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**DETERIORATION, SERVICE LIFE AND REPAIR OF CONCRETE  
STRUCTURES - KNOWLEDGE AND RESEARCH NEEDS**

**Göran Fagerlund**

Report TVBM-7112  
Lund, May 1997

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# DETERIORATION, SERVICE LIFE AND REPAIR OF CONCRETE STRUCTURES - KNOWLEDGE AND RESEARCH NEEDS

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## Abstract

The paper expresses the author's personal view concerning the present status within service life design of concrete structures. It is stated that service life design is still in an embryonal stage, possibly with the exception of reinforcement corrosion. For other types of attack, there have been almost no attempts to exchange the traditionally used but vague concept "durability" for the more precise concept "service life", by developing service life models that are based on a deep understanding of the destruction mechanisms.

A very short discussion is made on different types of attack, and comments are made on research needed in order to refine existing service life models, and to develop new models. Reinforcement corrosion has been the most intensely studied durability problem during the last decades, and it is also the durability problem for which service life modelling has been fairly well-developed, and also used in practice. Weak points in the present models are pointed out, especially the fact that the "moisture mechanics part" of the problem, until recently, has been more or less completely omitted, and that most work has been made on the, perhaps, rather trivial case, *defect-free concrete structure exposed to constant outer conditions*. It is observed, however, that research is now going on in order to cope with the considerably more complex, and much more normal case, *defect concrete structure exposed to varying outer conditions*.

The fact that two destruction types might influence each other is never considered in normal service life modelling. It is shown by an example that the effect of such synergetic effects could be very big.

For other types of attack than corrosion, such as frost, acid and chemical attack, possible ways of developing service life models are outlined very briefly.

Functional requirements in connection with repair are discussed. Examples showing the importance of selecting repair systems that do not harm the old concrete are presented. The use of high quality concrete for repair purposes is stressed.

The importance of involving the structural engineer in the assessment of the structural stability of a damaged concrete, and in defining the criterion for the minimum technical performance, is emphasized. This is especially important in conjunction with decisions concerning repair, and service life of the repaired structure.

Finally, the importance of using moisture mechanics, and using stochastic methods for service life design is emphasized. Attempts have previously been made to develop stochastic methods for corrosion, but we also need them for other types of attack. Stochastic methods must be based on realistic destruction models. This is not always the case with the models presented till now.

## 1. On the service life concept

Normally, the word *durability* is used for describing the ability of a material, or a structure, to withstand different types of "environmental aggression", such as frost attack, sulfate attack, acid attack, or reinforcement corrosion. One often differentiates between concrete with *low durability*, *average durability*, or *high durability*, or by using some other qualitative expressions. The judgement is normally based on some sort of laboratory tests in which the concrete is exposed to an accelerated exposure, or, in rare occasions, to a simulated natural environment. The effects of the exposure on the concrete are measured as surface erosion, or as expansion, or as decrease in some mechanical property, such as strength or E-modulus, etc. In other, just as rare occasions, the judgement is based on the long-term behaviour of the same type of concrete structure exposed to the real environment.

The concept *durability* based on such tests, or based on similar structures exposed to the natural exposure, however, is unprecise and unquantified. It will not tell very much about how long time a new structure to be built with the actual concrete, or with the actual concrete cover, will function in the actual environment, and it will not tell very much about how an existing structure will behave in the future. Besides, by only using the concept durability, it is not possible to compare different types of materials used in the same environment. One typical example is a sewage pipe. By accelerated tests one can obtain some information concerning the durability level of the actual product, but one will never be able to tell whether a certain concrete pipe will have a longer service life than a certain plastic pipe. Therefore, traditional durability tests can hardly be used in a materials choice situation. The same is true when it comes to a choice between one design of a concrete structure based on a low water/binder ratio combined with a small cover, and another design based on a higher water/binder ratio, but combined with a bigger cover. We will not be sure of the values of w/b and cover to be used for equal service life.

In traditional durability tests, unloaded crack-free specimens will always be used. In reality, the concrete is exposed to compressive, tensile, and fatigue load, and it will contain cracks. Besides, the lab-specimen is "perfect", while the real structure is defect due to bad curing or other types of mistreatment during the building phase. It might also contain such defects as uncompacted parts, insufficient bond to the reinforcement, etc. Therefore, most laboratory test methods are not representative of the real structure. They will merely give some information about the general durability level of the defect-free material.

These limitations in traditional durability tests, and in the durability concept, were pointed out more than 20 years ago, by professor Sven Gabriel Bergström, who was then head of the Swedish Cement and Concrete Research Institute in Stockholm. He, and maybe some other pioneer unknown to me, meant that durability research had entered a dead end, and that our knowledge on durability was so far advanced that one should instead of making more or less intelligent accelerated lab-tests, try to make *service life calculations* and *service life design*. He therefore advocated that durability studies should primarily be aimed at understanding the destruction mechanisms and the destruction kinetics in order to make service life predictions possible.

This means that we should try to analyze and divide the complex destruction process into the primitive partial processes occurring, or determining, destruction. These processes should be studied, and then be synthesized again into a model for the real destruction process. In this way it should, in principal at least, be possible to develop methods by which quantitative information about the durability of the structure could be gained. Thus, it should perhaps be possible to express durability in terms of years. This would facilitate the selection of good materials and good structural design. Besides, by using the concept service life, it would be possible to compare different types of materials in a fair manner. Effects on the service life of defects in the finished structure, such as cracks or bad compaction, could be taken care of by analyzing their effects on the different partial processes, such as the moisture content, the permeability, or the threshold chloride concentration.

Professor Bergström's idea was fruitful, and it was rapidly accepted. Some of the first applications were the detailed reinforcement corrosion model by Kyösti Tuutti [1], the theoretical/experimental model for acid attack by Lars Rombén [2] and the embryonal model for frost destruction developed by the author [3]. All three researchers were then active at CBI in Stockholm. Tuutti's model, and modifications of this, is the most widely spread, and the most cited of the three. Non-erosive acid attack, which is a comparably simple type of attack, was later subject to a similar service life analysis by Grube and Rechenberg [4]. Service life models with regard to frost attack has been further developed by the author [5] and by Vesikari [6]. The theories, however, are still rather primitive. For internal chemical attack there are in fact no existing service life models that can be used in the design phase.

## 2. Deterioration of concrete is more than reinforcement corrosion

One could without exaggerating too much state that in everyday speech service life of concrete has become synonymous with service life with regard to reinforcement corrosion. This is also the case at this conference. Almost all contributions deal with corrosion, despite the fact that the scope of the conference reasonably must be much wider. One might speculate about the reason. Maybe, it is because corrosion is a less complicated destruction type than other destructions like frost attack, alkali-aggregate reaction, or sulphate attack. Or, at least that one thinks that it is less complicated, because it seems to be possible to divide the corrosion problem into a number of seemingly simple partial processes that can be described by simplified diffusion processes to which there often exist analytical solutions. In fact, this seems to be an oversimplification that might lead to the wrong conclusions.

Another possible reason behind the overwhelmingly great interest in reinforcement corrosion might be that it is supposed to be the most frequent type of destruction. Maybe it is so in some countries. But in other countries, like Sweden, frost attack, however, has been much more frequent than corrosion. Frost attack is serious also for corrosion in that sense that if a structure is not frost-resistant, it is rather meaningless to talk about service life with regard to corrosion. Then, the concrete cover is probably lost by frost-erosion or cracking long before corrosion should have started if the cover had been intact.

It is remarkable that the powerful tool comprised in the service life concept has not been more exploited for other destruction types than reinforcement corrosion, such as internal chemical attack, like alkali-aggregate reaction or sulphate attack. Some ideas concerning how a service life concept might be used also for these destruction types are indicated below in paragraph 4.

It is also remarkable that the service life concept has not been used more in connection with repair of concrete structures that are damaged by frost, or corrosion, or any other type of destruction. A rational procedure should be the following: at first, one assesses the present status - e.g. the load-carrying capacity of the structure, and the risk of collapse. This is made on basis of testing in situ of critical sections of the structure. Then, one should try to estimate the actual deterioration rate in order to predict the residual service life if no repair is made. Finally, one should estimate the residual service life after different types of repair. In this work the structural engineer should also take part because the assessment almost always involves calculations and judgements of the structural stability and the risk of failure.

Such an approach is rational because it might turn out that much less repair is required than what is anticipated on basis of the visual appearance of the structure only. A methodology for such a service life assessment of the structural stability of a structure damaged by reinforcement corrosion, frost, or alkali-aggregate reaction has been outlined in a Brite-Euram project [5].

By a service life analysis, it might also turn out that the proposed repair technique actually is more harmful to the old structure than what could be accepted; more severe frost damage might occur, or the corrosion rate might be increased due to radical changes in the inner moisture level of the old concrete caused by the repair. Some examples are given below in paragraph 6.

### 3. General requirements for a service life assessment

As mentioned above, the service life concept is a powerful tool both in the design of new structures and in the repair of damaged structures. Detailed and quantified knowledge, however, is required concerning the following factors:

- 1: *Functional requirements.* These are fairly easy to formulate when it concerns load carrying capacity. Different parts of the structure, however, are more or less sensitive to damage. For instance, reinforcement at the beam support, and shear reinforcement are more sensitive than the reinforcement in the mid-span of a freely supported beam. This must be considered when the functional requirement is defined. Therefore, the structural engineer should be involved in defining such functional requirements that are coupled to the structural stability. The functional requirement might be very difficult to express in quantitative terms when it concerns such properties as aesthetics, etc.
- 2: *Quantified environmental properties.* Fundamental properties for all outdoor structures are, the RH-variation, the temperature variation, the amount and intensity of precipitation and driving rain, etc. For indoor structures, the RH-variation and temperature variation are fundamental properties. For structures exposed to chemical attack or de-icing, the concentration of aggressive substances primarily acids, or chloride, or sulphate are important. The outer RH-variation and exposure to precipitation or splash are transformed into an inner RH-variation. Methods for this transformation are given in [8] and [9]. Examples of the variation in internal RH close to the surface of different concretes are shown in Fig 1. Almost all destruction types are determined by the internal moisture level. Therefore, as further pointed out in paragraph 8, moisture mechanics is fundamental for service life calculations; [10].
- 3: *The destruction mechanism.* It can be defined as the influence of the actual environment on the concrete. Besides, eventual synergy between two or more destruction types acting simultaneously must be considered. One example is salt-frost scaling and reinforcement corrosion. This has been theoretically studied in [11]. Due to the gradual erosion caused by salt-frost attack, the concrete cover is reduced. This shortens the time it takes for reinforcement corrosion to start. One example is shown in Fig 2. Without erosion the service life for a cover of 30 mm is more than 100 years. With erosion it is only 40 years. There are also other types of synergetic effects that must be considered, such as combined ASR and frost.
- 4: *Test methods and/or calculation methods* by which knowledge according to points 1 to 3 above can be gained, and be transformed into a service life calculation. This means that one has to have test methods that are able to measure the different components of a destruction process; e.g. the relevant environmental and material properties. Or, one has to have access to test methods that are able to measure the total destruction process in a non-accelerated test. Principally, the service life calculation is a calculation of how the actual functional requirement changes with the exposure time. Normally, for a structural part, it should be a calculation of how the load carrying capacity changes with time. Principally, the structural engineer should always be involved in a service life calculation of a structure such as a bridge, and not only the materials engineer. This is however almost totally neglected in durability research and durability design. There seems to be watertight bulkheads between these two specialities.

The main point is the destruction mechanism since it determines the environmental and materials properties that are essential, and the test methods to be used.

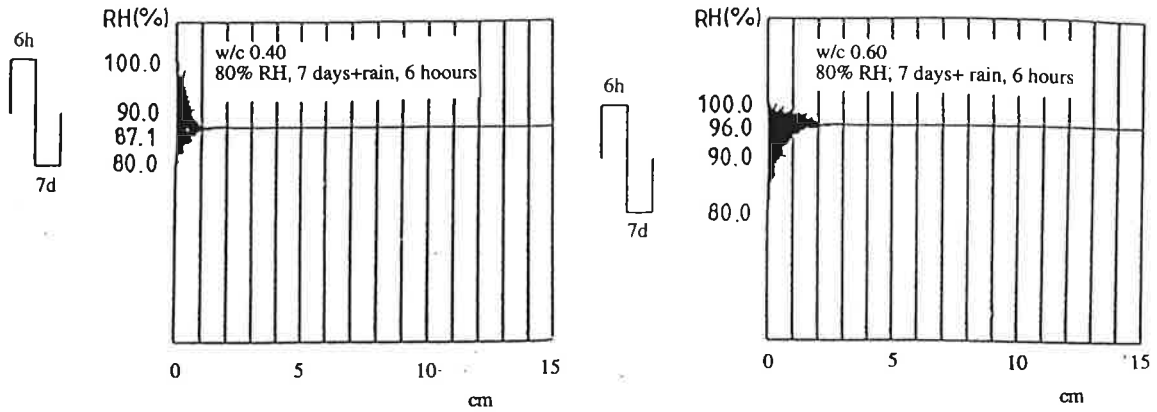


Fig 1. Examples of calculated moisture variations in the surface part of concrete exposed to short periods of rain mixed with longer periods of exposure to 80%. Left, w/c 0.40. Right, w/c 0.60; [8].

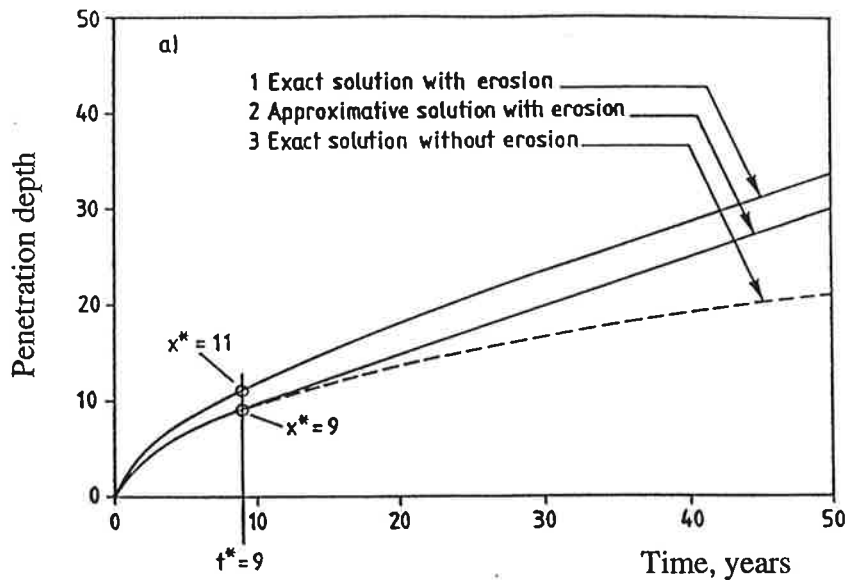


Fig 2. Example of the effect of combined salt-frost scaling and chloride-induced reinforcement corrosion; [11]. (Assumptions: frost-erosion  $5 \cdot 10^{-4}$  m/year, Penetration rate of the threshold concentration  $3 \cdot 10^{-3}$  m/year<sup>1/2</sup>)

## 4. Service life design in general. Research needs

### 4.1 Reinforcement corrosion due to chloride

As said above, most research concerning service life design has been spent on reinforcement corrosion, and almost all research has been made by materials specialists. The model used is normally the Tuutti model in which the process is divided into an initiation phase and a propagation phase. The time-dependency of both phases are described by simple diffusion processes with constant effective rate-determining coefficients. For a given situation the model can be expressed by two functions:

$$x_{Cl} = f(\delta_{Cl}; t) \quad (1)$$

$$x_{corr} = K \cdot t \quad (2)$$

where  $x_{Cl}$  is the penetration depth of the threshold penetration,  $\delta_{Cl}$  is a constant "effective" diffusion coefficient of chloride, and  $t$  is the exposure time.  $x_{corr}$  is the corrosion depth. The constant  $K$  depends on the resistivity of the concrete, the availability of oxygen, etc. It is not so much dependent of the concrete cover as such. The service life in this simple model will for a given situation only be a function of the threshold concentration and the concrete cover.

For the most studied case, chloride-induced corrosion, the ideal situation consisting of a water-saturated, uncracked concrete with homogeneous properties across the concrete cover, and exposed to constant outer chloride concentration, has been studied. Much research has been aimed at studies of the diffusion coefficient of chloride during this rather ideal condition. On basis of this model one has tried to find relations between the mix proportion, the type of concrete constituents and the service life. Many over-simplifications have been made in the past such as:

- \* A constant, more or less arbitrarily chosen, threshold concentration independent of the type of binder, the water/cement ratio, or the moisture condition in the cover. Besides, the threshold concentration has often been expressed in terms of total chloride instead of the more logical "corrosion active" chloride, or free chloride content.
- \* A constant, time-independent, outer chloride concentration also in cases where there is chloride enrichment in reality, such as in the splash zone, or when the concrete is exposed to de-icing salts. This leads to a steadily increasing inner chloride concentration and no chloride transport outwards.
- \* Linear chloride binding isotherm leading to the normal erfc-solution to Ficks law. In reality, however, there is a non-linear binding isotherm causing a chloride profile that might differ quite much from the erfc-solution; [12].
- \* Effective chloride diffusivity to be used in the service life calculations that are evaluated from the measured chloride profile of real structures or lab-specimens exposed to some sort of Cl-solution, whereby the evaluation is based on the traditional erfc-solution. This might lead to very big errors.
- \* Total chloride as the driving potential instead of the more logical potential, free chloride.
- \* Constant moisture condition (saturated) in the concrete cover also in cases where the moisture variations in reality are very big. Often, at the surface of a real structure, there is varying moisture conditions such as rain and drying cycles. Therefore, the moisture content in the cover will vary according to the principles in Fig 1. Hence, the chloride transport will occur with different rates at different times. Sometimes, even a convective transport of chloride might occur. Besides, when the moisture content is reduced, some of the physically bound - immobilized - chloride might become mobile. The rate of binding and release of chloride is important, but is normally neglected. In some dense concretes, the moisture content might be so low at a certain depth from the surface that chloride diffusion stops. Therefore, the defect-free concrete might often be immune towards chloride induced corrosion. This phenomenon is not considered in the simple diffusion approach.
- \* By not considering the moisture variation, the service life will theoretically be proportional to the concrete cover in square. Due to the probable effects of the moisture variations on the threshold concentration and on the chloride penetration, a certain increase of the cover will be much more positive than what is predicted by the square law. The relation between cover and service life ought to be studied.

- \* Constant concrete diffusivity also in cases where there is an ongoing hydration or densifying chemical effects at the surface, like in sea water. According to measurements in Norway [13] on concrete immersed for long time in sea water, there is a gradual reduction in the chloride diffusion constant with increasing exposure time. The reduction factor could be as high as 20 after 30 years of exposure. Similar observations have been made in Sweden; [14].
- \* Corrosion is the only destructive process considered. In reality there might also be frost attack or other types of attack changing the properties of the cover and thereby markedly change the service life.
- \* The concrete is defect-free; no cracks, perfect bond, no separation, no big voids.
- \* Consideration is seldom taken to effects of corrosion on the structural stability. In reality, different amount of corrosion can be accepted in different parts of the structure. Service life will therefore be different for different parts of the same structure, also in cases where the exposure and the properties of the structure are exactly the same over the entire structure.

These over-simplifications often lead to rather short predicted service lives. Therefore, they have led to the wide-spread opinion that almost every concrete structure exposed to sea water, or de-icing salt, is doomed to failure after a rather short period (maybe some decades) due to reinforcement corrosion. In reality, the "incubation time" before onset of corrosion is normally much higher than predicted, provided the concrete is of decent quality and defect-free. Therefore, it is quite evident that our rather crude predictions must be refined. We have to study the following factors in more detail :

- \* The condition for onset of corrosion primarily the critical free chloride concentration. Studies ought to be made of the effect on the threshold of the following factors: (i) the moisture and oxygen content, (ii) the chemistry of the pore solution, (iii) the steel type, (iv) defects such as voids at the steel surface, cracks in the cover, or debonding. It might be that the threshold concentration is so high that corrosion in normal environment is impossible in defect-free concrete, provided the concrete quality is decent and that the bars are located on a depth where the moisture conditions are constant. If this is the case, future corrosion research could be concentrated on the effect of defects.
- \* The true and effective diffusivity of chloride in the real environment including ageing effects. The mechanisms behind the difference between lab-diffusivity and field-diffusivity, [13], [14], ought to be explained theoretically.
- \* The relation between the moisture content and the chloride diffusivity is almost completely unknown. If one assumes that the diffusivity is directly proportional to the water content in the capillary pores one will obtain a relation between the chloride diffusivity and RH according to Fig 3, [15]. This hypothesis has to be verified, however.
- \* Convective flow in partly saturated concrete simultaneously with capillary flow. Also the effect of self-desiccation on chloride transport must be studied.
- \* The binding of chloride and the reversibility of binding including time, moisture and temperature effects should be investigated because it is needed for a detailed theoretical description of coupled moisture-chloride transport.
- \* The moisture level and variation in the real structure. Comparisons with theoretical modelling should be made. Examples of a modelling based on simplified assumptions concerning the outer moisture variations are seen in Fig 1. The simulation has been based on rather uncertain data for capillary suction. This type of moisture transport must be studied more, especially the re-distribution of moisture after terminated exposure to free water.
- \* The effects of the outer and inner moisture variation and level on the chloride profile. This requires a development of a theory for coupled chloride-moisture transport including

ding effects on the chloride diffusion. An example of such calculations including a certain time lag in binding is shown in Fig 4; [16]. A very valuable discussion of different aspects of chloride penetration is performed in the Danish HETEK project; [17].

- \* The effects of cracks and other defects on the chloride penetration. Healing effects in different types of concrete with different binders.
- \* Typical chloride levels at the concrete surface in different environments; splash zone, salt-spray zone, deicing zone.
- \* The effect of the chemical characteristics of the binder on all factors determining corrosion should be investigated - not only its effect on diffusion in saturated concrete with constant boundary conditions.
- \* Theoretical and experimental studies of the corrosion process in crack-free and cracked concrete of different composition, and with different types of binder, and different moisture condition in the cover, and different chloride content. It is not reasonable to assume that the corrosion rate is constant as is often assumed. Instead it varies with time and with changes in the outer conditions, because these factors influence the electrical potential of the bar.
- \* Development of a service life model for the initiation phase and the propagation phase considering all effects above.
- \* Systematic field studies for verifying the theoretical modelling of the initiation and propagation phases.
- \* Investigations of synergetic effects of other destruction types, or of mechanical stresses acting simultaneously with the normal processes determining corrosion.
- \* Structural analyses of the effect of corrosion on the structural stability and safety in different types of structures, and for different parts of the structure.

It is interesting to note that the defects of the "old" type of modelling are recognized more and more, and that there is a growing acceptance for the idea that more complex models are needed, and that all models require an experimental verification by field studies. Such work has started in many countries, which is clearly seen by contributions to this conference.

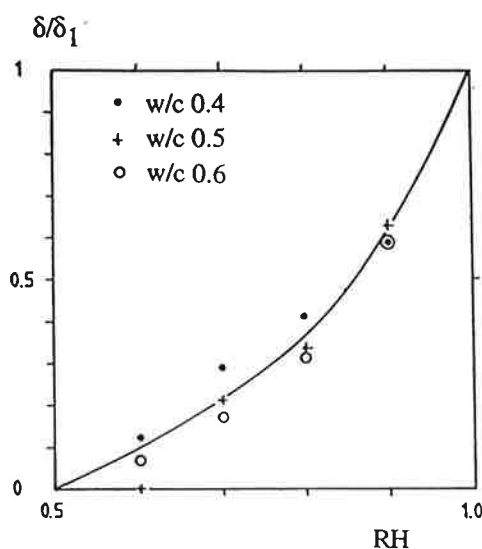


Fig 3. Assumed relation between chloride diffusivity and RH; [15]. The diffusivity is supposed to be proportional to the amount of water in the capillary pores.  $\delta_1$  is the diffusivity at saturation

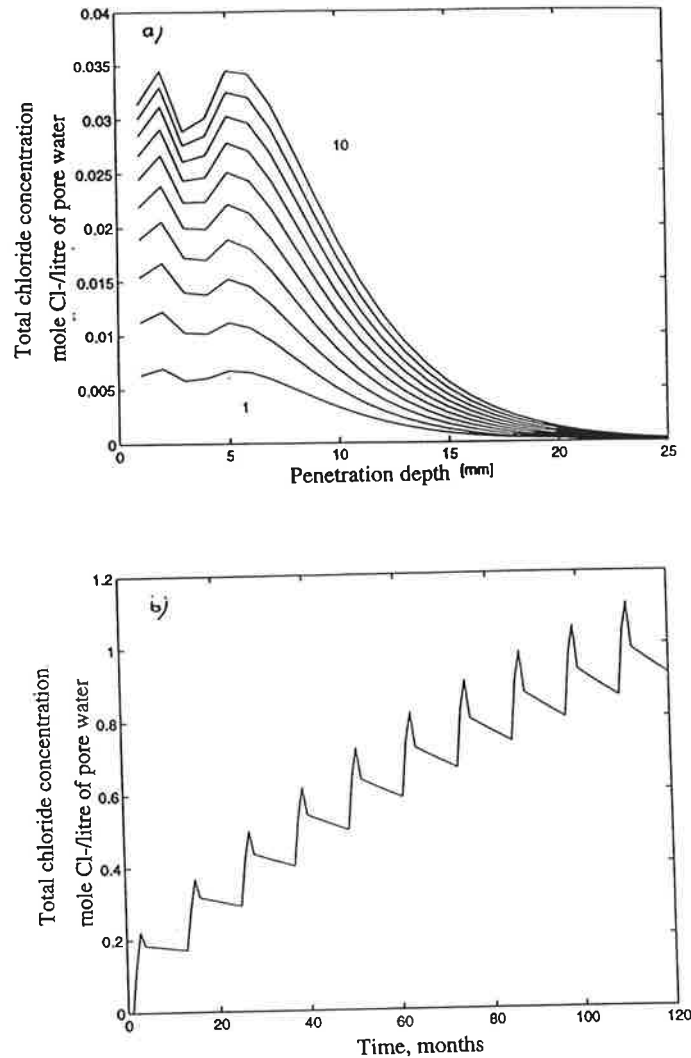


Fig 4. (a) Calculated free chloride profiles in concrete in June from year 1 to year 10. The concrete is exposed to intermittent de-icing during 5 months, and intermittent rain during 7 months; [16]. (b) The total chloride concentration on the depth 5 mm from the surface. The peaks correspond to winter conditions; [16].

#### 4.2 Reinforcement corrosion due to carbonation

Reinforcement corrosion due to carbonation is normally no problem in bridges and other advanced structures. On the other hand, carbonation releases bound chloride [1] and might therefore influence the initiation time for chloride-induced corrosion. Therefore it ought to be considered, at least for the drier part of the structure, like the super-structure of a bridge.

The carbonation process is also in most cases described as a simple diffusion process, which leads to the following expression:

$$x = \{2 \cdot \delta \cdot c_0 / C\}^{1/2} \cdot t^{1/2} \quad (3)$$

where  $x$  is the carbonation depth [m],  $\delta$  the diffusion coefficient of  $\text{CO}_2$  [ $\text{m}^2/\text{s}$ ],  $c_0$  is the outer  $\text{CO}_2$ -concentration (actually the  $\text{CO}_2$ -concentration in the pores) [ $\text{mole}/\text{m}^3$ ],  $C$  the amount of carbonationable material in the concrete [ $\text{mole}/\text{m}^3$ ], and  $t$  the exposure time [s].

The expression leads to a square-root-of-time relationship according to which corrosion will start sooner or later. There, however, are great over-simplifications in the model. The most important is that the diffusion coefficient of  $\text{CO}_2$  is supposed to be constant with time and depth from the surface. In reality, there is a wide variation of moisture contents over time, and this variation is different on different depths from the surface. According to few measurements by Tuutti [1] the diffusion of a gas is approximately directly proportional to the relative humidity; Fig 5. Therefore, due to the varying moisture contents, it is quite possible that carbonation comes to a more or less complete stop at a certain depth.

Another consequence of eq (3) is that carbonation occurs as a moving boundary. In reality, it might be more blunt depending on different reaction rates for different reacting components in the cement paste. A third simplification is that eventual counter-diffusion of alkalis towards the carbonation front is neglected. This might also lead to a stop of the carbonation front.

Thus, it seems as if the normally used simple square-root relation for carbonation is very much on the safe side. Consequently, there is need of a more nuanced prediction model.

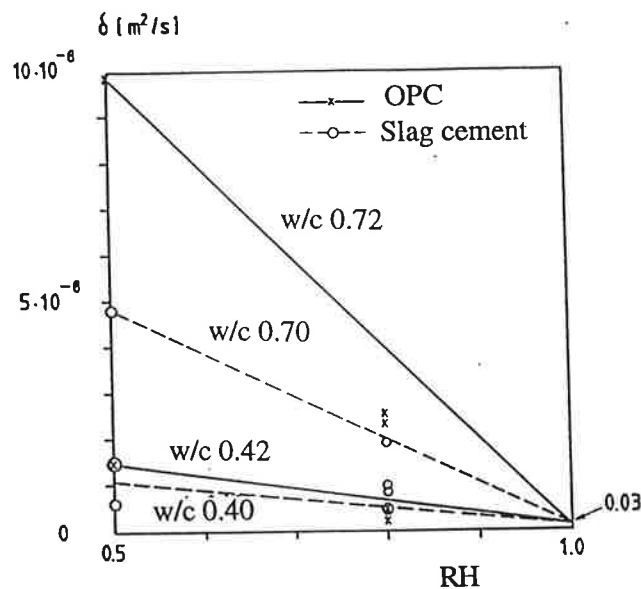


Fig 5. Relation between RH and the diffusion coefficient of oxygen; Data from [1] evaluated in [15].

### 4.3 Salt-frost scaling

Salt-frost scaling, or surface scaling, might be a comparably linear attack at which each year produces about the same amount of scaling. If so, one might, by investigation of the scaling of old structures, estimate the future rate of destruction; [18]. However, in the design phase one has no record of the behaviour of the structure in practice since it is not yet built. There are no possibilities available today of a calculation in advance of the future scaling. Therefore, one has to rely on a laboratory test of the scaling of the concrete mix to be used. The test is accelerated, and each cycle will often produce about the same amount of scaling. Assuming that the scaling is proportional to the minimum temperature in square, one might perhaps use the following expression for the scaling of the real structure:

$$S = (S_n/n) \cdot n_{\text{equiv}} \cdot t \quad (4)$$

where  $S$  is the scaling of the real structure after  $t$  years [ $\text{kg}/\text{m}^2$ ],  $S_n$  is the total scaling after  $n$  lab-cycles [ $\text{kg}/\text{m}^2$ ] and  $n_{\text{equiv}}$  is the number of "equivalent" freeze/thaw cycles in the real

structure [ $\text{year}^{-1}$ ]. By an "equivalent cycle" is meant the number of real cycles of a certain type that will produce the same scaling as one single lab-cycle. Thus:

$$n_{\text{equiv}} \approx \sum n_{\text{real}} \cdot (\theta_{\text{min}} / \theta_{\text{min,lab}})^2 \quad (5)$$

where  $n_{\text{real}}$  is the real number of freeze/thaw cycles a year with the minimum temperature  $\theta_{\text{min}}$  and  $\theta_{\text{min,lab}}$  is the minimum temperature used in the lab-cycle.

Thus, the service life can be written:

$$t_{\text{live}} = \{(\gamma \cdot n) / S_n \cdot n_{\text{equiv}}\} \cdot d_{\text{max}} \quad (6)$$

where  $\gamma$  is the density of the scaled material [ $\approx 2000 \text{ kg/m}^2$ ], and  $d_{\text{max}}$  is the maximum allowable scaling depth (e.g. based on aesthetic considerations, or on the risk of reinforcement corrosion.)

There are no experimental verifications of this equation. Therefore, there is an urgent need of obtaining a better and more scientifically based relation between a lab scaling test and reality. Such research has now started in Sweden in the so-called BTB-project (or, DCD, Durability of Concrete exposed to De-icing salts)

Another problem is that the translation between the result of a lab-scaling test and scaling of the real structure is to a great extent a function of the type of cement and of the manner in which the surface is treated before the frost test starts. It seems as if concrete with slag cement behaves fairly well in a lab-test but worse in the real structure. The reason might be that the surface properties are different in the two cases. This is a further indication that we have to find the basic mechanism behind salt-scaling.

Surface scaling is also very important for the service life with regard to reinforcement corrosion; see Fig 2.

#### 4.4 Internal frost attack

For internal frost attack, there is a service life theory, which is based on the critical degree of saturation hypothesis; [5]. Frost-damage will only occur when a critical moisture content is transgressed over a concrete volume that is big enough to produce observable destruction. Thus, the service life problem is reduced to the problem of calculating the moisture level in the concrete. The frost-resistance problem is therefore - as many other destruction types - to a great extent a moisture mechanics problem.

The critical moisture content can be determined by rather simple lab-tests, [19]. There are also possibilities to calculate the value theoretically on basis of the so called critical spacing factor, and the air-pore-size distribution curve; [5].

The real moisture content cannot be calculated theoretically because the critical moisture level will almost always occur above the hygroscopic or capillary range. Normally, frost-damage requires that some of the air-pores are water-filled, which will only occur due to other process than normal water diffusion, or suction caused by gradients in the water vapour concentration or capillary pressure. Such a mechanism is a dissolution-of-air and replacement-by-water process in which only the air-pores take part; [5].

One possibility to obtain some information about the future water content in the concrete is to measure the long-term absorption in the air-pore structure, and then extrapolate the absorption curve until the water content equals the critical water content. Theoretical analyses and expe-

rimental results indicate that the long-term water uptake can be described by an equation of the following type:

$$S(t) = A + B \cdot t^C \quad (7)$$

where  $S(t)$  is the degree of saturation after time  $t$  of uninterrupted water uptake, and  $A$ ,  $B$  and  $C$  are coefficients determined by the air content of the concrete (coefficient  $A$ ), by the diffusivity of dissolved air in pore water (coefficient  $B$ ), and the air-pore distribution (coefficient  $C$ ). All coefficients are evaluated from short-term water absorption experiments. Thereafter, a sort of potential service life can be defined by:

$$t_{\text{life}} = \{(S_{\text{cr}} - A)/B\}^{1/C} \quad (8)$$

where  $S_{\text{cr}}$  is the critical degree of saturation. By theoretical calculations of the effect of the water absorption on the critical degree of saturation one can theoretically calculate the required air content for a given service life; [20]. Such calculations indicate that it is often better to have an air-pore structure that is not too fine-porous in cases where the concrete is used in a very moist environment. The service life model described has to be verified by experiments and field studies. It is essential that water uptake before and during freeze/thaw is clarified.

Another important issue is the internal frost resistance of dense concrete, so-called HPC. One might assume that such a concrete is automatically frost-resistant due to the very low amount of freezable water. Besides, such concrete is often self-desiccated to a level where any freezing of water is impossible at normal outdoor temperatures. Nevertheless, by experiments it has been shown that severe salt scaling and internal damage might occur in concrete with water/binder ratios as low as 0.25 or even lower; [21], [22]. It can be shown theoretically that very little water is required for frost damage to occur; perhaps only 5 to 10 litres per cubic meter; [23]. It is not unreasonable to believe that such a low freezable water content appears in an aged HPC that has been exposed to free water for a very long time.

It is important to note that the model predicts that a concrete can shift from frost-resistant to non-frost-resistant if the moisture content is somewhat increased. This might happen when a concrete with a low potential service life is repaired by a material that changes its moisture level for example at the interface between the old concrete and the repair; see paragraph 6.

It must also be noted that severe frost damage will occur after rather few freeze/thaw cycles if the concrete is more than critically saturated. Therefore, in contrast to what many think, there is no real fatigue involved in frost damage. This means that a concrete with a low degree of frost-resistance runs the risk to be frost-damaged quite early in its life; viz. the destruction curve is not continuous, as in corrosion, but it makes a jump from undamaged to damaged when the critical moisture content is transgressed in connection with freezing temperatures. This means that a concrete is either frost-resistant, meaning that it will never be frost-damaged, or non-frost-resistant meaning that it might be frost-damaged already during the first winter season.

#### 4.5 Acid attack

Acid attack is important for structures exposed to acid ground water. Also moderately acid water will cause a surface erosion that could sometimes not be neglected.

As said above, there have been some attempts to find service life models for acid attack. There are many cases to consider; (i) the acid transport is rate-determining; (ii) the transport of dissolved material is rate-determining; (iii) stepwise erosive attack where the partly dissolved surface layer is eroded when dissolution has reached a certain stage; (iv) continuous erosive attack where erosion is going on with constant rate from time zero; (v) non-erosive attack, but a gradual dissolution of the partly dissolved layer; [2]. Only a few cases are treated below:

a) *Nonerosive attack. Diffusion of acid is rate-determining*

The depth of erosion is:

$$x = \{(m/a) \cdot (2 \cdot \delta_a \cdot c_0 / M)\}^{1/2} \cdot t^{1/2} = \text{const} \cdot t^{1/2} \quad (9)$$

where  $m$  is the number of moles of concrete and  $a$  the number of moles of acid participating in the reaction.  $\delta_a$  is the transport coefficient of the acid through the partly dissolved layer [ $\text{m}^2/\text{s}$ ],  $c_0$  is the outer concentration of acid [ $\text{mole}/\text{m}^3$ ], and  $M$  is the amount of dissolvable material [ $\text{mole}/\text{m}^3$ ]. The theory leads to a moving boundary where the thickness of the dissolved zone increases by the square-root of time.

b) *Continuous erosive attack. Diffusion of acid is rate-determining*

Erosion of the dissolved layer might be stepwise where the layer is falling off when the dissolution has reached a certain depth. Erosion might also be continuous, so that it occurs with a constant rate. Only the last condition is considered. Erosion is described by:

$$x_e = k \cdot t \quad (10)$$

where  $k$  is the constant erosion rate [ $\text{m}/\text{s}$ ]

The total destruction depth is:

$$x = x_0 + k \cdot (t - t_0) \quad (11)$$

where  $t_0$  is the exposure time when the erosion rate is the same as the dissolution rate.  $x_0$  is the corresponding destruction depth.

$$x_0 = (m/a) \cdot \{(\delta_a \cdot c_0) / (M \cdot k)\} \quad (12)$$

$$t_0 = (m/a) \cdot \{\delta_a \cdot c_0 / (2 \cdot M \cdot k^2)\} \quad (13)$$

Attack of this type is proportional to the square-root of time during the first time (years), but will then gradually approach a linear attack. Thus, erosive attack is more dangerous than non-erosive since the rate will not diminish with time.

c) *non-erosive attack, but a gradual dissolution of the partly dissolved layer*

Each acid will only dissolve certain components of the concrete. Other components remain as residues in a partly dissolved layer. This, however, might be slowly dissolved at the outer surface; [2]. Therefore, after a long time, it might be that the attack is not reduced according to the square-root law but becomes linear.

Similar expressions as those given above are also obtained when the diffusion of dissolved material is the rate-determining factor; [15].

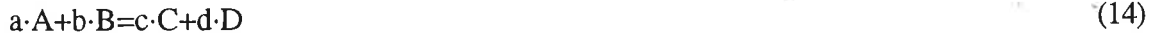
The models ought to be tested for different types of concrete, containing different types of binder. Different acids will behave differently, and mixtures of acids might behave differently from the pure acids of the same concentration. The rate-determining coefficients ought to be determined for frequently occurring cases. It is also important to investigate the conditions for erosive attack, and the slow dissolution of the residual layer. If erosion, or slow dissolution occurs, it means that the attack will sooner or later change from a square-root process to a linear. If this happens, a square-root extrapolation from a lab-test will under-estimate the future degradation, and over-estimate the service life.

#### 4.6 Internal chemical attack

There are few, if any, attempts to derive service life models for internal chemical attack, such as ASR, or sulphate attack. An attempt, however, has been made in [15]. There are different cases to consider, such as; (i) abundant amount of aggressive medium, internal reaction rate-determining; (ii) limited amount of reactants; (3) diffusion into the concrete rate-determining, immediate reaction; (iv) ditto, but time-dependent reaction.

##### a) Limited amount of reactants

The destructive reaction is:



where a, b, c and d are the number of moles of the substances A, B, C, and D. The components A and B reacts to create the destructive components C and D.

The reaction as function of time can be calculated by:

$$\frac{1}{(b/a) \cdot [A]_0 - [B]_0} \cdot \ln \left\{ \frac{[B]_0 \cdot ([A]_0 - [x])}{[A]_0 \cdot ([B]_0 - (b/a) \cdot [x])} \right\} = K \cdot t \quad (15)$$

where  $[A]_0$  and  $[B]_0$  are the initial concentrations of components A and B [e.g.  $C_3A$  and sulphate],  $[x]$  is the concentration of A that has reacted at time t. K is a rate determining constant that has a temperature dependency that can be described by the Arrhenius equation.

If the destruction is supposed to be linear - i.e. for each mole of reacted A, a certain constant volume change of the concrete is caused. It can be described by the following formula:

$$\Delta V/V_0 = \text{const} \cdot [X] \quad (16)$$

where  $[X]$  is the number of moles of A that has reacted with B in order to create the expansive components C and D.

This model could perhaps be used for many types of internal attack, provided the rate determining constant is known. The model predicts some characteristics of chemical attack that can also be observed in practise:

- \* It predicts a certain maximum possible damage. The extent of this depends on the amount of A and/or B. Such maximum damage levels have for instance been observed for ASR, see Fig 6; [24]. Probably the same is valid for delayed ettringite formation.
- \* At low amounts of A or B the reaction product is not sufficiently big to cause damage. This is also well-known from ASR.

##### b) Diffusion into the concrete is rate-determining. Immediate reaction

In this case we will obtain a moving boundary with an external, totally reacted, layer, and an inner, totally unaffected, material

$$x = \{(a/b) \cdot (2 \cdot \delta_A \cdot [A]_0 / M)\}^{1/2} \cdot t^{1/2} \quad (17)$$

where  $\delta_A$  is the transport coefficient of the aggressive substance A; e.g. sulphate.

The approach clearly indicates the importance of concrete permeability in internal chemical attack. It is also well-known that this is not only determined by the concentrations as described by eq (15) but also by the permeability.

c) *Diffusion into the concrete is rate-determining. Time-dependent reaction*

This is a more complicated case. The following differential equation describing the process can often be used; [25]:

$$d[A]/dt = \delta_a \cdot \{d^2[A]/dx^2\} - K \cdot [A] \quad (18)$$

where  $K$  is the reaction rate. The solution to this equation is rather complicated. It can be found in [25].

Physio-chemical service life models of this kind ought to be developed further and be verified by experiments in order to make possible quantitative service life assessments for chemical attack.

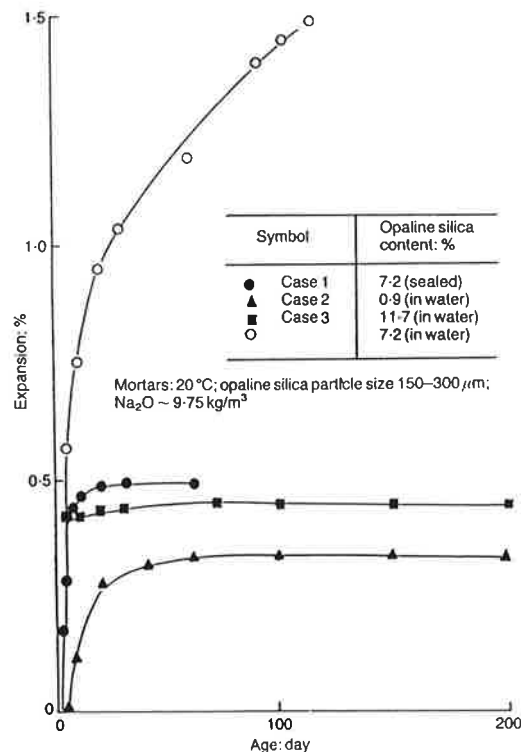


Fig 6. ASR as an example of a chemical reaction that reaches a limit; [24].

## 5. Repair of concrete in general

In paragraph 2, the basic principles were outlined for a methodology of using the service life concept also in connection with decisions concerning, (i) if to repair, and (ii) how to repair, a damaged structure. The repair strategy should be based on an assessment of the structural stability at the time of inspection (which is supposed to be the same as the time of decision concerning the repair strategy), but also on an assessment of the future change of the structural stability if no repair was made. Of course, one should also try to design the repair in such a way that the initial structural stability is restored to a sufficiently high degree. Besides, one should try to assess the service life of the repaired structure, which is a difficult task since it

involves not only the old concrete, but also the repair material and the interface between the old and new material. Since structural stability is involved it is important that the structural engineer and the materials engineer cooperate. In this paragraph some more general remarks concerning repair will be made.

The assessment of the actual destruction rate at the time of inspection (time of repair) can be based on direct observations of what has happened before. Therefore, one is on a much safer ground than in the design phase of a new structure. From data of the residual strength, and the corrosion depth in different parts of the structure, the structural engineer could estimate the risk of failure and the residual load-carrying capacity.

The assessment of the future deterioration rate of the unrepaired structure could be based on detailed observations of the structure. From these, one could obtain relevant material data, such as transport coefficients, to put in the normally used service life formulas. Even here, the structural engineer has a natural role.

Methods for such assessments of existing damaged structures have been studied in [7]. Three types of damage were investigated; frost attack, ASR and reinforcement corrosion.

Before a decision is made about how to repair, one has to make an analysis of the functional requirements of the repair. Let us consider a structure that is to be repaired by applying a new protective material on the surface. The requirements must be coupled to service life; i.e. the requirements must express the fact that there will be a gradual decay of the repaired structure. The purely static requirements coupled to the load-carrying capacity, such as a minimum bond strength between the old concrete and the new material are not considered explicitly below. These factors are of course also fundamental. Normally, they are fulfilled automatically if the requirements described below are fulfilled.

- 1: The repair shall protect the old concrete from frost damage. One could often assume that the old concrete is of rather low quality. Therefore, one might assume that the concrete is close to critical saturation even before it was repaired. Perhaps it was already frost-damaged. Therefore, it is important that the repair reduces the water inflow into the structure to such low levels that no frost-damage occurs. This is an extremely hard requirement, which is very difficult to fulfil. As will be shown below, a surface repair will often increase the risk of frost damage, especially when dense repair materials are used.
- 2: The repair shall protect the reinforcement in the old concrete from corrosion within the required service life. This means that it should reduce, or stop, carbonation and chloride ingress. It must be observed that a repair will often change the moisture level in the old concrete. It might be decreased, in which case the carbonation rate is increased. In other cases it might be increased, in which case the rate of chloride ingress is enhanced. In order to fulfil the requirement, the repair material ought to be "reactive" by being chloride absorbing and  $\text{CO}_2$ -absorbing. Thus, polymeric repair materials are less effective than cement based materials.

The retarding effect of a non-reactive coating on the penetration rate of  $\text{CO}_2$  or chloride might be very big. A reduction by a factor of 5 to 10 might occur; see [15]. The problem is that this effect is only existing as long as the coating is intact. Many polymeric coatings will not survive a long-term contact with the highly alkaline concrete.

- 3: The repair material shall be durable in itself, which means that it should have a service life that is at least as long as that required for the structure. If it is not durable, it will hardly fulfil its protective action (or its ability to take the required mechanical load). This means that the repair material shall have a high frost resistance, and it shall not undergo negative ageing, embrittlement etc during the entire required service life.
- 4: The repair material shall not cause any negative effects on the old structure. Such a negative effect might be that different thermal movements, or moisture movements, between the

repair and the concrete induces too big mechanical stresses in the concrete, or that the pore structure of the repair is such that the interface becomes frost sensitive.

## 6. Examples of durability problems with different types of repair

Some studies of the potential durability of different repair systems were made in Sweden during the seventies in conjunction with durability problems with concrete balconies; [26], [27], [28]. Some information from this study will be given. It has not been published outside Sweden.

In a field study where many structures were investigated a few years after repair, it was found that almost all "cosmetic" repairs made with different types of thin and dense polymeric coatings used directly on the frost damaged concrete, or on concrete with ongoing corrosion, were more or less a total failure already within a few years of exposure. Corrosion continued and so did frost attack. The latter was often increased after repair. Repair made with air-entrained concrete of high quality, or shotcrete, behaved much better. At least, they were not frost damaged.

A rather big laboratory study was made on the frost resistance of concrete repaired by different polymeric materials and with concrete of different type; [28]. The two "old" concretes, used as sub-base material, was of low quality with w/c-ratios of about 0.85 and 0.75. Two different freeze-test techniques were used; one "sealed", using specimens that had sucked water for a long time before sealing, and one "open" where the specimens were exposed to simulated rain during the thawing phase of the cycle. The results were rather unambiguous; thin and dense polymeric coatings were very destructive to the sub-base concrete - the two concretes were much more frost damaged than when they were frozen without any coating. Thick polymeric mortars behaved considerably better, and air-entrained concrete did not harm the sub-base concrete at all. Latex-modified cement mortar, however, gave fairly negative results. Some results are shown in Fig 7.

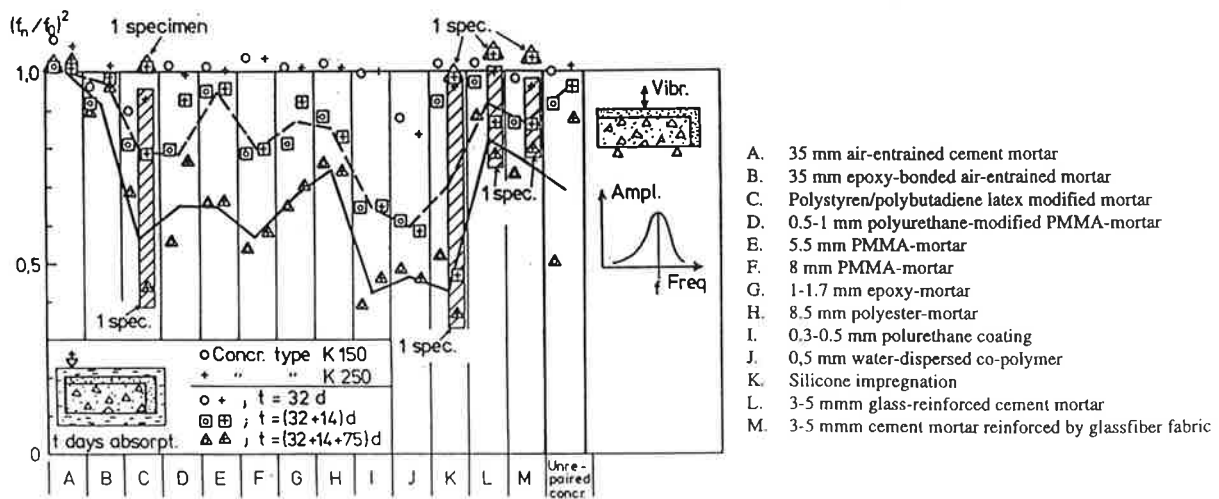


Fig 7. The relative fundamental frequency of repaired beams before and after freeze/thaw tests. Different repair systems on two sub-base concretes (K150 with w/c 0.85 and K250 with w/c 0.75). Before freezing that occurred in sealed condition the beams were stored for 32 days, or 46 days, or 121 days in water; [28]. Two unrepaired reference concretes of higher quality (K300 and K400) were also tested.

The reason behind this negative behaviour of thin coatings was not clarified. A hypothesis was put forward; see Fig 8. During the freezing phase, there is a temperature gradient across the coating causing moisture flow towards the coating. The thinner the coating, the more steep the temperature profile. Principally, moisture transport is proportional to the gradient in temperature. Therefore, a more rapid moisture redistribution, from the interior of the concrete towards the coating, occurs at a thin coating than at a thick repair. Thus, it is quite possible that critical saturation is reached at the interface to the coating. Expelled moisture from places where pore water freezes has no place to go because the concrete is saturated and the coating is non-porous. Therefore, frost-damage occurs. With a thick coating, or a repair made with concrete, the moisture redistribution is more slow. Besides, if the repair is porous like air-entrained concrete, water at the interface has space to freeze without causing damage.

A field study was made on the moisture content in balcony slabs covered by different coatings. It was never found in any slab that a dense polymeric coating reduced the moisture content in the concrete. On the contrary, it was found that the RH-level at the interface was more constant and at a higher level when the coating was dense than when concrete was used as repair material. Therefore, it is safe to conclude that the moisture content in an outdoor structure will seldom be reduced behind a dense coating. It will remain on a high level causing rapid corrosion and high risk of frost-damage.

Almost all experience also points to that a damaged concrete shall be repaired by a durable concrete of high quality and not by impermeable, non-porous organic coatings. If such a coating must be used it shall probably be thick, like a layer of asphalt or another thick polymer mortar with a very high durability in the alkaline environment.

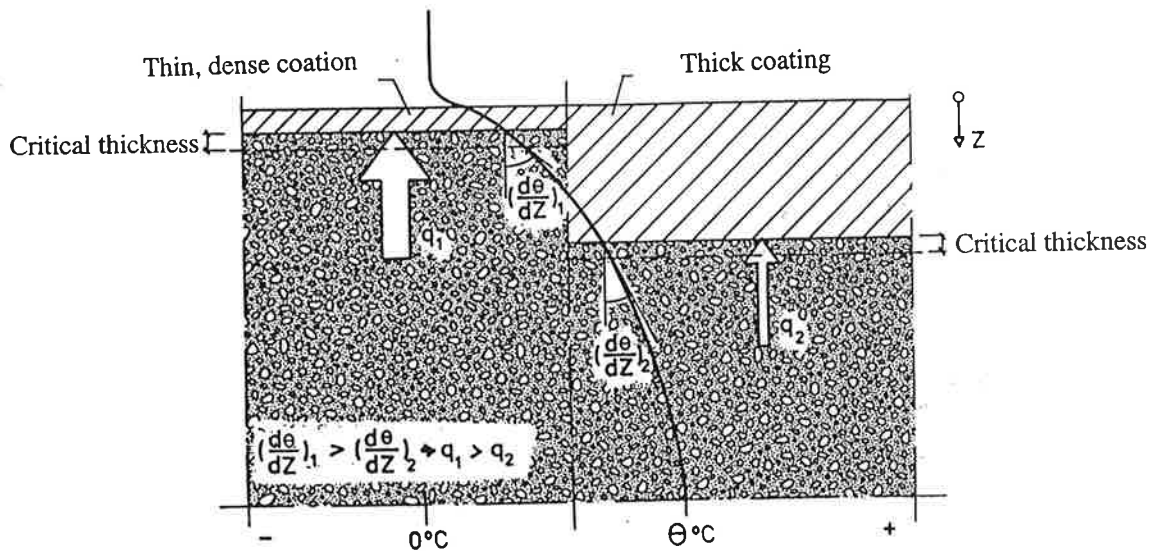


Fig 8. Principles for moisture redistribution behind a coated concrete in conjunction with freezing; [28].

## 7. Repair of concrete. Research needs

The prediction of the long-term function of a repaired concrete structure is a difficult problem. There are many unknown factors to consider, such as:

- 1: The durability (service life) of the bond between the repair material and the old concrete.
- 2: The moisture mechanics interaction between the new and the old material. Equilibrium moisture contents at the interface, and moisture transfer over the interface on the micro-scale in connection with freezing, etc.

- 3: The frost resistance of the "package", repair material and concrete. This is closely linked to problem 2:
- 4: The eventual chloride reducing effect in the old cover caused by an initially chloride-free new surface layer based on cement mortar or concrete.

There are also big problems connected with the decision situation when one shall decide about if and how to repair. We need much more information of the residual strength of a structure damaged by frost, corrosion or chemical attack. So for instance, there are no studies made of the load-carrying capacity of a frost-damaged concrete. Therefore, it is not so easy to predict the future reduction in the structural stability under the condition that no repair is made. This makes a well-founded rational decision concerning the repair strategy impossible. The consequence is that structures with fair structural stability will be repaired just because they look dangerous, not because they are dangerous.

## 8. Moisture mechanics and durability

In this paper, there are many examples showing that moisture is a basic factor in durability. It determines almost all forms of transport of harmful substances. It probably to a large extent influences the threshold concentration of chloride. It determines the electrical resistivity and therefore the corrosion rate. It certainly determines the risk of frost-damage, both of the unrepaired and of the repaired structure. There are many forms of moisture transport involved; (i) diffusion, (ii) liquid transport due to "suction", or under-pressure, in the pore water, (iii) liquid transport due to over-pressure, (iv) transport due to suction caused by dissolution of air initially enclosed in air-pores, (v) suction caused by ice formation. Therefore, it is very important that moisture transport and moisture fixation is studied in connection with durability research. Such studies are going on in Sweden, in Norway, and perhaps also elsewhere.

## 9. Stochastic methods

Traditionally, service life models are of a deterministic character where each property is supposed to have a certain assumed well-defined value. It is important that well-founded stochastic models are developed, so that the relative effect on the service life of different environmental factors and material parameters can be evaluated. Methods have been developed - e.g. [29], [30] -, but unfortunately such methods are in most cases more of a "philosophical nature" than operative. They are often based on models for the destruction process that are too simplified. By good stochastic methods, using the real measured variations in the different properties involved in the destruction model, we will have access to a much better tool for service life predictions. We will also, by using such methods, find out what factors are most important for a good service life design and what factors can be omitted.

It is also important that stochastic methods are developed and applied, not only to reinforcement corrosion, as is the case today, but also to other types of attack.

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