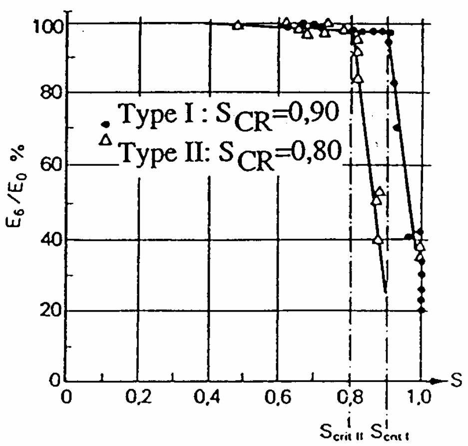

Figure 17: The critical degree of saturation of two concrete types; Type I, non-air-entrained, Type II, air-entrained; /20/.

4.2 Effect of the rate of freezing on the critical water content

It has been demonstrated experimentally, that the value of S_{CR} is almost completely independent on the rate of freezing, see Figure 18; /20/. Other experiments confirming this can be found in /21/. Normal variations in the freezing rates in the surface part of a concrete is between 1 and 3°C/h. This small variation has no important effect on the critical moisture content, and therefore it does not have to be considered.

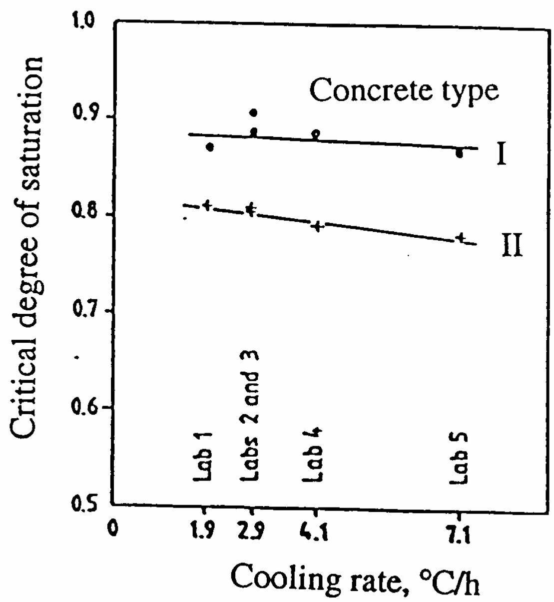

Figure 18: Effect of the rate of freezing (between 0°C and -10°C) on the critical degree of saturation of two types of concrete ; /22/.

In *freeze-thaw tests* it is often found that the freezing rate has a considerable effect on the result. There is no unambiguous answer, however, to whether an increased freezing rate is negative or positive. Such tests are always performed as ”open” tests, where water has the possibility to enter and leave the concrete during the test. Therefore, the water content in the concrete will be dependent on the way the test is performed. In a ”dry” test, in which the concrete has a possibility to dry during the freezing phase, an increased freezing rate is often negative, since the drying period is short. In a ”wet” test on the other hand, in which the concrete is stored in water (or ice) during the whole freeze-thaw cycle, a decreased freezing rate is negative, since the water content will increase steadily. These effects of the freezing rate have nothing to do with the critical moisture content, only with the moisture content reached in the concrete; the *actual moisture* part of the frost resistance problem, see paragraph 6.

4.3 Effect of the number of freeze-thaw cycles (fatigue) on the critical moisture content

The critical moisture content, like S_{CR}, is independent on the number of freeze-thaw cycles. This is seen in Figure 16 where the increase from 9 freeze-thaw cycles to 78 did not change the location of the breaking point. It is even more clearly seen by Figure 19 where the result of a freeze-thaw test of moisture sealed cement mortar specimens is shown. The specimens have been exposed to an increasing number of freeze-thaw cycles while the moisture content has been kept constant in each specimen.

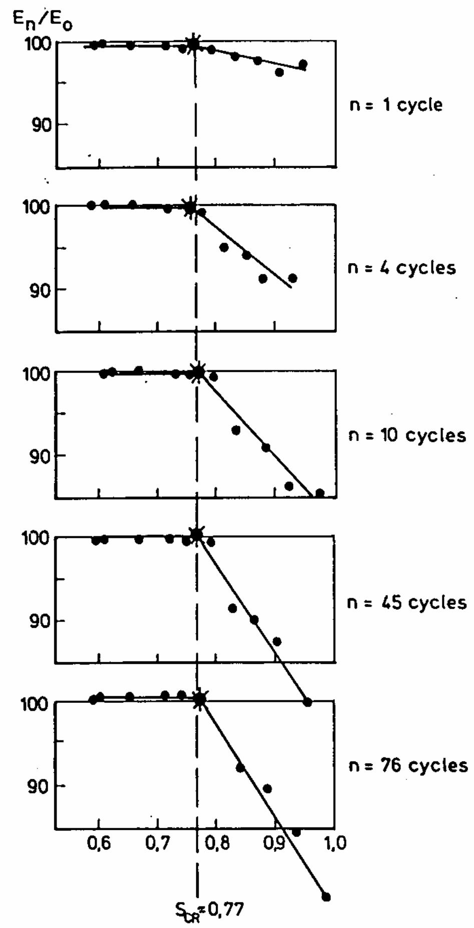

Figure 19: Effect of the number of freeze-thaw cycles and of the degree of saturation on the frost damage of cement mortar; /23/.

Another example is shown in Figure 20. The length change has been monitored during the freezing phase of a freeze-thaw test of a concrete specimen. The critical degree of saturation, which is defined as the degree of saturation below which there is no expansion, is uninfluenced by the number of freeze-thaw cycles.

Thus, there is no true fatigue involved in freeze-thaw damage, since only a few freeze-thaw cycles are able to destroy the concrete if the water content is high enough. There is a certain "low-cycle" fatigue effect, but only when $S>S_{CR}$. The fact that damage in a traditional "open" freeze-thaw test often increases with increasing number of freeze-thaw cycles is not depending on fatigue, but on a gradual water uptake. During each new cycle the moisture content is higher than it was in the previous cycle, especially in the surface part of the specimen. Therefore, damage is gradually increasing. This fatigue effect is discussed in /24/.

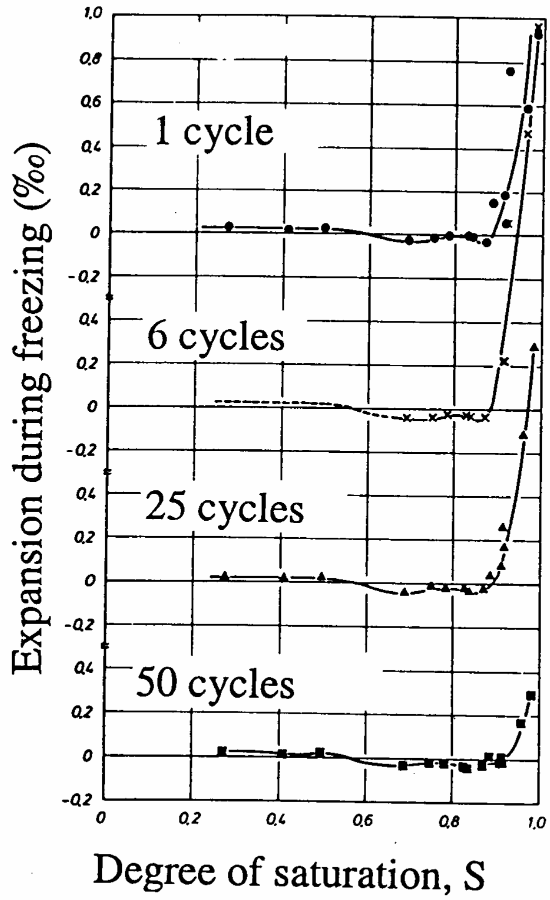

Figure 20: Effect of the number of freeze-thaw cycles and degree of saturation on expansion during closed freeze-thaw of a concrete; /21/.

Consequently, no frost damage occurs if $S<S_{CR}$, independently of the number of freeze-thaw cycles. At $S>S_{CR}$ damage is bigger the more S_{CR} is transgressed. For a given number of freeze-thaw cycles, damage seems to be directly proportional to the amount by which S transgresses S_{CR}. Thus, the following "damage" criterion can be used:

$$S \leq S_{CR}: \quad \text{Damage} = 0 \tag{30a}$$

$$S > S_{CR}: \quad \text{Damage} = K_N(S-S_{CR}) \tag{30b}$$

Where K_N a "coefficient of fatigue" which depends on the number of cycles with $S>S_{CR}$.

In Figure 21, data from Figure 19 have been used for defining K_N according to eqn. (30b). "Damage" is based on the relative change in dynamic E-modulus after N cycles. Thus, "damage", D, is defined as:

$$D=(E_o-E_N)/E_o \tag{31}$$

Where E_o the dynamic E-modulus before freeze-thaw

E_N the dynamic E-modulus after N freeze-thaw cycles.

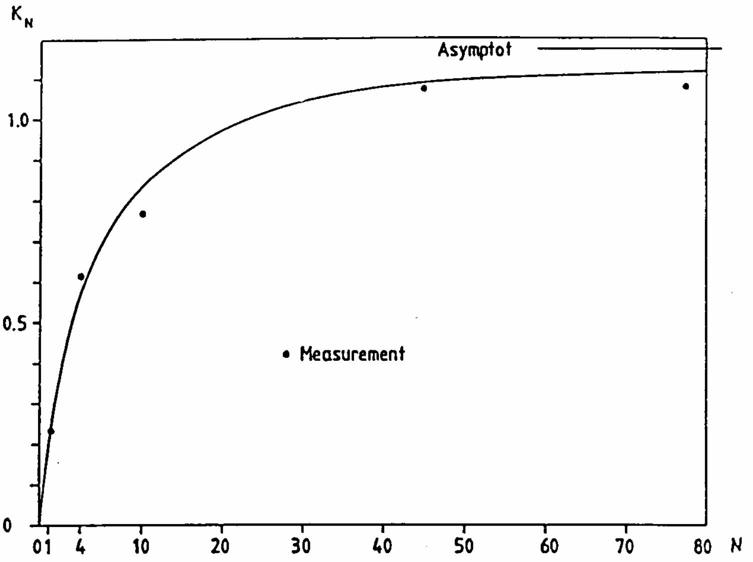

Figure 21: Plot of the coefficient of fatigue defined by eqn. (30b) based on the experimental data in figure 19.

Figure 21 shows that K_N is approaching a ”fatigue limit” within about 70 cycles. K_N can be described by an equation of the following type:

$$K_N=A\cdot N/(B+N) \tag{32}$$

Where, A the fatigue limit, giving the maximum possible damage when N is approaching infinity. A is different for different materials

B coefficient which is different for different materials

Probably, A and B are individual for each concrete. For the cement mortar in Figure 19, A=1.2 and B=4. For concrete, the coefficient A seems to be higher. A-values of the order 12 are reported in /24/. This gives considerably bigger damage after few cycles.

5. Water absorption in concrete

5.1 Water absorption and frost resistance

Fig 22 shows a representative unit volume inside a bigger concrete member. The unit volume is damaged if its actual moisture content transgresses the critical moisture content at the same time as freezing occurs. Macroscopic damage is observed when a sufficiently big number of unit volumes have been damaged.

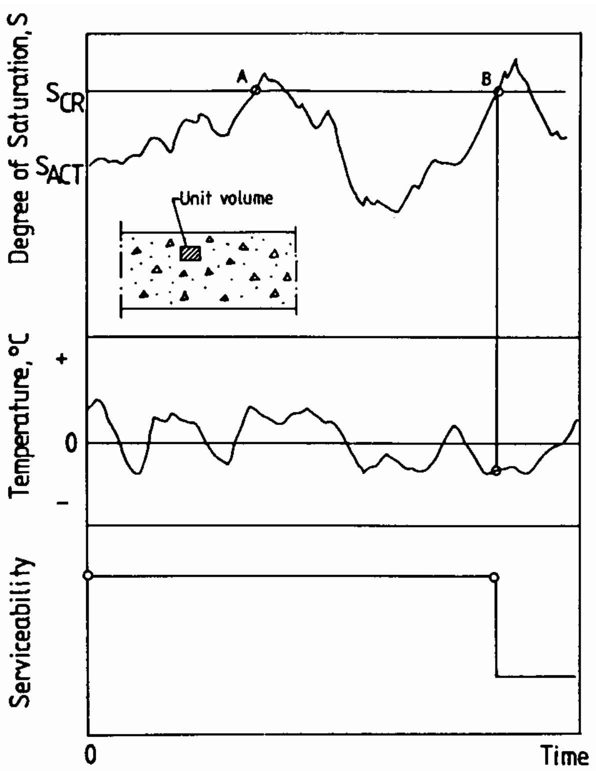

Figure 22: A representative volume inside a concrete structure. Evolution of the moisture content and frost damage in the volume.

This means that frost damage is related to the moisture conditions in the structure. Therefore, in order to estimate the risk of frost damage, it is necessary to predict the moisture field in the material over the years. The risk of frost damage, P_D, in a certain materials volume can be calculated by:

$$P_D = P\{W_{ACT} > W_{CR}\} = \int_{o}^{W_{max}} F(W_{CR}) \cdot f(W_{ACT}) \cdot dW \qquad (33)$$

where
- $F(W_{CR})$ the distribution function of the critical moisture content
- $f(W_{ACT})$ the frequency function of the moisture content in the structure
- W_{max} the moisture content at complete saturation (S=1)

Thus, both W_{CR} and W_{ACT} are assumed to be stochastic variables; W_{ACT} is of course much more variable than W_{CR}.

Eqn. (33) shows that *frost resistance is mainly a moisture mechanics problem*. Moisture influences both the 'load part', W_{ACT}, and the 'resistance part', W_{CR}, of the frost resistance problem. Eqn. (33) is illustrated in Figure 23.

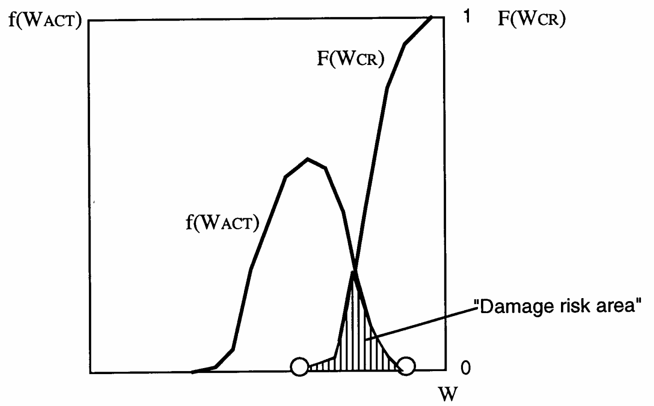

Figure 23: The frost damage risk defined by eqn. (33).

5.2 Water absorption before and between freezing

5.1.1 Water absorption in the capillary pore system

When concrete is exposed to liquid water, all gel pores and capillary pores in the surface part of the structure are filled almost immediately. After longer absorption times, also all capillary pores in the interior parts of the concrete will become water-filled by capillary suction.

One can therefore assume that the entire pore system in concrete, except the air-pores, are often water-filled during moist outdoor conditions. This is the reason why air-pores are needed to protect the concrete.

The water absorption properties of a concrete can be investigated by letting a thin slice of the concrete suck water from a free water surface. Then, a moisture uptake curve of the type shown in Figure 24 will be obtained. The breaking point in the diagram corresponds to the situation when all gel and capillary pores are water-filled. This is seen in Figure 25 where the theoretically calculated air-pore volume, based on the assumption that the breaking point corresponds to complete filling of all gel and capillary pores, is compared with the actual measured air-pore volume. The agreement is excellent.

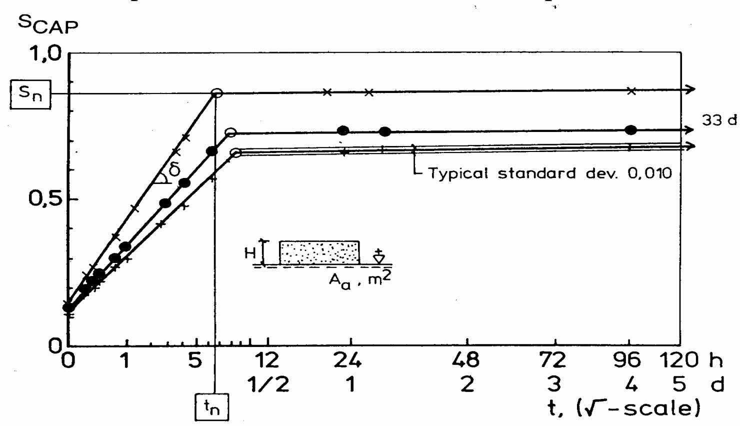

Fig 24: Water absorption curves in 25 mm thick slices of a number of concrete, /25/.

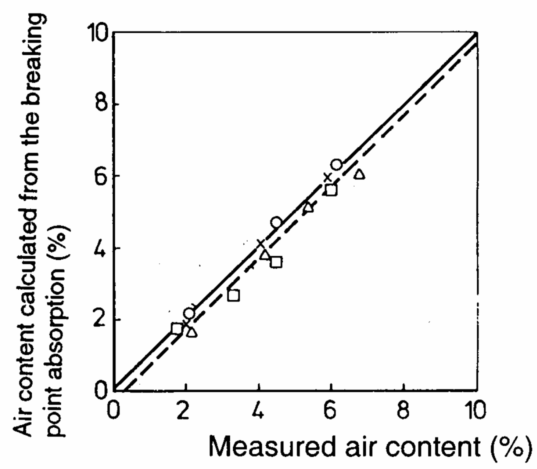

Figure 25: Experimental relation between measured air content and air content calculated from the breaking point in diagrams of type Figure 23 assuming the breaking point corresponds to complete filling of all gel and capillary pores.

5.1.2 Water absorption in the air-pore system

Not only gel and capillary pores can be water-filled, but also part of the air-pore system, provided the outer conditions are moist enough. Thus, the air-pore system becomes more or less inactivated with time. This was illustrated in Figure 15. The air-pore absorption can be treated theoretically, and also be observed experimentally. It is input in a service life prediction as described in paragraph 6.2, and in a prediction of the future frost destruction as described in Appendix C. The air-pore absorption mechanism will be described very briefly.

When the concrete is placed in water, as in a test illustrated by Figure 24, air will be entrapped in coarse pores surrounded by finer continuous pores; /19/. Thus, in all entrapped and entrained air-pores, being bigger than 1μm, an air-bubble is enclosed, since the air-pore is surrounded by a fine-porous cement paste matrix, in which the biggest capillary pore is less than 0.1 μm. The air bubble is exposed to an over-pressure caused by the curved meniscus between the air bubble and the water. The bubble will therefore be compressed. The relation between the volume of the compressed air bubble, and the total volume of the empty pore can be described by Boyle´s law. The following relation is obtained:

$$(V_1/V_o)^{1/3}\cdot(V_o/V_1-1)=2\cdot\sigma/(10^5\cdot R) \qquad (34)$$

where
V_o the pore volume [m^3]
V_1 the volume of the compressed air bubble inside the pore [m^3]
R the radius of the empty pore [m]
σ the surface tension between air and water [N/m]
10^5 the atmospheric pressure [Pa].

This equation shows that *pores with radii smaller than 0.1 mm (like capillary pores) will become completely water-filled almost directly when the concrete is exposed to water*; the residual volume of the compressed air-bubble is only 1.8% of the total pore volume. Therefore, such pores are almost always saturated during normally moist conditions.

An air-pore with radius bigger than 10 µm will stay air-filled for long time also when the concrete is stored in water; the volume of the compressed air-bubble in a pore with radius 10 µm is as high as 87% of the pore volume.

The over-pressure inside the air bubbles makes air gradually dissolve in the surrounding pore water, however. The solubility of air increases linearly with increased pressure. At +20°C the solubility, s, of air in water is $s=2,5\cdot 10^{-7}\cdot P$ kg/m^3 where P is the air pressure inside the air-bubble. This is given by the Laplace law

$$P=10^5+2\cdot\sigma/r \tag{35}$$

where 10^5 the normal atmospheric pressure [1 bar]
r the radius of the enclosed air-bubble [m]

This dissolution of enclosed air into the pore water implies that there will be a gradual and slow transfer of air from every bubble to adjacent coarser air bubbles. The driving force for this transport of air is the difference in air pressure, and hence the difference in the solubility of air. The smaller the bubble, the bigger the pressure and solubility. Consequently, there will be a complicated network of inter-pore diffusion of air. It can be shown that this inter-diffusion can cause a net reduction in air volume, /19/. Therefore, it causes a *water uptake* in the bulk material, even if no air is transferred to the surface of the material. However, there is also transfer of air to the outer surface. Principally, after long time, the total volume of water uptake is equal to the total air volume transferred to the surface and leaving this. The time process of air dissolution and water uptake is theoretically described in /19/.

The dissolution/absorption process is very slow. It occurs after the breaking point in a water absorption experiment of the type shown in Figure 24. Experimental observations and theoretical considerations indicate that the long-term absorption after the breaking point can be described by (in terms of degree of saturation):

$$S=A+Bt^C \tag{36}$$

where A the degree of saturation at the breaking point
B coefficient determined mainly by the diffusivity of dissolved air, and the air-pore size distribution (the higher the diffusivity, and the finer the air-pore system the higher the value of B)
C coefficient determined mainly by the air-pore distribution

For a concrete with non-porous aggregate, A corresponds to total water-filling of all gel and capillary pores; i.e.

$$A \approx P - a \tag{37}$$

Where P the total porosity [m^3/m^3]

a the total air-pore volume [m^3/m^3]

5.3 Water absorption during freeze-thaw and its effect on frost damage in a freeze-thaw test

When freeze-thaw takes part during very moist conditions, extra water might be forced into the concrete so that the next cycle will occur at a higher moisture level than the previous cycle. This has frequently been observed in freeze-thaw tests. Examples are given in Figure 26. It probably also occurs during natural freeze-thaw.

The mechanism behind this water absorption is not fully clarified. It, however, occurs in the initially air-filled pore system, since this is the only place where water can be accommodated. It also occurs in cracks formed during freeze-thaw. At constant temperature above zero, the water absorption is caused by the dissolution of air enclosed in ”air-pores”, described above; /19/.

During the thawing phase in water, this might be sucked into the material by different mechanisms. Since ice-bodies contained in bigger pores has lower free energy than unfrozen water in finer pores, there is a certain internal desiccation of the material during the freezing phase; see Damage Mechanism 2 in paragraph 2.3. During thawing, the material is therefore able to suck water in order to restore saturation of the desiccated pores. The mechanism is only active in fine-porous materials containing a large fraction of non-freezable water.

Another possibility is that water is sucked into the material when ice inside the pores melts. The volume of the melted water is lower than the initial volume of ice.

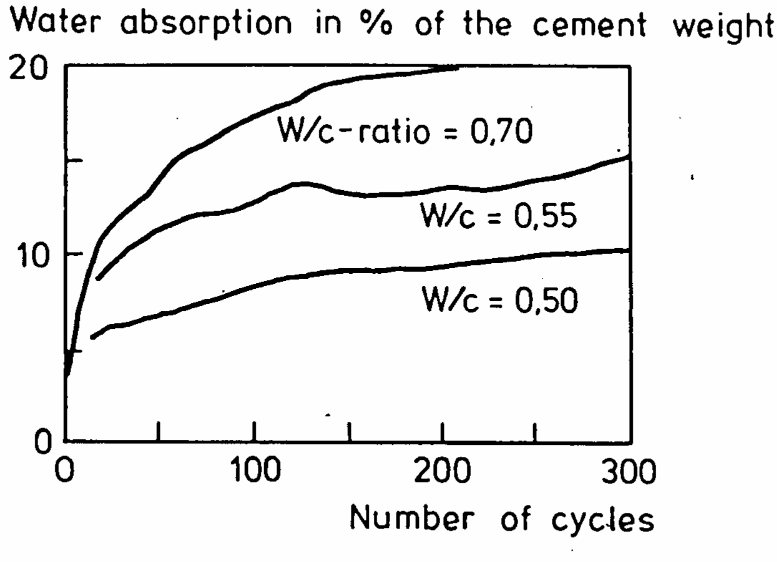

Figure 26: Water absorption in three concrete types at freeze-thaw in water; /26/.

Another mechanism for water absorption during thawing is that, when the specimen warms from its lowest temperature, the contraction of the ice phase is bigger than the contraction of the solid material. Therefore, theoretically, there is a possibility of a certain absorption, provided the specimen is placed in unfrozen water; /27/.

When the material is frost damaged, the cracks formed will be rapidly filled by water. Therefore, one can assume that the absorption rate is somewhat bigger in the frost damaged material than it was before frost damage occurred.
A special case is salt frost scaling, where the material surface is exposed to a salt solution, normally NaCl. Then, ice bodies formed close to the surface will absorb water from the solution, since this contains unfrozen liquid down to the eutectic temperature (-21°C for NaCl.) The driving force for water absorption is the free energy differential between liquid and ice. Salt scaling is presented in ANNEX B.

In a traditional freeze-thaw test, and sometimes also in the real structure, the water content in the specimen will therefore gradually increase. Thereby the internal stresses during freeze-thaw will gradually increase, provided the critical moisture condition is transgressed. Examples of the gradual increase in damage during traditional moist freeze-thaw test of cement mortars are shown in Figure 27. For some mortars, deterioration starts already at the first cycle For other mortars, a certain number of freeze-thaw cycles are needed.
The result in Figure 27 can be interpreted in the following way; see Figure 28 and /24/.

1: When the test starts, the initial moisture content, S_o, is below the critical value. Consequently no damage occurs during the first cycles; points 1, 2, 3 ,4 in Figure 28.
2: Due to water uptake during and between the freeze-thaw cycles, the water content is gradually increasing; Figure 28(a). Finally, the critical moisture content is reached in the whole or parts of the material, point 5 in Figure 28. The time and cycles needed for this to happen depends on the water uptake during each cycle, ΔS, and on the difference between the initial moisture content S_o and the critical moisture content S_{CR}. Thus, a higher number of cycles are needed for materials which are highly frost resistant than for materials with low degree of frost resistance. This explains why the different materials in Figure 27 start to deteriorate after different number of cycles.
3: Already one cycle after the critical moisture content has been reached, the water content is so high that frost damage occurs; point 6 in Figure 28. The amount of frost damage depends on the amount by which the critical water content is transgressed, and is given by the expression $D=K_N(S-S_{CR})=[A\cdot 1/(B+1)]\cdot(S-S_{CR})$.
4: Due to frost damage, the water absorption during each cycle probably increases in comparison with the absorption before the critical moisture content was reached. This is visualised by the steeper water uptake curve in Figure 28(a).
5: Due to the extra water absorption, and the increasing number of cycles, damage after each number of cycles is determined by different damage lines; point 7 is on the line for 10 cycles, point 8 is on the line for 20 cycles, point 9 is on the line for 50 cycles.

Thus, the gradual damage observed in open freeze-thaw tests is not a consequence of fatigue, but of a gradual increase in moisture content above the critical. There is a certain *low-cycle fatigue*, but it is limited to the first cycles.

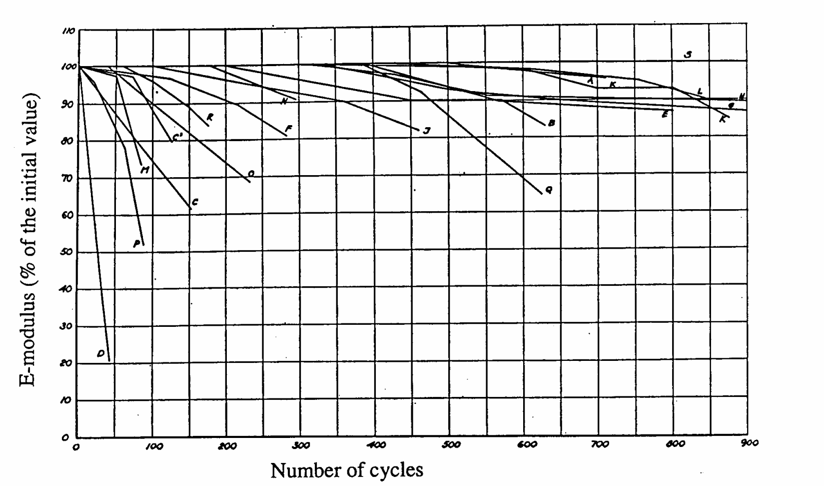

Figure 27: Reduction in E-modulus of cement mortar specimens repeatedly frozen in air to -15°C and thawed in water at +5°C. 2 cycles per day; /28/.

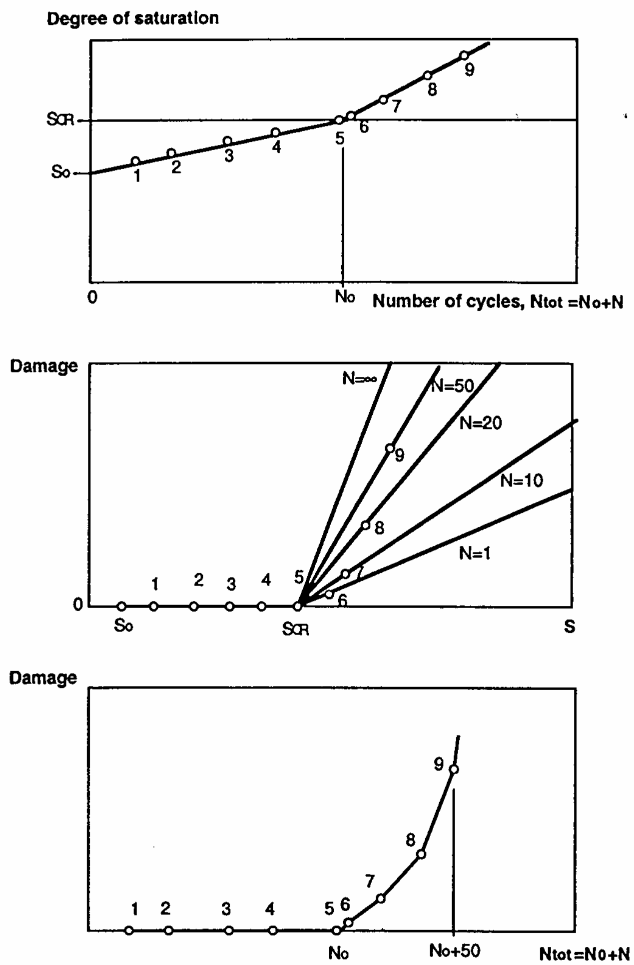

Figure 28: Principles of water absorption and development of damage in an open freeze-thaw test; /24/.

6. Service life with regard to frost

6.1 The service life before frost damage occurs-a general definition

Internal frost damage negatively affects most mechanical properties. It causes reduced compressive and tensile strength, reduced bond strength of the reinforcement, loss in E-modulus. Besides, it causes aesthetic damage by cracking. The effect of frost damage on mechanical properties of concrete is discussed in ANNEX E.

The effect of frost damage on the structural capacity and safety depends on which part of the structure is damaged, how big volume is damaged, and the extent of frost damage. In the following, only one single unit (representative) volume is considered. One damaged unit volume will not seriously affect an entire structure. Normally, however, many unit volumes are damaged at the same time. Therefore, by analysing the service life of one unit volume in a crucial section, the service life of the entire structure can be approximately evaluated.

In practice, the concrete structure will take up water. The water content can be expressed in terms of a degree of saturation S_{ACT} of the concrete as a whole, as defined by eqn. (29). When the actual degree of saturation exceeds the critical degree of saturation over a sufficiently big portion of the structure, frost damage will be observed. As shown by Figure 16 and 17 considerable frost damage occurs already after a few freeze-thaw cycles when $S_{ACT}>S_{CR}$. This means that one can define service life as the point of time when S_{CR} is transgressed for the first time (provided the concrete is at the same time exposed to freezing temperatures). The service life of a unit volume inside the structure can therefore be defined by one of the conditions:

$$w(t_{life})=w_{CR} \tag{38a}$$

$$U(t_{life})=U_{CR} \tag{38b}$$

$$S(t_{life})=S_{CR} \tag{38c}$$

Where w_{CR}, U_{CR} and S_{CR} are the critical moisture content, the critical moisture ratio and the critical degree of saturation as defined by eqn (27), (28), and (29). $w(t_{life})$, $U(t_{life})$ and $S(t_{life})$ are the actual moisture contents in the unit volume when the critical moisture content is reached for the first time, i.e. when $t=t_{life}$, where t is the total exposure time from erection of the structure.

Note:
The definition above is based on the assumption that frost damage is very big if the critical moisture content is transgressed once. Thus, the first time this happens, severe frost damage is supposed to occur. This is not altogether correct, since a certain number of transgressions are needed in order to obtain a big damage; see Figure 19. Using eqn. (30b) the service life criterion is instead (when moisture content is expressed in S):

$$S(t)\geq D_{max}/K_N+S_{CR}=D_{max}(B+N)/A\cdot N+S_{CR} \tag{39}$$

Where D_{max} the maximum tolerable reduction in E-modulus (or strength) $(0\leq D\leq 1)$
K_N the "coefficient of fatigue"

Example:

If D_{max} is 0.3 (30% destruction) S_{CR}=0.77 and the coefficients A and B defining K_N (see eqn. (30b)) are 12 and 4, service life is ended when:

$$S(t) \geq 0.3(4+N)/12 \cdot N + 0.77$$

Thus if there are 10 cycles during which S transgresses S_{CR}, service life is ended when the average degree of saturation during these cycles is 0.805 instead of 0.77 given by eqn. (38c). If the number of cycles is 1000 service life is ended when S=0.795. Thus, the effect of the number of freeze-thaw cycles is not so big. Therefore, eqn (38) can be used as a criterion on "the safe side".

In the general case, both S_{ACT} and S_{CR} are functions of time. For S_{ACT} this is self-evident, since the moisture content fluctuates all the time. For S_{CR} it is not so evident. Experiments indicate that a certain increase in the critical moisture content occurs during the first days and weeks. However, for an old concrete it can be assumed to be constant.

Service life defined by eqn. (38) is also visualised in Figure 29. It is of course almost impossible to predict exactly when the condition (38) occurs for the first time. For this to be possible, one has to be able to predict the future moisture variation exactly. A stochastic approach can be used as shown by eqn. (33). For degree of saturation as a measure of moisture, the condition for frost damage is described by:

$$P\{S_{CR}<S_{ACT}\} = \int_0^1 F(S_{CR}(t)) \cdot f(S_{ACT}(t)) \cdot dS \qquad (33a)$$

Where $P\{S_{CR}<S_{ACT}\}$ the probability of frost damage, 0=P=1

$F(S_{CR}(t))$ the distribution function of S_{CR} at time t

$f(S_{ACT}(t))$ the frequency function of S_{ACT} at time t

dS interval in the degree of saturation S.

Thus, if the variations in S_{CR} and S_{ACT} as a function of time can be expressed quantitatively, eqn. (33a) can be used for calculating the risk of frost damage at a certain point of time.

In an old structure, that is not frost damaged, or in which damage has a certain limited value, there is no reason to assume a different variation in inner moisture content in the future than the "historic" variation. Therefore, there is no reason to assume a bigger risk of frost damage in the future, unless the climate is radically changed.

The service life of the entire structure will be a function of the service life of the different unit volumes building up the structure, and can only be evaluated by evaluating the entire moisture-time field in all unit volumes across the entire structure, together with an evaluation of the distribution of the critical moisture condition in all unit volumes across the entire structure. A stochastic approach described by eqn. (33) can then be used for all parts of the structure. Then, using normal structural analysis, the time process of deterioration of the structural stability of the entire structure can be made. The method of a structural analysis is also discussed in ANNEX H.

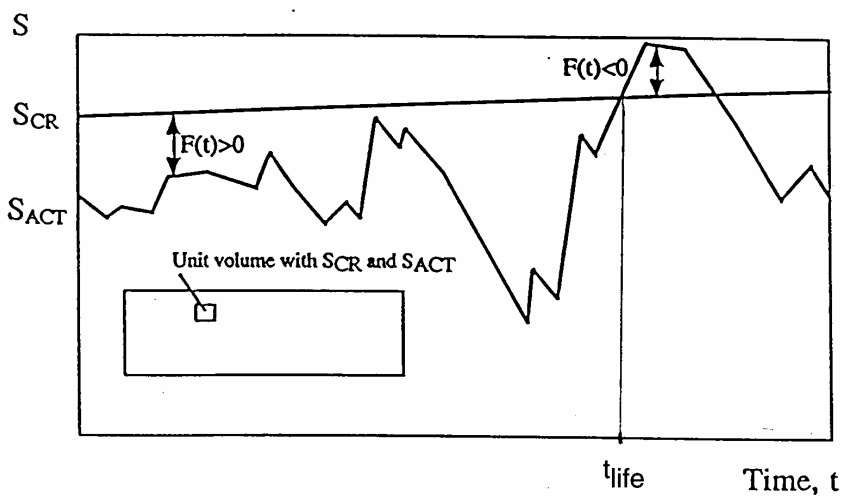

Figure 29: Definition of frost resistance, $F=S_{CR}-S_{ACT}$, and service life, t_{life}

6.2 The service life before frost damage occurs-the potential service life

Instead of using the really occurring degree of saturation for calculating service life one can use a definition based on a simple water absorption test. The most simple test is the continuous water absorption within a thin slice of the concrete. The slice is put in contact with a free water surface. Evaporation from the specimen is hindered by an impermeable lid on the water uptake vessel. Examples of the result of such tests are shown in Figure 24. Normally S_{CR} corresponds to a water uptake beyond the breaking point in the diagram; i.e. a certain air-pore absorption is required.

One can define a sort of *potential service life* as the time it takes for the capillary uptake specimen to reach the critical water absorption. Thus, the potential service life, tp,life, is defined as (when S is used as measure of moisture):

$$S_{CAP}(t_{p,life})=S_{CR} \qquad (40)$$

This definition is visualised in Figure 30. It can be proven by an analytical analysis of the water uptake process in air-pores, that the long-term absorption in the air-pores can be described by an equation of the following type; see paragraph 5.1.2 and /19/:

$$S_{CAP}(t) = A+B\cdot t^{C} \qquad (36a)$$

Where A, B, C coefficients determining the rate of capillary water uptake
t absorption time expressed in hours

This means that the potential service life is calculated by:

$$t_{p,life} = \{(S_{CR}-A)/B\}^{1/C} \qquad (41)$$

Principally, each real environment corresponds to a certain point on the long-term absorption curve. Some examples can be given:

1. *Vertical surface (e.g. a façade).* The water uptake normally corresponds to the breaking point, i.e S_{CAP}=A. Then, since S_{CR} normally is bigger than A, this means that:

$$t_{p,life} = \infty$$

2. *Road surface*. The surface might be very moist for months. Let us assume that the maximum moisture content corresponds to 3 months (2160 hours) in the capillary water uptake test. Then, the level of frost resistance after this time is:

$$F(3\text{ months}) = S_{CR}\text{-}S_{CAP}(3\text{ months}) = S_{CR} -\{A+B\cdot 2160^{C}\}$$

If this expression is negative, the service life is lower than 3 months, and the concrete is not frost resistant in its environment. If the expression is positive;

$$t_{p,life}=\infty$$

3. *Bridge pier at the water line*. The concrete will have no chance to dry. Thus, eqn. (41) can be used directly for estimating the service life. The problem is that the coefficients B and C must be known with very high precision because of the very long time-extrapolation required. Moreover, it might very well be that the type of equation used -eqn. (41)- cannot be used for very long extrapolation.

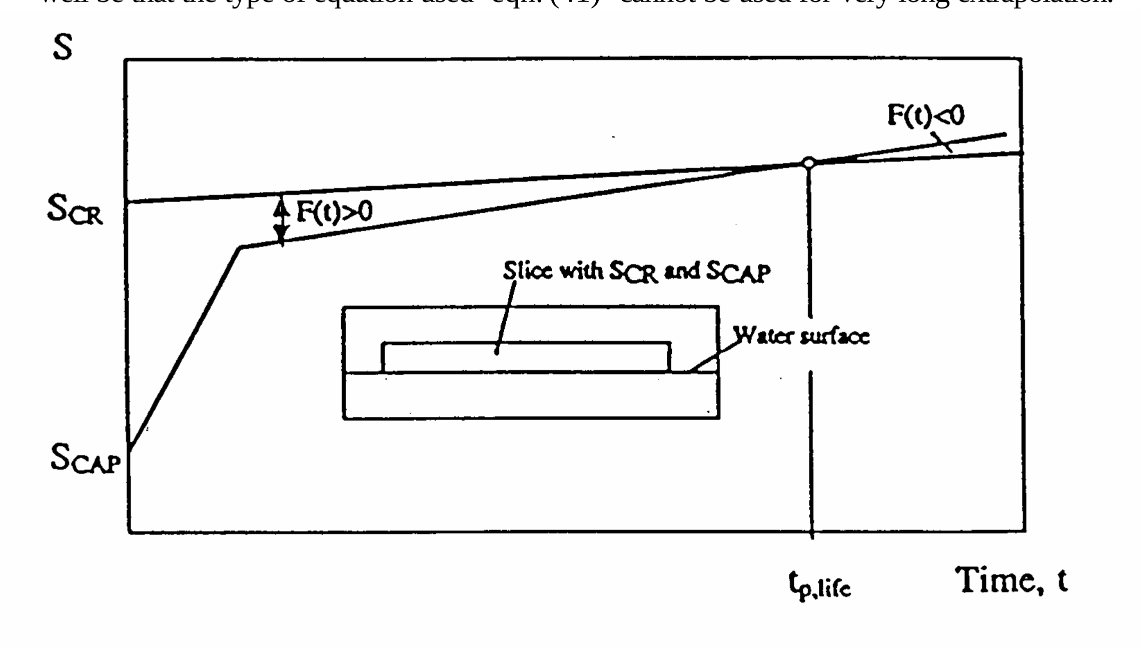

Figure 30: Definition of the "potential service life", $t_{p,life}$.

7. Concrete technological factors and internal frost resistance

7.1 Air content

By increasing the air content in a concrete, the spacing between air-pores is reduced. Therefore, the resistance to internal frost damage increases with increased air content. An example of the relation between air content and internal frost resistance measured by the American ASTM method C666 is shown in Figure 31.

The more moist the environment, the bigger the air content required, since some air pores gradually absorb water in moist environment; paragraph 5.1.2. A non-air-entrained concrete can therefore be suspected to have low frost resistance in a moist environment.

In older concrete, the internal frost resistance might be quite high also when it has no air-entrainment. The reason is that such concrete often had quite a stiff consistency when it was cast. In almost no concrete made before the 1960:ies plasticising agents were used.

In concrete cast before about 1940 internal vibration was not used. Instead the concrete mass was compacted by hand-held poles or stamps. Consequently, older concrete often contains a big amount of compaction pores caused by insufficient compaction. Such pores can be effective in producing frost resistance, provided their volume is above 2.5 to 3%, and the environment is not too moist. In a very moist environment a higher air content is often needed (3.5 to 4%).

In more "modern" concrete, super-plasticizing agents, or water reducing agents, were frequently used. Consequently, the consistency was more fluid, and as a consequence the "natural" compaction pores was often very much reduced. Non-air-entrained concrete of this type often has low degree of internal frost resistance.

Theoretically, the air content shall be so high that the critical moisture content corresponds to at least the water content that the concrete reaches in its environment. Thus, if the environment gives a moisture content that corresponds to 1 week of water absorption the minimum air content can be lower than if the environment gives a water content that corresponds to 6 months of water absorption.

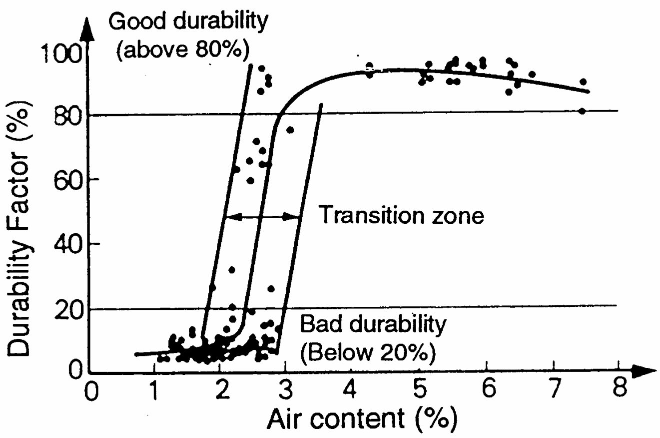

Figure 31: Effect of air content on the internal frost resistance; /29/.

7.2 Spacing factor

Normally, the internal frost resistance increases with decreasing Powers spacing factor. The ”true” critical spacing factor considering *only air-filled air-pores* should be below about 0.35 to 0.40 mm; see paragraph 3.2. This spacing factor is difficult to measure experimentally. Therefore, a spacing factor based on all air-pores, also the water-filled air-pores, is always used as a measure of frost resistance. It has been found that during the very moist conditions prevailing in the test method ASTM C666 this spacing factor shall be below 0,23 to 0.25 mm. Examples are shown in Figure 32.

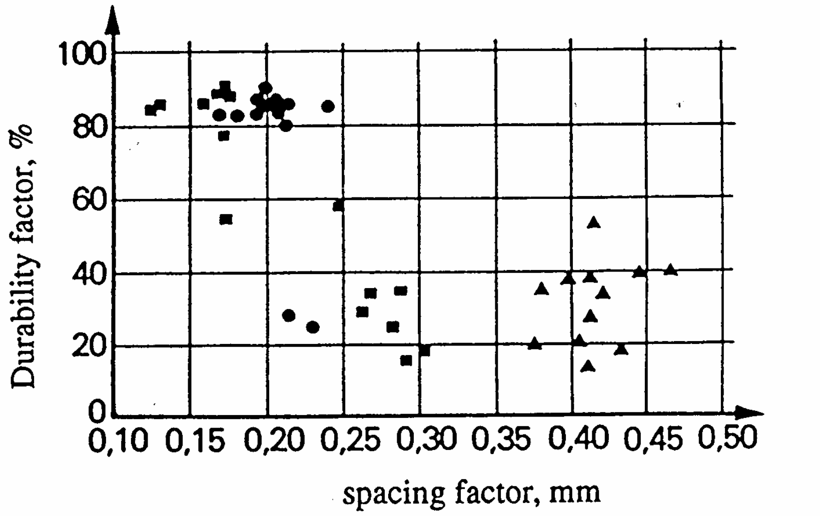

Figure 32: Effect of the Powers spacing factor, considering all air-pores, on the result of a freeze-thaw test using the method ASTM C666, /30/.

7.3 Water-binder ratio

The internal frost resistance increases with decreasing water-binder ratio (w/b-ratio). This is natural since a reduced w/b-ratio makes the concrete more impermeable to water ingress. It also seems as if a reduced w/b-ratio causes a reduction in the average air pore size and spacing; see Figure 33.

An example of the relation between the water/cement ratio and the number of freeze-thaw cycles required for 25% loss in weight at a certain test method is shown in Figure 34. This figure shows that the effect of w/c-ratio is important only when the concrete is air-entrained. The positive effect of a lower w/c-ratio is probably caused by the combined effect of a lower water absorption and a more favourable air-pore structure.

For non-air-entrained concrete, the effect of a lower w/c-ratio is remarkably small. It is only for w/c-ratios below 0.40 that a marked positive effect of a lowered w/c-ratio is noticed.

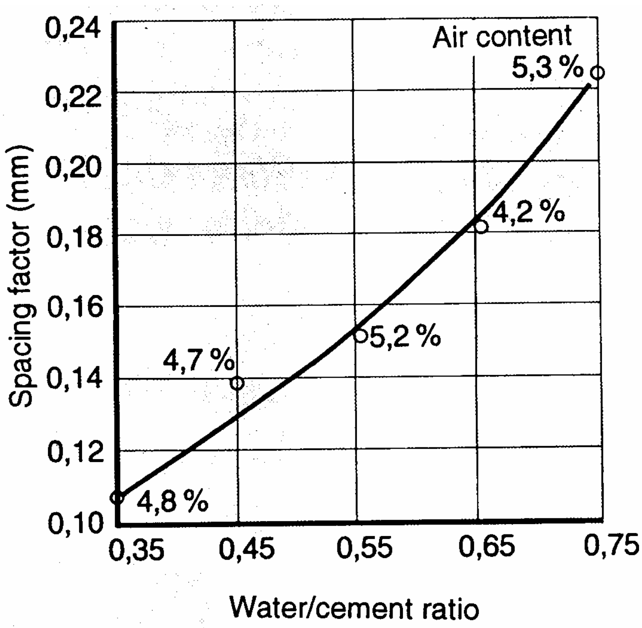

Figure 33: Effect of the water/cement ratio on the Powers spacing factor (all air-pores included), /31/.

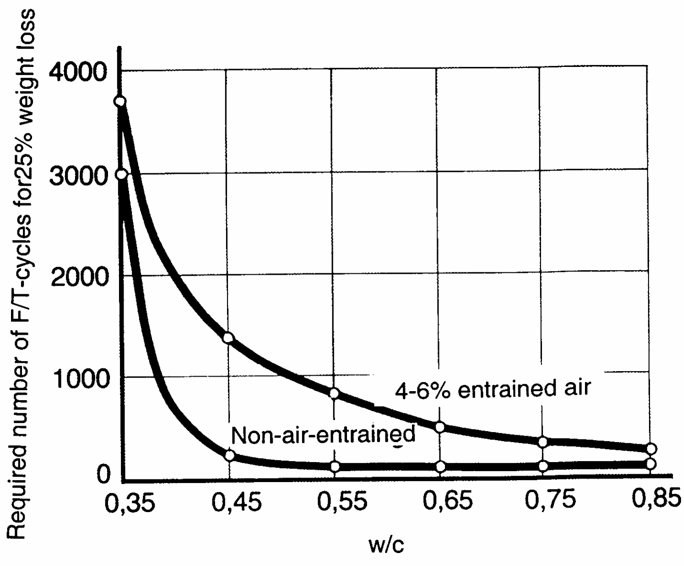

Figure 34: Relation between the result of a freeze-thaw test and the water-cement ratio; /32/.

7.4 Type of cement

The cement type has no bigger influence on the internal frost resistance, provided the air content and spacing factor are not affected.

The alkalinity of the cement seems to affect the frost resistance because it affects the air-pore formation. A higher water soluble alkali content often tends to increase the spacing factor. Examples are seen in Figure 35.

It has turned out that incorporation of fly ash in cement, or in concrete, might reduce the natural air content in non-air-entrained concrete. This will have very negative effect on the frost resistance: One example is seen in Figure 36.

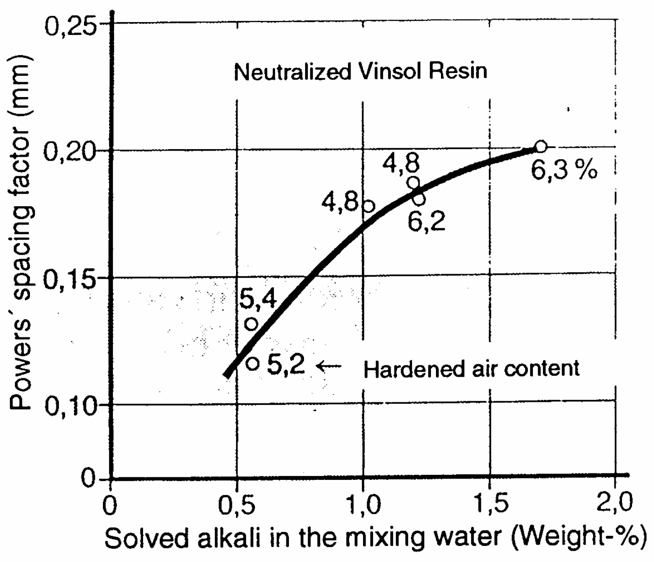

Figure 35: Effect of the alkali content in the pore water on the spacing factor of concrete having about the same air content; /33/.

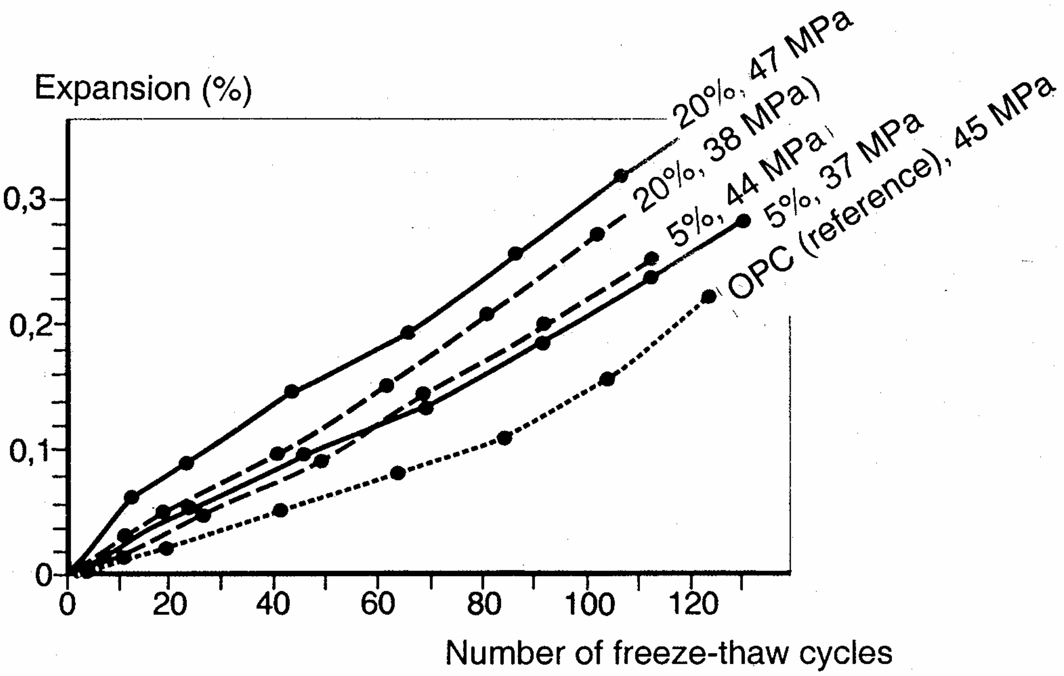

Figure 36: Example of the effect of incorporation of 5 or 20% fly ash in the cement on the frost resistance of concrete; /34/.

7.5 Type of aggregate

Porous *coarse* aggregate particles, that can become fully saturated when embedded in the cement paste, will cause substantial damage, unless their porosity is below about 1 to 2%. Such particles will freeze as saturated closed containers according to destruction mechanism 1 in paragraph 2.1. Therefore, the only possibility there is, that these particles will not destroy concrete is that the cast-in particles cannot become critically saturated in the real outdoor condition. If they are very fine-porous this possibility can be excluded when the environment is very moist.

Examples of the relation between the vacuum-saturated porosity of coarse aggregate and the expansion during freeze-thaw of concrete made with the aggregate is shown in Figure 3. There is a limit between good and average frost resistance at a porosity corresponding to about 1 to 2weight-% water content at complete saturation.

It is only fine-porous aggregate, like limestone, slate or sandstone, that can become fully saturated when embedded in concrete, and which are therefore dangerous with regard to frost damage. Coarse-porous aggregate, like expanded clay and other light-weight aggregate, can seldom become saturated to a dangerous condition when used in concrete. Therefore, they will normally not cause frost damage.

If the fine-porous aggregate is small, it will not be able to cause frost damage in the concrete. The pressure caused by the freezing aggregate is small, because water can be expelled from the aggregate particles when these freeze. Therefore, there is an increased risk of frost damage with increased aggregate size; see Figure 37, which shows the results of freezing tests of concrete with aggregate of different size.

A more detailed discussion of the effect of porous aggregate on the internal frost resistance of concrete is performed in /6/.

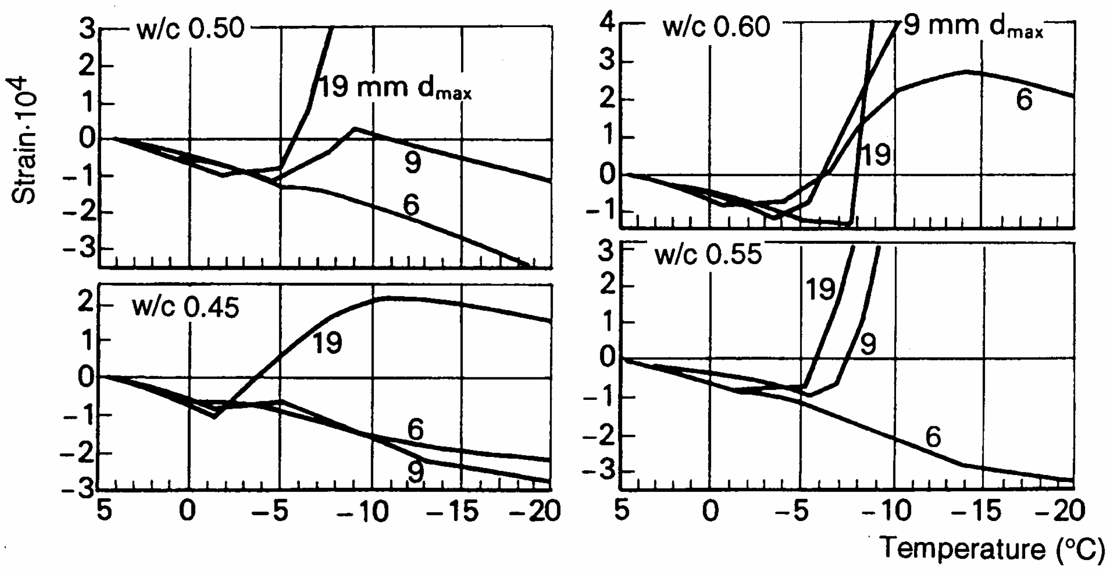

Figure 37: Expansion during freezing of concrete containing porous natural aggregate. Effect of aggregate size and water-cement ratio; /35/.

7.6 Salt concentration of the pore water

If the concrete is exposed to saline water for a very long time, the pore water might become saline. This might have an effect on the internal frost resistance, but probably this effect is not very big. It might even be positive, since the amount of freezable water is reduced when the pore water is saline.

Specimens that have been stored for a very long time in NaCl-solutions with concentration up till 6% have not been more frost damaged, when freeze-thaw tested in pure water, than companion specimens stored for the same time in pure water. These results are shown in ANNEX B.

According to other tests, no difference in the critical degree of saturation could be observed for a number of concrete tested with different salinity of the pore water. Thus, it seems as if the internal salt concentration does not significantly affect the internal stresses occurring during sealed freeze-thaw.

All these tests indicate that the effect of salinity of the pore water on the internal frost resistance can be neglected. Also the effect on salt frost scaling of the internal salt concentration is marginal; see ANNEX B.

8. Test methods for internal frost resistance

8.1 Freeze-test methods

8.1.1 The Critical Degree of Saturation method, /18/.

The Critical Degree of Saturation, S_{CR}, is determined by freeze-thaw of isolated specimens pre-conditioned to different moisture contents. Curves of type Figure 16 and 17 are obtained.

The Capillary Degree of Saturation, S_{CAP}, is determined by uni-directional water uptake in thin concrete slices. Curves of type Figure 24 are obtained. The long-term capillary water uptake is described by:

$$S_{CAP}(t)=A+B\cdot t^{C} \qquad (36a)$$

Where the coefficients A, B and C are determined by linear regression of the experimental long-term absorption curve.

The Potential Frost Resistance, F; and the Potential Service Life, $t_{life,p}$, are obtained by comparing S_{CR} with S_{CAP}:

$$F=S_{CR}-S_{CAP}(t) \qquad (42)$$

$$t_{life,p}= [(S_{CR}-A)/B]^{1/C} \qquad (41)$$

If F<0 after short water absorption time, the concrete has low frost resistance. The bigger the value of $t_{life,p}$, the higher the frost resistance.

8.1.2 Open freeze-thaw tests

In an open freeze-thaw experiment, the concrete is repeatedly frozen and thawed in water or air. The damage is measured as loss in a mechanical property, normally the dynamic E-modulus, or as expansion. The number of cycles might vary from less than 100 to more than 300. The freeze-thaw cycles might vary as well, from a very rapid performed many times a day to day-long cycles.

The most common method is *ASTM C666 Procedure A* in which a maximum of 300 cycles are performed on immersed concrete cylinders or prisms. The cycle length may vary from 2 to 5 hours. The test is interrupted when the loss in dynamic E-modulus is above 40%. A Durability Factor, DF, is defined as:

$$DF=(E_i/E_o)^2 \cdot N/300 \tag{43}$$

Where E_o the initial dynamic E-modulus
E_N the E-modulus after N cycles
N the number of cycles when the test is stopped (either 300 or when the E-modulus is reduced by 40%)

A *new open freeze-thaw test metod* is based on the traditional salt scaling test (Swedish test method SS 13 72 44) where the salt solution on top of the specimen is exchanged for pure water. The cycle length is 24 hours. Damage is measured, either by change in the speed of sound parallel to the exposed surface, or by change in the frequency of transverse vibration. Both measurements give the change in the dynamic E-modulus. Destruction curves of the type shown in Figure 38 are obtained; /36/. The bigger the number of freeze-thaw cycles until frost damage is observed, the higher the frost resistance.

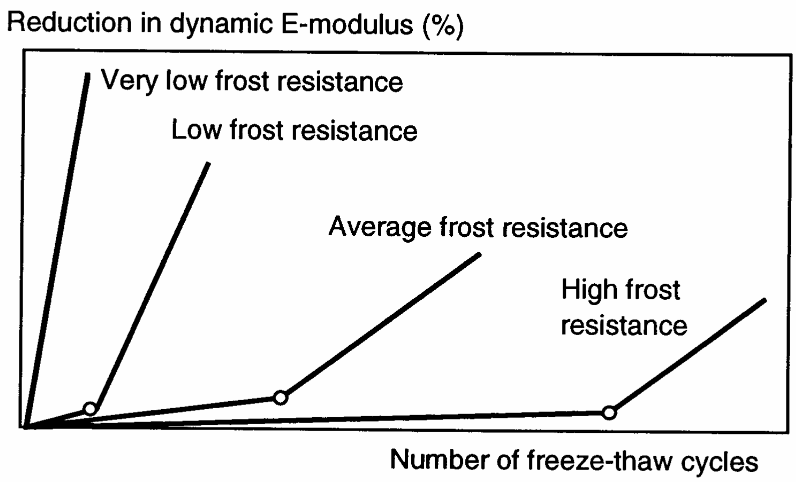

Figure 38: Dynamic E-modulus versus the number of freeze-thaw cycles. Hypothetical curves.

8.1.3 Dilation during isolated freeze-thaw

The specimen is stored in water for a certain time. Then, it is sealed and exposed to one (or 2) freeze-thaw cycles. The length change is measured during the entire cycle. If the specimen contracts during the test as shown in Figure 39 curve A, it is frost resistant at the actual moisture content. If it expands more than the thermal contraction, Curve B in Figure 39, it is non-resistant. The bigger the expansion, the lower the frost resistance.

By pre-storing the specimen in a manner that corresponds to the water content the actual concrete will obtain in practice, or by taking a core from the actual structure, and test it at the moisture content it had in the structure, a measure is obtained for the actual degree of frost resistance.

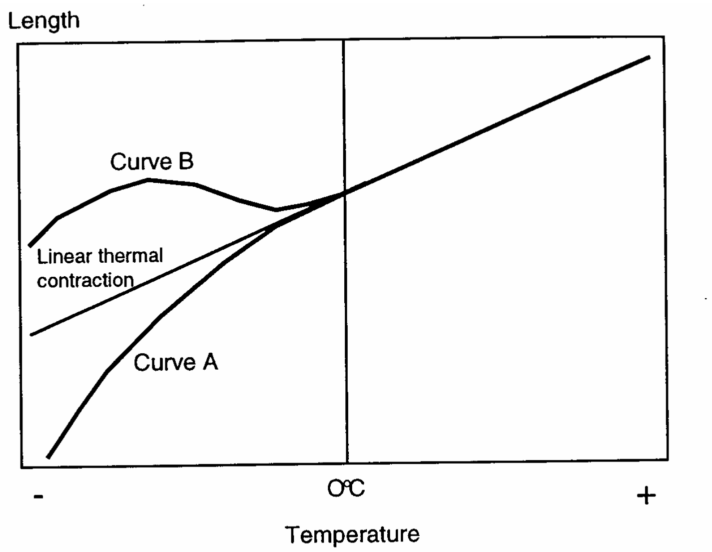

Figure 39: Hypothetical results of a dilation freeze-thaw test of an isolated specimen.

8.2 The air content

The effective air content can be obtained by measuring both the water-filled pore volume, and the total pore volume in a specimen with known volume. The effective air content is obtained by:

$$A_{eff}=(V_p-V_w)V \tag{44}$$

Where V_p the total pore volume [litres]

V_w the water-filled pore volume at natural water absorption [litres]

V the total specimen volume [litres]

The values V and V_p are determined by weighing the specimen in dried condition (+105°C), and in vacuum-saturated condition in air, and immersed in water.

$$V=Q_{sat,a}-Q_{sat,w} \tag{45}$$

$$V_p=Q_{sat,a}-Q_{dry} \tag{46}$$

Where Q_{dry} the weight after drying at +105°C [kg]
$Q_{sat,a}$ the weight in air of the vacuum-saturated specimen [kg]
$Q_{sat,w}$ the weight in water of the vacuum-saturated specimen [kg]

The value of V_w is depending on how long time the specimen has absorbed water. Normally 1 week can be sufficient for a normal-sized specimen. Then, V_w is calculated by:

$$V_w = Q_w - Q_{dry} \tag{47}$$

Where Q_w the weight after natural water absorption [kg]

Note:
Principally the water storage time shall be so high that the water content corresponds to the water content in practice. However, since one only wants a rough measure of the effective air content, it is sufficient to use an absorption time that is long enough to secure that the breaking point in a water absorption curve of the type shown in Figure 24 is reached.

8.2 The air-pore structure. The spacing factor

The spacing factor, and the air-pore size distribution, can be obtained by optical techniques. Normally the spacing factor is determined by the so-called linear traverse technique applied to polished concrete surfaces. Lines are arbitrarily placed on the surface. The lines intersect air-pore chords. The number of intersected pores, and the chord size distribution are measured microscopically, either by the naked eye or by an image analysis technique. On basis of this information the air content and the Powers spacing factor can be determined. The technique is described in /37/.

A measure can also be obtained for the air-pore size distribution. The method by Lord and Willis is used, /38/.

There are other possibilities of measuring and calculating the air-pore structure using the same polished concrete surface. One possibility is to measure the size distribution of the individual pore calottes intersected by the polished plane. This can be done by image analysis. Thereafter, the spacing factor, the air content and the air-pore size distribution can be calculated. The mathematical procedure is described in /39/.

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ANNEX B
Salt frost scaling

Contents Page

ANNEX B: Salt scaling

Types of frost damage

There are two types of frost damage:

1: *Internal damage* caused by freezing of water inside the concrete. The damage is always confined to such parts of the concrete where the degree of water saturation exceeds the critical value. The damage causes loss in compressive and tensile strength, loss in E-modulus, and loss in bond strength; see ANNEX E.

2. *Surface scaling* caused by freezing of the concrete surface when it stays in contact with saline solutions of weak concentration. The initial scaling occurs in the cement paste phase while the aggregate grains are intact. Due to the gradually deeper and deeper scaling also coarser aggregate grains are lost. In serious cases a big portion of, or the whole cover can be eroded which has very big effect on the service life with regard to reinforcement corrosion. See ANNEX F.

Freezing of very moist concrete (also concrete containing salt water in its pores) but with no external salt water or pure water in contact with the surface seldom leads to surface scaling but more often to internal damage.

Freezing of concrete in presence of an external salt solution at the surface seldom leads to internal damage, but often to surface scaling.

Salt frost scaling is described in this ANNEX.
Internal damage is described in ANNEX A.

1. Effect of the internal and external salt concentration

A special type of frost damage, surface scaling, can occur when a concrete surface is exposed to frost at the same time as it is also exposed to a weak solution. It has been found that all types of solutions might cause such scaling, not only chloride solutions, but also weak solutions of urea and alcohol. Examples are shown in Figure 1. It appears that the most severe damage occurs at rather weak solutions, while strong solutions cause less destruction as seen in Figure 1.

> Note:
> Some salts like $CaCl_2$ also cause chemical damage besides frost damage. Chemical damage increases with increasing concentration. Therefore, for these salts there are often two concentration ranges that cause damage; one low range causing salt-frost damage, and one high range causing chemical attack. Between these ranges, the salt is more or less harmless.

Not only concrete can be attacked, but also natural stone. Examples are shown in Figure 2 which shows the result of a salt-frost test of some sandstone and limestone.

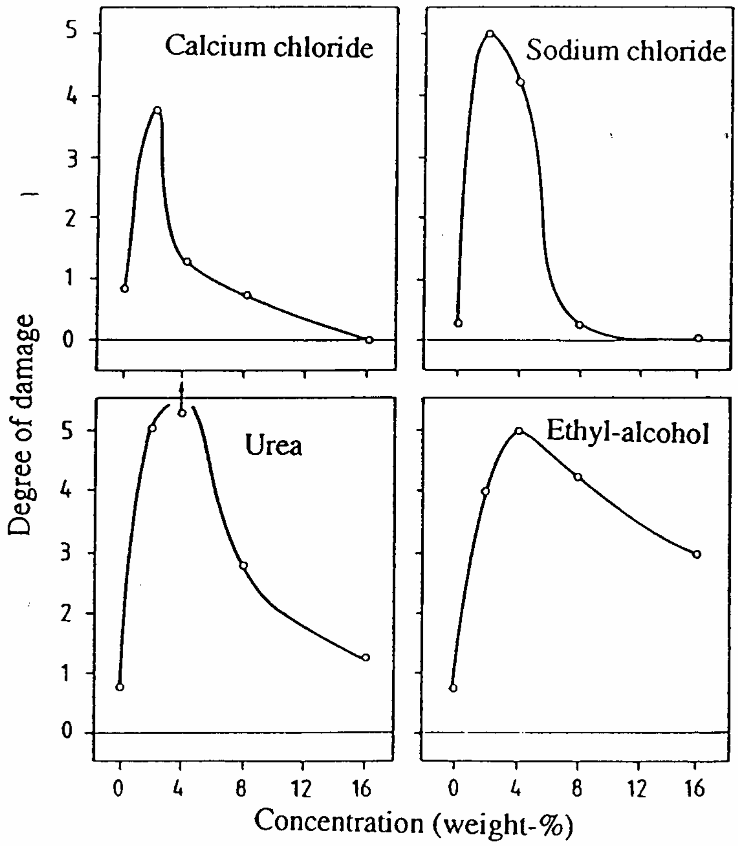

Figure 1: Effect of the concentration of different de-icing agents applied to the concrete surface on the degree of frost damage observed; /1/.

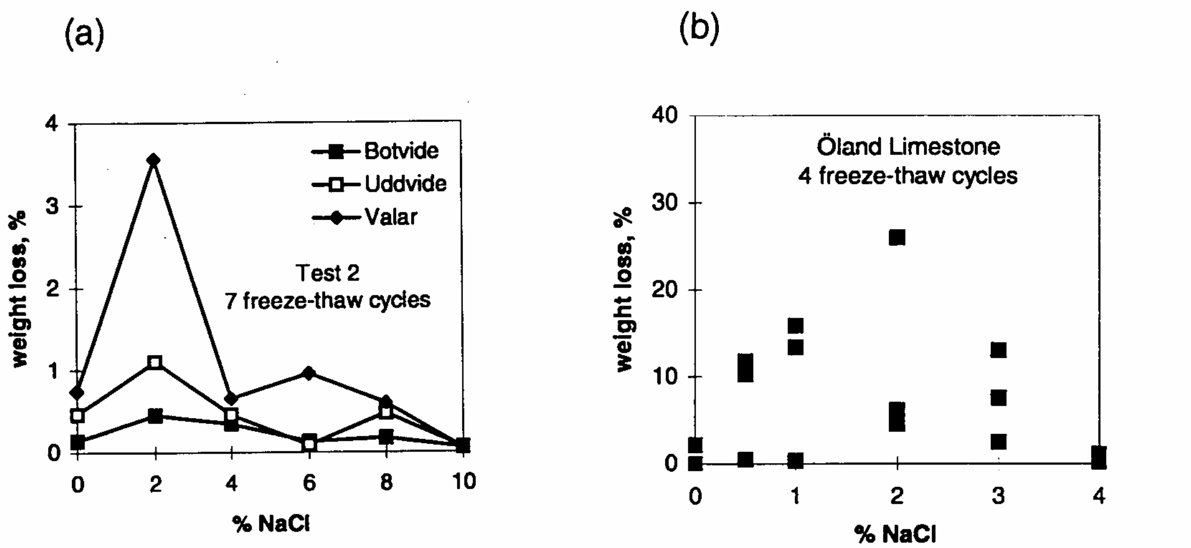

Figure 2: Effect of the concentration of NaCl on the frost damage of natural stone. (a) Sandstone. (b) Limestone. /2/.

For concrete, the most dangerous outer concentration seems to be around 2 to 4%. This is valid for the concrete in Figure 1, but also for the natural stone in Figure 2.

The result of a systematic study of the effect of different combinations of the internal and external salt concentrations on the salt frost resistance is shown in Figure 3-6; /3/. The internal salt concentrations were obtained by storing the specimens for half a year in salt solution. Three different salt concentrations were used; 0%, 3% and 6%. Three different minimum freezing temperatures were used; -7°C, -14°C and -22°C. Two rates of freezing were used; "rapid" and "slow". The freeze/thaw cycles used are shown in Figure 7. The results indicate the existence of a "pessimum" external salt solution, and this seems to be independent of the inner salt concentration, the rate of freezing, and the lowest freezing temperature. In the actual study, the pessimum concentration was 3%. It is, however, not possible to know, from these experiments, if 2%, or 4%, or any other concentration, had been "more pessimal".

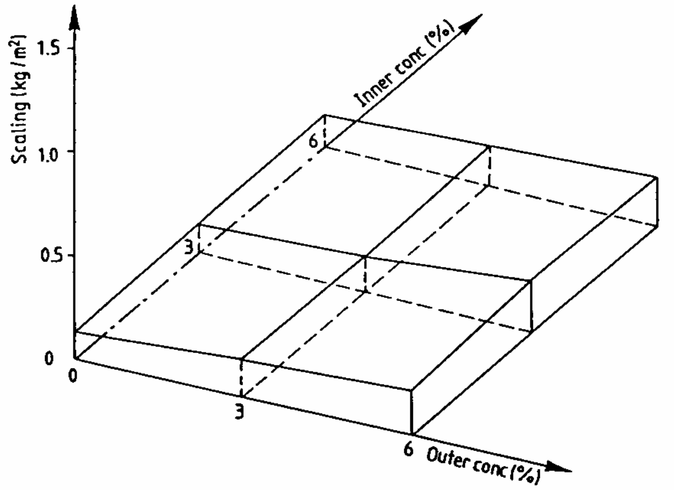

Figure 3: The total salt scaling after 56 freeze/thaw cycles of an air-entrained concrete with the water/cement ratio 0,40 tested with the minimum temperature -7°C and "rapid freezing", curve A in Figure 7; /3/.

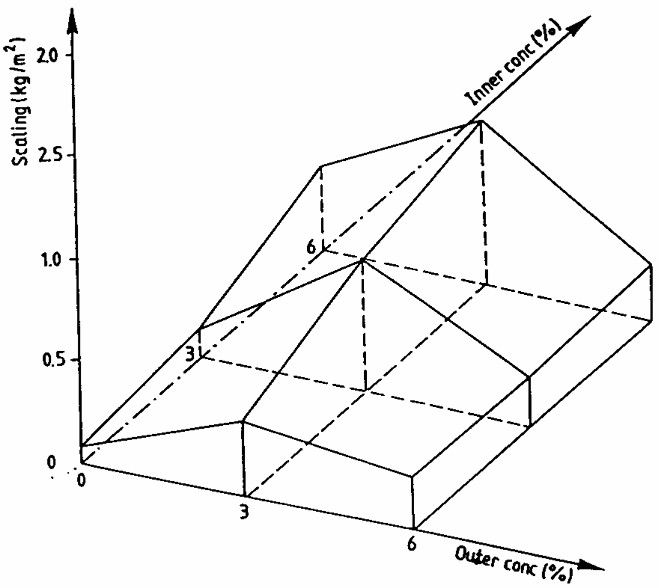

Figure 4: The total salt scaling after 56 freeze/thaw cycles of an air-entrained concrete with the water/cement ratio 0,40 tested with the minimum temperature -14°C and "rapid freezing", curve C in Figure 7; /3/.

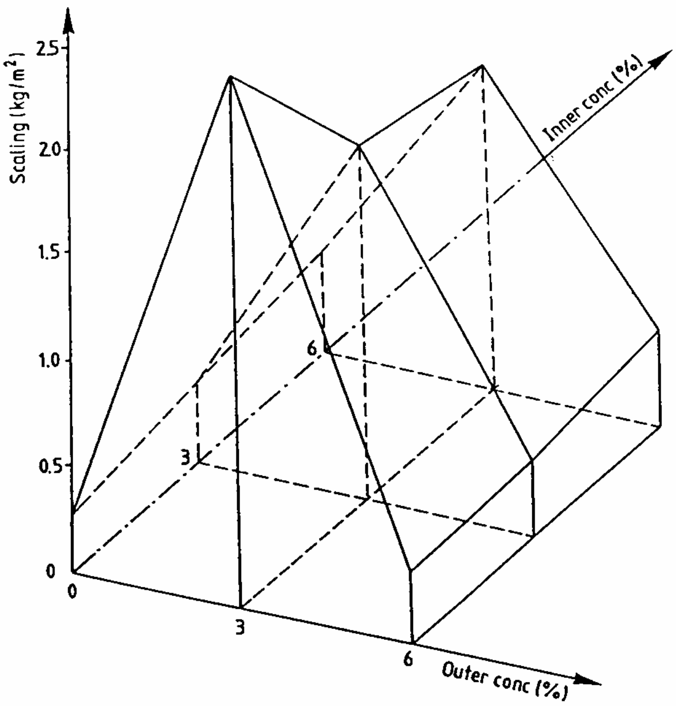

Figure 5: The total salt scaling after 56 freeze/thaw cycles of an air-entrained concrete with the water/cement ratio 0,40 tested with the minimum temperature -22°C and "rapid freezing", curve E in Figure 7; /3/.

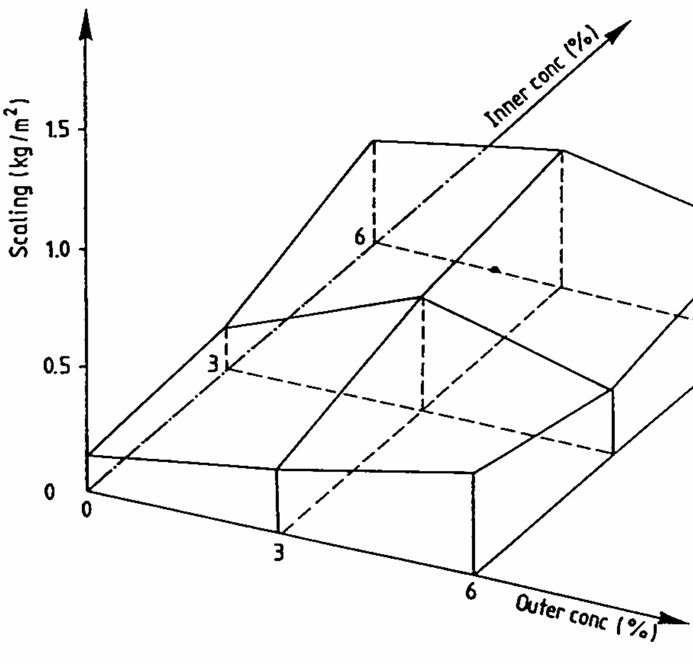

Figure 6: The total salt scaling after 56 freeze/thaw cycles of an air-entrained concrete with the water/cement ratio 0,40 tested with the minimum temperature -22°C and "slow freezing", curve F in Figure 7; /3/.

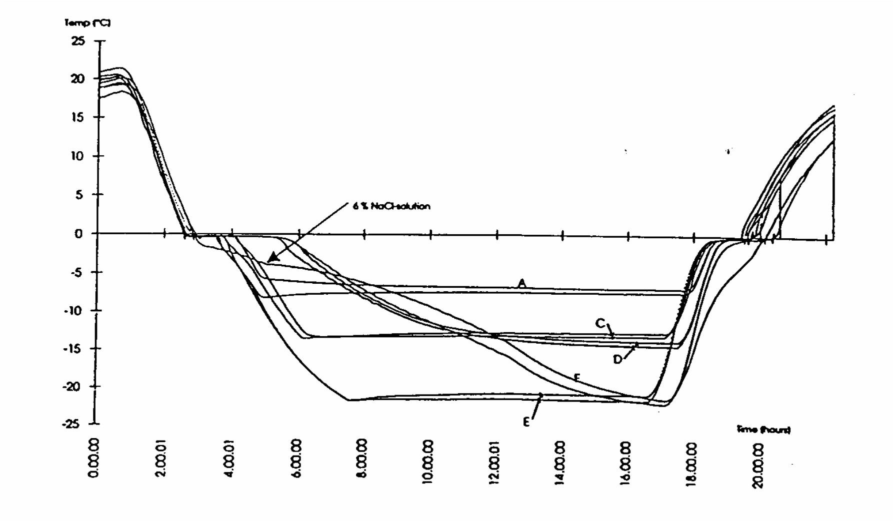

Figure 7: The freeze/thaw cycles used for the tests shown in Figure 3-6; /3/.

The results are important, because they indicate that an ageing of the concrete in the form of a gradual salt absorption in the pores, will not make the concrete more vulnerable to salt scaling; it is the *outer salt concentration* that counts.

The study also indicates, that the rate of freezing, (or rather the duration of freezing temperatures) is of great importance; the more rapid the freezing (or rather, the longer the freezing period), the bigger the scaling. The study also shows, that the minimum freezing temperature is crucial; -7°C and -14°C being much less harmful than -22°C. The following approximate relation between the lowest temperature reached during the freezing phase, θ_{min}, and the scaling S can be used:

$$S = constant \cdot (\theta_{min})^2 \qquad (1)$$

2. The damage mechanism

So far, salt scaling cannot be fully theoretically explained in a satisfying way. It is quite clear, however, that salt frost scaling is a very complex phenomenon, that involves moisture and salt migration before, during and after freezing, as well as internal pressure caused by many superimposed mechanisms.

There are many, partly conflicting, mechanisms presented. Recently, a logical mechanism was suggested and to a certain extent experimentally verified; /4/. It is based on thermodynamic considerations concerning the energy balance between unfrozen solution, pore liquid, and ice in pores and solution. It explains most phenomena observed. The mechanism is illustrated in Figure 8.

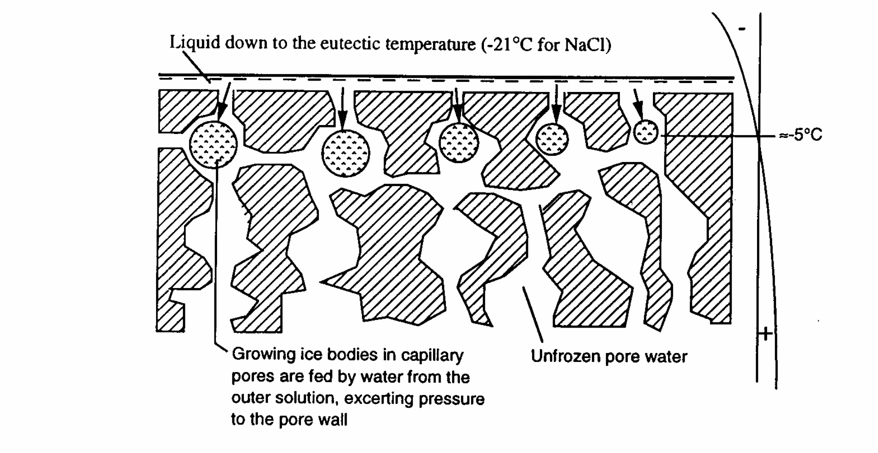

Figure 8: The salt scaling mechanism according to /4/.

The most fundamental feature of the destruction mechanism is that water from the outer solution is sucked into the surface part of the concrete during freeze-thaw. As long as the temperature is above the eutectic temperature of the outer solution (-21°C for NaCl-solution) there is always liquid at top of the concrete which can be sucked in. The driving force for water inflow is the free energy unbalance between outer liquid and pore ice. The lower the temperature, the bigger the driving force, and the longer the frost period, the bigger the time for moisture inflow. Thus, the mechanism is favoured by low temperature and long frost periods.

The mechanism is almost identical to frost heave in soil (see Mechanism 4 in ANNEX A), but in that case water is taken from the ground water reservoir, and not from an outer salt solution reservoir. The mechanism is also similar to the microscopic ice lens segregation mechanism (see Mechanism 3 in ANNEX A), but in that case water is taken from the finest pores containing unfrozen water. This causes an internal desiccation that gradually stops the process. In the case of an outer salt solution, no drying will occur, and therefore the process can go on as long as there is unfrozen liquid at the surface. Big pressure can be built up.

Due to this inflow of water across the surface, ice-bodies in the surface part of the material will grow and expose the material to tensile stresses perpendicular to the surface. If these stresses are bigger than the tensile strength scaling of the surface will occur. The thickness of the scaled material at each cycle is of the order 0.5 mm. With repeated freeze-thaw cycles repeated scaling will occur and consequently the surface becomes gradually eroded.

According to the theory, water uptake through the surface should increase when the first ice bodies are formed in the surface part. This behaviour has also been observed in carefully executed experiments. One example is seen in Figure 9. Thin cement mortar discs were first submerged in salt solution of different concentration, and the weight change at the temperature +21°C was monitored, Figure 9(a). The salt solution had a drying effect, which was shown by a gradual weight loss. When later the same discs were immersed in salt solution at a temperature that was equal to the freezing temperature of the liquid, there was a marked weight gain; Figure 9(b). The most plausible explanation is that ice bodies formed at the surface of the mortar sucked water from the solution. Moreover, in some cases also scaling occurred.
There are other absorption experiments presented in /4/ that strengthen the 'frost heave' hypothesis.

The ice bodies will grow as long as there is liquid on the surface, and the compressive stress in the ice bodies, has not grown big enough to stop the process. But before this happens, the concrete is already damaged. If the concrete contains air-pores, ice bodies within them will grow unhindered by the pore-walls. Therefore, water migration from the outer solution will rather go to ice-bodies in air-pores, and therefore the growth of ice-bodies in the capillaries will stop before the concrete is damaged. The lower the spacing between air-pores, the lower the stresses. Therefore, there is a *critical spacing* between air-pores also for salt scaling.

The fact that air-pores can save the concrete from salt frost damage is seen in Figure 10. The very big damage occurring at about 3% salt solution is considerably reduced when the concrete contains entrained air.

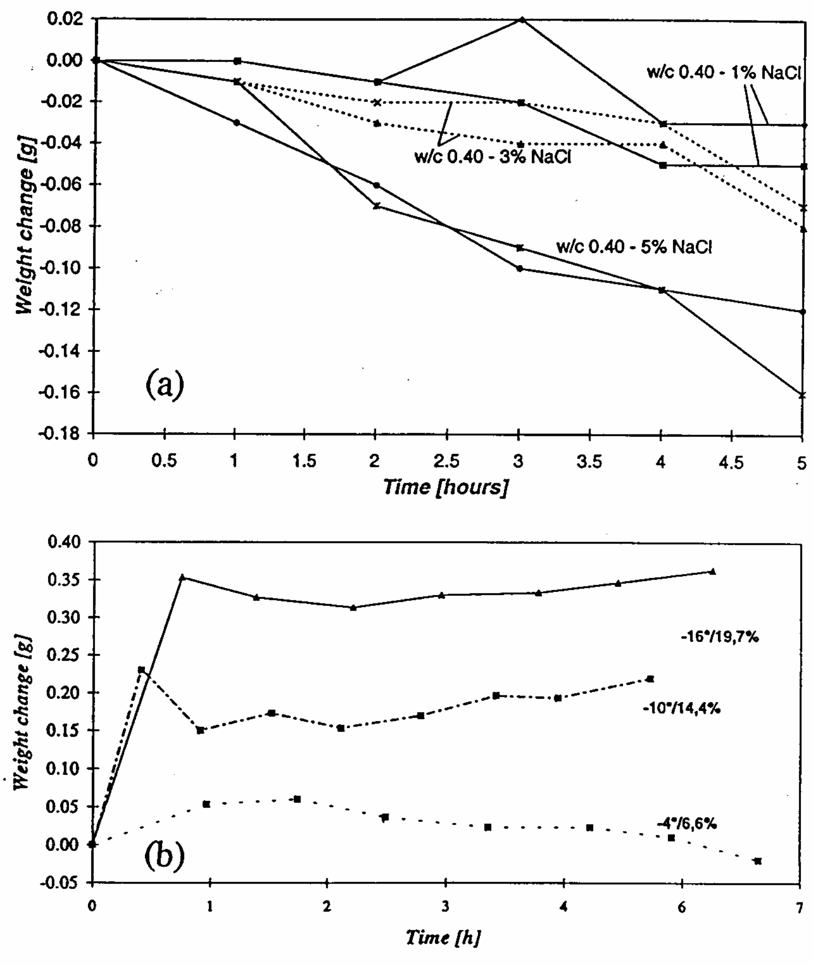

Figure 9: (a) Weight change in concrete discs (w/c 0.40) stored in NaCl-solution at +21°C. (b) Weight change of concrete discs (w/c 0.40) stored in NaCl-solutions at different freezing temperatures. The salt concentrations are 'temperature matched' so that the concentration corresponds to the freezing temperature. Time 0 corresponds to the time when the specimens were placed in the solution, /4/.

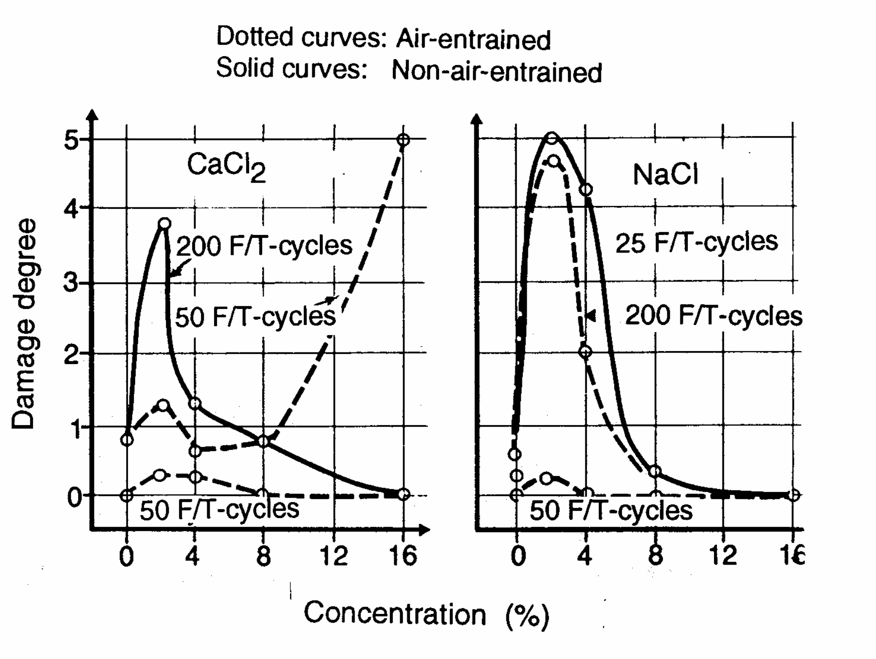

Figure 10: Example of the effect of salt concentration and air content on the salt frost damage of concrete; /1/.

3. The progression of salt frost scaling

According to the destruction mechanism, every new freeze-thaw cycle ought to produce the same damage, provided the cycle has the same characteristics as regards freezing rate, duration, minimum temperature, and salinity. Therefore, salt scaling of a homogeneous material like cement paste, or a cement mortar, is almost directly proportional to the number of equal freeze-thaw cycles. There might be a certain retarded scaling in cases where the surface part has lower salt scaling resistance than the interior parts, due to separation of the fresh mix. There might also be cases where there is an accelerated scaling because the degree of saturation is gradually increasing with time. Normally, however, for cement paste and mortar that are not of very bad quality, the scaling after a certain initial period can be considered *linear*.

In concrete with durable coarse aggregate there can be jumps in the scaling curve, because coarse grains are gradually undermined and lost after a certain numbers of freeze-thaw cycles. Seen over a longer period the scaling is more or less linear, however. In Figure 11 typical scaling curves obtained at a laboratory salt scaling test are shown, /5/.

For concrete with aggregate that is itself prone to salt frost scaling, like limestone or other fine-porous aggregate, but in which the cement mortar phase is durable, the scaling curve can be expected to be similar to the case with durable aggregate but non-durable cement mortar. The only difference is that the cement mortar is undermined instead of the aggregate grains. The different scaling curves are visualised in Figure 12.

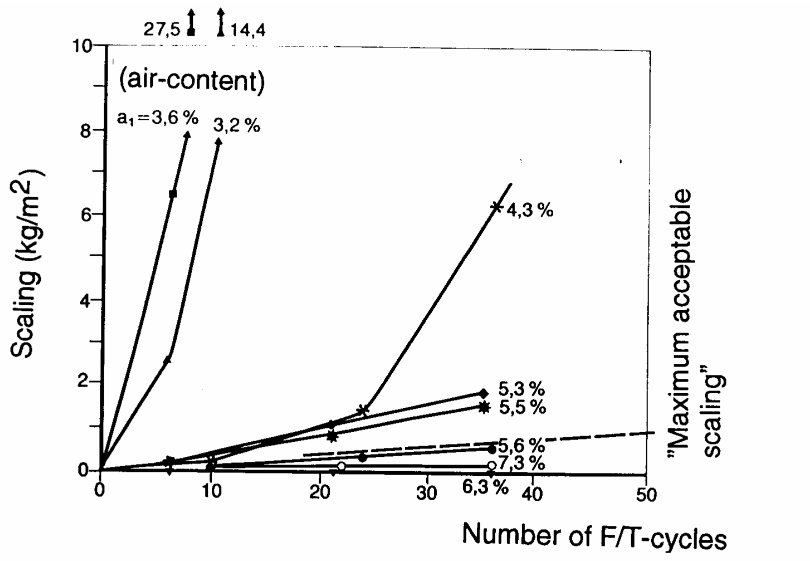

Figure 11: Results of a salt frost scaling test performed by the Swedish test method SS 13 72 44, /5/

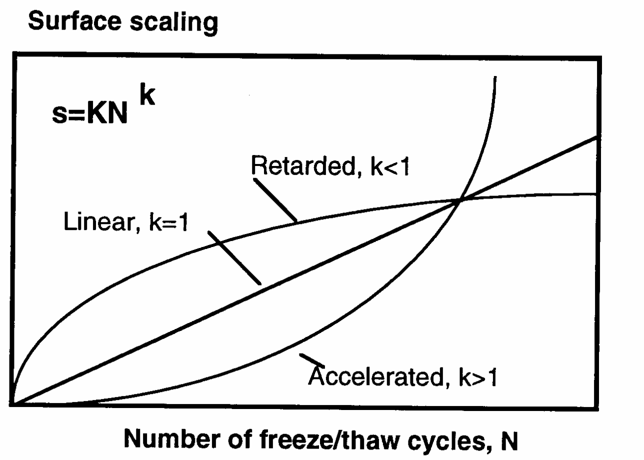

Figure 12: Different salt frost scaling curves-principles

4. Service life with regard to salt frost scaling

Salt scaling has a number of negative effects:

1: Scaling causes aesthetic damage
2: Scaling causes reduction of the effective cross-section of the concrete member
3: Scaling reduces the anchorage capacity of the reinforcement
4: Scaling reduces the service life with regard to reinforcement corrosion by reducing the concrete cover

Only the last three effects are harmful to the structural stability.
If the scaling rate in different parts of a structure is known, it is possible to assess the effect of

scaling on the time process of the stability and safety of the structure. Normal methods for structural analysis can be used, at which the reduced cover and cross-section are inserted in the normal design formulas; ANNEX H.

The effect of scaling on the service life with regard to reinforcement corrosion can be estimated according to the method described in ANNEX F.

5. Concrete technology factors affecting salt frost scaling

The same factors, that were important for internal frost resistance, are also relevant for salt scaling; see ANNEX A paragraph 7. For salt frost scaling there are some modifications to the critical value of some parameters. There are also some additional important parameters. New values and parameters are described below.

5.1 Air content

If a concrete shall be resistant to salt frost scaling it is normally important to have a bit higher air content than what is required for internal frost damage. The reason is that the critical spacing between air-pores seems to be a bit lower.

It is not possible to find a general value of the air content above which a concrete is always salt frost resistant. Examples that this is not possible are shown in Figure 13 where the result of salt frost scaling tests of a big number of concrete is plotted versus the air content of the fresh mix. There are concrete with air contents above 9.5% that were completely destroyed during the test, while other concrete with air content below 5% were undamaged. The reason behind this very big scattering in results is that different air-pore systems have very big differences in ”quality”. Some air-pore systems are unstable during the casting phase. Consequently the pore system collapses more or less completely during the production phase. In many cases the air-pore system forms more or less continuous channels that are able to suck water. Such air-pore systems are not effective in protecting the concrete. On the contrary, they cause damage when water in the ”air-pore channels” freezes.

Those unstable air-pore systems are often found when the cement has high alkali content, and when the air-entraining admixture is mixed with a water reducing (plasticising) admixture, with which it is not compatible, /5/. If only those concrete are selected from Figure 13 that contain a pure air-entraining agent of high quality and no plasticising agent, it is found that an air content of 5.5% seems to be high enough to create concrete with high salt frost resistance.

As a consequence of this big effect of factors influencing the air-pore stability, it is not possible to say in advance that a certain air content of the fresh (or hardened) concrete will create a concrete with high salt frost resistance.

The required air content in a concrete to be cast must therefore always be determined by a salt scaling test. Similarly, the potential salt scaling resistance of an existing structure can only be assessed by salt frost tests of cored specimens, not on the air content of the concrete.

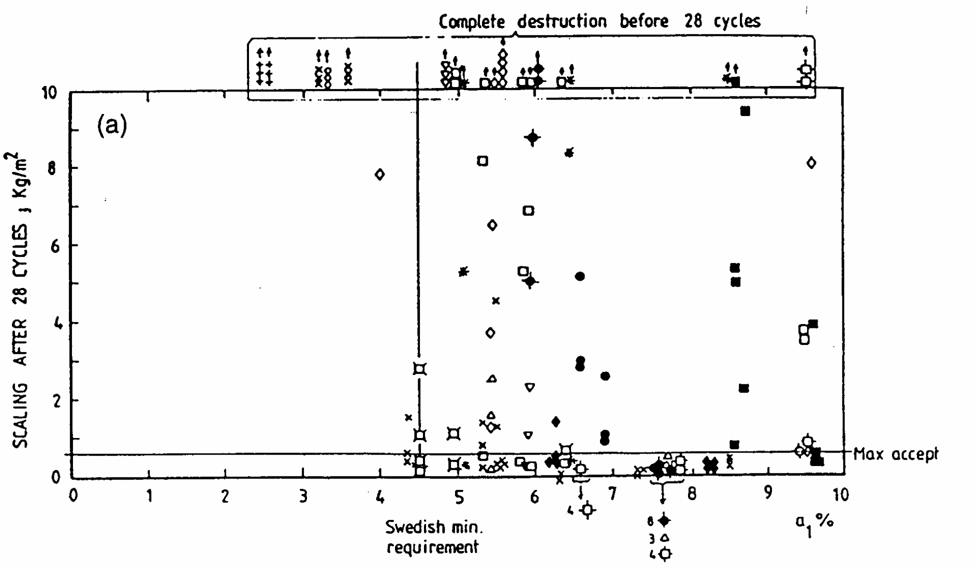

Figure 13: Relation between the air content of the fresh concrete mix, and the salt scaling resistance in the laboratory. Many concrete types (all with w/c-ratio 0.45) produced with different types of air-entraining agents combined with different plasticising agents; /5/.

5.2 Spacing factor

The critical Powers spacing factor for salt scaling, all air-pores included in the calculation, is not very well-known. There are indications that it is of the order 0.18 to 0.20 mm; see Figure 14.

Other tests in which more than 50 concrete types were produced did not reveal the same good agreement between salt scaling and spacing factor as in Figure 14. In fact no correlation was found; /5/. These observations, therefore, strengthen the conclusion that the best way of assessing the potential salt frost scaling resistance of a concrete is to perform a freeze-thaw test in the laboratory.

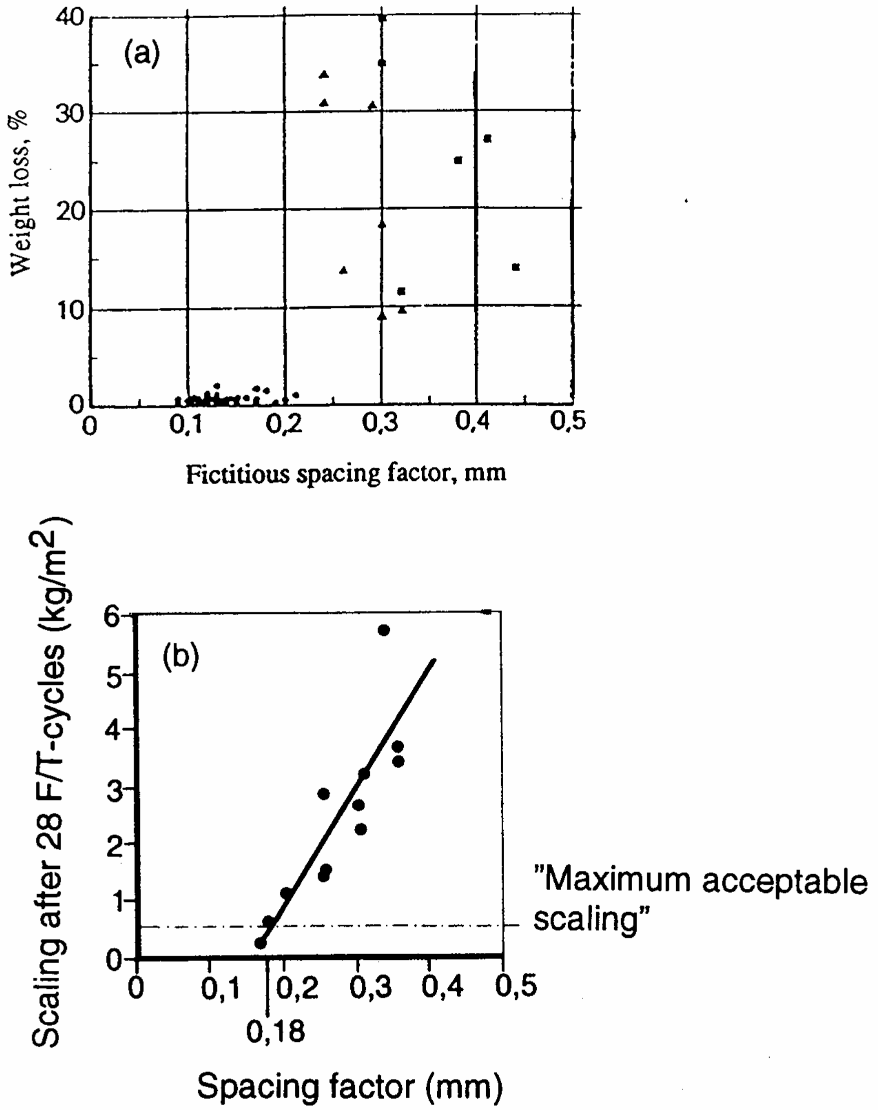

Figure 14: Effect of the Powers spacing factor including all air-pores on the salt scaling resistance. (a) From /6/. (b) From /7/.

5.3 Water-binder ratio

One can assume that the salt scaling resistance increases with decreased water-binder ratio. Normally a w/b-ratio below 0.45 to 0.50 is required. It has been found, however, that a low water cement ratio alone cannot give a concrete with high salt frost resistance unless it is below 0.30; /8/.

5.4 Type of cement

5.4.1 Type of portland cement

The type of Portland cement is of big importance for the possibility of producing concrete with high salt frost resistance. Swedish experience show that portland cement with low alkali content and high sulfate resistance (low C3A) are more suitable for salt frost resistant concrete than normal portland cement types; /5/, /9/.

In Figure 15, a, comparison is made between two cements produced in Sweden. Concrete with different air content, and two types of air-entraining admixtures (C88L and AerL) were produced. The cement of type "Degerh" which is low alkali and low C_3A, produced concrete with high salt scaling resistance with air contents that are as low as 4%. The cement of type "Slite", which is high alkali, and high C3A, produced concrete that required 5,5 to 6% of air in order to obtain an acceptable salt scaling resistance.

Thus, it seems as if the chemical properties of the portland cement plays a fundamental rôle for the salt scaling resistance. This is most probably an effect of the air-pore structure produced. It was shown by Mielentz et al. /10/, that an increased alkali content produced a more coarse air-pore structure.

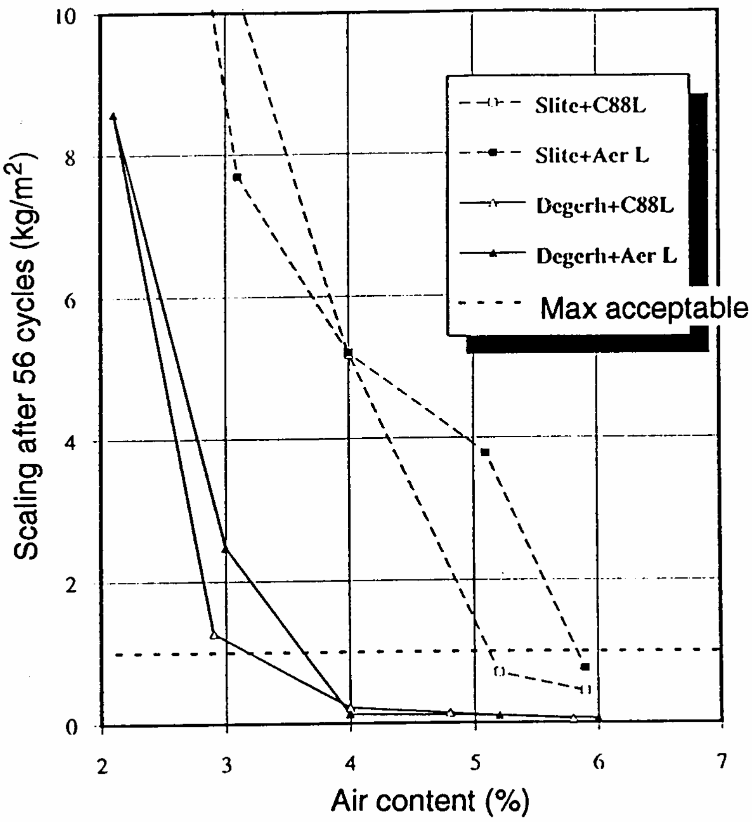

Figure 15: The effect of the "fresh" air content on the saltfrost scaling of concrete made with two types of cement ("Slite" and "Degerh" and 2 types of air-entraining admixtures (C88L and Aer L); /9/.

5.4.2 Mineral admixtures

Mineral admixtures might affect the salt frost scaling resistance. Mineral admixtures of interest are

* Fly ash from coal burning
* Silica fume (or ”Microsilica”)
* Ground granulated blast furnace slag

Fly ash

There are some negative effects caused by fly ash. The first effect to consider is an increased variation in the air content of the fresh mix. Small variations in the loss on ignition of the ash, and in the so-called ”organic content”, cause large variations in the required dosage of the air-entraining admixture. This means that the variation in the frost resistance of concrete containing fly ash can be bigger than normal.

A possible, second, negative effect, is a bad long-term performance. This was found in a big study at Treat Island in Canada, where concrete containing different amount of fly ash, slag and silica fume were exposed to natural weathering by tidal Atlantic water and freeze/thaw; /11/. Some results are shown in Figure 16 for concretes with high air content. The general trend is, that an increased amount of fly ash impairs the long-term behaviour.

Another example from the Treat Island investigation is shown in Figure 17. The fly ash was mixed with slag. The negative effect on the long term behaviour is obvious. The frost resistance after a short time of exposure is, however, about the same for all concretes.

The reason behind this behaviour can be twofold, (i) concrete containing large amounts of fly ash, and/or slag, has a reduced possibility of self healing cracks and other defects, that appear during the long-term exposure; (ii) the air-pore system is more rapidly filled by water, and thereby inactivated, during long-term exposure to water, than concrete with portland cement.

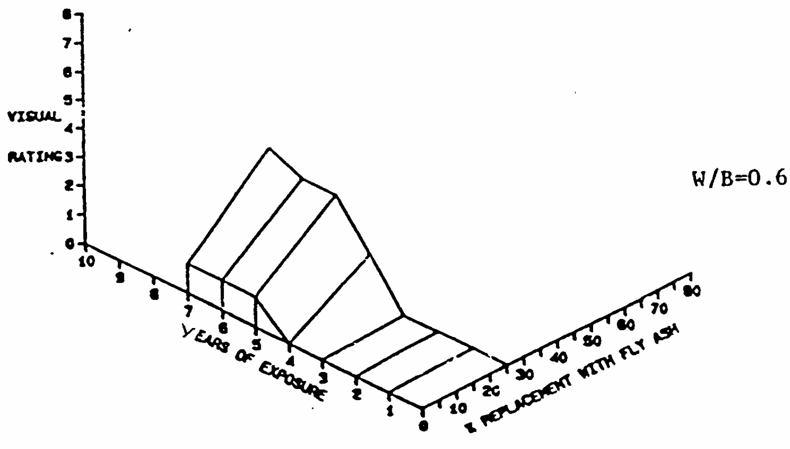

Figure 16: Result of exposure tests at Treat Island in Canada on the long-term behaviour of air-entrained concrete containing fly ash; /11/.

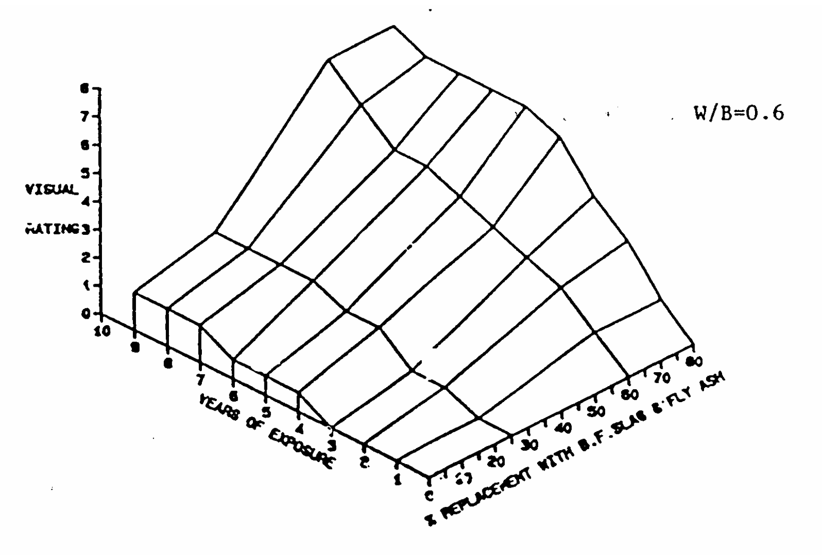

Figure 17: Result of exposure tests at Treat Island in Canada on the long-term behaviour of air-entrained concrete containing a combination of fly ash and blast furnace slag; /11/.

Silica fume

Silica fume was not used in concrete until about 20 years ago, and until recently mostly in the Scandinacvian countries and in Canada. It is normally used in quantities not exceeding 10% of the cement weight, and often only 5%. Therefore, its effect on the frost resistance seems to be marginal, provided the air content is maintained on a high level. Besides, the long-term tests at Treat Island indicate, that a negative effect occurs only when the amount of silica fume exceeds 10%; see Figure 18.

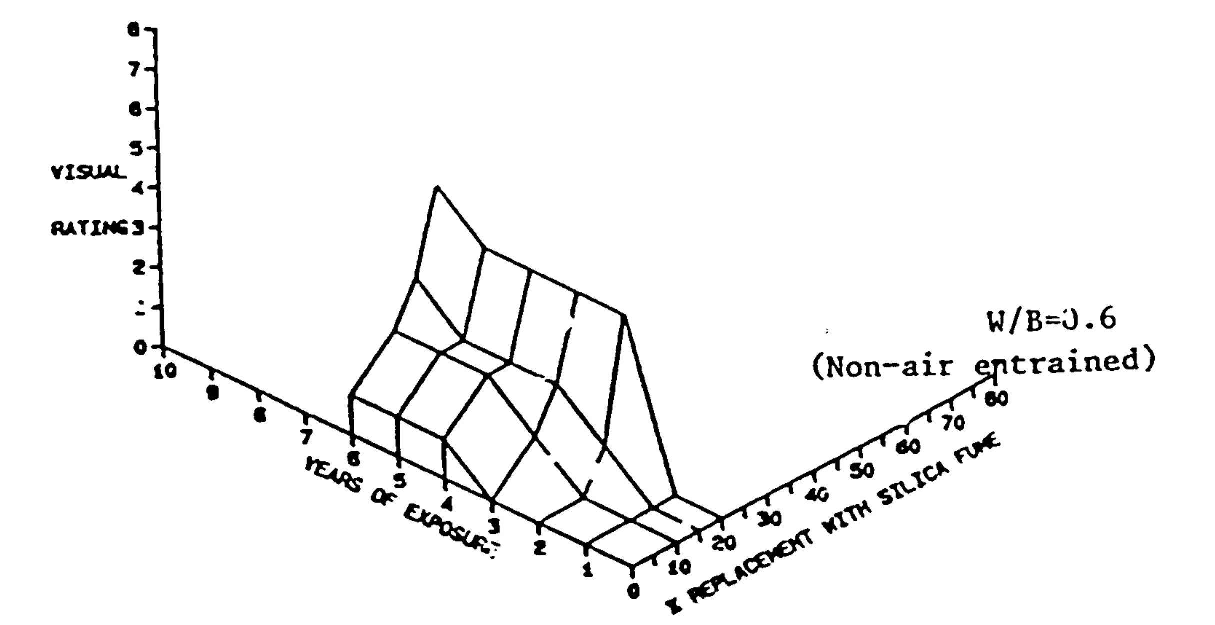

Figure 18: Result of exposure tests at Treat Island in Canada on the long-term behaviour of air-entrained concrete containing silica fume; /11/.

It has been shown, that the salt scaling in a standard test of concrete containing silica fume, might become accelerated after some time; /12/. This behaviour has not been explained. One possibility is, that the air-pore structure in concrete with silica fume is so fine, that the water uptake in the air-pores becomes more rapid. Therefore, the critical water content is reached after a shorter time than for a comparable concrete without silica fume. This explanation has not been experimentally verified, however.

Ground granulated blast furnace slag

It is doubtful whether a concrete containing slag will ever obtain the same good salt frost resistance as concrete containing the pure portland cement on which the slag cement is based. It might be that the short-term durability is the same, but it seems as if the ageing properties of slag cement concretes are rather bad. This was observed in a study in Finland where concrete containing different types of binders were first frost tested in a virgin state. Then, the concrete was exposed to two types of natural ageing, and were frost tested once again; /13/. In the virgin state, the slag cement behaved just as well as the portland cement, or even better. After ageing, the salt scaling resistance of the slag cement concrete was reduced. The pure Portland cement concrete, on the other hand, was improved, or unchanged, after the ageing; see Figure 19.

The negative ageing of concrete with slag cement is believed to depend on carbonation that creates a carbonated layer with low salt scaling resistance; /14/, /15/. For OPC concrete, on the other hand, carbonation seems to improve the salt frost scaling resistance; /16/.

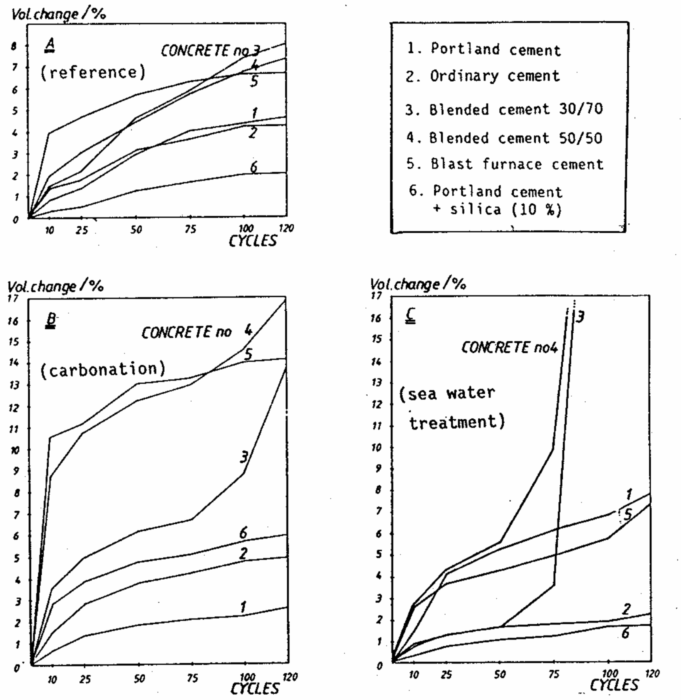

Figure 19: Effect of natural ageing on the salt scaling resistance of concrete containing different types of mineral admixtures, or pure portland cement; (A) before outdoor exposure, (B) after 200 days of exposure to 70% RH, (C) after 30 cycles of exposure to sea water and drying (each cycle 7 days); /13/. (”Ordinary cement” contains 17% fly ash and 6% blast furnace slag;”Blended cement 30/70” contains 30% blast furnace slag; “Bended cement 50/50” contains 50% blast furnace slag: “Blast furnace cement” contains 70% blast furnace slag; Silica (10%)” is 10% silica fume in the mix.)

A similar, negative long-term behaviour of slag cement concretes was observed in the Treat Island test. This is shown by Fig 17, where the slag is mixed with fly ash, but also by Figure 20, where the slag is used alone. The larger the slag content, the lower the durability.

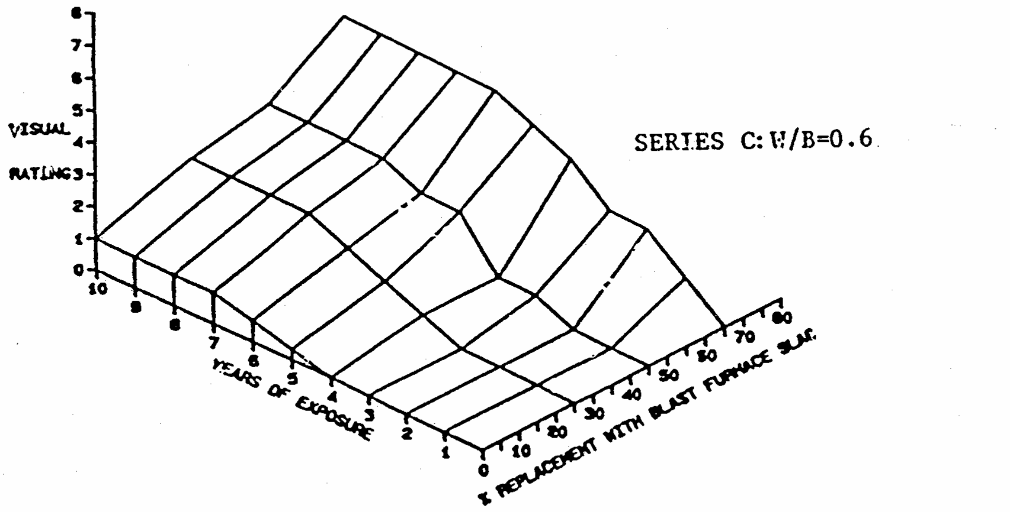

Figure 20: Result of exposure tests at Treat Island in Canada on the long-term behaviour of air-entrained concrete containing ground granulated blast furnace slag; /11/.

5.5 Type of aggregate

Coarse aggregate will only affect the salt scaling resistance and salt scaling process if itself is vulnerable to salt frost scaling. Such aggregate is sandstone, limestone and other fine-porous stone types. Normally, the aggregate phase is more resistant to salt frost scaling than the cement mortar phase.

6. Test methods for salt frost scaling

6.1 General viewpoints

The salt scaling test used for obtaining the results in Figure 1-6, and other similar tests show that salt frost scaling is most severe at a certain pessimum outer salt concentration, and that it is more severe the lower the temperature, and the longer the duration of freezing temperatures. In reality, a concrete structure that is exposed to de-icing salts, can be exposed to any salt concentration; from zero to saturated. A salt scaling test must therefore be designed in such a way, that the surrounding solution has the pessimum concentration, and in such a way, that the most severe freeze/thaw cycle occurring in practice is used.

It will never be possible, however, to design a test method that exactly reproduces the real conditions. Therefore, the result of scaling tests are only indicative. They can be a good complement to field observations of the actual scaling, and then be used for extrapolation of the future scaling. See ANNEX D.

6.2 Salt scaling test

The Swedish test method for salt scaling SS 13 72 44; ”Concrete testing-Hardened concrete-Frost resistance”, also called ”The Borås Method” is recommended. It has been in use in Sweden since about 15 years. From 1988 it is a mandatory method for control of concrete to be used in Swedish bridges. The principles of the method are as follows. For details refer to the text of the Standard:

* The specimens are prepared from 150 mm cubes that are cast and cured in a standardized way. Alternatively the specimen is drilled from a structure. A 50 mm thick slice is cut from the cube or the core. The surface to be exposed to salt in the test can be a saw-cut or the cast surface. All sides except the exposed side are moisture sealed by a rubber membrane.

* At the specified time and curing, the concrete slice is placed in a heat insulated box. The insulation covers all sides of the specimen, except the exposed surface. This is covered by a 3% NaCl-solution, to 3 mm depth. The solution is covered by a thick polyethylene foil, so that evaporation is hindered.

* The freeze-test is run for 56 cycles. Each cycle lasts for 24 hours and consists of about 16 hours of freezing and 8 hours of thawing. The required temperature in the solution is specified.

* The material, that is scaled off from the specimen, is collected during the thawing phase, at different intervals. It is dried, and weighed, and expressed in kg per m^2 of exposed area. The salt solution is renewed before the new series of cycles starts.

* The scaling can be plotted versus the number of cycles. A diagram of type Figure 11 is obtained. The result can, according to the Standard, be assessed according to the following table.

Salt scaling resistance	Requirement
Very good	No specimen is scaled more than 0,10 kg/m^2 after 56 cycles
Good	The mean value for the scaling after 56 cycles is less than 0,50 kg/m^2 and $m_{56}/m_{28}<2$ 1)
Acceptable	The mean value of the scaling after 56 cycles is less than 1,0 kg/m^2 and $m_{56}/m_{28}<2$
Unacceptable	If the requirements for acceptable salt scaling resistance are not fulfilled

1) m_{28} and m_{56} is the mean scaling after 28 and 56 cycles.

A scaling depth of 1 kg/m^2 corresponds to a scaling depth of about 0,5 mm (average density of the scaled material is 2000 kg/m^3). Thus, the criterion for acceptable salt scaling resistance, 1 kg/m^2 after 56 cycles, approximately corresponds to a maximum scaling of about 10 mm after

100 years of exposure, provided there are 10 ”severe” freezing cycles each year. By ”severe” is meant a cycle that is just as severe as a test cycle.

Comment:
The pre-treatment of the specimens seems to have a big influence on the test result. If the concrete is never dried, or if it is exposed to hard drying, the salt scaling seems to be enhanced. Therefore, in the test method, a moderate drying is used, which might represent the real condition in a reasonably good manner. If the concrete is never dried, it contains water in interfaces and in other defects. This water might cause trouble. This water is lost even at a mild drying, but it is not regained when the dried concrete is put in water once again. If the concrete is dried hard, the freezable water is increased, perhaps to an unnaturally high level; See Figure 1 in ANNEX A.

6.3 Determination of air content

The effective air content is determined according to the principles in paragraph 8.2 in ANNEX A

6.4 Determination of air-pore structure and spacing factor

The spacing factor and the air-pore size distribution are determined according to the principles described in paragraph 8.3 in ANNEX A.

References

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ANNEX C
The future internal frost damage

Contents Page

ANNEX C: The future internal frost damage

1. Internal damage

Internal frost damage causes loss in compressive strength and tensile strength, loss in E-modulus, loss in bond strength, and loss in anchorage capacity. These types of damage will have a big effect on the load-carrying capacity and the structural safety of the structure at the present time, and in the future.

There are in most cases no linear relationships between the different types of mechanical damage. So for example, loss in compressive strength is often conside-rably smaller than the loss in E-modulus, or tensile trength. Therefore, all important mechanical properties must be measured, which is best done on drilled-out cores. The loss in bond can be correlated with the loss in tensile strength; see ANNEX E.

> *Comments on synergy:*
> Other destruction mechanisms than frost might affect the future internal frost damage. One example is leaching of lime that might increase the inflow of moisture, thereby increasing the future damage rate. Synergy is treated in ANNEX F.

2. Different principles of extrapolation of damage

From measurements of actual values of relevant mechanical properties in different parts of the structure, the amount of damage in different parts can be assessed. The *future deterioration* can then be estimated according to the principles described below. From the predicted future destruction curves of the mechanical properties, the future development of the structural capacity can be assessed; ANNEX H

There is also an imaginable case where internal frost damage has not yet occurred, but might do so in the future. This case can also be analysed by an approximate method described below.

An extrapolation of the future evolution of damage can be done by two different methods:

* Observation of destruction, at the time of inspection, and extrapolation based on known destruction mechanisms. This method is described below.

* Measurements of the actual level of frost resistance, using freeze-thaw test methods, and extrapolation of the future degradation based on the test result. A method is described below.

Generally, extrapolation based on observations is the best method.

3. Estimation of the present damage

Extrapolation of frost damage according to the principles below is based on the amount of destruction at the time of inspection; the present damage. Therefore, not only the actual strength level, or the actual E-modulus, must be measured, but the damage must also be quantified.

Damage, D, is defined

$$D=\Delta R/R_o = 1-R/R_o \quad (1)$$

Where R the present value of the property of interest (e.g. compressive strength or E-modulus)

R_o the present value of the property of interest had no frost damaged occurred. (normally somewhat higher than the value directly after erection of the structure since hydration takes place.)

ΔR The present change in the property of interest

This means that the strength and E-modulus of the *undamaged* structure must be assessed. In order to do this, the initial strength of the structure adjusted for normal time effects and climatic effects, had no damage occurred, must be estimated. This is not an easy task since there is normally no record of the concrete mix used. In many cases, one can find seemingly undamaged parts of the structure. Measurements of cores taken from such parts are used for determination of the values R_o; ANNEX G. Other possibilities are to use test data for concrete strength from the building phase of the structure, or requirements from norms and standards used when the structure was erected.

4. Effect of moisture and number of freeze-thaw cycles on extrapolation of damage

The extrapolation of damage is based on the following experimentally-based theory which is described in more detail in ANNEX A, paragraph 4.3.

If internal frost damage in a unit volume of the structure shall occur or not depends on whether the outer moisture conditions are such, that the concrete in the unit volume can be more than critically saturated, or not. Frost damage, D, can be expressed in the following way:

$$S_{ACT} < S_{CR}: \quad D = 0 \quad (2a)$$

$$S_{ACT} > S_{CR}: \quad D = K_N \cdot (S_{ACT} - S_{CR}) \quad (2b)$$

Where D ”frost damage” expressed in terms of loss of any type of strength, or E-modulus, in relation to the initial values before frost action; see eqn. (1). (0<D<1)

S_{ACT} the actually occurring degree of saturation, in the concrete volume considered

K_N ‘coefficient of fatigue’, which depends on the number of F/T-cycles

This means that no damage will occur in a concrete, that is less than critically saturated, irrespectively of the number of freeze/thaw cycles. For a given number of freeze/thaw cycles, the amount of damage is only a function of the amount by which the critical degree of saturation is transgressed.

The coefficient K_N can be expressed by an equation of the following type:

$$K_N = A \cdot N/(B+N) \qquad (3)$$

Where N the number of freeze-thaw cycles
A, B empirical constants

A is the 'fatigue limit', expressing the damage after an infinite number of freeze/thaw cycles with a *constant degree of saturation*. B has been found to be of the order 4. Therefore, since N>>B the coefficient B is negligible. The coefficient A is often of the order 12 for a concrete.

> *Example:*
> Eqn. (2b) combined with eqn. (3) predicts that a concrete with an S_{ACT}-value that transgresses the S_{CR}-value by 0,05, will obtain a reduction in strength of 55% after 100 severe freeze/thaw cycles, corresponding to about 10 years of exposure.

5. Extrapolation of observed damage

5.1 Introduction

According to eqn. (2b) the future evolution of frost damage in a given unit volume of a structure will depend on:
1: The future moisture content in the volume.
2: The number of freeze thaw cycles in the future

The number of freeze thaw cycles each year is supposed to be the same in the future as it has been in the past. Consequently, the only factor that influences the future internal frost damage is *the future moisture level*. This means that a prediction of the future deterioration of a given part of the structure must be based on a prediction of the future moisture level in the same part. Three different moisture conditions (or "moisture classes") are considered:

Moisture condition 1; "Moist".
Concrete not permanently exposed to moisture. Moist periods alternating with drying periods. No future change in moisture content.

Moisture condition 2; "Very moist".
Concrete not permanently exposed to moisture. Moist periods alternating with drying periods. A certain gradual increase in the moisture content with time, due to gradual deterioration by freeze-thaw.

Moisture condition 3; "Extremely moist".
Concrete permanently exposed tp moisture. No drying periods. Gradual increase in the moisture content.

These three conditions will lead to three different damage curves; see Figure 1. The same curves can be used for all types of mechanical degradation:

* Compressive strength
* Tensile strength
* Bond strength
* E-modulus

5.2 Moisture condition 1; ”moist”

Many structures are exposed to intermittent ‘moisture load’. Between each rain, or other exposure to free water, the concrete has rather long time to dry. In such a case the moisture level and variation inside the concrete will be the same in the future as it has been in the past; the fact that the concrete has become frost damaged will not cause any increase in moisture uptake. Then, since the fatigue limit is reached after rather few F/T-cycles, one can safely assume that the fatigue limit has been obtained long before the time of inspection (the present time), and that there will be no further frost damage in the future. Thus, the future damage, D(t), is equal to the present damage, D(o), and the future mechanical property (compressive, tensile, bond strength, or E-modulus), R(t), is equal to the present strength, R(o):

$$\mathbf{D(t) = D(o) \quad and \quad R(t) = R(o)} \tag{4}$$

This is the ‘*time-independent damage function*’. It is illustrated in Figure 2, Curve 1.

Structures belonging to this group are exposed to rather few freeze-thaw cycles with high moisture condition, but many cycles with low moisture conditions. The structures dry occasionally. Typical examples of this type of structure are normal façades and other structures with fairly low moisture load, although the number of freeze-thaw cycles might be high.

5.3 Moisture condition 2; ”very moist”

When the conditions are somewhat more moist than in moisture condition 1, there might be be a certain increase in water absorption in frost damaged parts of the concrete during and between each freeze/thaw period. This also occurs when the concrete has a possibility to dry between the cycles, and even in a case where the absorption time before different freeze/thaw cycles is unchanged. The reason is, that frost action causes a gradual cracking of the specimen. This gives an increase in the diffusivity of air from air-pores, and an increase in the diffusivity of water, entering the concrete. A theoretical analysis shows that the water absorption is almost directly proportional to the square-root of the diffusivity. Therefore, the water absorption between each cycle, ΔS_{ACT}, can be assumed to be proportional to the square-root of the number of freeze/thaw cycles:

$$\Delta S_{ACT} = \alpha \cdot N^{1/2} \tag{5}$$

Where α is a constant.

Then, the degree of saturation of the specimen becomes:

$$S_{ACT} = S_{CR} + \Delta S_{ACT} = S_{CR} + \alpha \cdot N^{1/2} \qquad (6)$$

Using eqn. (6), and assuming that the number of F/T-cycles is β each year, the following approximative relation for the development of the damage parameter D(t) can be derived (Note: $\beta \cdot t >> B$ in eqn. (3). Therefore, B is neglected):

$$D(t) = \{A \cdot \alpha \cdot (\beta \cdot t\}^{3/2}\}/(B+\beta \cdot t) = A \cdot \alpha \cdot (\beta \cdot t)^{1/2} = Const \cdot t^{1/2} \qquad (7)$$

This is the *'square-root damage function'*. It is illustrated in Figure 2, Curve 2. The coefficient α is obtained from the observed damage D(o) at the present time, t_o:

$$\alpha \approx D(o)/(A(\beta \cdot t_o)^{1/2}) \qquad (8)$$

Inserting this in eqn. (7) gives the following "damage curve":

$$\mathbf{D(t)=D(o)(t/t_o)^{1/2}=D(o)(1+\Delta t/t_o)^{1/2}} \qquad (9)$$

Or, expressed in terms of the material property considered:

$$\mathbf{R(t)=R_o - [R_o - R(o)] \cdot (1+\Delta t/t_o)^{1/2}} \qquad (10)$$

Where Δt the extrapolation time

Structures belonging to this group are exposed to many freeze-thaw cycles with high moisture condition. The structures dry occasionally. Typical examples of this type of structure are horizontal unprotected slabs (without de-icing salts) like balcony slabs, hydraulic structures some meter above the water line, etc.

5.4 Moisture condition 3; "extremely moist"

A concrete structure can be constantly exposed to moisture with no possibility to dry. Then, the moisture content will gradually increase with time. An analysis of the water absorption process in the air-pore system of a concrete indicates that the water absorption can be approximately described by a square-root relationship:

$$S_{CAP} = S_n + a \cdot t^b = S_{CR} + a \cdot t^b \approx S_{CR} + a \cdot t^{1/2} \qquad (11)$$

Where the exponent b depends on the air-pore distribution. A theoretical analysis indicates that b<1/2. The value 1/2 therefore is on "the safe side" giving the biggest possible damage. S_n is the degree of saturation corresponding to the rapid water absorption in capillary pores and other fine pores. Since the concrete is frost damaged, the value of S_{CR} is obviously transgressed.

Thus, S_n is replaced by S_{CR} which means that the time t is in fact counted from the instant when S_{CR} was reached for the first time. This normally occurred shortly after erection of the structure.

Therefore t in eqn. (11) is almost equal to the total time. a is a coefficient, that depends on the permeability of the concrete. It is individual for each concrete. It is assumed to be independent of the concrete age. As said above, the gradual damage will, however, increase the diffusivity of the concrete. This means, that the degree of saturation increases more rapidly than expressed by eqn. (11). The following relation is used, cf. eqn (5):

$$S_{CAP} = S_{CR} + a \cdot t^{1/2} \cdot \alpha \cdot N^{1/2} = S_{CR} + \varepsilon \cdot t^{1/2} \cdot N^{1/2} \qquad (12)$$

Where $\varepsilon = \alpha \cdot a$, which is a constant that is individual for each concrete. S_{CR} was assumed to be exceeded already at the first freeze/thaw cycle. Therefore, eqn. (12) can be written

$$D(t) = K_N \cdot \varepsilon \cdot t^{1/2} \cdot N^{1/2} \qquad (13)$$

Or, after inserting the fatigue function (3):

$$D(t)=A \cdot \varepsilon \cdot \beta^{3/2} \cdot t^2/(B+\beta \cdot t) \approx A \cdot \varepsilon \cdot \beta^{1/2} \cdot t \qquad (14)$$

This is the *'linear damage function'*. It is illustrated in Figure 2, Curve 3. The coefficient α is obtained from the observed damage D(o) at the present time, t_o:

$$\varepsilon = D(o)/(A \cdot \beta^{1/2} \cdot t_o) \qquad (15)$$

Inserting this in eqn. (14) gives the "damage curve":

$$\mathbf{D(t)=D(o)(t/t_o)} \qquad (16)$$

Or, expressed in terms of the material property considered:

$$\mathbf{R(t)=R_0 - [R_0 - R(o)] \cdot (1+\Delta t/t_o)} \qquad (17)$$

Structures belonging to this group are exposed to many freeze-thaw cycles with very high moisture condition. Typical structures are hydraulic structures, or bridge piers in, and immediately above, the splash zone in freshwater, and structures sucking water from a water reservoir, or from the ground-water.

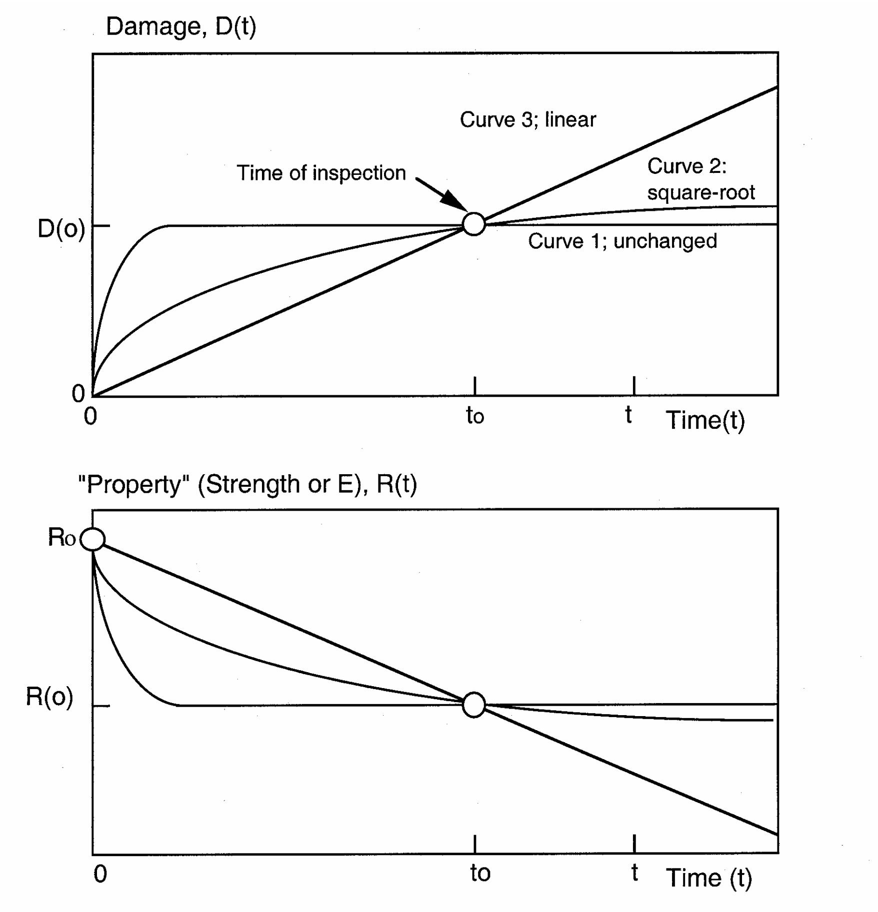

Figure 1: Extrapolation of internal frost damage (D), and strength or E-modulus (R).

6. Future damage of a concrete that is not yet frost damaged

6.1 Extrapolation based on water absorption test

Sometimes it might be, that a concrete is not yet damaged by frost, but is placed in a very wet environment *(Moisture condition 3)*, which means that there might be frost damage in the future. This case can be analysed by the method described below.

Cores are drilled out of the structure and used for two types of test:

1: The critical degree of saturation, S_{CR}, is determined by the so-called S_{CR}-method described in ANNEX G, paragraph 5.4.

2: The capillary degree of saturation, S_{CAP}, is determined by a capillary absorption test on slices sawn from the cores. This test is run for about 1 month. The test is described in ANNEX G, paragraph 5.3. The result of a test can be described:

$$S_{CAP} = a + b \cdot t^{c} \tag{18}$$

Where a, b, c coefficients, that are determined by a regression analysis of the water uptake curve. The coefficients are assumed to be valid also for absorption times far longer than the test period

t total absorption time (time from erection of the structure)

No damage occurs until $S_{CAP}>S_{CR}$ corresponding to the total time t* which is

$$t^{*}=[(S_{CR}-a)/b]^{1/c} \tag{19}$$

$t^{*}>t_{o}$ since frost damage has not yet occurred at the time of inspection.

Then, the future evolution of frost damage can be described by (using eqn (3), and neglecing the coefficient B):

$$\mathbf{D=0 \qquad \text{for } t<t^{*}} \tag{20a}$$

$$\mathbf{D = A \cdot [(a+b \cdot t^{c})-S_{CR}] \qquad \text{for } t>t^{*}} \tag{20b}$$

The residual time before damage is

$$D\mathbf{t=[(S_{CR}-a)/b]^{1/c} - t_{o}} \tag{21}$$

The principles are described in Figure 2.

6.2 Extrapolation based on a combination of the real moisture condition and a moisture absorption test

It might be, that the real moisture content of the structure at the present time t_{o} is different from the calculated according to 6.1, due to the fact that the latter is based on rather unrealistic conditions of constant temperature and uninterrupted water uptake. The reason behind this discrepancy is that the outer conditions are not the same in reality, as they are in the capillary test. In such a case, there is another possibility of calculating the future development of damage, taking the real moisture conditions into consideration.

The age t_{o} of the structure is known. The theoretical present degree of saturation determined by the continuous suction test described above is:

$$(S_{CAP})_{o,test} = a + b \cdot t_{o}^{c} \tag{22}$$

The real moisture content is determined for the structure at inspection, and is translated to a degree of saturation, $(S_{CAP})_{o,real}$.

If the concrete had been exposed to free water all the time from erection of the structure, the following relation should have been valid:

$$(S_{CAP})_{o,real} = (S_{CAP})_{o,test} \tag{23}$$

Deviations indicate that the real water uptake occurs in another manner than in an isothermal suction test. On basis of the water content at inspection, one can calculate the real water uptake by the following equation:

$$(S_{CAP})_{real} = a + b_{real} \cdot t^{c} \tag{24}$$

Where the coefficients a and c, which are mostly depending on the air-pore structure and diffusivity, are *taken from the absorption experiment*. The coefficient b, on the other hand, is the rate-determining constant, that is supposed to depend mostly on the outer moisture conditions.

This means, that the coefficient b_{real} can be evaluated by:

$$b_{real} = [(S_{CAP})_{o,real} - a]/t_{o}^{c} \tag{25}$$

Where $(S_{CAP})_{o,real}$ the degree of saturation at the inspection.

No damage occurs until $S_{CAP} > S_{CR}$ corresponding to the total time t*, which is

$$t^* = [(S_{CR} - a)/b_{real}]^{1/c} \tag{26}$$

t*>to since frost damage has not yet occurred at the time of inspection.

Then, the evolution of frost damage can be described by (using eqn (3), and neglecting the coefficient B):

$$\mathbf{D=0} \qquad \textbf{for } \mathbf{t<t^*} \tag{27a}$$

$$\mathbf{D= A\cdot[(a+b_{real}\cdot t^{c})-S_{CR}]} \qquad \textbf{for } \mathbf{t>t^*} \tag{27b}$$

The residual time until damage is:

$$D\mathbf{t=[(S_{CR}-a)/b_{real}]^{1/c} - t_{o}} \tag{28}$$

The principles of this extrapolation of damage are described in Figure 2.

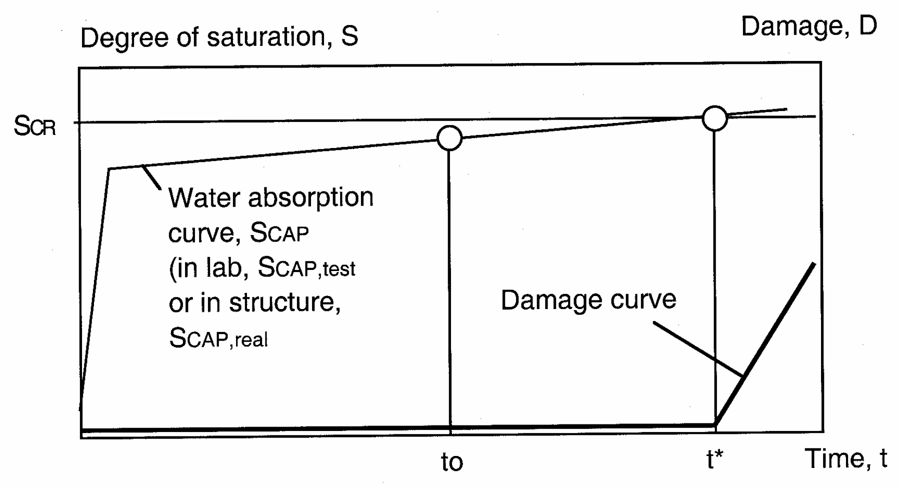

Figure 2: Assessment of the future evolution of damage in a structure for which frost damage has not yet occurred.

7. Estimation of the future internal frost damage based on freeze-thaw tests

The most safe prediction is based on a measurement of the actual damage, and use the extrapolation techniques described in paragraph 5 above.

There is no good, traditional, freeze/thaw method by which one can obtain the service life. The most widely used method is the American method ASTM C666. In this, cores are exposed to rapid freezing and thawing in water, or freezing in air and thawing in water. One measures the loss in fundamental frequency of transverse vibration (which is a function of the loss in weight and E-modulus). The method has, however, no known relation with the destruction in the field. It will only give an indication of whether the concrete is ”excellent”, ”good”, ”fair”, ”average” or ”bad”. The way it behaves in practice, depends on the wetness around the concrete. A concrete, that is ”bad” in the test, might be ”good” in reality, simply because the real environment is more dry than in the test.

The S_{CR}-method is a non-traditional test method, which gives information of the so-called potential service life. The principles of the method are described in ANNEX G.

A determination of the critical degree of saturation, and of the capillary absorption process, gives information of the time the concrete can absorb water without obtaining frost damage. This is the *”potential service life”*; see ANNEX A, paragraph 6.2. This time can be compared with the length of the periods during which the actual concrete takes up water without any possibility to dry. If the real absorption time is much shorter than the potential service life, one can assume, that the concrete will not be harmed by frost in the future. If the suction times are of the same order of size as the potential service life, one can assume that frost damage might occur during periods which are both wet and cold.

If the actual absorption times are much higher than the potential service life, one can assume that the concrete is vulnerable to frost damage in the future.

Additional information is provided by measurements of the moisture content of the concrete in-situ. If the actual degree of saturation is close to the critical degree of saturation, or if it is higher, the risk of frost damage is very big. In such a case, the structure is, however, probably already frost damaged, and the extrapolation methods described in paragraph 5 above can be used.

ANNEX D
The future salt frost scaling

Contents Page

ANNEX D: The future salt frost scaling

1. Introduction

Salt frost scaling, only affects the surface of the concrete, while the interior is unharmed in most cases. If internal damage also occurs, the extrapolation of this is treated as normal internal damage according to ANNEX C.

Salt frost scaling is obvious, and the extent of damage can be estimated visually. Salt scaling causes reduced effective cross section, reduced bond, and reduced anchorage capacity. It reduces the service life with regard to corrosion of the reinforcement, and it causes troubles with aesthetics.

The scaling depth is determined. From this the future scaling can be assessed. Consideration can be taken to possible changes in the saline environment.

On the basis of scaling depth versus time the future structural capacity can be assessed.

> *Comments on synergy:*
> Salt scaling will have consequences for the future service life with regard to reinforcement corrosion. In order to estimate this effect, one must consider the following factors:
>
> 1: The previous salt scaling before the time of inspection.
> 2: The future salt scaling
>
> Theories for how to consider these synergetic effects between frost and other destruction types are described in ANNEX F.

2. Different principles of extrapolation of damage

From measurements of the actual scaling depth, the future scaling can be predicted. The future scaling curve can be used for a prediction of the future load-carrying capacity.

An extrapolation of the future evolution of scaling can be done by two different methods:

* Observation of scaling, at the time of inspection, and extrapolation based on known destruction mechanisms. This method is described below.

* Measurements of the actual level of salt frost resistance, using a salt scaling test method, and extrapolation of the future degradation based on the test result. A method is described below.

Generally, extrapolation based on observations is the most reliable method.

3. Estimation of the present scaling

In order to predict the future scaling, one must be able to determine the actual scaling, S(o), at the time of inspection. The only method of doing this is to try to identify the initial surface, either directly on the structure itself, or from information given in the original building documents. In ideal cases, one can use parts of the structure that are unharmed. The scaling depth of the mortar phase next to coarse aggregate will also give quantitative information of the scaling depth of the cement mortar phase.

Surface scaling is the scaling depth in a point, calculated from the initial surface above the same point. Therefore, since the scaling front is not perfectly smooth, the scaling will be different in different points of the same concrete cover. Normally, it is the deepest scaling, i.e. the scaling in the cement mortar phase, that is determining the load carrying capacity, and other properties of importance for the structural stability.

4. Extrapolation of observed scaling

4.1 Introduction. Scaling characteristics

Salt scaling is a surface erosion, caused by simultaneously acting salt solution at the surface, and freezing temperature. Scaling is caused by a mechanism that is similar to frost heave in the ground; see ANNEX B and /1/. In salt scaling, the growing ice lenses are microscopic instead of macroscopic, as they are in frost heave, and are very close to the surface. They are fed by unfrozen water at the surface. As long as there is unfrozen solution, i.e. as long as temperature is above -21°C for NaCl, the ice lenses can grow. Only a very small amount of salt is required for salt scaling to occur. If the solution is strong, the destructive action decreases due to a reduced amount of freezable water. Air-pores act as stress relievers by allowing ice to grow without causing stress in the material; /2/.

According to this mechanism, each new freeze-thaw cycle will cause the same amount of scaling, provided the cycle is unchanged; unchanged freezing temperature, freezing rate, salt concentration, and moisture condition. Thus, scaling ought to be more or less linear with time. In a scaling test, however, and often in a real structure, there are three types of scaling observed; *retarded, linear and accelerated*; see Figure 1. Scaling as function of the number of freeze-thaw cycles is described by:

$$S \approx K \cdot N^{k} \qquad (1)$$

Where
- S — scaling
- K — coefficient
- k — exponent (k<1 for retarded scaling, k=1 for linear scaling, k>1 for accelerated scaling)
- N — the number of identical freeze-thaw cycles

Normally, concrete with very low degree of salt scaling resistance has accelerated scaling, probably because the water content increases between each cycle. Therefore, such concrete normally is so severely damaged after rather few years (or cycles in a test), that a service life prediction, or a prediction of of the future structural stability, is hardly relevant. Therefore, only linear or retarded scaling are of practical interest.

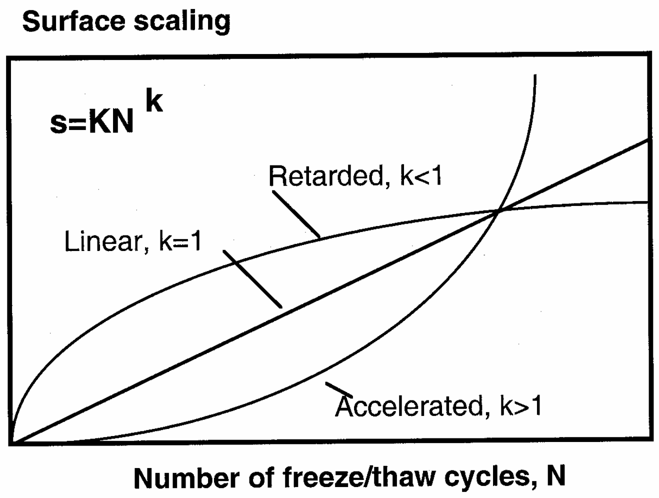

Figure 1: Linear, retarded and accelerated salt scaling in a salt scaling test.

The amount of scaling depends on the salt concentration, a concentration of the outer solution of about 2 to 4% being the most aggressive for normal concrete, but also smaller scaling, as low as 0.5%, can cause considerable scaling. Scaling is also depending on the minimum temperature during a freeze/thaw cycle. Research indicates that the temperature dependency can be described by the following approximate formula; (from data in /1/):

$$S_1 = \text{constant} \cdot \theta_{min}^2 \qquad (2)$$

Where S_1 the scaling caused by one F/T cycle

θ_{min} the lowest temperature reached during the cycle

This means that 10 cycles to a temperature of -20°C correspond to about 50 cycles to -10°C, and about 150 cycles to -5°C. When the aggregate grains are dense, such as granite, quartzite, etc, salt scaling only affects the cement paste phase. Porous aggregate, like limestone or shale, can be severely scaled, however.

The future deterioration depends on the future environment, especially the exposure to salt. For a marine structure, the salt concentration will be unchanged, and, therefore, the rate of deterioration will be maintained at the same level as before. For a structure exposed to de-icing salts, a reduction in the amount of salt used will completely change the future degradation. If the use of de-icing salts is stopped, there will be almost no more scaling. If it is unchanged, or only slightly increased, the rate of scaling is maintained at about the same level as before.

4.2 Homogeneous concrete

If the concrete is completely homogeneous, so that there is no separation layer at the surface, and the air-pore structure is the same across the entire cross section of the concrete, one can assume that the 'historic' scaling has proceeded along a linear path. Certainly, the scaling is not completely linear. There will be 'jumps' in the scaling curve; loss of a small grain will cause a small jump, and the loss of a coarse grain a big jump. Seen over a longer period, and/or over a larger surface area, the average scaling expressed in terms of weight loss (kg/m^2) will, however, be fairly linear. See Figure 2.

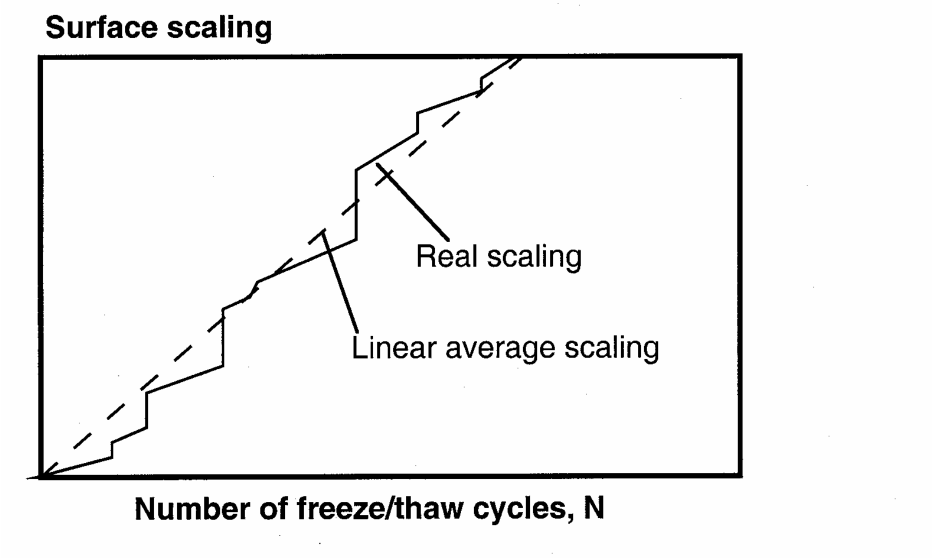

Figure 2: Jumps in a "linear" salt scaling curve, caused by the loosening of aggregate particles.

This means that the future scaling can be extrapolated from the present scaling observed at the inspection. The increase in scaling depth after time Δt counted from the time of inspection will be:

$$\mathbf{\Delta S(t) \approx (S(o)/ t_0)\cdot \Delta t} \qquad (3)$$

Where $\Delta S(t)$ the additional scaling after the additional time Δt
$S(o)$ the scaling at inspection (the present age of the structure)
t_0 the age of the structure at inspection

And, the total scaling from the time the structure was first exposed to de-icing salts (normally the age of the structure) will be

$$\mathbf{S(t)=S(o)\cdot(t/t_0)} \qquad (4)$$

Where $S(t)$ the scaling at time t (the age of the structure)
In reality, the scaling front is not sharp. One might assume, however, that the scaling rate in each point will be about the same as the 'historic value'. Thus, the general shape of the 'scaling profile' will be maintained.

The extrapolation according to eqn (4) is illustrated in Figure 3.

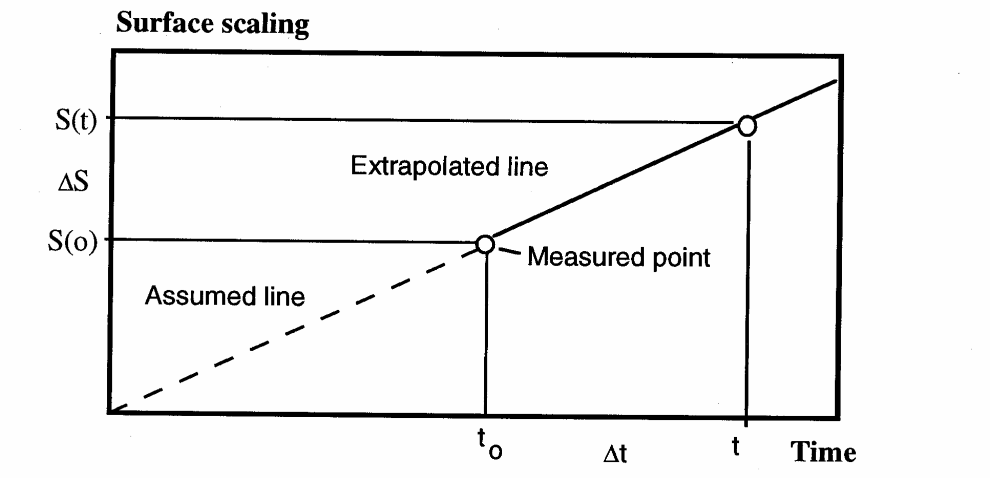

Figure 3: Extrapolation of the observed salt scaling

4.3 Heterogeneous concrete

Normally, concrete is non-homogeneous. There are at least three types of heterogeneity, which might affect the extrapolation of the scaling curve:

* Water separation, and cement mortar segregation.
* Different air-pore structure at the surface; normally there is a lower air content, which depends on air loss to the surface during compaction.
* Defect curing of the concrete surface.

Cement mortar separation can be quite big. Therefore the scaling front ought to be more smooth at the beginning, and more rough later on. Thus, if the scaling depth is counted from the upper level of the aggregate, the rate of this type of scaling has gradually decreased with time. This means that the future scaling, occurring after the time of inspection, will be somewhat exaggerated at linear extrapolation; therefore, eqn. (4) is on 'the safe side'.

An imperfect *air-pore structure* in the surface part causes a reduced salt scaling resistance. Thus, there will be a gradually retarded scaling, and the use of eqn. (4) will lead to a pessimistic prediction of the future scaling rate.

Imperfect curing has limited effect on frost resistance and salt scaling resistance. If there is an effect, it will be negative. This will lead to retarded scaling.

The conclusion is that all types of non-homogeneity ought to lead to retarded salt scaling. Thus, eqn. (3) and (4) can be used also for the case of non-homogeneous concrete. It will lead to an extrapolated residual service life on the 'safe side'.

$$\mathbf{\Delta S(t) \approx (S(o)/t_0)\cdot \Delta t} \qquad (3)$$

$$\mathbf{S(t) \approx S(o)\cdot(t/t_0)} \qquad (4)$$

5. Extrapolation of the future salt frost scaling based on scaling tests

There are cases where the ”historic” scaling cannot be used, or will be very misleading; e.g. when the future environmental conditions will be changed, or when the conditions were changed some time before the time of inspection. Three important cases can be observed:

Case 1. No scaling is observed, but might come in the future

De-icing salts has not been used in the past, but will be used in the future. Then, a concrete that is not scaled at the inspection, might obtain very severe damage in the future. An ‘extrapolation of history’ is very misleading.

Case 2: Scaling is observed but will not continue

De-icing salts have been used during the entire life of the structure, but will be used no more in the future. Then, extrapolation of the ‘historic’ scaling curve is too pessimistic.

Case 3: Scaling is observed, and the average scaling rate will increase in the future

De-icing salts have been used only during a period immediately preceding the time of inspection, but not when the concrete was younger. Extrapolation of the observed scaling from time zero will be too optimistic in a case where salt will be used in the future.

For cases 1 and 3 there is a certain possibility of estimating the future salt scaling from the results of a salt scaling test. Cores with diameter not less than 10 cm are taken from the concrete. The specimens are exposed to a salt scaling test, e.g. the Swedish Standard Test, SS 13 72 44 in which the surface is exposed to 56 (or 112) 24-hour long freeze/thaw cycles in 3% NaCl-solution, or in any other relevant solution, such as sea water, or pure water. The gradual weight loss is measured. As mentioned, the minimum freezing temperature is important. Therefore, one shall use temperatures that are realistic for the actual environment (normally a temperature of -18 to -20°C is used for Scandinavian conditions. A temperature of -10 to -15°C might be more realistic for less harsh climate).

If the concrete is fairly resistant, there will be a fairly linear scaling which is expressed in terms of dry weight loss, Q (kg/m^2). The future scaling in the real environment is:

$$Q(t)=(Q_n/n)\cdot n_{equiv}\cdot t \qquad (5)$$

Where	$Q(t)$	the total scaling after t years in the real environment [kg/m^2]
	Q_n	the measured scaling in the scaling test after n F/T-cycles
	n_{equiv}	the average number of F/T-cycles per year in the real environment
	t	the exposure time (the age of the structure) [years]

n_{equiv} is calculated by; see eqn. (2):

$$n_{equiv}=\Sigma\, n_i\cdot\{(\theta_{min})_i/(\theta_{min})_{test}\}^2 \quad \text{for} \quad \theta_{min} < \pm 0°C \qquad (6)$$

Where n_i annual number of F/T-cycles with minimum temperature $(\theta_{min})_i$
$(\theta_{min})_{test}$ the minimum temperature used in the scaling test.

The weight of the scaled material can be transformed into the average scaling depth. The scaling depth at time t is

$$\mathbf{d(t)=(1/\gamma)\cdot(Q_n/n)\cdot n_{equiv}\cdot t} \qquad (7)$$

Where d(t) the scaling depth after t years [m]
γ the density of the scaled material [kg/m^3]
(a normal value is 2000 kg/m^3)

Example:
In a salt scaling test to -22°C, total scaling after 56 cycles is 1.5 kg/m^2. The annual F/T spectrum is: 6 F/T to -18°C; 15 F/T to -15°C; 22 F/T to -10°C; 40 F/T to -7°C; 100 F/T to -3°C. The average density of the scaled material is 2000 kg/m^3.
Then, $n_{equiv}=6(18/22)^2+...+100(3/22)^2=21$ cycles/year. The scaling depth after 20 years is:

$d(20)=(1/2000)\cdot(1.5/56)\cdot 21\cdot 20=0.006$ m (6 mm)

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/1/ Lindmark, S.: Mechanisms of salt frost scaling of portland cement-bound materials: Studies and hypothesis. Div. Building Materials, Lund Institute of Technology, Report TVBM-1017, 1998.

/2/ Powers, T.C., Helmuth, R.A.: Theory of volume changes in hardened portland cement paste during freezing. Highway Research Board, Proceedings, No 32, 1953.

ANNEX E
Effect of internal frost damage on mechanical properties of concrete

Contents Page

ANNEX E: Effect of internal frost damage on mechanical properties of concrete

1. Introduction

Internal frost damage manifests itself as

* Reduction in compressive strength
* Reduction in tensile strength
* Reduction in bond strength
* Reduction in E-modulus

The bigger the transgression of the critical moisture content, the bigger the reduction in the mechanical properties.

There are no simple relations between reduction of different properties. Thus, a certain relative reduction in tensile strength, or bond strength, can be much bigger than the relative reduction in compressive strength. The same is valid for E-modulus; the reduction in stiffness is often much bigger than the reduction in compressive strength.

The reduction in bond strength is particularly complicated, since it to a very high extent dependent on how the load is applied to the bar, and on how the bar is enclosed by stirrups. Bond tests cannot be performed in-situ on the structure. Therefore, one has to use a substituting property, preferably the tensile strength.

2. Laboratory investigations

The effect of frost damage has been investigated; /1,2/. A number of specimens made with different w/c-ratio and with cast-in reinforcement bars (ribbed and plain) using different bond length (130, 100, 72 and 48 mm) were exposed to very severe freeze-thaw. All mechanical properties of interest were determined before and after freeze-thaw.

In order to obtain maximum possible frost damage, that might occur in practice, the specimens were pre-saturated using vacuum-treatment. By adjusting the residual air-pressure during the vacuum-treatment, different degrees of saturation in the specimen could be achieved. For each combination of concrete quality, bar type and bond length, 4 specimens were used. One of each specimen was not exposed to frost. The other 3 specimens were freeze-thaw tested with moisture contents well above the critical. The maximum moisture content used corresponds to almost complete saturation. It is hardly possible to obtain more frost damage in a real structure, than that obtained in these saturated specimens.

The results from this investigation is the basis for the data given in this ANNEX.

3. Frost damage and compressive strength

The relation between the compressive strength of frost damaged concrete and undamaged concrete is shown in Figure 1. There is a fairly good correlation. A regression analysis gives the following *mean* relation:

$$\mathbf{f_{c,damaged} = 0.96 \cdot f_{c,o} - 9.1 \qquad r^2 = 079} \qquad (1)$$

Where $f_{c,damaged}$ the compressive strength of frost damaged concrete [MPa]
$f_{c,o}$ the compressive strength of undamaged concrete [MPa]

Thus, the *mean* loss in compressive strength is about 10 MPa. The *mean relative* loss in compressive strength is therefore bigger for lower strength grades. Eqn (1) gives the following *mean* relation:

$$\Delta \mathbf{f_c/f_{c,o} \approx 10/f_{c,o}} \qquad (2)$$

Where Δf_c the loss in compressive strength due to frost damage [MPa]

As a *lower bound* relation can be used

$$\mathbf{f_{c,damaged} = f_{c,o} - 20} \qquad \text{for } f_{c,o} > 35 \text{ MPa} \qquad (3)$$

The lowest compressive strength observed is 24 MPa valid for a concrete with initial strength 35 MPa. The biggest strength reduction observed is 35%.

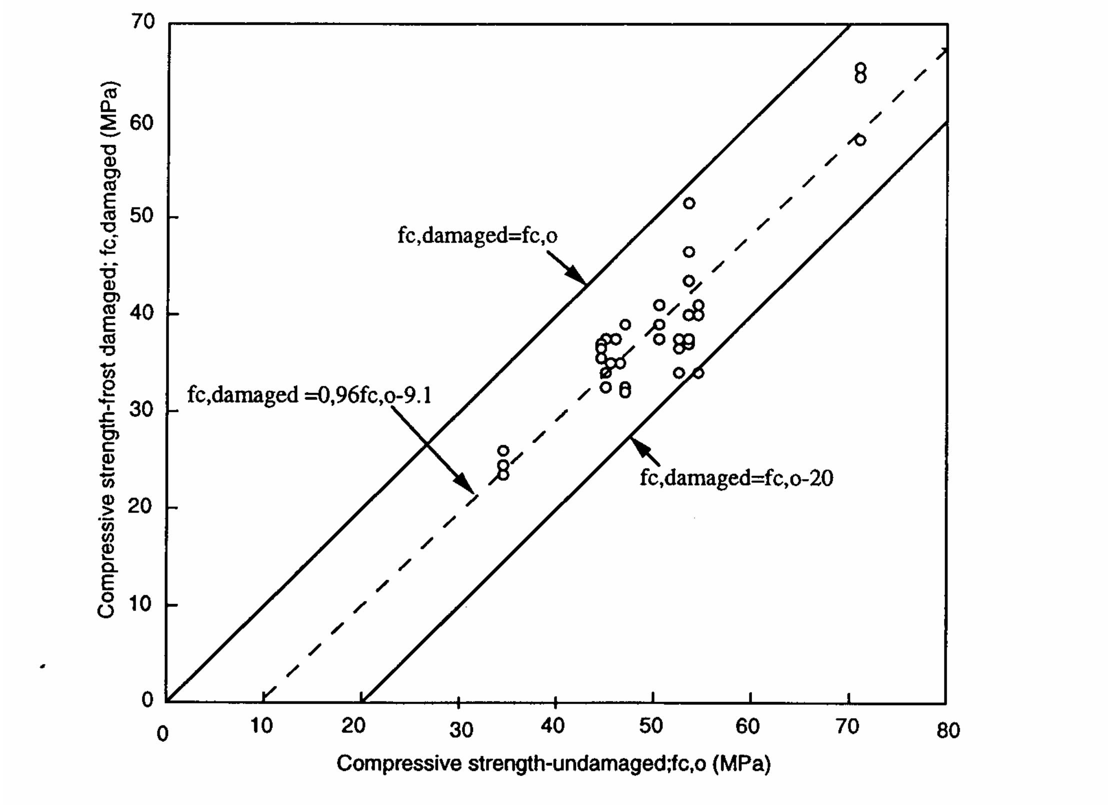

Figure 1: Relation between the compressive strength of undamaged and frost damaged concrete; /1/.

4. Frost damage and split tensile strength

4.1 Reduction is split tensile strength

The relation between the split tensile strength of frost damaged concrete and undamaged concrete is shown in Figure 2. A regression analysis gives the following *mean* relation:

$$\mathbf{f_{t,damaged} = 1.2 \cdot f_{t,o} - 3 \qquad r^2 = 0.47} \qquad (4)$$

Where $f_{t,damaged}$ the split tensile strength of frost damaged concrete [MPa]
$f_{t,o}$ the split tensile strength of undamaged concrete [MPa]

According to this equation the tensile strength of frost damage concrete is zero for concrete with an initial tensile strength below 2.5 MPa.

The *mean relative* loss in split tensile strength is:

$$\mathbf{\Delta f_t / f_{t,o} \approx 3/f_{t,o} - 0.2} \qquad (5)$$

Where Δf_t the loss in split tensile strength due to frost damage [MPa]

As a *lower bound* relation can be used

$$f_{t,damaged} = 3 \cdot f_{t,o} - 11 \quad (6)$$

According to this equation the tensile strength is zero for a severely damaged concrete with an initial tensile strength below 3.7 MPa.

The results show that frost attack might cause very big reductions in tensile strength. The lowest tensile strength observed is 1 MPa valid for a concrete with an initial strength above 3 MPa. The biggest strength reduction observed is 70%.

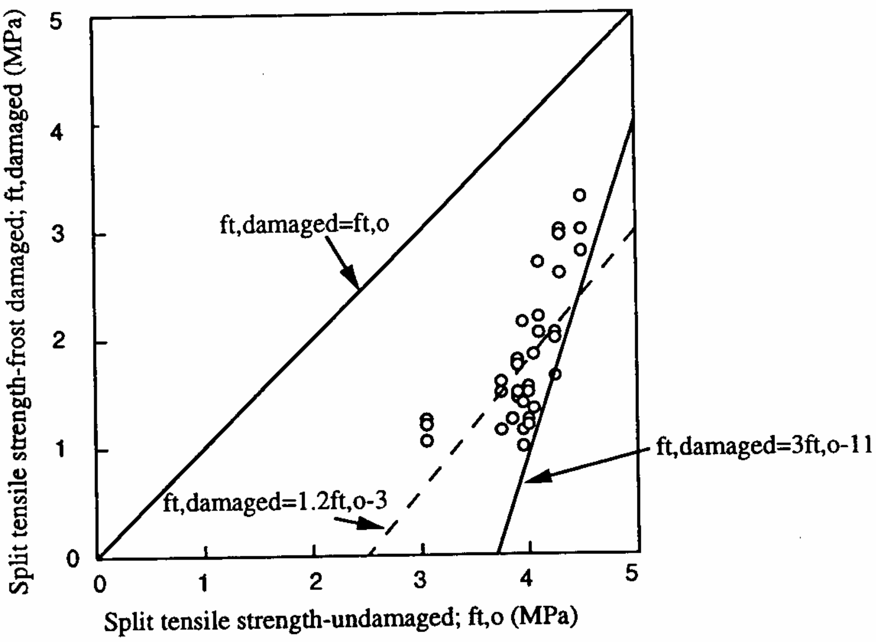

Figure 2: Relation between the split tensile strength of undamaged and frost damaged concrete.

4.2 Relation between reduction in tensile strength and reduction in compressive strength

Frost attack causes much bigger reduction in tensile strength than in compressive strength; see Figure 3 and eqn (7).

The mean relation is:

$$\Delta f_t/f_{t,o} = 1.1\cdot(\Delta f_c/f_{c,o}) + 29 \qquad r^2=0.55 \qquad (7)$$

Where $\Delta f_t/f_{t,o}$ the relative change in split tensile strength [%]

$\Delta f_c/f_{c,o}$ the relative change in compressive strength [%]

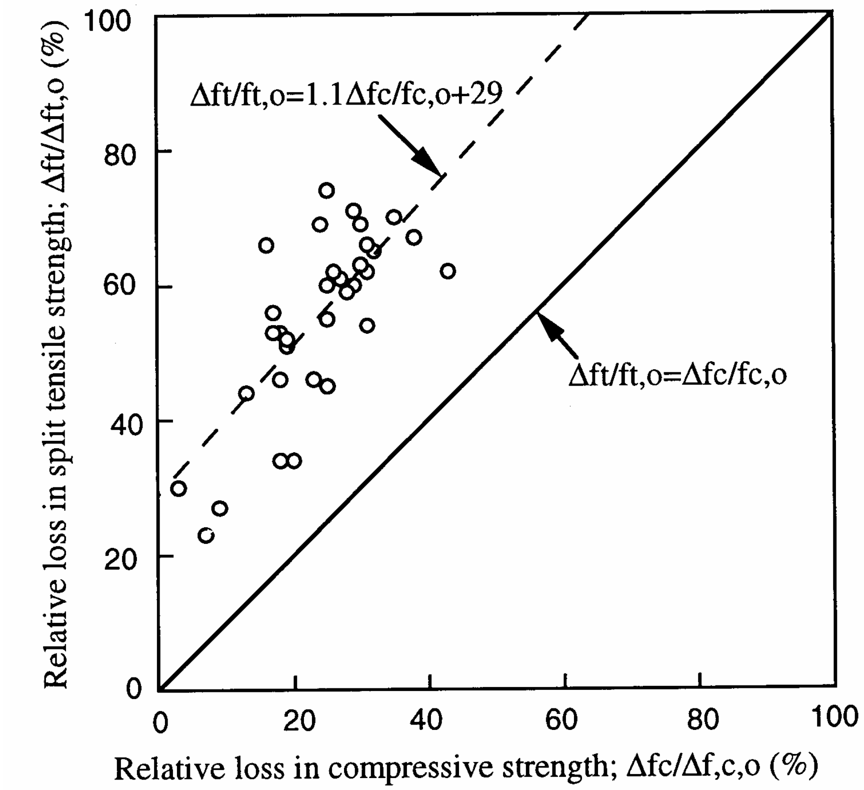

Figure 3: Relation between the relative loss in tensile strength and the relative loss in compressive strength; /1/.

5. Frost damage and E-modulus

5.1 Reduction in E-modulus

The relation between E-modulus of frost damaged concrete and E-modulus of undamaged concrete is shown in Figure 4.

It is not possible to find any well-defined relation. The loss in E-modulus can be very big. In some concrete, the loss in E-modulus is almost total. The lowest E-modulus measured is 5 GPa.

The E-modulus was calculated from the fundamental frequency of transverse vibration, and therefore is the dynamic E-modulus. A concrete which has lost most of its cohesion, cannot propagate transverse vibration. Therefore, the static E-modulus might have been higher than the dynamic, which is indicated by the much more limited reduction in compressive strength; Figure 1. The dynamic E-modulus, however, ought to represent the stiffness of the structure in a better way than the static E-modulus, because the effect of damage (internal cracks) in all directions of the structure are included in E_{dyn}.

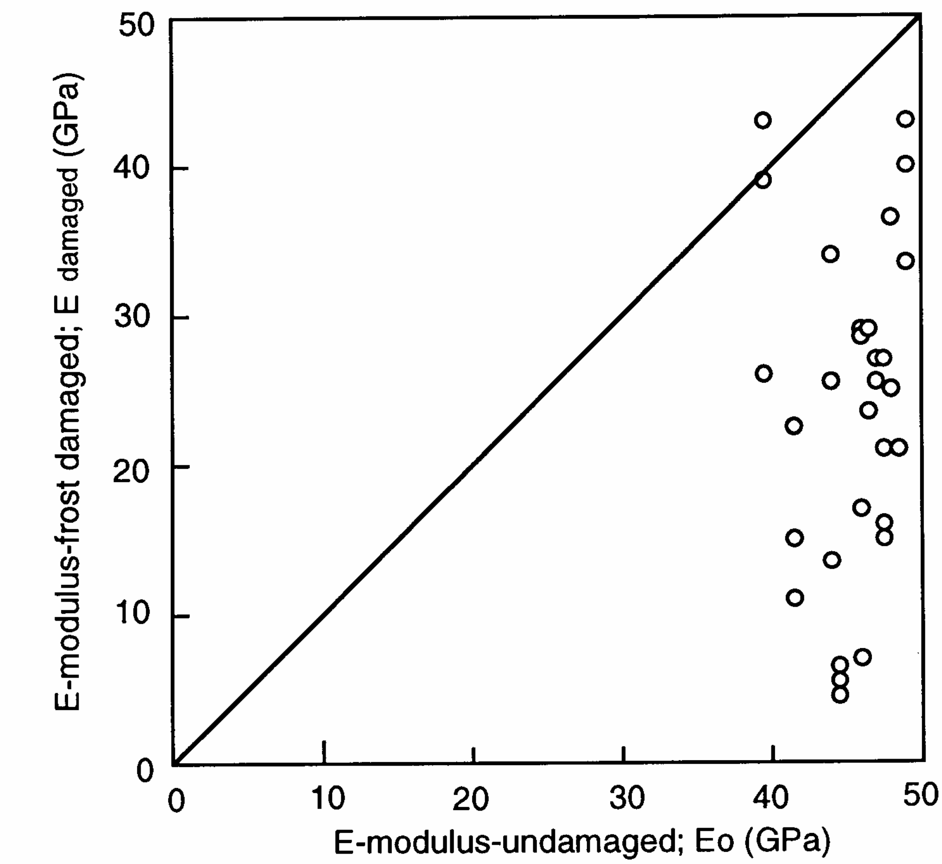

Figure 4: Relation between the dynamic E-modulus of undamaged concrete and frost damaged concrete; /1/.

5.2 Relation between loss in concrete strength and loss in E-modulus

In Figure 5 the relative loss in E-modulus is plotted versus the relative loss in compressive strength. In almost all cases the loss in E-modulus (stiffness) is considerably bigger than the loss in compressive strength.

In Figure 6 the relative loss in E is plotted versus the relative loss in split tensile strength. There is no well-defined relation, but the average relative loss in E is equal to the average relative loss in tensile strength. Therefore, changes in dynamic E-modulus is better correlated with changes in tensile strength than with changes in compressive strength.

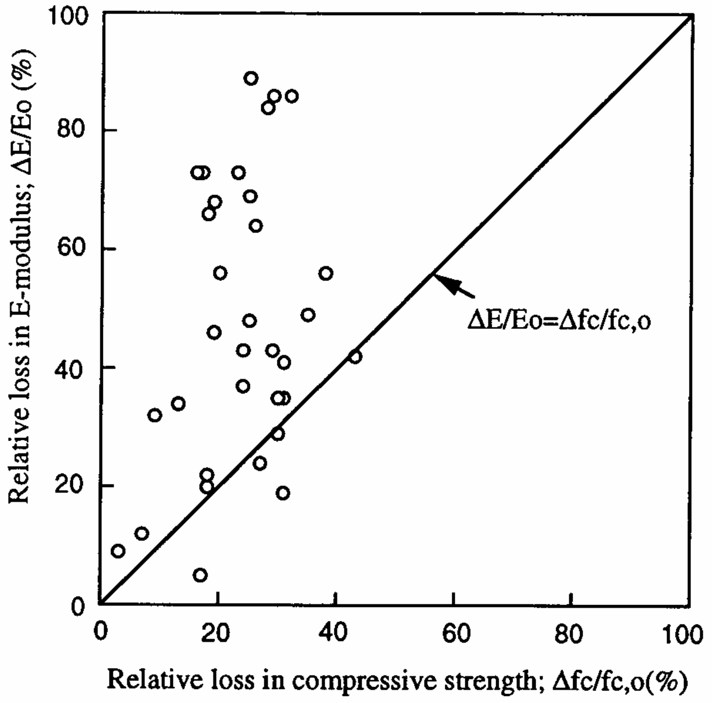

Figure 5: Relation between the relative loss in E-modulus and the relative loss in compressive strength.

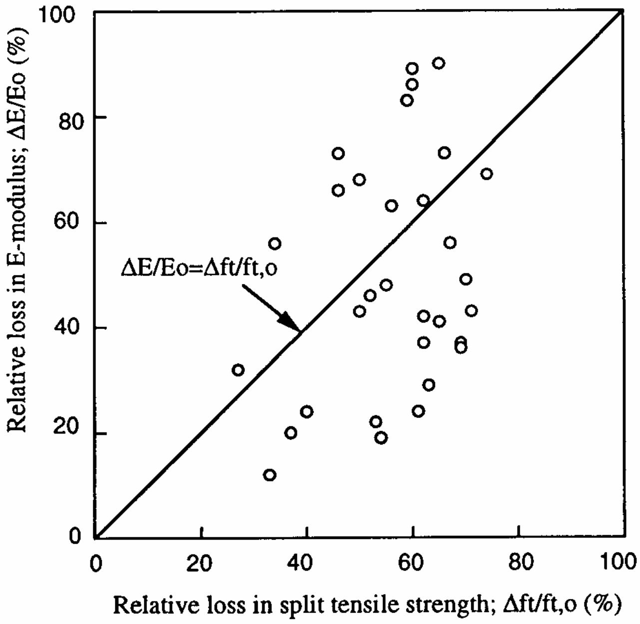

Figure 6: Relation between the relative loss in E-modulus and the relative loss in split tensile strength

6. Frost damage and bond strength - ribbed bars, no stirrups

6.1 Reduction in bond strength

The bond force is defined as the maximum force in the force-displacement diagram obtained when a reinforcement bar is pulled out of a concrete specimen with no consideration taken to cover or confinement by stirrups. Typical force-displacement diagrams are shown in Figure 7.

Relations between the bond force needed to pull a bar out from undamaged concrete and the bond force for frost damaged concrete is shown in Figure 8. The mean relation (excluding specimens with stirrups) is:

$$\mathbf{F_{b,damaged} \approx 0.5 \cdot F_{b,o}} \tag{8}$$

Where $F_{b,damaged}$ the bond force in frost damaged concrete [kN]
$F_{b,o}$ the bond force for the same bar in undamaged concrete [kN]

Since the bond length, bar type and bar diameter is the same in the compared data in Figure 7, an equation of type (8) can also be used for the effect of frost on bond strength.

$$\mathbf{f_{b,damaged} \approx 0.5 \cdot f_{b,o}} \qquad (9)$$

Where $f_{b,damaged}$ the bond strength in frost damaged concrete [MPa]
$f_{b,o}$ the bond strength in undamaged concrete [MPa]

The *mean relative* reduction on bond strength is:

$$\Delta \mathbf{f_b / f_{b,o} \approx 0.5} \qquad (10)$$

This means that in average 50% of the bond strength is lost.

The *lower bound* for absolute bond strength is:

$$\mathbf{f_{b,damaged} \approx 0.30 \cdot f_{b,o}} \qquad (11)$$

And the *lower bound* for relative loss in bond strength is:

$$\Delta \mathbf{f_b / f_{b,o} \approx 0.70} \qquad (12)$$

Thus, the maximum reduction in bond is about 70%. The lowest bond strength observed is 3.4 MPa.

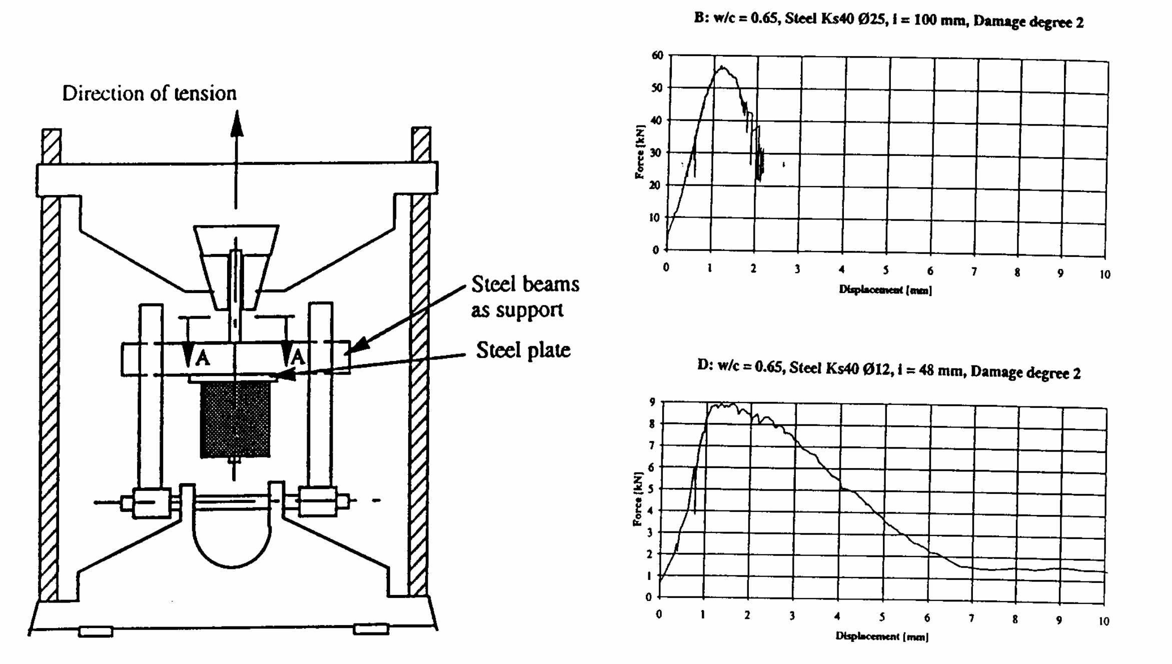

Figure 7: Pull-out test used for determination of the bond strength data shown in Figure 8. Examples of force-displacement diagrams; /1/.

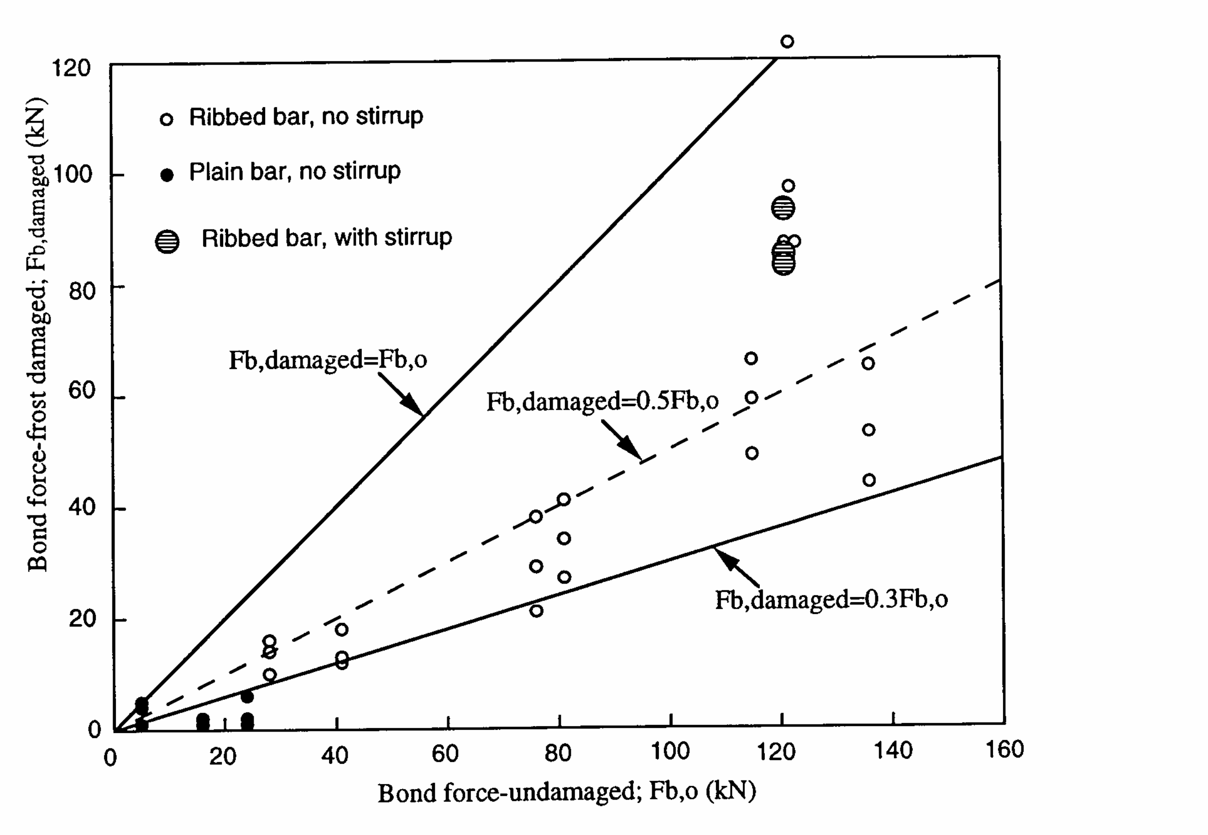

Figure 8: Relation between the bond force at undamaged concrete and the bond force at damaged concrete; /1/.

6.2 Relation between loss in bond strength and loss in split tensile strength

In Figure 9 the relative loss in bond strength is plotted versus the relative loss in split tensile strength. The average trend is that the two types of destruction follow each other; a big reduction in tensile strength causes a big reduction in bond strength. The *mean* relation is:

$$\Delta f_b/f_{b,o} = \Delta f_t/f_{t,o} - 5 \approx \Delta f_{t,o}/f_{t,o} \qquad (13)$$

Where $\Delta f_b/f_{b,o}$ the relative loss in bond strength [%]

$\Delta f_t/f_{t,o}$ the relative loss in split tensile strength [%]

Thus, in average the loss in bond strength is the same as the loss in tensile strength. This can therefore be used as a measure of the residual bond strength in a frost damaged structure.

The maximum observed loss in bond strength of ribbed bars is 70 %.

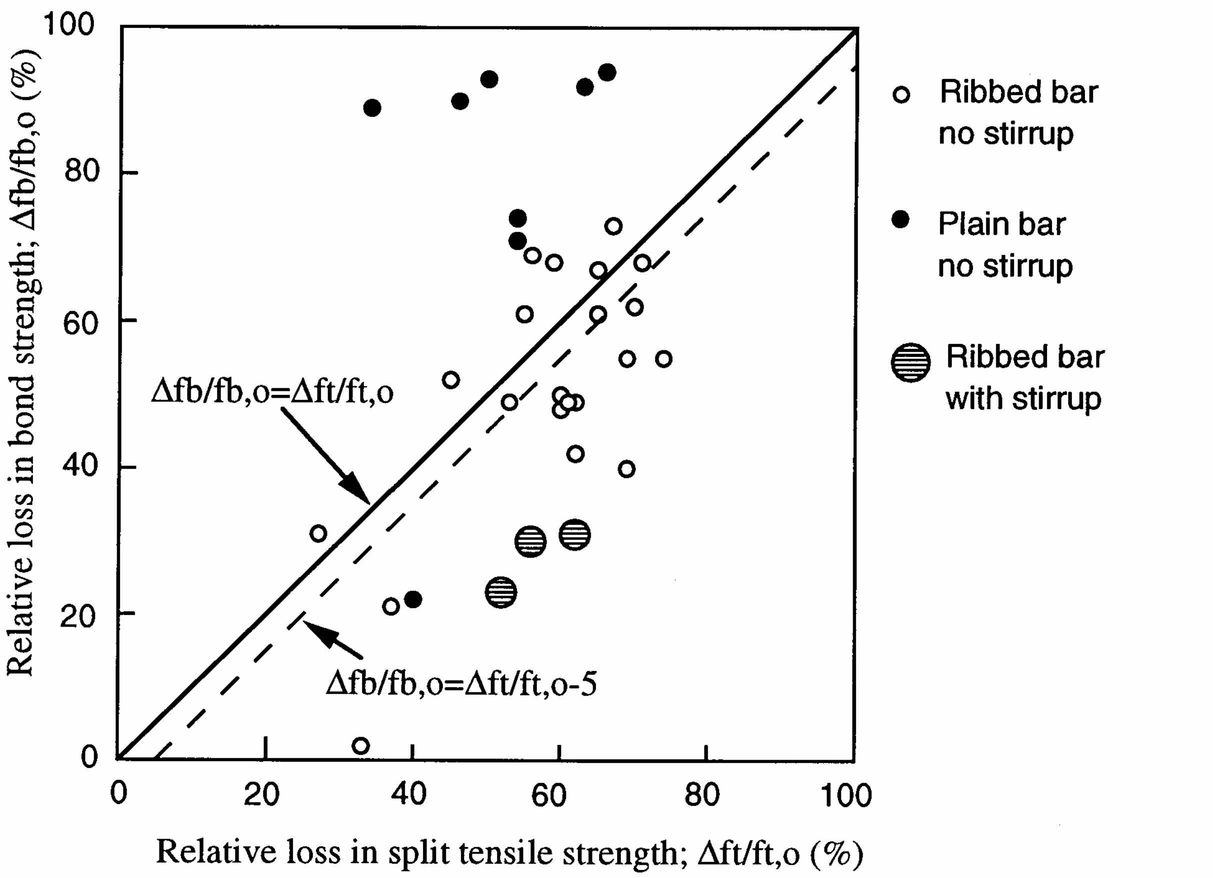

Figure 9: Relation between the relative loss in bond strength and the relative loss in split tensile strength; /1/.

6.3. Absolute bond strength versus the split tensile strength

In Figure 10 all measurements of the bond strength are plotted versus the split tensile strength. Values for frost damaged and undamaged specimens are included in the figure. Three values of the w/c-ratio are included (0.50, 0.65, 0.80)

The following mean regression curve is valid for ribbed bars:

$$\mathbf{f_b = 3.3 \cdot f_t + 1.3 \qquad r^2 = 0.92} \qquad (14)$$

The lower bound is

$$\mathbf{F_b \approx 3 \cdot f_t} \quad (15)$$

These equations can be used for estimation of bond strength of a frost damaged concrete. The only information needed for assessment of the bond strength is the split tensile strength of the concrete.

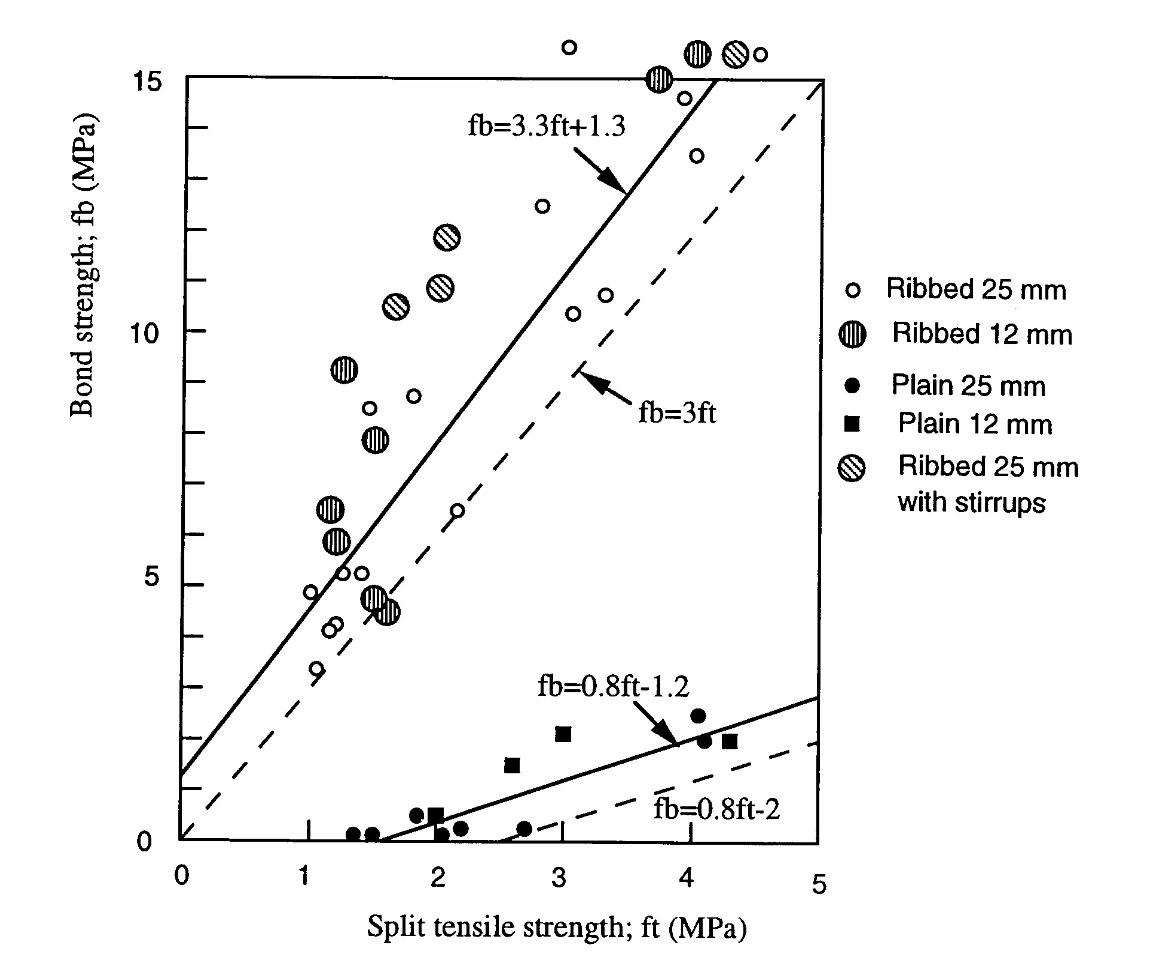

Figure 10: Relation between the split tensile strength and the bond strength; /1/.

7. Frost damage and bond strength - plain bars, no stirrups

The bond strength of plain bars is almost totally lost when the concrete is frost damaged. This is seen in Figure 9. Normal reductions in bond strength for plain bars is 90% or more. The lowest reduction observed is 70%. This means that the residual bond to plain bars after frost damage is almost zero.

The mean relation between bond strength and split tensile strength is:

$$\mathbf{f_b = 0.8 \cdot f_t - 1.2} \qquad (16)$$

The lower bound relation is:

$$f_b = 0.8 \cdot f_t - 2 \qquad (17)$$

8. Frost damage and bond strength - effect of stirrups

There are very few tests made of the effect of stirrups on the bond strength of frost damaged concrete. The few results in Figure 8 and 9 indicate that the maximum loss is restricted to about 30%. In this case, three stirrups with diameter 4 mm and spacing 30 mm were used; see Figure 11. The bar was placed eccentrically in the specimen.

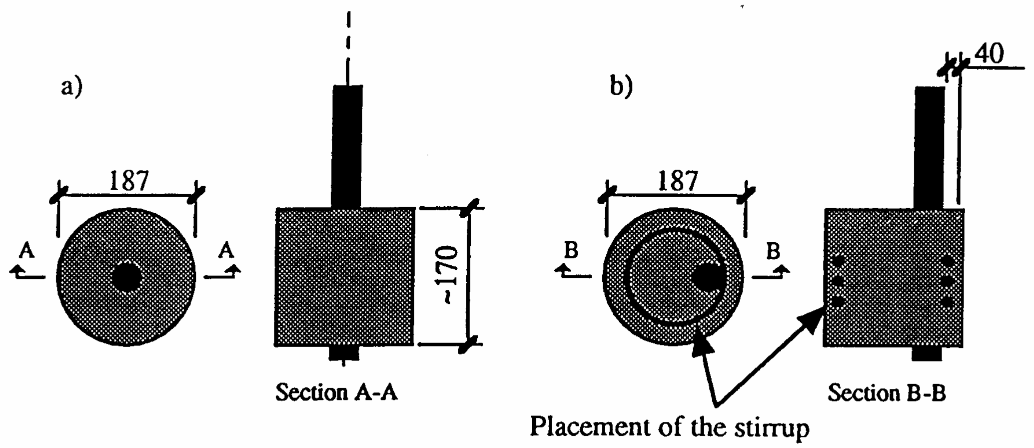

Figure 11: Arrangement of longitudinal bar and stirrups in the bond test; /1/.

The results are also plotted in Figure 12. The regression curve is:

$$f_b = 1.8 \cdot f_t + 7.6 \qquad r^2 = 0.97 \qquad (18)$$

This equation can be compared with eqn (14) which is valid for bond without stirrups. For high tensile strength, they give results that are similar.

Examples:

f_t=4 MPa

without stirrups: f_b=14.5 MPa

with stirrups: f_b=14.8 MPa

f_t=3 MPa

without stirrups: f_b=11.2 MPa

with stirrups: f_b=13.0 MPa

f_t=2MPa

without stirrups: f_b=7.9 MPa

with stirrups: f_b=11.2 MPa

Stirrups evidently increase the effective bond when the initial tensile strength is low or reduced due to frost damage.

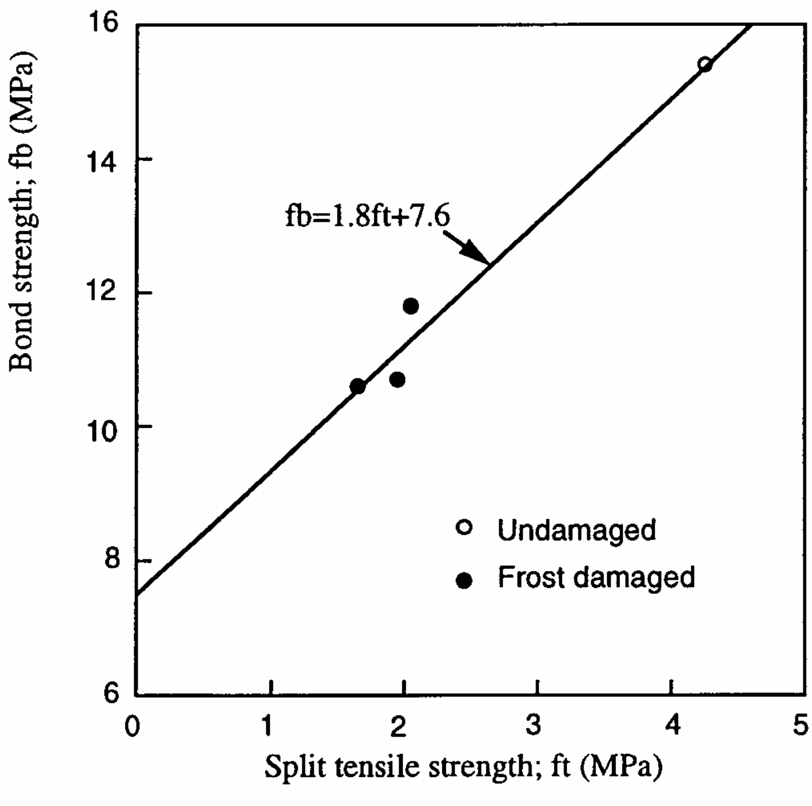

Figure 12: Relation between split tensile strength and bond strength in specimens with stirrups; /1/.

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/2/ Janz,M., Johannesson,B.: Influence of frost damage on bond between reinforcement and concrete. Div. Building Materials, Lund Institute of Technology, Report TVBM-7061, 1994.
(In Swedish)

ANNEX F
Synergy between different destruction types

Contents | Page

ANNEX F: Synergy between different destruction types

1. Introduction

Two or more destruction mechanisms acting simultaneously might strengthen each other. Such synergy cannot be neglected. Examples of synergetic effects are:

1. Surface erosion by salt frost scaling (or mechanical erosion) reduces the service life with regard to reinforcement corrosion, through a gradual reduction of the concrete cover; /1/.

2. ASR opens the structure to inflow of moisture, reducing the resistance to internal frost damage; /2/.

3. Leaching of lime by streaming water flowing through the structure increases the moisture uptake and, therefore, reduces the internal frost resistance; /3/.

4. Leaching of lime from the concrete cover increases the rate of carbonation and the diffusivity of chloride, and reduces the threshold chloride concentration for onset of corrosion. Thereby it decreases service life with regard to reinforcement corrosion; /3/

None of the synergies 1, 3, 4 have been studied experimentally. Therefore, only a simplified theoretical analysis can be made.

Synergy 2 has been studied in a very limited investigation. There seems to be a negative effect of ASR on internal frost resistance, but the results do not allow a quantitative calculation of the effect.

When the inspection indicates that such synergy exists it must be considered in the assessment of the future structural stability.

2. Effect of salt frost scaling on reinforcement corrosion

2.1 Effect on the initiation time

2.1.1 Approximate method

Salt scaling causes a gradual erosion of the surface. The erosion rate is often linear and can be described by:

$$x_e = C_e \cdot t \qquad (1)$$

or $$\mathbf{dx_e/dt = C_e} \qquad (1a)$$

Where x_e the erosion depth at time t (the age of the structure) [m]

C_e the erosion coefficient [m/year]

The penetration of the threshold chloride concentration, or the penetration of the carbonation front in a cover with intact thickness, is described by:

$$x_c = C_c \cdot t^{1/2} \qquad (2)$$

or

$$\mathbf{dx_c/dt = C_c/(2 \cdot t^{1/2})} \qquad (2a)$$

Where x_c the carbonation depth, or the depth of the critical chloride concentration, counted from the non-eroded surface, at time t (normally the age of the structure) [m]

C_c the penetration coefficient [m/year$^{1/2}$]

After a certain total exposure time (t*) and penetration depth (x*), the rate of erosion (dx_e/dt) is equal to the rate of penetration (dx_c/dt). t* and x_c* can be calculated by:

$$\mathbf{t^* = [C_c/(2 \cdot C_e)]^2} \qquad (3)$$

$$\mathbf{x_c^* = C_c \cdot t^{*1/2}} \qquad (4)$$

After this time, the penetration rate is constant and described by eqn (1). The total penetration depth after time t > t* is

$$t = t^* + (x_c - x_c^*)/C_e \qquad (5)$$

The total exposure time (age) until start of corrosion is:

$$t_{initiation} = t^* + (T - x_c^*)/C_e \qquad (6)$$

Where T the initial concrete cover before any erosion [m]

The residual service life (Δt) until start of corrosion is :

$$D\mathbf{t = (T - x_c^*)/C_e} \qquad (7)$$

This equation can be used when the two coefficients x* and C_e are known.
C_e is estimated from the erosion depth, using *non-eroded* parts of the surface as reference.

x_c* can be calculated from measurements of the penetration depth of carbonation, or the penetration depth of the threshold concentration for *non-eroded* parts of the structure, assuming that the time t* is the same as the age of the structure at inspection, t_o. This is not correct, but will give a value of the residual initiation time on the safe side (One can also use the penetration depth in eroded parts and add the erosion depth ($x^*=x_e+x'_c$ where x'_c is the penetration depth from the *eroded* surface.)

The extrapolation of penetration is illustrated in Figure 1.

Example:
The measured erosion depth in a 20 year old structure is 14 mm. The initial concrete cover is 35 mm. The penetration depth of the carbonation front is 15 mm from the eroded surface.

The erosion coefficient is: $C_e=0.014/20=7\cdot10^{-4}$ m/year

The residual time until start of corrosion is:

$\Delta t =$ [0.035-(0.014+0.015)]/$7\cdot10^{-4}$ = **8.5 years**

(If erosion is neglected the cover is believed to be 35-14=21 mm and the penetration depth 15 mm. This gives a believed coefficient of penetration $C_c=3.35\cdot10^{-3}$ m/year$^{1/2}$; see Eqn (2). Thus, corrosion is believed to start when the structure is 39 years old, which gives a residual service life of **19 years**. The service life will be overestimated by 10 years)

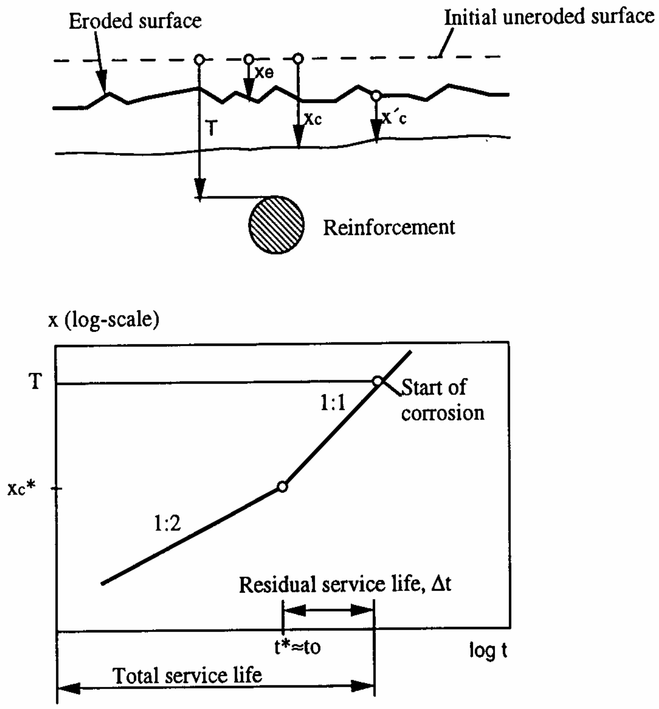

Figure 1: Extrapolation of the penetration depth with simultaneous erosion by salt scaling.

2.1.2 Exact method

A more exact method of assessing the effect of surface erosion is to use the following equation that considers both erosion (like salt frost scaling) and penetration of the carbonation front or threshold chloride front; /1/:

$$(x-C_e \cdot t)\cdot(dx_c/dt)= C_c^2/2 \qquad (8)$$

This equation can be solved numerically. It gives somewhat shorter residual service life than the simpler equation (7). The difference between the two equations is however not so big. The uncertainties in determination of the governing parameters C_e and C_c are so big that the use of the complicated Eqn (8) is hardly justified.

2.2 Effect on the corrosion rate

Theoretically, the corrosion rate ought to be somewhat influenced by the cover thickness. Therefore, surface erosion ought to have a certain accelerating effect on the corrosion rate, provided the moisture content around the bar is not changed by erosion, which it probably is.

The net effect of erosion on the corrosion rate can probably be neglected in most cases.

3. Effect of ASR on internal frost resistance

3.1 Introduction

The following *negative factors* can be 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 become water-filled and cause frost damage. Non-air-entrained concrete ought to be more vulnerable.

3: Gel produced from ASR contains large amounts of water, part of which might be freezable.

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

The following *positive factor* can be identified:

5: The gel caused by ASR makes the cement paste matrix more dense, leading to a reduced ingress of water into the cement paste structure. It might be, that this positive factor in certain cases compensates 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; /2/. It is uncertain, however, if the same negative effect takes place in the real structure.

3.2 The future frost damage

At present there is no possibility to express the synergy between ASR and frost in a quantitative way. A simplified way of doing this is to use the concept potential service life described in ANNEX A, paragraph 6.2.

It is obvious that the first three negative factors described in paragraph 3.1 causes a gradual increase in the inner moisture content (W_{ACT}). The fourth negative factor causes a reduction in the critical moisture content (W_{CR}).

This means that the *potential frost resistance* (F) expressed in terms of the difference between W_{CR} and W_{ACT} decreases with time.

$$F=W_{CR}(t) - W_{ACT}(t) \qquad (9)$$

The parameters W_{CR} and W_{ACT} at the time of inspection can be determined on drilled-out cores from the ASR-affected structure. W_{CR} is determined by a freeze-thaw test described in ANNEX , paragraph 5.4. W_{ACT} at the time of inspection is determined by drying and weighing; ANNEX G, paragraph 4.3. There are two possibilities:

1: If $W_{ACT}>W_{CR}$, the structure is already frost damaged, or very close to become frost damaged. ASR will only accelerate the future deterioration process by increasing the moisture uptake. The future deterioration due to frost can be extrapolated by a linear time function:

$$D(t) = D(o)(t/to) \qquad (10)$$

Where	D(t)	damage (e.g. tensile strength) at time t
	D(o)	present damage
	t	the age of the structure
	t_o	the present age of the structure

In order to find a value of D(o) one has to find parts of the structure that are unharmed by frost and ASR; ANNEX G, paragraph 4.1.

2: If $W_{ACT}<W_{CR}$ the structure might be unharmed by frost. It could be, however, that the structure is drier at the inspection than it might be during more moist conditions. Therefore, a capillary absorption test can be performed, and the breaking point in a diagram where moisture

content is plotted versus the square-root of time be found (see ANNEX G, Figure 3):

$$W_{CAP} = A + B\cdot t^{C} \tag{11}$$

Where W_{CAP} the water content in the capillary suction test
A the breaking point
B and C coefficients determining the long-term water absorption

If $A<W_{CR}$ synergy can probably be neglected. There will probably be no future frost damage.
If A is only somewhat below W_{CR}, frost attack might occur in the future. An estimation of how long time it may take until there is severe frost damage can be obtained by using the concept potential service life (ANNEX A paragraph 6.2): The potential residual service life is:

$$t_{p,residual} = [(W_{CR}-A)/B]^{1/C} \tag{12}$$

This equation tells how long time it will take for the structure before it is damaged by frost, provided it is exposed to water without any possibility to dry. The value of $t_{p,residual}$ can be compared with the water absorption time of the actual structure. If this is shorter than the potential service life, no damage will probably occur in the future.

4. Effect of leaching on the internal frost resistance

Water flowing trough the concrete structure will dissolve lime. The increase in porosity caused by leaching can be very high. An increase by 25% of the initial porosity is possible. This will severely affect the internal frost resistance. Big leaching only occurs in concrete of low quality, which in itself has low, or average, initial potential frost resistance. Leaching makes it still more vulnerable to frost attack.

Unfortunately there are no tests made on the effect of leaching on frost resistance. Therefore only a qualitative analysis is made below.

The internal frost resistance of concrete is determined by two factors:

1: The *critical water content* which is mainly a function of material properties.
2: The *actual water content* which is a function both of the material structure (mainly the air-pore volume and the air-pore structure) and environmental properties, mainly the ”wetness” of the environment.

The risk of frost damage is calculated by

$$R=\int_{0}^{W_{saturated}} F(W_{CR})\cdot f(W_{ACT})\cdot dW \tag{13}$$

where R the risk of frost damage ($0 \leq R \leq 1$)
$F(W_{CR})$ the distribution function of the critical water content
$f(W_{ACT})$ the frequency function of the actually occurring water content
$W_{saturated}$ the water content at total saturation of the concrete

In eqn. (13) it is assumed that neither W_{CR} nor W_{ACT} has a distinct, deterministic value, but that there is a certain variation depending on variations in material and environment.

Leaching will affect both W_{CR} and W_{ACT}. The effect can be assumed to be such that the risk of frost damage increases. W_{CR} will be reduced, since leaching will cause a coarsening of the pore system, thereby increasing the amount of freezable water. W_{ACT} will be increased since the concrete will absorb water more readily. Besides, the coarsening of the pore system will increase the diffusivity of dissolved air that emanates from enclosed air-bubbles in air-pores. Therefore, the air-pore system will become more rapidly inactivated due to a gradual water absorption.

The effect of leaching on the frost resistance can be quantified by determining the values of W_{CR} and W_{ACT} on cores taken from different parts of the structure, non-leached and leached. The technique for such testing is known; ANNEX G. Thereafter, the increased risk of frost damage can be calculated by eqn. (13). A method for extrapolation of the future frost deterioration is described in ANNEX C. The extrapolation depends on the "wetness" of the environment. As one extreme, in very moist conditions, destruction is supposed to be linearly growing with time. As the other extreme, in more dry conditions, destruction is supposed to be unchanged, and maintain the same level as at it has at the time of inspection.

The effect of leaching on the risk of internal frost attack is illustrated in Figure 2.

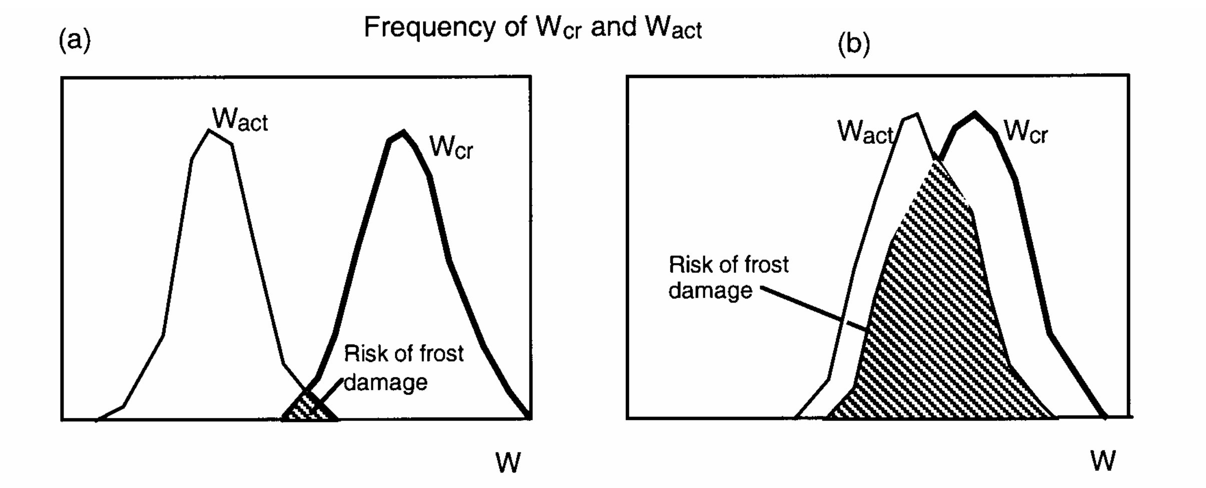

Figure 2: Effect of leaching on the risk of frost damage - principles; /3/.
(a) Before leaching. (b) After leaching.

5. Effect of leaching on salt frost scaling

A concrete that has a high scaling resistance when non-leached might possibly obtain a reduced scaling resistance after leaching, since the amount of freezable water increases due to the increased porosity. The relation between leaching and scaling resistance is unknown, however.

The scaling rate can be estimated by a laboratory freeze-thaw experiment in salt solution; ANNEX G, paragraph 5.5. On basis of results from this test the future scaling rate in the real environment can be extrapolated; ANNEX D.

6. Effect of leaching on reinforcement corrosion

6.1 Introduction

A concrete surface exposed to streaming water will be gradually dissolved. This will increase the diffusion coefficient to carbon dioxide and chloride, and it will reduce the threshold chloride concentration for start of corrosion. Therefore, it might have an effect on the service life with regard to the *initiation time* of corrosion. The effect has not been studied experimentally. Therefore, only a qualitative analysis is made below.

Reinforcement corrosion starts when the passive conditions around the steel disappears due to carbonation, or due to the influx of chloride ions.

Carbonation is extremely slow in water saturated concrete. Therefore, corrosion due to carbonation should be a minor problem in most structures exposed to leaching. There are unsaturated parts in such structures, however, where corrosion might be a problem. In such parts leaching should normally be of less importance, however. If leaching has occurred, for example at the downstream face of a lamellae dam, or in a cooling tower, it will increase the carbonation rate, thereby reducing the service life.

Chloride induced corrosion is not relevant for structures placed in normal water. In structures exposed to sea water, that is penetrating the structure, chloride ions will enter the concrete both by convection and diffusion. Leaching will have two negative effects with regard to chloride induced corrosion:

1: It increases the permeability to chloride ions
2: It reduces the threshold chloride concentration for onset of corrosion

It is shown below that the synergetic effects can be very big and negative. Therefore, in structures where leaching occurs one must be very cautious in extrapolating observed "historic" penetration rates and be cautious in using threshold chloride concentrations that are valid for non-leached concrete.

6.2 Effect of leaching on corrosion initiated by carbonation

6.2.1 The residual initiation time

The carbonation rate in a given environment is determined by two factors that are highly influenced by leaching:

1: The amount of lime that is able to carbonate
2: The permeability of the concrete cover to penetrating carbon dioxide

The carbonation process can be described by the following equation

$$x=[2 \cdot \delta_c \cdot c_c/C]^{1/2} \cdot t^{1/2} \tag{14}$$

where
- x the depth of the carbonation front [m]
- δ_c the diffusivity of carbon dioxide in the concrete cover [m^2/s]
- c_c the concentration of CO_2 in air surrounding the structure [mol/m^3]
- C the amount of material able to carbonate [mol/m^3]
- t the exposure time (the age of the structure) [s]

The carbonation rate is found by derivation of eqn. (14)

$$dx/dt= [\delta_c \cdot c_c/(C \cdot 2)]^{1/2} \cdot t^{-1/2} \tag{15}$$

Therefore, the relation between the amount of leached lime and carbonation rate is

$$\mathbf{(dx/dt)_l/(dx/dt)_o=[1/(1-\gamma)]^{1/2}} \tag{16}$$

where
- $(dx/dt)_1$ the carbonation rate of the leached concrete [m/s]
- $(dx/dt)_o$ the carbonation rate of the unleached concrete [m/s]
- γ the fraction of total lime that is leached [-]

This means that the carbonation rate is increased by a factor 1.3 when 40% of the lime is leached, and by a factor 1.85 when 70% of the lime is leached.

Eqn. (16) implies that the entire cover is leached to the same extent, and that the diffusivity of CO_2 is unaffected by leaching.

The effect of leaching on the diffusivity of CO_2 is not known. One simplification is to assume that the diffusivity is directly proportional to the cement paste porosity. This increases in direct proportion to the leaching. Thus the diffusivity is:

$$\delta_{c,l}=\delta_{c,o}(1+\Delta P/P_o) \tag{17}$$

where $\delta_{c,l}$ the diffusivity after leaching [m^2/s]

$\delta_{c,o}$ the diffusivity before leaching [m^2/s]

ΔP the increase in porosity due to leaching [m^3/m^3)

P_o the porosity before leaching [m^3/m^3]

The volume of leached material is directly proportional to its weight

$$\Delta P=v_s \cdot Q_v \tag{18}$$

where v_s the specific volume of leached lime [m^3/kg]

Q_v the weight of dissolved lime [kg/m^3]

According to eqn. (17) the relation between the carbonation rates of the leached and non-leached concrete is

$$\mathbf{(dx/dt)_l/(dx/dt)_o=\{(1+\Delta P/P_o)/(1-\gamma)\}^{1/2}} \tag{19}$$

Leaching of all $Ca(OH)_2$ from the concrete with a w/c-ratio of 0.60 corresponds to an increase in the porosity of about 25%, i.e. $\Delta P/P_o \approx 0.25$. The fraction of leaching γ is about 40%. Thus, the carbonation rate is increased by a factor of about 1.45 instead of 1.3, a value which is valid if the diffusivity was kept constant.

The increased rate of carbonation means that the time until start of corrosion is reduced in the same proportion. *Thus, one can conclude that leaching causes a considerable reduction in the service life with regard to corrosion induced by carbonation.*

The effect of leaching on carbonation and time to onset of corrosion is illustrated in Figure 3.

In order to be able to analyse the effect of leaching on the future deterioration, one has to measure the rate of leaching, i.e. the rate of increase in porosity. This can be done by analysing the water flux and the lime concentration of the leaching water.

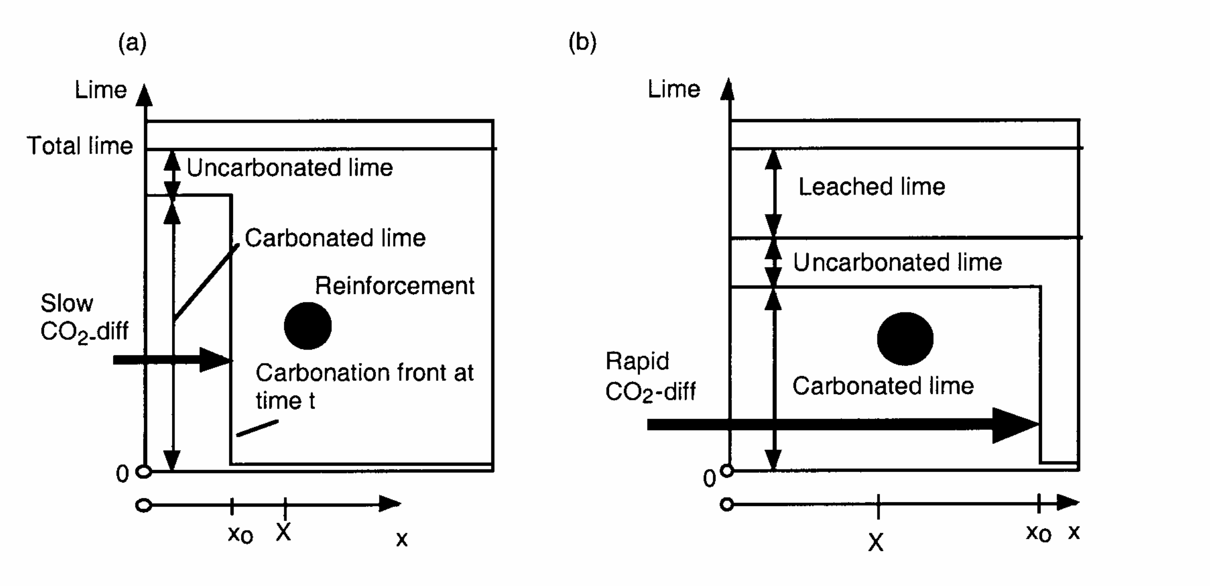

Figure 3: Effect of leaching on corrosion induced by carbonation - principles; /3/.
(a) Before leaching. (b) After leaching.

6.2.2 The corrosion stage

Probably leaching has marginal effects on the corrosion rate once this has been initiated.

6.3 Effect of leaching on corrosion initiated by chloride

6.3.1 The residual initiation time

The service life with regard to chloride induced corrosion can be described by the following equation, provided the outer chloride concentration is constant and theinflow of chloride ions is determined by diffusion with a diffusivity that is unchanged with time.

$$c_{thr}/c_o = \mathrm{erfc}\{T/[4 \cdot \delta_{eff} \cdot t]^{1/2}\} \qquad (20)$$

where
- c_{thr} the threshold concentration of free chloride ions [mol/l]
- c_o the chloride ion concentration at the surface [mol/l]
- T the concrete cover [m]
- δ_{eff} the effective chloride diffusivity considering chemical and physical binding of chlorides [m^2/s]
- t the exposure time (the age of the structure) [s]
- erfc the complementary error function

The effective diffusivity is (provided chloride binding is linear):

$$\delta_{eff} = \delta/(c_b/c+1) = \delta/(\rho+1) \qquad (21)$$

where δ the diffusivity not considering chloride binding (i.e. the "steady state diffusivity") [m^2/s]
c_b the bound (immobile) chloride ion concentration [mol/l]
c the concentration of chloride that is free to move [mol/l]
ρ the ratio between bound and free chloride [-]

Chloride diffusivity is defined by Fick´s law

$$q_c = \delta \cdot d[c]/dt \qquad (22)$$

where q_c the flux of free chloride [$mol/(m^2 \cdot s)$]
δ the diffusivity of chloride [m^2/s]
$d[c]/dx$ the gradient in free chloride concentration [$mol/(m^3 \cdot m)$]

The threshold concentration is largely unknown. It certainly depends on the alkalinity of the pore water, but it also depends on the permeability of the cement paste, and on the oxygen concentration at the steel surface.

Leaching will have many negative effects with regard to chloride induced corrosion. They are illustrated in Figure 4.

1: Leaching will increase the steady-state diffusivity δ by increasing the porosity.
2: Leaching might decrease the amount of bound chloride by lowering the alkalinity of concrete . This will increase the effective chloride diffusivity, δ_{eff}
3: Leaching will decrease the alkalinity of the concrete (alkali hydroxides are leached at first, and thereafter all calcium hydroxide). This will decrease the threshold chloride concentration c_{thr} , which is a very negative effect.

The total effect of all these effects of leaching can be very negative and must be considered. This is not easy because the effects of leaching on the diffusivity, the threshold concentration, and the binding of chloride are unknown.

It is clear that "historic values" for penetration rate of chloride cannot be used for prediction of the future penetration and prediction of the residual initiation time. One has to be conservative in the prediction.

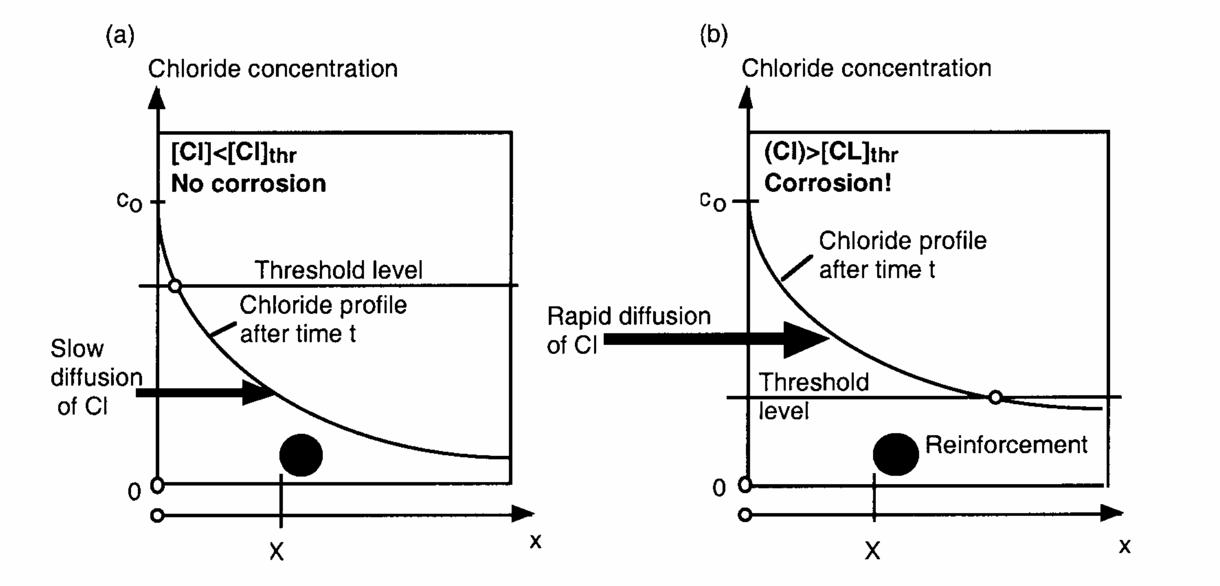

Figure 4: Effect of leaching on corrosion induced by chloride-principles; /3/
(a) Before leaching. (b) After leaching

6.3.2 The corrosion stage

Probably leaching will have marginal effects on the rate of corrosion.

References

/1/ Fagerlund, G., Somerville, G., Tuutti, K.: 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 (An extended version is, *Report TVBM-7071*, from Div. Building Materials, Lund Institute of Technology, 1994.)

/2/ Trägårdh, J., Lagerblad, B.: Influence of ASR expansion on the frost resistance of concrete. Internal report, Swedish Cement and Concrete Research Institute, 1994.

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ANNEX G
Diagnosis and inspection

Contents Page

ANNEX G: Diagnosis and inspection

1. Introduction

A quantitative structural assessment has two objectives:
1: To estimate the *actual* status of the structure at the time of inspection.
2: To estimate the *future* status of the structure if no repair is made.

An assessment, therefore, requires actual data of parameters of importance for structural stability.

Internal frost attack
For a structure damaged by internal frost attack the following data are required for vital parts of the structure:

1: For the actual structural stability
 * The actual compressive strength
 * The actual tensile strength
 * The actual bond strength
 * The actual E-modulus

2: For the future structural stability
 * The present damage in terms of strength and E-modulus
 * Predicted evolution of compressive strength
 * Predicted evolution of tensile strength
 * Predicted evolution of bond strength
 * Predicted evolution of E-modulus

The prediction (the future damage curve) is made according to the principles in ANNEX C. The future damage curves depend on the wetness of the environment around the actual part of the structure. There are three ”moisture classes”:

 * Moist
 * Very moist
 * Extremely moist

Salt frost scaling
For a structure damaged by salt frost scaling the following data are required:

1: For the actual structural stability
 * The actual scaling depth in vital parts

2: For the future structural stability
 * The future scaling depth in vital parts

The prediction is based on the principles described in ANNEX D.

2. Diagnosis

2.1 Internal frost damage

It is sometimes difficult to distinguish internal *frost damage* from *alkali silica reaction*. In both cases the concrete surface has a more or less dense crack pattern indicating internal disintegration due to expansion. Other destruction types causing similar cracking due to internal expansion are *sulphate attack* and *delayed (secondary) ettringite formation.* Often a petrography analysis is needed to establish the cause of damage. *It is very important that the diagnosis is correct, since the evolution of damage is quite different in a frost damaged structure than in a structure damaged by ASR, sulphate, or delayed ettringite.*

Internal frost damage only occurs when the structure has access to large amounts of free water. Thus, it never occurs where the environment is only ”humid” but not ”wet”. Typical structures/environments causing frost attack are:

* structures sucking ground water
* structures in contact with moist back-fill, like support walls
* structures constantly exposed to water, like hydraulic structures, dams etc.
* railway bridge troughs filled with moist ballast
* structures more or less constantly exposed to drainage water

Thus, in order to estimate if frost can be the cause of damage, it is important to clarify the wetness of the environment surrounding the structure.

Photographs of cores taken from frost damaged structures are shown in Figure 1. In Figure 2, the crack pattern in a core from a frost damaged concrete is shown. There is a random system of cracks in the heart of the concrete together with cracks parallel to the surface of the concrete. Therefore, in the case of frost attack, the internal cracking close to the surface is directed *perpendicular to the heat flow* during freezing, while they further into the concrete are directed in more or less all directions. There are often cracks running through the interface between cement mortar and coarse aggregate, but seldom cracks within the coarse aggregate grains (unless these are porous).

In many cases there are also long cracks parallel to joints and edges of the concrete, or emanating from corners. These cracks can be observed on the exposed surface. They are sometimes called *D-line cracking*.

2.2 Salt frost scaling

A concrete surface that is exposed to de-icing salts, or any other water solution, at the same time as it is exposed to frost, might obtain a more or less deep surface scaling. The surface is often rough, since the mortar phase is more eroded than the coarse aggregate phase. The scaling is not only restricted to such parts of the surface that are directly exposed to salt, but also to other parts of the structure. The reason is that there is often a ”salt-mist” around the entire structure when this is exposed to de-icing salt, or when it is located in a marine environment.

If scaling is observed, and one knows that there is a saline environment around the structure, one can be quite sure that combined salt-frost attack is the reason. Other causes of scaling might be mechanical erosion, but is has quite another appearance, and only occurs where the concrete is exposed to big mechanical erosive load.

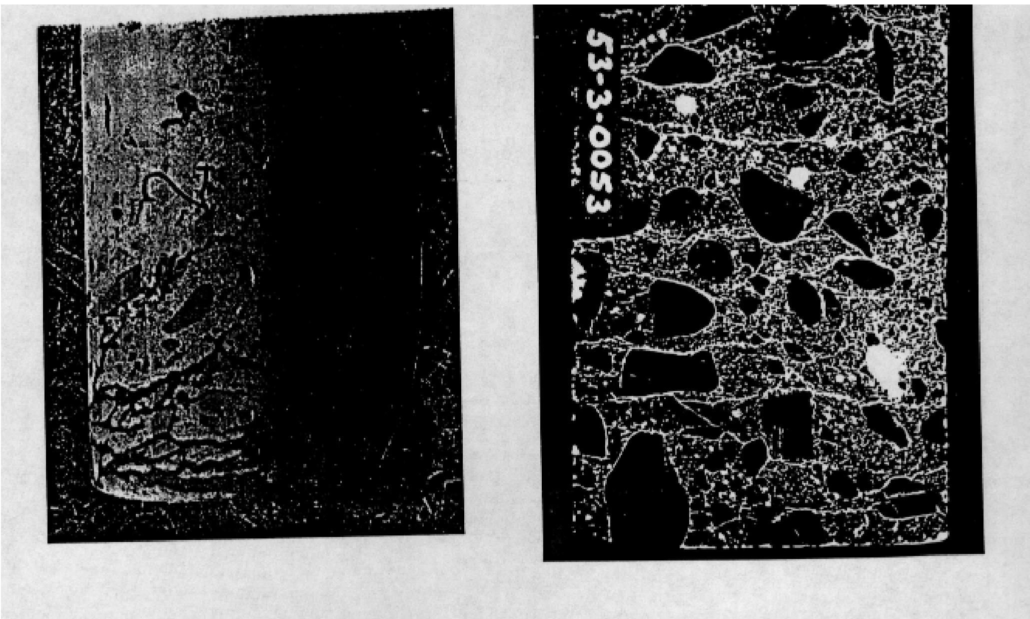

Figure 1: (a) Core from a UK structure showing internal frost damage. (b) Ditto from a Swedish structure.

Figure 2: Typical internal crack pattern induced by frost action.

3. Inspection and testing the structure with regard to assessment of the actual status

In the following, the exact number of specimens for each test, or the location where to measure or drill out specimens, is not considered. It will be individual for each structure, and *must be determined by the experienced investigator in collaboration with the structural engineer who is going to make the structural assessment.*

The amount of damage can vary from place to place in the same structure. Therefore, in most cases, series of data from many different places are required. Only in special cases can data taken from one place be used for the entire structure.

3.1 The visual appearance - scaling and cracking:

Aim:
To find proof of the existence of frost damage, and the extent of this. Such proof is:

1: *Surface scaling*
The current scaling depth is an important input for an assessment of the future scaling; ANNEX D. It is also required for an assessment of the penetration of chloride, or carbonation. This synergy between salt scaling and reinforcement corrosion is described in ANNEX F.

2: *Internal cracking and loss of cohesion*
This is used as an indication that it is frost attack that has caused the damage, and not another type of attack. This is important for the prediction of future damage.

Method:

Surface scaling:
Surface scaling is measured in-situ, by some sort of gauge, such as a measuring clock mounted on a bar. The scaling depth is counted from a reference initial surface, that can sometimes be difficult to define. The best is to use non-scaled parts of the structure. If such parts do not exist, one might base the measurements on the initial drawings used at the construction phase assuming these reflects the structure built. The scaling front will not be perfectly smooth. The actual profile is determined.

Internal damage:
Traditional techniques are used, such as visual analyses of thin-sections, petrographic analyses etc. By an analysis of the crack pattern, and other morphological characteristics of the concrete, an experienced petrographer will be able to see whether damage is due to frost attack, or to any other types of internal attack, such as alkali-silica reaction, secondary cement reactions, sulphate attack, etc. The petrographic techniques are not described here.
Salt deposits, and precipitation of crystals (e.g. secondary ettringite or calcite) in the air-pore system is quantified. It gives information of the amount of the inactivated air-pore volume; i.e. the air-pore volume that cannot protect the concrete from frost damage.

Note:
An air-pore, that is filled with secondary cement reaction products, or with calcite, is not an active air-pore. Evidently, it can be filled with water, which was a pre-requisite for the nucleation of the crystals that have been formed. Therefore, air-pores that are filled by crystals have not been functioning as a recipients for excess water that is displaced during freezing even before crystallisation occurred.

3. 2 Strength of the concrete

Aim:
To make possible an assessment of the current structural stability.

Method:
Compressive strength:

Only direct testing of cores can be used. Indirect measurements of speed of sound, and other indirect measurements, cannot be used due to very uncertain relations between strength and E-modulus of frost damaged specimens (ANNEX E).

Cores for testing of compressive strength are taken from cross sections that are vital for the load-carrying capacity.

Normal, standardised methods for testing and assessment of drilled-out cores, are used. The results are used for a determination of the *characteristic strength* to be used in the structural analysis (the strength of 15 cm cylinders tested wet). Therefore, for each core, the measured value is corrected for slenderness, size, and moisture condition.

*No correction for slenderness is made for cores with length-diameter above 2. The following correction factor is used for length-diameter below 2:
- length/diameter ratio 1.5: correction factor 0.95
- Length/diameter ratio 1: correction factor 0.85
- Linear interpolation is used for other values of the slenderness.

* No correction for size is made if the diameter of the core is above 100 mm. For diameter 5 cm a correction factor of 0.9 is used. For diameters between 5 and 10 cm linear interpolation is used.

* The specimens shall be tested wet. Therefore, there is no correction for moisture.

The mean value and standard variation of the (corrected) strength values are calculated and used for transformation of the measured values to characteristic values according to ANNEX H. Therefore, at least 3, and preferably 6 or more, specimens are required for each investigated section of the structure.

Split tensile strength
Tensile strength is important for assessing bond strength, and must therefore be known, especially for sections where shear and anchorage are important. The tensile strength cannot be calculated from the compressive strength, because it is often more affected by frost than compressive strength. It has to be measured on cores using standard procedure.
Cores (preferably with diameter ≥10 cm) are taken from crucial cross sections.
The cores are tested according to the standardised test method. Unevenness in the specimen surface caused by drilling can severely influence the result, and must be removed by grinding. The specimens are tested wet.

Mean value and standard deviation are determined.

Bond strength:
The residual bond strength is evaluated from the residual tensile strength, according to relations presented in ANNEX E and ANNEX H.

3.3 E-modulus

Aim:
To make possible an assessment of the structural stability and serviceability. The E-modulus is important for the distribution of moments and forces in a deteriorated structure. It affects the effectiveness of pre-stressing bars, and it determines deformations and cracking.

Method:
There are two types of E-modulus; static and dynamic.

Static E-modulus:
The static E-modulus is determined on cores loaded slowly by compression. The static E-modulus is defined and determined according to the design code used. Normally a cylindrical specimen with slenderness of at least 2 is used. The deformation parallel to the load is measured in the middle part of the specimen. The E-modulus is normally defined as the secant modulus from zero load to 1/3 of the fracture load

Dynamic E-modulus from the speed of sound:
The dynamic E-modulus can be determined from the speed of sound. The measurement is made in-situ, or on cores:

$$E_{dyn} = \alpha \cdot \gamma \cdot v^2 \qquad (1)$$

Where E_{dyn} the dynamic E-modulus [GPa]
α factor depending on Poisson´s ratio
γ the density of concrete [kg/m^3] (normally 2400 kg/m^3)
v the speed of sound (longitudinal waves) [m/s]

The factor α is:

$$\alpha=(1+\nu)\cdot(1-2\nu)/(1-\nu) \qquad (2)$$

Where ν Poisson´s ratio

For undamaged concrete $\nu\approx0.17$. Hence, $\alpha\approx0.93$. For severely damaged concrete ν is bigger. A value of 0.3 can be assumed if cracking corresponds to what occurs at the proportionality limit of concrete. Hence $\alpha\approx0.74$. This value is used for frost damaged concrete.

Dynamic E-modulus from the natural frequency of transverse vibration:
The testing is made in the laboratory on cores which must have a length/width ratio (slenderness) of at least 2 and preferably 3, or more. There are two possibilities:

1: *Forced vibration* perpendicular to the length axis of the specimen. The applied frequency is gradually increased (or decreased) until maximum amplitude of the specimen is reached. This is the natural frequency (Note: the ground mode of vibration must be searched, since the value of β in eqn. (4) is based on that vibration mode.)

2: The specimen is excited by a hammer. The natural frequency of the specimen is measured by a sensor on the specimen, or in the hammer.

The dynamic E-modulus is calculated by:

$$E_{dyn} = \beta\cdot G\cdot f^2 \qquad (3)$$

Where E_{dyn} the dynamic E-modulus [GPa]
β coefficient depending on the shape and size of the specimen
G the mass of the specimen [kg]
f the natural frequency of transverse vibration [s^{-1}]

The coefficient β is

General shape: $\beta=7.9\cdot10^{-11}\cdot(L^3/I)\cdot T$ (4)
Prism: $\beta=9.5\cdot10^{-10}(L^3/(b\cdot h^3)\cdot T$ (4a)
Cylinder: $\beta= 1.6\cdot10^{-9}\cdot(L^3/d^4)\cdot T$ (4b)

Where L the specimen length [m]
I the moment of inertia of the specimen [m^4]
T correction factor which depends mainly on the length/width ratio of the specimen (also to a certain extent on Poisson´s ratio) [-]
b width of a prism [m]
h height of a prism, parallel to vibration [m]
d diameter of a cylinder [m]

For a given value of Poisson´s ratio, the correction factor T only depends on the ratio between the radius of gyration of the specimen (r_g) and the specimen length (L). Values are given in the table for Poisson´s ratio ν=0.17 (undamaged concrete) and ν=0.30 (damaged concrete).

r_g/L	ν=0.18 T	ν=0.30 T
0.00	1	1
0.02	1.03	1.03
0.04	1.13	1.14
0.06	1.28	1.30
0.08	1.48	1.50
0.10	1.73	1.77
0.12	2.03	2.09
0.14	2.36	2.59

Prism: r_g=h/3.46. Cylinder: r_g=d/4
Example: For a cylinder with diameter 10 cm and length 25 cm, r_g/L=0.10, and T=1.73.

3.3 Amount of reinforcement and concrete cover

Aim:

1: The amount of reinforcement in the structure is fundamental for a detailed structural assessment. It can be different from what is shown on the drawings. In many cases, no drawings exist.

2: The concrete cover is important for an assessment of the anchorage capacity of the reinforcement bars and, consequently, for the structural stability. It is also important for an assessment of the residual service life with regard to reinforcement corrosion.

Method:
The location of reinforcement, the concrete cover in intact surfaces, and the residual concrete cover in scaled surfaces, shall be located by a cover-meter with the possibility to assess also the diameter of the bars.

4. Inspection and testing the structure with regard to assessment of the future status

4.1 The actual internal mechanical damage

Aim:

1: To make possible a quantitative determination of the actual degree of frost damage of the concrete (loss in compressive strength, loss in tensile strength, loss in bond strength, loss in E-modulus). This information is needed for a prognosis of the future structural stability.

2: To make possible an extrapolation of the future degradation of the structural stability according to the principles in ANNEX H.

Method:

Cores are taken from *undamaged* parts of the structure. Normal, standardised methods for testing and assessment of drilled-out cores, are used. These values are compared with values taken from *damaged* parts of the structure; paragraph 3.2. Damage (D) is defined

$$D = 1 - R_{damaged}/R_{undamaged} \tag{5}$$

Where $R_{damaged}$ strength or E-modulus of damaged parts

$R_{undamaged}$ strength or E-modulus of undamaged parts

4.2 Characterisation of the micro- and macro-climate around the structure

Aim:

It is important, for an evaluation of the future degradation, to have some information of the surrounding climate; see ANNEX C and D.

Important parameters for *internal frost attack* is:

* Access to free water (amount of precipitation, water splash, water accumulation in badly drained parts of the structure, etc.

Important parameters for *salt frost attack* are:

* Minimum freezing temperatures and frequency of low freezing temperatures
  This is required for design of a scaling test; paragraph 5.5.
* Use of de-icing salts; duration and amount. Indication of the presence of salt spray on parts of the structure, not directly exposed to salts, can be obtained by an analysis of the chloride profile in the concrete.
* Exposure to sea water

Method:

1: Detailed visual observations of the structure.
2: Collection of climatic data.
3: Collection of data from management of the structure.

4.3 The moisture level inside the concrete

Aim:

To find the actual moisture content of the concrete, W_{ACT} in order to estimate the risk of internal frost damage.

By comparing the observed value W_{ACT} with the water content $W_{CAP}(t')$ reached in a laboratory absorption test of the same concrete after an absorption time, which is representative for the actual type of structure (see ANNEX A, paragraph 6.2), one can estimate the risk of frost damage, and the observed behaviour of the structure can get an explanation.

One can also compare the observed value W_{ACT} with the water content, W_n, obtained at the breaking-point in the capillary absorption experiment. If the observed value is higher than the breaking-point absorption W_n there is an imminent risk of frost damage.

Low frost resistance:

$$W_{ACT} > W_{CAP}(t')$$
$$W_{ACT} > W_n$$

Fair (or good) frost resistance:

$$W_{ACT} < W_{CAP}(t')$$
$$W_{ACT} < W_n$$

Method:

Cores with diameter not above 6 cm are taken from different places of the structure, and preferably at different climatic conditions. This must be done without adding moisture, and without drying out moisture. One method is to drill small holes in a circle, and break loose the ”core”. This must be sealed immediately, in order to protect it from moisture changes. The core is treated in the following way; see Figure 3.

> Note:
> In the following, the moisture content W is exchanged for the total weight Q. This can be done since there is a linear relation between the two values; $Q=Q_{dry}+W$ where Q_{dry} is the dry weight of the specimen.

1: The core is weighed in its natural moisture condition, Q_{ACT}

2: The core is immersed in water for some time (at least 2 weeks) and the weight gain is monitored by *careful weighing* of the surface-dry specimen at different intervals.

The *long-term* absorption curve is described by:

$$Q_{CAP}(t) = Q_n + B \cdot t^C \quad (6)$$

Where Q_n, B, C constants obtained from the test

Since the specimen is very moist initially, it will be difficult to find a precise value of the breaking point, Q_n. Then the following additional procedure in points 3: and 4:, directly following points 1: and 2:, gives a safer value of the breaking-point:

3: The core is dried at room temperature to a moisture level far below the breaking-point.

4: The core is weighed and immersed in water for some time (at least 2 weeks), and the weight gain is monitored by *careful weighing* of the surface-dry specimen at different intervals. The long-term absorption curve is described by eqn. (7). The *short-term* absorption (the steep absorption curve) is described by:

$$Q_{CAP}(t) = k \cdot t^{1/2} \quad (7)$$

Where k the absorption coefficient; see eqn. (11)

Q_n is the breaking-point in a diagram where the two curves (6) and (7) intersect.

5: The weight Q is transformed to water content, W, by drying the specimen at +105°C.

($W = Q - Q_{dry}$ and $W_n = Q_n - Q_{dry}$ where Q_{dry} is the dry weight of the specimen.)

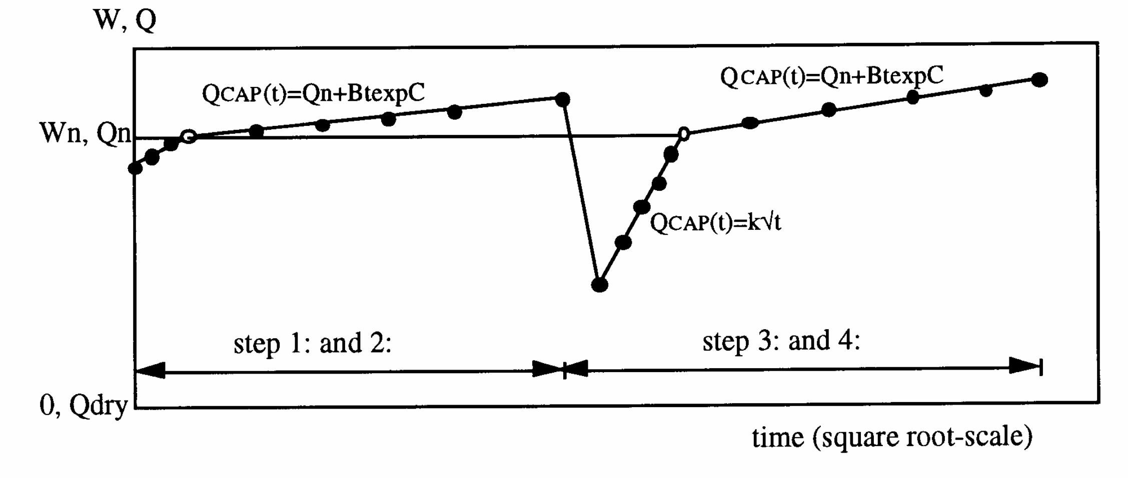

Figure 3: Determination of the actual moisture content.

5. Additional investigations

Besides the tests described above it can be important to know more about the concrete, and its *potential to resist frost damage*. Some methods are described below.

5.1 The air-pore structure

Aim:
To determine the ”quality” of the air-pore structure, expressed in terms of the following parameters:

1: The total air content
It is related to the internal frost resistance; ANNEX A, paragraph 7.1.

2: The specific surface area of the air-pore system
It gives an information about the fineness of the air-pore system. Normally, frost resistance increases with increasing specific surface area.

3: The Powers spacing factor for the entire air-pore system.
It combines the total air content and the specific surface area. Normally, but not always, the frost resistance increases with decreasing spacing factor

4: The size distribution of air-pores
It can be used for a theoretical service life modelling of internal frost resistance; /1/.

Test method:
Theoretical background:
The theoretical principles behind the determination of the air-content, the specific area and the spacing factor is based on the so-called *linear traverse method* described in ASTM C457. By adding the *Lord-Willis method* for analysing chord intercepts of the observed air-pores, the air- pore size distribution is also obtained by the same experi-mental technique; /2/.
Practical methods:
The test can be made by measuring the air-pore size distribution on a polished, concrete sample. The air-pores can either be observed ”manually” by microscope, as it is described in ASTM C457, or the observation can be made by a TV-camera mounted on a microscope, and coupled to an automatic image processing system. Even in the latter case, the software is based on the theoretical background behind the linear traverse method, described in ASTM C457, and the Lord-Willis method; /2/.

The test can also be made on smaller thin-sections. In this case, many sections are needed in order to obtain the necessary precision. The diameter distribution of the air-pore calottes is measured. A mathematical transformation of the observed calotte diameter distribution is required, in order to find the pore diameter distribution. If this transformation is not made in the mathematically correct way, the spacing factor will not be the same in the two methods. A method of transformation is described in /3/.

5.2 The effective air-pore volume

Aim:

Only air-pores that stay air-filled in the practical situation will function as stress relievers during the freezing process. Air-pores can become inactivated due to water-uptake. This occurs when the air-pores are inter-connected, so that they form continuous channels. It also occurs when the air-pores are so small that they take up water due to air-dissolution.

The effective air-pore volume gives information of the *potential frost resistance*; see Figure 5. It also is a fundamental parameter for a theoretical service life analysis.

Air-pores can also be filled with salt crystals. This is seen in the petrographic analysis described in paragraph 3.1.

Test method:

Cores with diameter 10 to 15 cm are drilled out of the structure. 20 to 25 mm thick slices are cut from the cores. The slices are treated in the following way:

1: Pre-drying in room-climate, protected from carbonation, during at least 2 weeks.

2: Weighing immediately before the suction test. Weight Q_o.

3: Placing in a tray containing water. Only 1-2 mm of the thickness is immersed.

4: Taking up and weighing at different time intervals; 10, 20, 30 minutes, 1, 2, 4, 6 hours, 1, 2, 3 ,4 ,5 etc days, until about 28 days. Before weighing, the bottom surface is wiped with a moist sponge. Directly after weighing, the specimen is once again placed in the tray. This is covered by an impermeable lid in order to protect the specimen from evaporation. The weights are called Q_t.

5: After terminated water uptake, the specimens are dried at +105°C to equilibrium and weighed, Q_{dry}

6: Vacuum-filling by pre-evacuation at maximum 2 bar during at least 24 hours, followed by water uptake with the pump running for about 1 hour, and finally water storage at ordinary atmospheric pressure during at least 1 week. Weighing in air and immersed in water, $Q_{sat,a}$ and $Q_{sat,w}$.

7: Calculating the degree of saturation S_{CAP} versus the square-root of time. Degree of saturation is defined:

$$S_{CAP} = \text{total water/total porosity} = (Q_t - Q_{dry})/(Q_{sat,a} - Q_{dry}) \qquad (8)$$

8: Defining the breaking point on the absorption curve (time in square-root scale) where the early rapid absorption goes over to a slow absorption; see Figure 4. This breaking point absorption is called Q_n, or in terms of degree of saturation, $S_{CAP,n}$

9: Calculating the total porosity P:

$$P = (Q_{sat,a} - Q_{dry})/(Q_{sat,a} - Q_{sat,w}) \tag{9}$$

10: Calculating the effective air content a_{eff}. This is defined:

$$a_{eff} = (1 - S_{CAP,n}) \cdot P \tag{10}$$

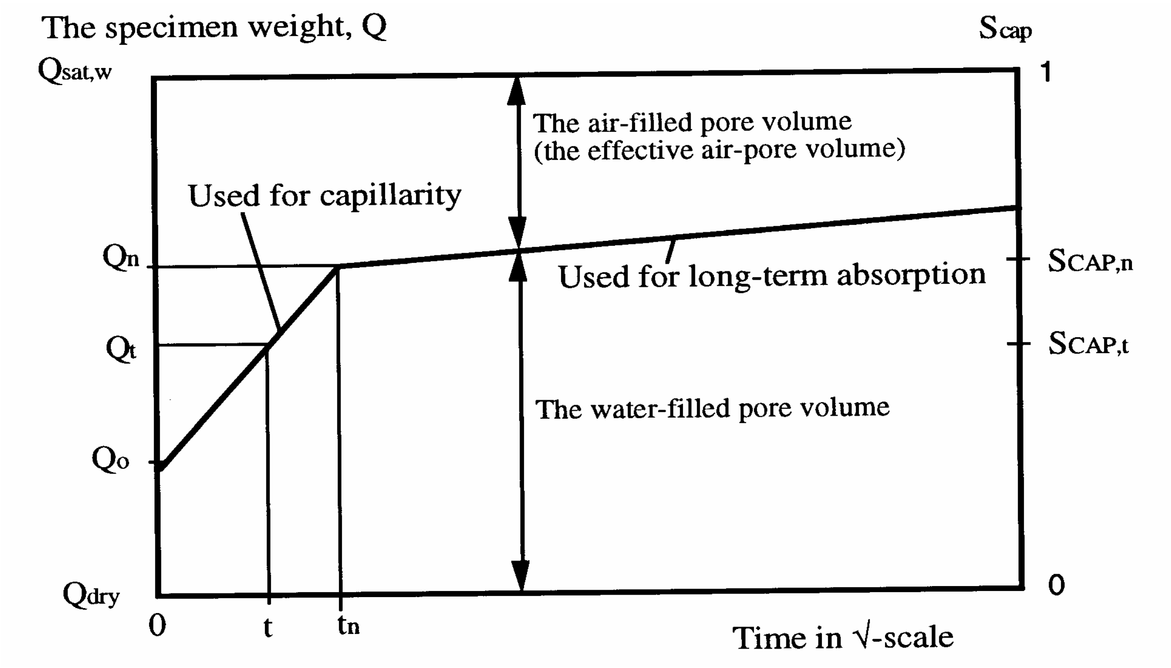

Figure 4: Illustration showing the result of a water absorption test used for determination of the effective air content, the capillarity, and the long-term absorption.

5.3 The capillarity and the long-term water absorption

Aim:

1: The capillarity gives information of the speed by which the concrete responds to exposure to free water. Two types of data are obtained:

a: *The absorption coefficient* telling the rate of water absorption. The value depends on the water content when suction starts. The absorption coefficient can be used for calculation of the moisture-time fields in the surface part of concrete, and therefore it is important for the prediction of the service life with regard to reinforcement corrosion. It is of less importance for analysing frost resistance.

b: *The resistance coefficient* telling the rate by which the water front penetrates the concrete. It is sensitive to inner damage. For an undamaged concrete, the value shall be within the range $5\cdot10^6$ to 10^8 s/m^2. Lower values are signs of internal damage. On the other hand, a high value is not a guarantee for undamaged concrete.

2: The long-term absorption gives information of the potential service life, and can be used for a prediction of the potential service life; paragraph 5.4 below.

Test method:

The same test method, as that used for determination of the effective air content, is used; see paragraph 5.2 above and Figure 4. The procedure is as follows.

Capillarity:

The absorption coefficient, k (kg/(m$^2\cdot$s$^{1/2}$), is defined by the slope of the *initial part* of the absorption-square-root of time curve; from time zero until the breaking point. This slope can be obtained by linear regression. The following definition is used:

$$k = dQ/d(t^{1/2})/A \qquad (11)$$

Where $dQ/d(t^{1/2})$ the slope of the water absorption curve
A the cross section of the specimen.

Note:
It might be that the absorption curve does not completely follow a square-root relationship. It might also be that the absorption curve is non-linear in a square-root time-scale. A typical case is when the initial water content is very high. In such a case, a new test with lower water contents can be made; see Figure 4. One can also normally find a certain linear portion of the curve for which the definition of k can be applied.

The result depends on the initial water content, expressed in terms of the initial weight Q_o; the lower the initial water content, the higher the absorption coefficient.

From the test, the so-called resistance coefficient, m [s/m^2], can be determined. It is defined:

$$t = m \cdot z^2 \quad (12)$$

Where t the suction time[s]
z the depth of penetration of the moisture front [m]

Thus, m is a measure of the resistance offered by the concrete to moisture penetration. m is much less dependent of the initial moisture content than k.

The value of m is obtained from the breaking point absorption:

$$m = t_n / H^2 \qquad (13)$$

Where t_n the nick-point time [s]
H the thickness of the specimen [m]

Long-term water absorption:
This is determined from the second part of the absorption curve; after the breaking point has been obtained.

The following expression is determined by linear regression; c.f eqn. (7):

$$S_{CAP}(t) = A + B \cdot t^C \qquad (7a)$$

Where S_{CAP} the capillary degree of saturation. It is calculated from the experimental points by the method that was described in the test method for the effective air content, paragraph 5.2.
A,B, C coefficients found by the regression analysis.

5.4 The potential internal frost resistance

Aim:
1: To obtain information of the general level of frost resistance of the concrete

2: To make possible an estimation of the potential residual service life of a structure that is susceptible to be frost damaged, but that is not yet damaged.

Test methods:
One can distinguish between methods that yield *qualitative* information only, and methods that yield *quantitative* information concerning service life. See also ANNEX A, paragraph 8.1.

Qualitative test methods:
In these methods, the specimen is exposed to repeated freeze/thaw cycles in a manner, that is not always representative of the practical situation. Information is obtained of the general level of frost resistance, but not of the frost resistance in the real case. The information obtained is often expressed in terms of qualitative judgements such as:

* Excellent frost resistance
* Good ....
* Fair ....
* Average ....
* Poor ...

There are many test methods to select between. The most widely used method is the American ASTM C666. This is a rapid freezing and thawing in water, or in water and air. Damage is measured by the loss in fundamental frequency of transverse vibration (a function of the loss in the E-modulus, and the weight loss). Also the scaling, and the length change can be measured. There are two variants of the method:

* *Procedure A:* ”Rapid water”. In this, the specimen is constantly stored in water during all the 300 cycles that are specified.

* *Procedure B*: ”Rapid air”. In this, freezing is made in air, but thawing in water.

It seems reasonable to use Procedure A for structures, that are used in very wet environments, and Procedure B for more dry structures. It must be noted, that *small changes in the test procedure, might give big changes in the observed frost resistance.*

The test is performed on drilled out specimens.

Quantitative test method:
Information of the potential service life can be obtained by the so-called ”Critical Degree of Saturation Method”, which is a Tentative RILEM-method; /4/.

The principles of the method are as follows; see Figure 5.

1: The critical degree of saturation, S_{CR}, is determined by a freeze/thaw test. This can be performed as a multi-cycle freeze thaw on many specimens at the same time, or as a single freeze/thaw experiment, with one specimen.

2: The long-term capillary degree of saturation, S_{CAP}, is determined by the method described for determination of the effective air content, paragraph 5.2. Three constants are determined for the long-term water absorption, A, B, C. The capillary degree of saturation is defined by eqn (8).

Then , the potential service life, t_{pot}, is calculated by:

$$t_{pot} = \{(S_{CR} - A)/B\}^{1/C} \tag{15}$$

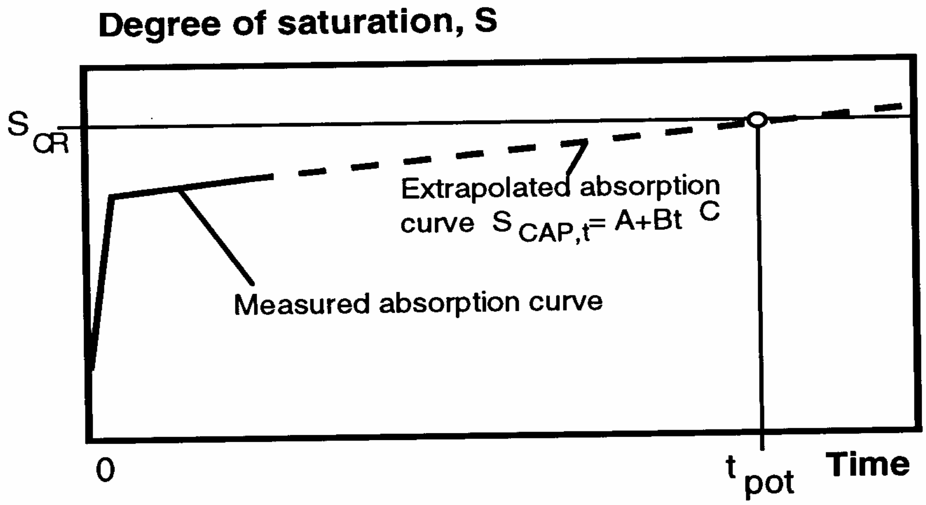

Figure 6: Illustration showing the definition of the potential service life.

5.5 The potential salt frost scaling resistance

Aim:

1: To find the general level of the scaling resistance of the concrete

2: To find the salt scaling resistance of a concrete, that has not been exposed to salts in the past, but might be so in the future

3: To find the scaling resistance of a concrete, that has been exposed to salts in the past, but will not be so in the future

Test method:

The method, most widely used, is the Swedish test method SS 13 72 44. In this, cast specimens, or cores, are exposed to, either 3% NaCl-solution, or pure water. Each freeze/thaw cycle has a duration of 24 hours. In total 56 to 112 cycles are performed. Useful information is, however, obtained already after the first 10 cycles.

In the actual assessment case, drilled out cores shall be used. The diameter shall be at least 10 cm and preferably 15 cm. The length shall be at least 10 cm. The surface, exposed in the test, shall be the real outer surface of the core.

For a concrete, that is exposed to de-icing salts in practice, a 3% NaCl-solution shall be used as freeze/thaw medium. For concrete, that is not exposed to any salts, pure water shall be used. For concrete in sea water, this might be used in the test.

5.6 The internal relative humidity

Aim:

The aim is to obtain general information of the moisture level of the structure. This information is, however, more important for an assessment of reinforcement corrosion, or ASR, than for an assessment of frost damage. This always occurs when RH=100% inside the concrete.

Test method:

The test shall be made with RH-sensors, that are carefully calibrated, and with a deviation from the true value of less than ±1% in RH. The moisture capacity of the sensor (the sensors own moisture uptake) shall be as small as possible in order to make the measurement rapid.

The measurements can be made in three ways; see Figure 6.

1: In holes drilled in the structure. The hole is lined with a plastic tube. The space between the tube and the concrete is sealed. Moisture is only allowed to enter the hole, at its bottom end. The RH-probe must be designed in such a way, that a small space can be enclosed at the end of the tube, in which the gauge can be located. Holes are drilled to different depths, so that the RH-profile can be obtained.

2: In drilled holes without lining. Then, the space between the probe and the hole must be sealed. The measurement depth is less well-defined than in method 1.

3: At the laboratory, in test tubes containing pieces of the concrete taken from the structure. The pieces are taken out of the structure in a manner that does not affect the moisture content. The same technique as for determination of free moisture can be used; see paragraph 4.3. Pieces are taken from different depths, so that the moisture profile can be obtained. Moisture is measured by the RH-probe inserted in the test tube.

Reliable results can only be obtained after a certain measurement time. The reason is, that it takes time for moisture to leave the concrete, and bring the surrounding air, and sensor, into equilibrium. Normally, 24 hours is sufficient. The RH-sensors must be calibrated, both before and after the measurement. It is also important that the sensor is pre-conditioned at a RH, that is lower than that of the structure. The sensors often have a significant hysteresis i.e. the same RH gives a different signal if the sensor is on its desorption curve, than if it is on its absorption curve.

Compensation must be made for temperature. If the temperature is higher at the measurement than in the structure (this is often the case when the measurement is made in the laboratory), the RH-value observed will be some % higher than the real value. The reason is that the equilibrium moisture curve (the sorption isotherm) of concrete is depending on the temperature; the lower the temperature, the higher the equilibrium moisture content. A diagram for this adjustment is shown in Figure 8. The correction is maximum at about 80% RH. At this RH-level, the measured temperature shall be corrected by about 0,4% in RH, for each degree of temperature difference between the structure and the laboratory.

Note:
The correction is important which is shown by the following example: The temperature of the structure is +10°C. The temperature in the laboratory, where the RH-measurement is made, is +22°C. The measured RH-value is 80%. Then, the real RH in the structure is $80-12\cdot0,4 = 75\%$. The correction is big and will have a significant influence on the estimated corrosion rate. The temperature effect also implies that RH inside a concrete will change when the temperature of the structure is rapidly changed; viz. the response of a concrete to temperature changes is normally so rapid that the concrete has not time to come to a new moisture equilibrium. Slow seasonal temperature changes will not have the same effect on RH. Then, the concrete has time to reach the new moisture equilibrium.

The technique for RH-measurement and its precision, and techniques for calibration of the gauges, are described in /5/.

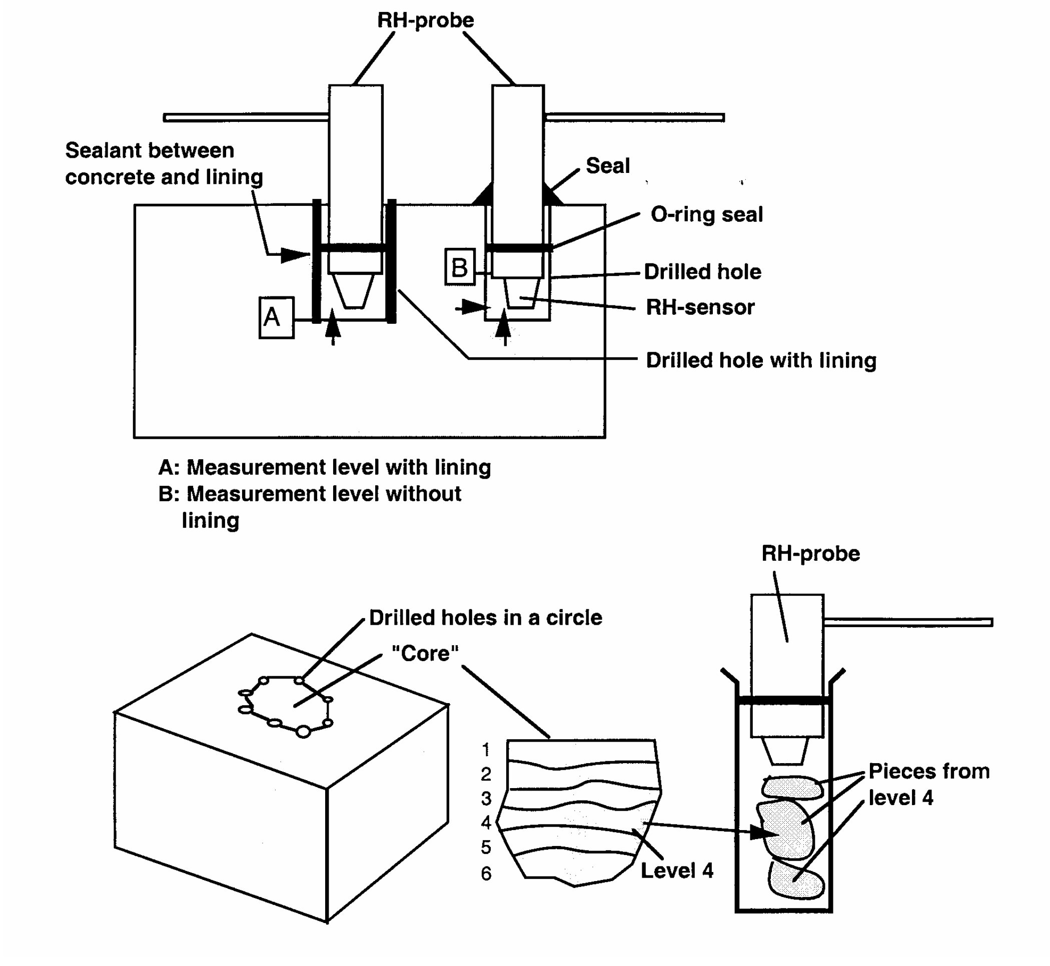

Figure 6: Different technique for measuring RH.

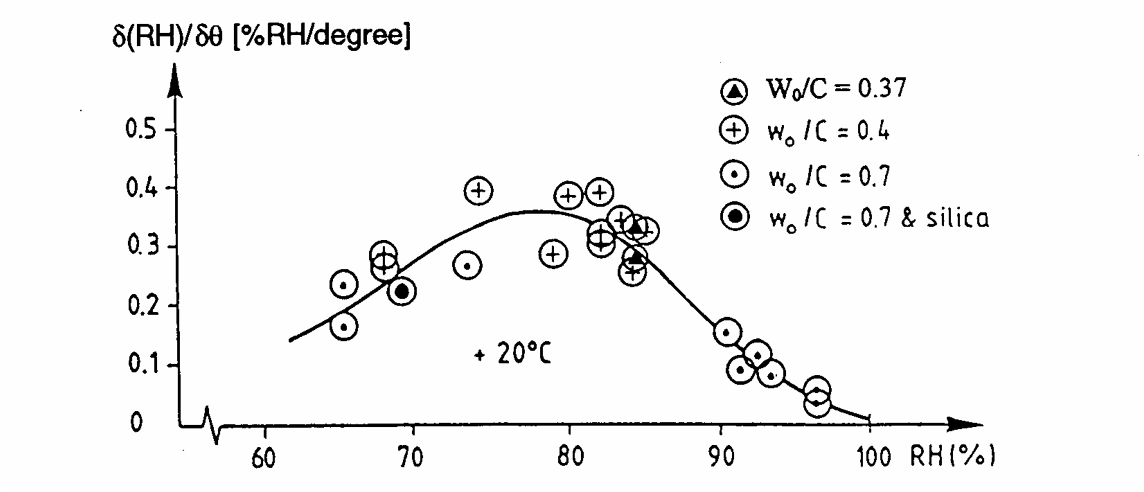

Figure 7: Change in RH for a certain change in temperature, assuming the total internal moisture content being constant. The average temperature is +22°C. The figure expresses the influence of temperature on the equilibrium moisture curves (sorption isotherms) The curve is taken from /6/.

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ANNEX H
Structural assessment-principles

Contents Page

ANNEX H: Structural assessment-principles

1. Effects of frost damage. Introductory remarks

Internal frost only affects (reduces) the mechanical properties of concrete in the structure. There is no effect on reinforcement (except for reduction in bond strength), and there is no internal stresses built up as in ASR. Therefore, the effect of internal frost damage can be rather easily handled in a structural assessment. The new reduced strength values, and the reduced E-modulus, are used as input in a re-design of the structure, using the same design codes as are used for design of a new structure, or as were used at the time when the structure was originally designed. Information from the structure of load and dimensions can be used in order to reduce the partial coefficients.

Note:
There might be synergetic effects between internal frost and other destruction mechanisms acting simultaneously. Examples are:

1: ASR, gradually opening the structure to inflow of moisture, thereby increasing the risk of frost damage.

2: Leaching of lime, gradually making the concrete more porous and permeable. The effect is a reduced internal frost resistance.

Both synergetic effects are treated in ANNEX F.

Salt frost scaling only affects the effective cross-section of the structure, and the cover over the reinforcement. It does not cause any internal damage. Therefore, its effects on the structural capacity can be assessed by introducing the reduced section and reduced cover in a control in critical sections of, (i) the moment capacity, (ii) the compression capacity, (iii) the shear capacity, (iv) the anchorage capacity.

Note:
Surface scaling will affect the residual initiation time until start of reinforcement corrosion, and possibly also the corrosion rate. Also leaching of lime will have an effect of the initiation time, and possibly also on the corrosion rate.

These types of synergetic effects must be considered when the structural stability with regard to reinforcement corrosion is assessed

Both synergetic effects are treated in ANNEX F.

2. Preliminary and detailed assessment

A *preliminary* (or simplified) assessment is based on:

1: Documents from the building phase of the structure
2: Visual inspection of the structure
3: General lower bound material data for frost damaged concrete

The preliminary assessment can be made on two levels:

1: Qualitative, including no (or few) structural calculations
2: Quantitative, making use of lower bound material data. This assessment is similar to, but less detailed than, the detailed assessment,

A *detailed* assessment is based on the same data as a simplified assessment supplemented by:

1: In-situ measurements of dimensions, surface erosion ,and some mechanical properties that can be measured in-situ, like E-modulus.
2: Laboratory measurements of concrete strength and stiffness using cores from the structure

This ANNEX H mainly deals with the detailed assessment. Material data based on specimens taken from the structure or based on testing this in-situ are described in this ANNEX. Lower bound data for a quantitative preliminary assessment are however also given in this ANNEX.

Design principles for different types of action (bending, shear, compression etc) and for different structural members are described an ANNEX I, J and K.

3. Present and future structural capacity

Two types of detailed assessment of the structural capacity are made:

1: Assessment of the *present* structural capacity valid for the time of inspection. Material and load data obtained from the structure are used as input in a traditional structural analysis (re-design). Material data from cores drilled out of the structure are transformed into characteristic values to be used in design, using well-established transformation technique considering moisture content, slenderness, and size of the core.

Alternatively, general lower bound data for mechanical properties of frost damaged concrete are used. These are based on test results shown in ANNEX E.

2: Assessment of the *future* structural capacity. The material data from the structure are extrapolated in time, and used as input in new structural assessments (re-designs) valid for different points of time in the future. In this way, the gradual degradation of the structural capacity and safety can be assessed. For a prediction of the future structural stability, the future changes in mechanical properties and cover are estimated according to the principles in ANNEX C and D.

Note:
When scaling has reduced the entire cover, reinforcement corrosion will be determining the structural capacity. In these cases, design formulas given in the Manual on Corrosion can be used for calculating the structural effects of loss of cover and loss of steel section.

The assessment (re-design) shall be made for all crucial cross-sections. This means that material data must be known for all these sections.

Principally, the assessment (both for present and future structural capacity) can be based on any design code; either the code used when the structure was erected, or the actual national or international code. The choice of code is important, and will normally be made by the Owner. Assessment procedure based on Eurocode EC2 are described in ANNEX I, J and K.

The load to be used in the design is a crucial factor. Normally, it must be given by the Owner.

The principles for an assessment are illustrated in Figure 1. Material data used in the assessment are illustrated in this Figure.

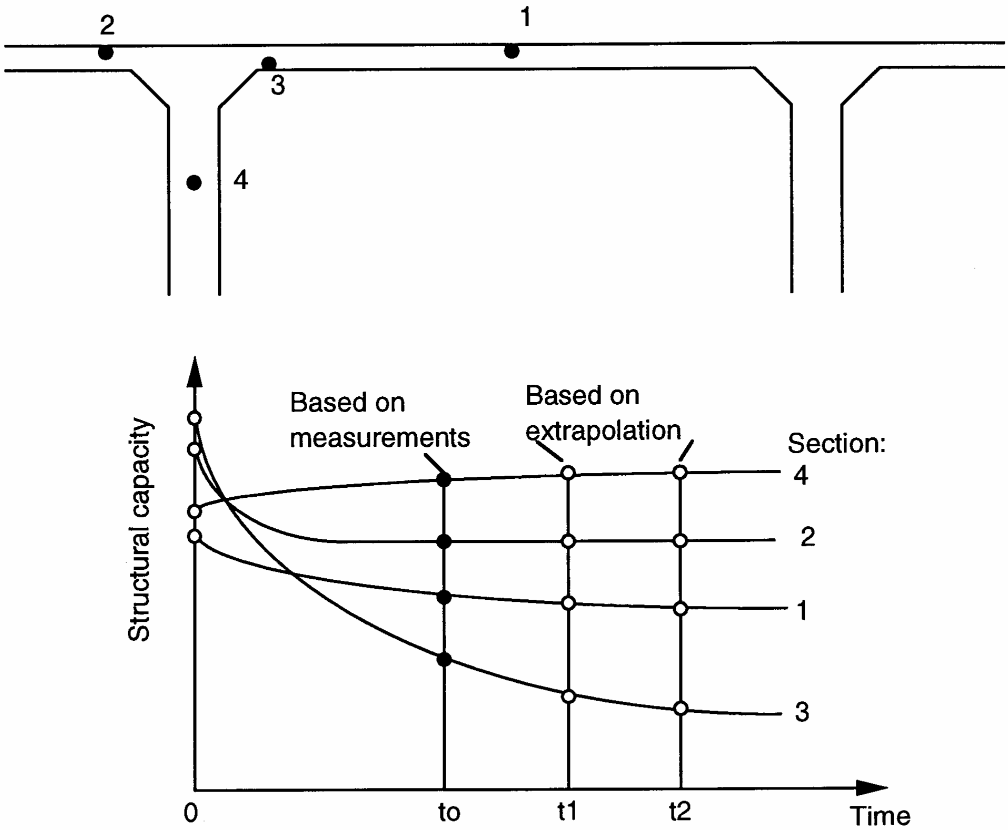

Figure 1: Assessment of the structural capacity of frost damaged structure; principles.

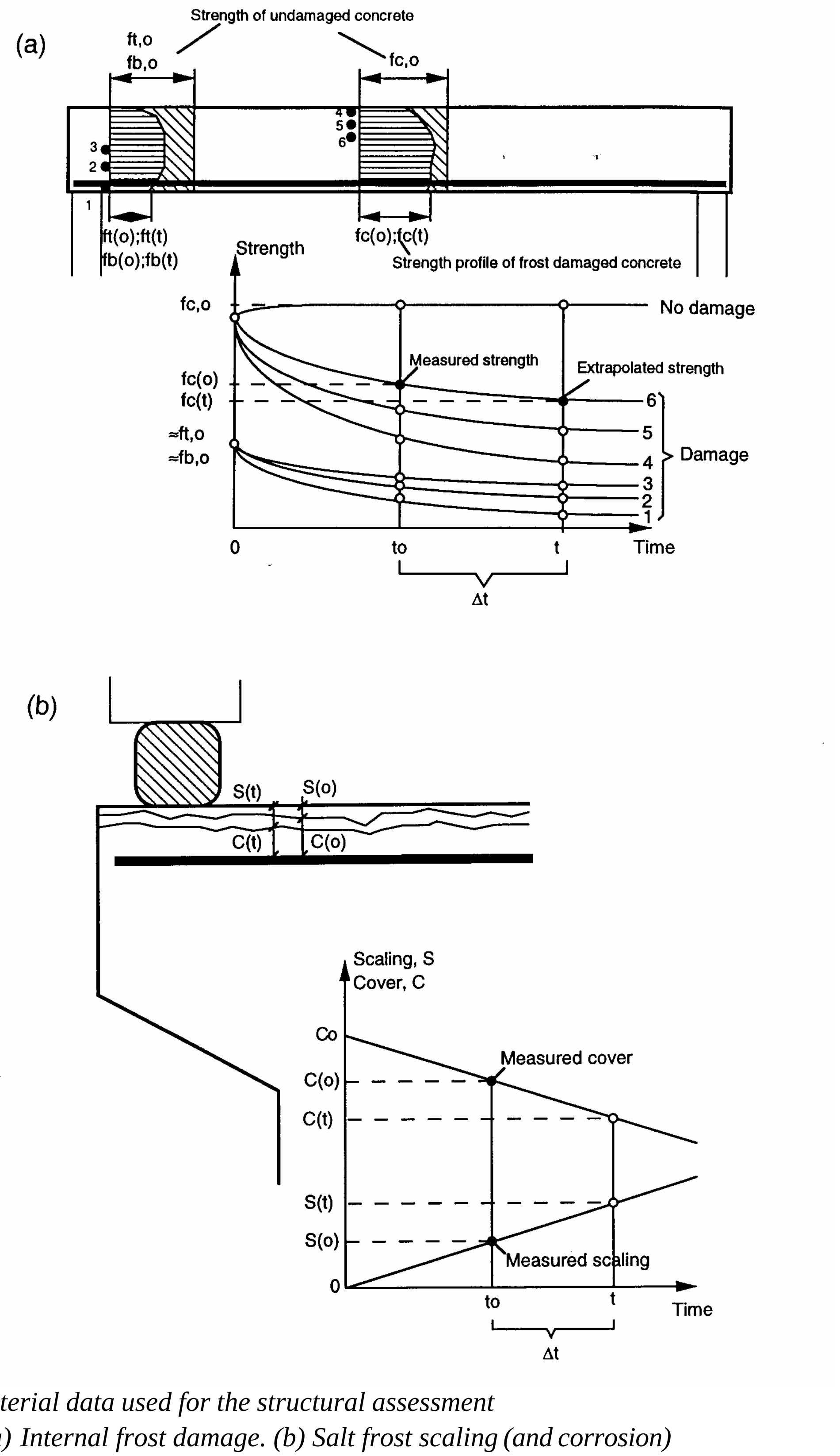

Figure 2: Material data used for the structural assessment
(a) Internal frost damage. (b) Salt frost scaling (and corrosion)

4. The present structural capacity

4.1 Assessment principles

The structural assessment is made as a re-design of the structure using actual loads, and actual material data for the deteriorated structure. The re-design is made for crucial sections with regard the structural stability and safety.

Normally only data for *concrete* must be considered. In a case where other deterioration mechanisms are going on simultaneously, these must also be considered in the design. Thus, if reinforcement corrosion is taking place in the structure, both the reduction in strength of concrete, and in bond due to frost, and the reduction in cross-section of the bar due to corrosion must be considered.

In the assessment (re-design), consideration shall be taken to possible changes in the stiffness of the structure caused by internal frost damage. Such changes will affect the moment and force distribution within the structure making some sections more stressed than before damage.

The design is made by a design code prescribed, or accepted, by the Owner.

Only structures with unstressed reinforcement are considered. Internal frost damage might cause a very big reduction in E-modulus (ANNEX E). Therefore, *pre-stressed concrete* with internal frost damage might have lost a considerable fraction of the initial compressive force. This is a dangerous situation, that has to be considered by careful analyses of the residual strength and stiffness of the structure.

Salt scaling will probably not affect a pre-stressed structure more than an ordinary structure. Thus the assessment principles below apply.

4.2 Material data for concrete

4.2.1 Introduction; acquisition of data; partial coefficients

Material data to be used in the structural assessment can either be determined by *testing the structure* according to ANNEX G, or be lower bound data obtained from *laboratory testing* of severely frost damaged concrete. Such data are given below, based on tests described in ANNEX E.

In-situ testing, preferably using cores from damaged and undamaged parts of the structure, is recommended since the lower bound data are so low that it is often not possible to save the structure on basis of these.

In the assessment case, all material data are normally based on cores, which means that any difference in "quality" between a test specimen used for quality control during the erection phase of the structure, and the structure is automatically considered. Long-term effects on strength have already taken place, which means that the observed strength is equal to the long term strength. No consideration need therefore to be taken to "static fatigue".

The core shall be tested in the same condition as it has in the structure. Therefore, the partial coefficient for strength and E-modulus can be reduced to 1.

The amount of damage can vary from place to place in the same structure. Therefore, in the normal case, many series of data from different places are required. Only in special cases can data taken from one place be used for the entire structure.

4.2.2 Characteristic concrete compressive strength

Based on measurements
The strength values for cores, corrected for slenderness, size, and wetness, according to the principles in ANNEX G, are transformed into characteristic values using the observed spread in the results.

The characteristic strength is:

$$\mathbf{f_{ch}=m -1.65}\cdot s \qquad (1)$$

Where f_{ch} the characteristic strength
m the mean value
σ the standard deviation

Based on lower bound values
A lower bound relation for compressive strength of severely frost damaged concrete is;
ANNEX E:

$$\mathbf{f_{c}= f_{c,o} - 20} \qquad (2)$$

Where f_c the characteristic compressive strength of frost damaged concrete
$f_{c,o}$ the characteristic compressive strength of undamage concrete

$f_{c,o}$ is based on cores taken from undamaged parts of the structure, or from test data obtained during the erection of the structure.

4.2.3 Characteristic concrete split tensile strength

Based on measurements
The measured split tensile strength values are used for determination of the characteristic tensile strength, using eqn (1).

Based on lower bound values
A lower bound relation for split tensile strength of frost damaged concrete is;
ANNEX E:

$$\mathbf{f_t = 3 \cdot f_{t,o} - 11} \qquad (3)$$

Where f_t the characteristic split tensile strength of frost damaged concrete
$f_{t,o}$ the characteristic split tensile strength of undamaged concrete

$f_{t,o}$ is based on cores taken from undamaged parts of the structure, or from test data obtained during erection of the structure. Equation (3) gives so low tensile strengths that it can hardly be used for practical design. Therefore, coring will normally be required.

The *uniaxial* tensile strength is 80% of the split tensile strength.

4.2.4 Bond strength. Definition

Bond strength is here defined as the fracture shear stress between a reinforcement bar and concrete, when the bar is pulled out of a big concrete volume. Thus, no conside-ration is taken to confinement by stirrups or to cover/diameter ratio. Bond strength is therefore a sort of ”intrinsic” strength, which is a pure material property, but which is depending on the surface characteristics of the bar.

This bond strength is used in conventional formulae in the design code for calculation of the anchorage capacity. In these formulae, structural characteristics such as bar diameter, presence of stirrups, and cover thickness are considered.

4.2.5 Characteristic bond strength, ribbed bars

Based on measurements of tensile strength

There are two alternatives to use the measured split tensile strength for estimation of the bond strength. The alternative that gives the lowest strength is used in the design.

Alternative 1: The average loss in bond strength is proportional to the average loss in tensile strength; ANNEX E. Therefore, the bond strength can be estimated by the following formula:

$$\mathbf{f_b = f_{b,o} \cdot (f_t / f_{t,o})} \qquad (4)$$

Where f_b characteristic bond strength of frost damaged concrete
$f_{b,o}$ characteristic bond strength of undamaged concrete
f_t characteristic tensile strength of frost damaged concrete
$f_{t,o}$ characteristic tensile strength of undamaged concrete

The characteristic bond strength of undamaged concrete ($f_{b,o}$) is calculated from the "undamaged" tensile strength $f_{t,o}$ according to rules in the design code used for the assessment.

Alternative 2: The lower bound relation between bond strength and measured split tensile strength is used directly

$$\mathbf{f_b=3 \cdot f_t} \tag{5}$$

Where f_t the measured characteristic split tensile strength.

Based on lower bound values for split tensile strength

The relation between the mean bond strength as function of the tensile strength, and the lower bound of tensile strength of frost damaged concrete is used.

The mean bond strength is (ANNEX E):

$$f_b= 3.3 \cdot f_t + 1.3 \tag{6}$$

After insertion of eqn. (3) for the lower bound of tensile strength:

$$f_b= 3.3 \cdot (3 \cdot f_{t,o} -11)+1.3 \tag{7}$$

or:

$$\mathbf{f_b = 10 \cdot f_{t,o} - 35} \tag{7a}$$

Where $f_{t,o}$ is the characteristic split tensile strength of undamaged concrete

This equation leads to very low values of the bond strength (or zero bond strength), unless the initial split tensile strength is high.

4.2.6 Characteristic bond strength, plain bars

Based on measurements of tensile strength

The lower bound relation between the split tensile strength and bond strength is used directly

$$\mathbf{f_b= 0.8 \cdot f_t - 2} \tag{8}$$

Where f_t the measured present characteristic split tensile strength

Based on lower bound values for split tensile strength

The relation between the mean bond strength and the lower bound of tensile strength of frost damaged concrete is used:

$$f_b= 0.8 \cdot f_t - 1.2 \tag{9}$$

After insertion of eqn. (3):

$$f_b = 0.8 \cdot (3 \cdot f_{t,o} - 11) - 1.2 \qquad (10)$$

or

$$\mathbf{f_b = 2.4 \cdot f_{t,o} - 10} \qquad (10a)$$

Where $f_{t,o}$ is the characteristic split tensile strength of undamaged concrete.

This equation normally leads to zero bond strength.

4.2.7 Characteristic E-modulus

Based on measurements

Static E

The E-modulus to be used in calculation of deformation cannot be based on the compressive strength, since this is normally affected by frost in another way than E. see ANNEX E. Therefore, cores are taken and tested mechanically in a compression test, according to the standard procedure.

In the static E-modulus, as defined here, creep is not included. Normally, creep can be neglected since the structure is almost always fairly old.

Dynamic E

Measurements in-situ of the speed of sound can be used for calculating E_{dyn}.
E_{dyn} can also be calculated from the natural frequency of transverse vibration of cores; see ANNEX E.

E_{dyn} can be translated to E_{stat} by:

$$\mathbf{E_{stat} = 0.85 \cdot E_{dyn}} \qquad (11)$$

Based on lower bound values

It has not been possible to find a general relation between the initial E-modulus and the E-modulus after frost damage. Any reduction in E seems to be possible; see ANNEX E.

The lowest observed value of E is **5 GPa**. This value is so low that it can hardly be used for a structural assessment. Therefore, in-situ measurements are required.

4.3 Surface scaling; concrete cover; dimensions of cross-section

The depth of surface scaling and residual cross-section area are measured in-situ in cross-sections that are crucial for structural stability and safety. The data are used as input in a calculation of the moment capacity, shear capacity, and compression capacity, using conventional formulae in the actual design code.

The residual concrete cover is measured in the same sections as scaling. The data are used as input in calculations of the anchorage capacity of reinforcement bars.

5. The future structural capacity

5.1 Principles

All material data, and data for scaling, residual cover, and cross-section, used at the assessment of the present structural status, are extrapolated in time, using the extrapolation principles described in:

* ANNEX C for internal frost damage
* ANNEX D for salt frost scaling

A structural assessment can then be made for any time in the future, using the same design code as for the assessment of the present status.

5.2 Internal frost damage

5.2.1 The structure is frost damaged

The extrapolation of material data depends on the environmental conditions. These are estimated on basis of an analysis of the actual moisture content and the climate around the structure. There are three moisture conditions (”moisture classes”); ANNEX C:

1: Moist
2: Very moist
3: Extremely moist

Each moisture class corresponds to a certain time extrapolation; ANNEX C.
For different parts of the same structure, different time extrapolations should often be used since the moisture condition is different in different parts.

Example:
A hydraulic structure like the front wall of a lamellae dam.

The top of the dam:	”Moist”
The downstream surface:	”Very moist”
The upstream surface:	”Extremely moist”

The degree of damage of different parts of a structure, and the actual moisture level, will often tell which environment is most applicable.

Moist:

Drying periods alternating with liquid water uptake periods, but no increase in internal moisture in the future.

All material data will be unchanged in the future.

$$\mathbf{R(t) = R(o)} \qquad (12)$$

Where R(t) the future value at time t of the actual property (strength or E-modulus)

R(o) the present value of the property R

Very moist:

Long liquid water uptake periods interrupted by shorter drying periods. A certain gradual increase in moisture in the future.

All material data follow a square-root time relation:

$$\mathbf{R(t) = R_o - [R_o - R(o)] \cdot (1 + D\,t/t_o)^{1/2}} \qquad (13)$$

Or, expressed in terms of ”damage”:

$$\mathbf{R(t) = R_o\{1 - D(o) \cdot (1 + D\,t/t_o)^{1/2}\}} \qquad (14)$$

Where R_o the present value of undamaged concrete

t_o the present age of the structure

Δt the extrapolation time [Δ=t(t-t_o)]

D the present damage

”Damage” is defined

$$\mathbf{D = [R_o - R(o)]/R_o} \qquad (15)$$

Extremely moist:

Continuous liquid water uptake. No drying periods. A gradual increase in moisture in the future.

All material data follow a linear time relation:

$$\mathbf{R(t) = R_o - [R_o - R(o)] \cdot (1 + D\,t/t_o)} \qquad (16)$$

Or, expressed in terms of damage:

$$\mathbf{R(t) = R_o\{1 - D(o)\cdot(1+\Delta t/t_o)\}} \tag{17}$$

5.2.2 The structure is not yet frost damaged. The residual time to frost damage

There are structures, or parts of a damaged structure, that is not yet frost damaged, but for which one might suspect that damage might come in the future. Examples are structures affected by ongoing leaching, or any other deteriorating process. In these cases, the extrapolation procedure described in ANNEX C, paragraph 6.6 might be used. It is based on freeze-thaw testing of the concrete combined with an extrapolated moisture absorption test. The extrapolation only gives the residual time until start of frost damage, but not the extent of damage. It, however, gives an indication of when a new inspection and assessment ought to be made.

5.3 Salt frost scaling. Concrete cover

5.3.1 Extrapolation of scaling based on measured scaling

A linear extrapolation is made of the present scaling:

$$\mathbf{S(t)=S(o)\cdot(1+\Delta t/t_o)} \tag{18}$$

Where
S(t) the future scaling depth at additional time Δt
S(o) the present scaling depth
Δt the extrapolation time
t_o the present age of the structure

The extrapolated concrete cover is

$$\mathbf{C(t) = C(o)-S(o)\cdot(\Delta t/t_o)} \tag{19}$$

Where
C(t) the future cover
C(o) the present cover

5.3.2 Extrapolation of scaling based on scaling tests

If scaling has not yet occurred, but might do so in the future because de-icing salt will be used, a measure of the expected scaling can be obtained by a salt scaling test. The extrapolation technique is described in ANNEX D, paragraph 5.

ANNEX I
Effects of frost on load-carrying capacity

Contents Page

ANNEX I: Effects of frost on load-carrying capacity

1. General

In this section, an attempt has been made to present guidance on the assessment process in a prescriptive format. Nine phenomena have been considered and guidelines are given for adjustment of national codes to fit the damaged situation.

Frost damages influences the load carrying capacity in two different ways.

- Frost creates loss of concrete area, influencing the concrete cover, effective depths of beams etc.
- Frost can also lead to reduced concrete strengths, the different parameters such as compressive and tensile strength and elasticity modulus are affected differently. Therefore it is important to measure the strength parameters that are to be used in a detailed assessment procedure since ordinary strength relationships are no longer valid.

The large uncertainties of the actual concrete strength must be born in mind during assessment of frost damaged concrete structures. Even if the strength is referred to as a constant value in the following text it can have a large variation from element to element and even within the same element.

During assessment of damaged structures it is of large importance to remember that the decisive failure mode can change due to the damages. A bending failure can become a shear failure or the other way around.

Reference is made to ANNEX E concerning the effect of frost on mechanical properties and to ANNEX H concerning basic principles for the structural assessment.

2. Bending

2.1 Surface scaling

Reduction of width and height of the affected structural element take surface scaling into account. The influence on bond strength must be checked, as available concrete cover is one of the main parameters for the bond strength. Another possible drawback from loss of cover is that the cover concrete may not be effective in resisting compression and it would then tend to act as a strut and possibly buckle.

However, Clark /1/ has calculated that the main bars would buckle rather than yield only if the ratio of link spacing to main bar diameter exceeded 44 and 32 for mild and high yield steel respectively. Thus, for the majority of practical situations main bar buckling is unlikely to be a problem provided that the links are anchored adequately (a ratio of 32 implies a link spacing of 800 mm for a 25 mm diameter high yield reinforcing bar). This investigation should be done on national basis.

2.3 Internal frost damage

Internal frost damages leads to reductions of compressive strength and tensile strength as well as the elasticity modulus. These reductions of concrete parameters make it necessary to check the following:

- Recalculation of the bending capacity with observed compressive strength. No investigations exist regarding the maximum compressive strain, therefore strain distribution as for ordinary concrete may be assumed.
- Verification of available bond forces based on reduced tensile strength must be made for the longitudinal reinforcement.
- In case of statically indeterminate structures, redistribution of internal forces due to reduced elasticity modulus must be checked.

3. Column behaviour

3.1 Surface scaling

As with beams in flexure, the compressive strength used in any assessment should be based on the uniaxial compressive strength. Reducing the cover concrete changes the cross-section and slenderness ratio. This modified slenderness ratio should be used in the assessment calculations to check buckling and to classify the column as either short or slender.

The check on longitudinal reinforcement buckling can also be omitted for most practical situations, where the ratios of link spacing to longitudinal reinforcement diameter are less than 44 and 32 for mild and high yield steel respectively and where links are anchored adequately.

3.2 Internal frost damage

A reduction in compressive strength is considered using measured values. Changes in elasticity modulus have to be taken into account for all columns; the bending stiffness is of large importance for second order effects, normally considered by methods such as the moment magnifier method.

4. Shear

At the moment no existing test data are available. Tests of the effect of frost damage on shear strength of beams are performed at Div. of Building materials at Lund Institute of Technology under the guidance of Dr. M. Hassanzadeh. The tests will be terminated during year 2001.

4.1 Surface scaling

Shear capacity can be calculated taking into account the same reductions as discussed for flexural beams, regarding the breadth and effective depth of the cross section. A possible reduction in tensile reinforcement ratio must also be done with regards to reduced bond. This also means that other failure modes can be decisive.

4.2Internal frost damage

When internal frost damage is present the reinforcement ratio is recalculated due to reduced bond strength and the observed tensile strength can be used in the design equations. This is so far a hypothesis that won't be validated until Dr Hazzanzadeh has finished his experiments.

5. Punching shear

No data is available for punching shear of frost damaged reinforced concrete structures.

6. Torsion

No data is available for torsional shear of frost damaged reinforced concrete structures.

The torsion design and assessment methods are all based on the space truss analogy. This space truss analogy consists of longitudinal bars acting as stringers, link legs acting as posts and the concrete between the cracks acting as compression diagonals. Clark(1) suggests that provided the member is torsionally under-reinforced, the torsional capacity is independent of the concrete strength (i.e. the reinforcement yields before the concrete crushes). Clark thus proposed for ASR affected structures that an extra equilibrium equation be used so that the stress in the concrete compression struts can be checked to ascertain the possibility of premature crushing. This approach might be possible for frost damaged concrete as well.

7. Bearing

No test data is available for bearing of frost damaged concrete.

Bearing is not affected directly by surface scaling.

Standard design equations based on the observed tensile strength of the reinforced concrete with internal frost damages can be used to calculate the bearing capacity for frost damaged structures.

8. Fatigue

No data is available for cyclic testing of frost damaged reinforced concrete structures.

9. Bond

Bond is dependent on the surrounding, i.e. confinement, links and the thickness of the cover. The tensile strength of concrete is used in many design equations when calculating the available bond force. Results from /2/ and /3/ are used to verify the use of the proposed equation for design bond stress from Model Code 1990 /4/.

9.1 Surface scaling

No test data is available for this case but it is suggested that loss of cover from surface scaling be taken into account as reduced concrete cover in design equations for bond and anchorage lengths.

9.2Internal frost damage

Calculations made with results from /2/ further analyzed in /3/ indicates that the design bond strength equations suggested in Model Code 1990 /4/ can be used to predict the bond stress for ribbed bars if the observed design tensile strength for the frost damaged concrete is used. For plain bars there is no correlation between the tensile strength and the bond stress and therefore no recommendations are made for this case.

The design bond strength according to Model Code 1990 /4/ is calculated as for ribbed bars.

$$f_{bd} = h_1 h_2 h_3 f_{ctd}$$

where;

f_{ctd} design value of observed concrete tensile strength

η_1 considers the type of reinforcement

η_1=1,0 for plain bars

η_1=1,4 for intended bars

η_1=2,25 for ribbed bars

η_2 considers the position of the bar during concreting

η_2=1,0 when good bond conditions are obtained, as for all bars with inclination of 45°-90° to the horizontal concreting. Good bond conditions also include all bars with an inclination less than 45° to the horizontal, which are up to 250 mm from the bottom or at least 300 mm from the top of the concrete layer during concreting.

η_2=0,7 for all other cases and for bars in structural parts built with slip forms

η_3 consider the bar diameter

η_3=1,0 for Ø ≤ 32 mm

$h_3 = \frac{132 - Ø}{100}$ for Ø > 32 mm

The results in /2/ and /3/ are mainly based on ribbed bars with diameter 25 mm or less, the bars where standing with an angel of 90° towards the concreting surface. Based on these facts and by using the mean value of the tensile strength from test specimens subjected to the same freeze thaw cycles as the test specimens with bars the results shown in Figure 1 and Figure 2.

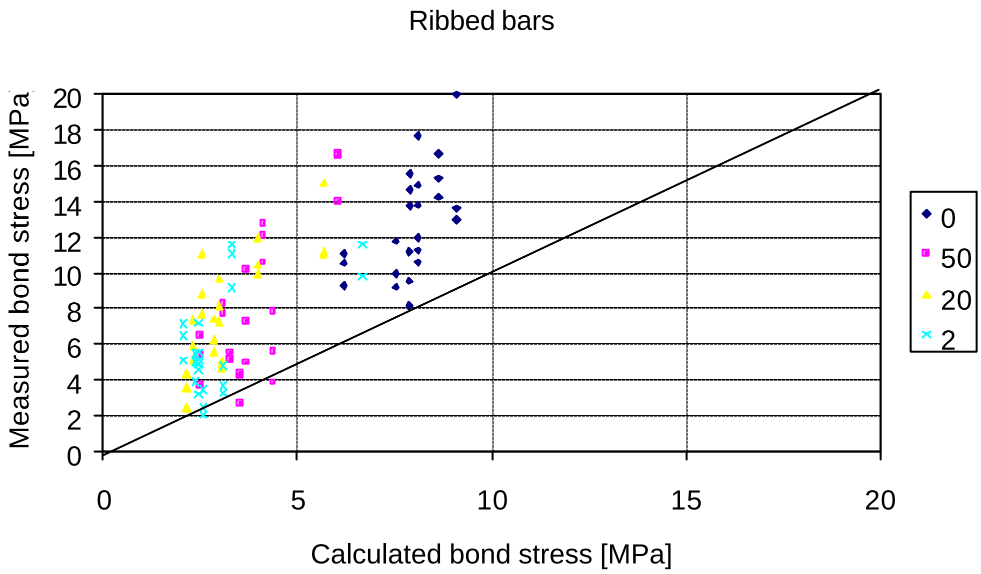

Figure 1 *Relation between calculated bond stress and measured bond stress for ribbed bars with different confinement. 0 indicates zero damage, 50, 20 and 2 indicate damage levels where 2 are the most severe damage.*

Proposed equation is also tried for plain bars with the result shown in Figure 2. The result indicates the equation is not suitable for plain bars and should not be used.

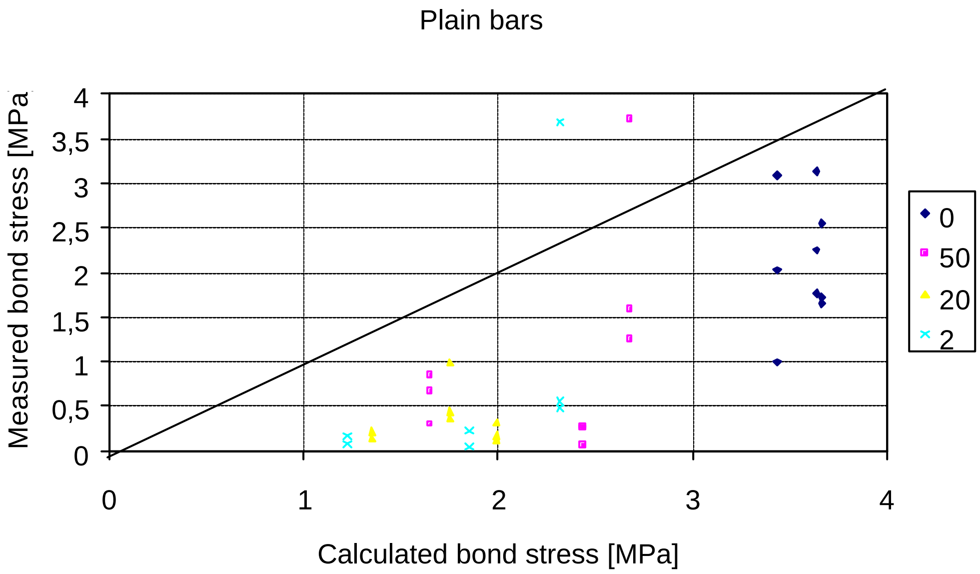

Figure 2 *Relation between calculated bond stress and measured bond stress for plain bars with different confinement. 0 indicates zero damage, 50, 20 and 2 indicate damage levels where 2 are the most severe damage.*

References

/1/ Clark, L. A.: Critical review of the structural implications of the alkali-silica reaction in concrete, Transport Research Laboratory, *Contractor Report 169*, Crowthorne, 1989.

/2/ Janz M, Johannesson B. Influence of frost damage on bond between reinforcement and concrete'. (Only in Swedish: Inverkan av frostskador på vidhäftningen mellan armering och betong – Resultatredovisning.), Division of Building Materials,Lund Institute of Technology, Report TVBM-7061, Lund 1994.

/3/ Fagerlund, G., Janz, M., Johannesson,B.: Effect of frost damage on the bond between reinforcement and concrete. Division of Building Materials, Lund Institute of Technology, Report TVBM-9016, Lund 1994.

/4/ CEB-FIP Model Code (1990) 'Bulletin d'Information 213/214', Lausanne, Switzerland May 1993, 437pp.

ANNEX J
Behaviour at the serviceability limit state

Contents Page

ANNEX J: Behaviour at the serviceability limit state

1. General

Assessments are usually only carried out at the ultimate limit state. However, there are occasions when the serviceability limit state is of interest to the owner of a structure. A typical example would be where a bridge is assessed for its suitability for inclusion in a new (or widened) highway scheme. In such a case the owner would want a fully compliant structure that is satisfactory at both the ultimate and serviceability limit states.

Much of the research work on which this manual is based has been carried out at the ultimate limit state in order to ascertain the load-carrying capacity of various members. However, sufficient data is available to provide outline guidance on assessing a reinforced concrete structure at the serviceability limit state.

2. Stiffness

The stiffness of a member is defined by expressions containing multipliers of EA/L, EI/L, EI/L^2, EI/L^3 and GJ/L. Any change in stiffness will be a functions of changes in the elastic modulus (E and G) and the concrete cross-section (A, I and J). Any change in member length will be insignificant in comparison to changes in E, I, A and J and can be ignored.

The section properties will have to be chosen to reflect the condition of the members under consideration. Cracked, uncracked or partially cracked section properties are permitted in Codes of Practice for serviceability calculations. The modular ratio should be based on the elastic modulus corrected for frost otherwise the concrete contribution to the section properties will be underestimated.

The relation between elastic modulus for frost damaged concrete and undamaged concrete is very uncertain. No predictions can be made of the effects of internal frost damage on elastic modulus; see ANNEX E. Therefore, the actual E-modulus must be determined experimentally according to the principles described in ANNEX G.

ANNEX K
Load-carrying capacity according to ENV 1992-1-1:1991

Contents | Page

ANNEX K: Load-carrying capacity according to ENV 1992-1-1:1991

1. General

In this section, an attempt has been made to present guidance on the assessment process when using ENV 1992-1-1:1991. Nine phenomena have been considered and rules based on tests and theoretical studies are presented. The detailed guidance is presented in the form of modifications to ENV 1992-1-1:1991 to allow amend it for use with the assessment of frost-affected structure.

It is important that the User updates this guidance to reflect any updates to ENV 1992-1-1:1991.

The procedures are best implemented as spreadsheets. In that way a series of 'what if' assessments can be undertaken to investigate the sensitivity of structures and different deterioration scenarios. For example, checks could be made on what level of material deterioration still gives acceptable load-carrying capacities within the structure.

2. Bending (EC 2 Section 4.3.1)

The amendments given in Table 1 should be made to ENV 1992-1-1:1991.

Table 1: Amendments to ENV 1992-1-1:1991 required to cater for the effects of frost on flexure

Clause	*Amendment*
P1(iv)	The stresses in the concrete compression are preferably derived from test on the damaged concrete. If this is not possible the design stress-strain curve in Figure 4.2 or Figure 4.3 can be used.
P1(viii)	For cross-sections not fully in compression, the limiting compressive strain is taken as the measured value. If no possible the limiting compressive strain is taken as 00,0035.

3. Column behaviour (EC 2 Section 4.3.5)

The amendments given in Table 2 should be made to ENV 1992-1-1:1991.

Table 2: Amendments to ENV 1992-1-1:1991 required to cater for the effects of frost on column behaviour

Clause	*Amendment*
P(3)	Observed material properties should be used since the deviation from the design properties can be large. Also the restraint of the column should be taken under consideration with respect to the change in E-modulus that can occur in frost damaged concrete structures.

4. Shear (EC 2 Section 4.3.23)

It is of extreme importance that measured concrete strengths are used since the correlation assumed between for instance tensile and compressive strength no longer is valid.

The amendments given in Table 3 should be made to ENV 1992-1-1:1991.

Table 3: Amendments to ENV 1992-1-1:1991 required to cater for the effects of frost on shear

Clause	*Amendment*
4.3.2.3	Replace the definition of f_{ctk} with: “f_{ctk} shall be taken as the concrete tensile strength, determined by considering the deleterious effects of frost”
Table 4.8	The correlation between tensile and compressive strength is no longer valid and this table can not be used.
4.3.2.3	Replace the definition of f_{ck} with: “f_{ck} shall be taken as the concrete compressive strength, determined by considering the deleterious effects of frost”

5. Torsion

There appears to be no test data available on the effects of frost on the torsional capacity of concrete members. Hence, no definitive guidance can be given.

6. Punching shear

There appears to be no test data available on the effects of frost on the punching shear capacity of concrete members. Hence, no definitive guidance can be given.

7. Bearing

There appears to be no test data available on the effects of frost on the bearing capacity of concrete members. Hence, no definitive guidance can be given.

8. Fatigue

There appears to be little test data available on the effect of frost on the fatigue strength of concrete. Hence, no definitive guidance can be given.

9. Bond (EC 2 section 5.2.2)

Section 5.2.2 shall be replaced with this section of the Manual.

9.1 Ribbed bars. Internal frost damage

The design bond strength is calculated as:

$$f_{bd} = h_1 h_2 h_3 f_{ctd}$$

where;

f_{ctd} design value of tensile strength for frost damaged concrete

η_1 consider the type of reinforcement

- η_1 1,0 for plain bars
- η_1 1,4 for intended bars
- η_1 2,25 for ribbed bars

η_2 consider the position of the bar during concreting

- η_2 1,0 when good bond conditions are obtained, as for all bars with inclination of 45°-90° to the horizontal concreting. Good bond conditions also include all bars with an inclination less than 45° to the horizontal, which are up to 250mm from the bottom or at least 300mm from the top of the concrete layer during concreting.

- η_2 0,7 for all other cases and for bars in structural parts built with slip forms

η_3 consider the bar diameter

- η_3=1,0 for Ø ≤ 32 mm
- $h_3 = \frac{132 - Ø}{100}$ for Ø > 32 mm

9.2 Plain bars. Internal frost damage

No recommendations given, consider the bond strength to be zero.