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APPROACHES FOR MODELLING THE RESIDUAL SERVICE LIFE OF MARINE CONCRETE STRUCTURES AFTER REPAIR

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Abstract

This work provides an introduction to the methodology of the service life design of reinforced concrete structures in a marine environment after application of a concrete repair measure. A simplified mathematical model of chloride diffusion in a two-layer system is presented. Preliminary numerical calculations demonstrate the effect of various conditions on the residual service life. First studies on chloride diffusion in a two-layer system have been conducted using the Finite Element Method. Results of a long-term exposure test are presented.

The content and gradient of residual chlorides along with the thickness and the chloride ingress resistances of both remaining and new layers of cover will determine the residual service life of repaired structures.

Key Words: Concrete, Chloride, Corrosion, Repair, Diffusion, 2-Layer System, Service Life, Long-Term Exposure Test, Modelling
1. INTRODUCTION

In a marine environment chloride induced corrosion is the major problem with respect to the durability of reinforced concrete structures. Reinforcement corrosion is initiated when a critical chloride content at the surface of the rebars is reached which can disrupt their protective passive layer. In most cases, this initiation time defines the service life of structures, since the subsequent deterioration period is, by comparison, negligibly short. A traditional and feasible repair method is to remove the chloride contaminated concrete layer and replace the removed volume by cement-based repair mortars or concretes to repassivate the rebars.

In recent years probabilistic service life design models have been developed for new structures, using time-dependent models to describe the chloride ingress into concrete. Besides empirical models, Fick’s second law of diffusion is also applied with some modifications as diffusion is the most active mechanism in chloride transport in concrete. For repaired structures, the transport model first has to be adapted to a two-layer model considering both the layer of repair material and the remaining concrete layer as well as their interface. Where chloride ions are present in the remaining concrete layer, a possible back-diffusion into the repair phase as well as further diffusion towards the inside of the structure both have to be considered.

2. SERVICE LIFE DESIGN OF (NEW) CONCRETE STRUCTURES DUE TO CHLORIDE-INDUCED CORROSION

To satisfy the durability of concrete structures one has to respect deemed-to-satisfy provisions which are based on application experience. These rules are given in current standards and guidelines. By contrast to this prescriptive methodology of standards, performance-based probabilistic design models are being developed. The “fib Model Code for Service Life Design” [1] presents a full probabilistic concept for the service life design of uncracked concrete structures for the case of chloride induced corrosion. The concept is based on the following:

- definition of limit states
- models which are able to describe the time-dependent transport and deterioration mechanisms
- statistical definition of actions
- measurement and statistical quantification of building component resistances
- assumed permissible failure probabilities

The limit state is determined by the initiation of reinforcement corrosion and reached when a critical chloride content is exceeded at the rebar surface, disrupting the passive layer protecting the rebar. At the end of the initiation period, no corrosion-induced deterioration will have yet occurred in the structure. In marine environments generally, high corrosion rates are observed so that the deterioration period is comparatively short. Moreover, there is as yet no commonly used model to describe the mechanism of reinforcement corrosion and its effect on the structures. Therefore the deterioration period is neglected in this concept and the service life is considered as the initiation period.

To assess the initiation period, a mathematical model, equation (1), is used to estimate the time- and depth-dependent chloride concentration, C(x,t), in the concrete, based on the error
function solution of Fick’s second law of diffusion in the one-dimensional form. This diffusion-controlled assessment of chloride ingress is only a simplification, since other transport mechanisms are involved in the chloride ingress in concrete, in particular, capillary absorption and permeation. However, diffusion is the most active and important mechanism related to the initiation of corrosion in concrete structures.

\[ C(x, t) = C_0 + (C_{s,\Delta x} - C_0) \cdot \left[ 1 - \text{erf} \left( \frac{x - \Delta x}{2 \cdot \sqrt{D_{\text{app,C}}(t) \cdot t}} \right) \right] \]  

\[ C_0 \] is the initial chloride content of the concrete [wt.-%/c]; \( \Delta x \) is the depth of the convection zone (concrete layer, up to which the process of chloride penetration differs from Fick’s law due to the frequent wetting and drying in the tidal and splash zones) [mm]; \( C_{s,\Delta x} \) is the chloride content at the depth \( \Delta x \) and a certain point of time [wt.-%/c]. Although \( C_{s,\Delta x} \) theoretically is a time-dependent variable, it will be considered as time-independent in order to fulfill the basic condition to solve the differential equation of Fick’s second law of diffusion. The apparent chloride diffusion coefficient, \( D_{\text{app,C}}(t) \) [m²/s] is time-dependent and is to be determined by means of equation (2):

\[ D_{\text{app,C}}(t) = \exp \left( b \left( \frac{1}{T_{\text{ref}}} - \frac{1}{T_{\text{real}}} \right) \right) \cdot D_{\text{RCM,0}} \cdot k_t \cdot \left( \frac{t_0}{t} \right)^\alpha \]  

where \( b_e \) is the temperature coefficient [K]; \( T_{\text{ref}} \) is the reference temperature (for laboratory tests) [K]; \( T_{\text{real}} \) is the temperature of the structural element or the ambient air [K]; \( D_{\text{RCM,0}} \) is the chloride migration coefficient [m²/s]; \( k_t \) is the transfer parameter with a constant value of 1 [-]; \( t_0 \) is a reference point of time [s] and \( \alpha \) is the ageing exponent [-].

\( D_{\text{RCM,0}} \) and the ageing exponent, \( \alpha \), are the governing parameters for the description of the material properties. While \( D_{\text{RCM,0}} \) is simply determined by Rapid Chloride Migration (RCM) test methods [2, 3], the ageing exponent, which operates the decrease of \( D_{\text{app,C}}(t) \) over time, is both material- and exposure-dependent and cannot be determined by laboratory tests alone. To determine the ageing factor the long-term behavior of \( D_{\text{app,C}}(t) \) of existing structures has to be considered by processing the chloride profiles over time. Gehlen [4] quantified the statistical distribution of the ageing exponent of three different types of cement, considering both chloride profiling data of several existing structures and the RCM test results at the reference time (28 days), \( D_{\text{RCM,0}} \).

The limit state equation is obtained by comparing the calculated chloride content in the concrete cover (position of rebars) at the time \( t \), \( C(x=a, t) \), and the critical chloride content, \( C_{\text{crit}} \): 

\[ g(C_{\text{crit}}, C(x=a, t)) = C_{\text{crit}} - C(x=a, t) < 0 \]  

Although only the free chloride ions that have dissolved in the pore solution of the concrete result in depassivation of the reinforcement, the overall chloride content is included in the model as it is difficult to determine the free chloride content in the concrete. The critical chloride content, \( C_{\text{crit}} \), is to be quantified statistically and inserted into the model. In [1], some stochastic values for the input parameters are recommended (e.g., figure 1, right). A reliability analysis is performed using the limit state equation (3) and by specifying a minimum
reliability index, $\beta$. There is a correlation between the reliability index, $\beta$, and the probability of failure, $P_f$. In EN 1990 [5], ISO 2394 [14] and [15] there are recommendations for reliability index for serviceability limit states in dependence of the necessary expenses to risk minimization from $\beta = 0$ to $\beta = 2.3$. The diagram in figure 1 is an example of a reliability analysis used to predict the service life of a structural element. A target reliability index of $\beta = 1.5$ ($P_f = 6.7\%$) is expected after a 25 years’ period of exposure to severe tidal conditions (common marine structural element, CEM III/B, w/c = 0.50, a = 60 mm). A lower target reliability index of $\beta = 1.0$ ($P_f = 16\%$) is expected at an age of 55 years. The input parameters are illustrated in figure 1, right.

![Figure 1: Development of reliability index and probability of corrosion over time of a marine structural element (left), the input parameters (right)](image)

<table>
<thead>
<tr>
<th>parameter</th>
<th>distribution &amp; values</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{RCM}$ $[10^{-12}m^2/s]$</td>
<td>ND (2.8 / 0.56)</td>
</tr>
<tr>
<td>$t_0$ [s]</td>
<td>constant 2,419,200 (28 d)</td>
</tr>
<tr>
<td>ageing factor $\alpha$ [-]</td>
<td>BetaD (0.45/0.20/0/1)</td>
</tr>
<tr>
<td>$T_{ref}$ [K]</td>
<td>constant 273</td>
</tr>
<tr>
<td>$T_{real}$ [K]</td>
<td>ND (283 / 7)</td>
</tr>
<tr>
<td>$b_x$ [K]</td>
<td>ND (4800/700)</td>
</tr>
<tr>
<td>$a$ [mm]</td>
<td>ND (60/6)</td>
</tr>
<tr>
<td>$C_{\Delta,s}$ [wt.-%/c]</td>
<td>BetaD (8.9/5.6/0/50)</td>
</tr>
<tr>
<td>$\Delta x$ [mm]</td>
<td>BetaD (0.60/0.15/0.2/2)</td>
</tr>
</tbody>
</table>

This model is applied during the design stage of new structures by inserting input parameters of the material properties, of the cover depth and of actions into the equation (3).

The model can also be used to assess the durability of existing structures and to predict their residual service life by determining the current material properties and the actual actions statistically and inserting them into the model. However, some modifications are required for modelling the residual service life design after a repair measure, e.g. application of repair mortars, since the chloride ingress model first has to be adapted to a two-layer model considering both the repair and remaining concrete layers and their interface. This subject is dealt with in the next chapter.

3. MATHEMATICAL DESCRIPTION OF CHLORIDE PENETRATION IN A 2-LAYER SYSTEM

Three different cases need to be considered when carrying out repairs which involve replacing sections of the concrete with a repair mortar or concrete:

**Case no. 1:** The concrete cover is removed entirely and replaced with a repair material. The remaining layer of concrete behind the reinforcement is not affected by chloride ions.

In this case, the design of the service life of a structure with regard to chloride-induced corrosion of the reinforcement is based on a common 1-layer system. The design model described in section 2 can be used to determine the chloride penetration into the concrete and
to calculate the residual service life of a structural element, stating the characteristics of the repair material.

Case no. 2: The concrete cover is only partially removed and replaced with a repair material. The remaining layer of concrete in the cover and behind the reinforcement is not affected by chloride ions, see figure 2, left.

In this case, the concrete cover comprises two layers with different material characteristics; a new layer (new) and the remaining layer (remain). The initiation period depends on the chloride penetration behavior of the two layers. The mathematical modelling of the penetration behavior of chloride ions in such a 2-layer system can be determined as follows using the diffusion equations developed by Carslaw & Jaeger [6]:

\[
C_{\text{new}}(x,t) = C_{s,0} \sum_{n=0}^{\infty} \alpha^n \left\{ \text{erfc} \left( \frac{(2n+1) \cdot a_{\text{new}} + x}{2 \sqrt{D_{\text{app,new}}(t)} \cdot t} \right) - \alpha \cdot \text{erfc} \left( \frac{(2n+1) \cdot a_{\text{new}} - x}{2 \sqrt{D_{\text{app,new}} \cdot t}} \right) \right\}
\]

\[
C_{\text{remain}}(x,t) = \frac{2k \cdot C_{s,0}}{k+1} \sum_{n=0}^{\infty} \alpha^n \cdot \text{erfc} \left( \frac{(2n+1) \cdot a_{\text{new}} + k \cdot x}{2 \sqrt{D_{\text{app,new}} \cdot t}} \right)
\]

with:

\[
k = \frac{D_{\text{app,new}}}{D_{\text{app,remain}}}
\]

and

\[
\alpha = \frac{1-k}{1+k}
\]

where \(\text{erfc}(x)\) is the complementary error function (1-\(\text{erf}(x)\)). The chloride ion concentration in the layer of repair material, \(C_{\text{new}}(x,t)\), is described using equation (4) and that of the remaining layer of concrete, \(C_{\text{remain}}(x,t)\), using equation (5). The boundary conditions for the new layer are a constant surface chloride concentration, \((C_{s,0} = \text{const.})\), and an equilibrium concentration at the interface, \(C_{\text{new}}(x=0,t) = C_{\text{remain}}(x=0,t)\). Figure 2, right, shows a typical graph for the development of the chloride concentration in a 2-layer system.

![Figure 2: Chloride penetration in a repaired concrete element – 2-layer system](image)

Any interfacial resistance at the repair material / concrete layer is disregarded here. The interfacial resistance can be caused by the incoherence of the pore structure of the two
materials (pore blocking) and by the large proportion of impermeable aggregates in a layer. In this case, the boundary condition of the equilibrium concentration at the interface is not satisfied in the above approach.

**Case no. 3:** The concrete cover is only partially removed and replaced with repair material. The remaining layer of concrete contains (residual) chlorides. The same situation occurs when the contaminated concrete cover is left in place and topped with a layer of repair material.

In this case, the result is also a 2-layer system except that, by contrast with case no. 2, the residual chlorides are redistributed in the new layer and within the structural element in addition to chloride ingress, see figure 3. The redistribution of the residual chlorides cannot be described mathematically by the error function of Fick’s second law of diffusion as the required boundary condition of a constant concentration at the phase boundary, \( \partial C_s / \partial t = 0 \), no longer applies. The diffusion process can be described by means of equation (9) if the concentration at the phase boundary varies at a known rate of concentration change (eq. (8)) [6].

\[
C_s = C_0 - kt \quad (8)
\]

\[
C(x, t) = C_0 \text{erfc} \frac{x}{2\sqrt{Dt}} - 4kt \text{i} \text{erfc} \frac{x}{2\sqrt{Dt}} \quad (9)
\]

where \( i \) is the imaginary unit and \( k \) the rate of concentration change per unit time. In order to model the redistribution of the residual chlorides, the rate of concentration change, \( k \), must therefore be determined first. After application of the repair material it can be assumed that the residual chlorides will initially penetrate into the new layer by capillary action owing to the wetness of that layer. Studies by Martin [7] of repaired concrete beams contaminated by chlorides have shown that the most significant migration of the residual chlorides into the layer of repair material occurred shortly after the layer had been applied. The later diffusion-controlled transport of chlorides took place at a much slower rate. The studies also showed diffusion of the residual chlorides within the beams [7].

A mathematical model for describing chloride penetration in 2-layer systems with residual chlorides does not yet exist. Ongoing studies dealing with the description and modelling of the various mechanisms of chloride transport in 2-layer systems are presented in section 7.
4. APPROXIMATED CALCULATION OF RESIDUAL SERVICE LIFE

One of the most important preconditions for prediction the service life of a structure is the availability of mathematical models to describe the relevant transport and / or deterioration mechanisms (see section 2). However, those models needed to calculate the residual service life of repaired structural elements, as described above in section 3, do not yet exist. The main problem is the presence of the residual chlorides in the remaining layer of concrete. Nonetheless, the residual service life can be approximated initially by means of the simplified approach described below (see also [13]).

The mathematical model used is the approach proposed by Carslaw et al. (eq. (4) and (5)). The residual chloride content at the surface of the reinforcement, \( C_r \), is determined / specified. The limit state is taken to be the point at which a chloride content defined as the difference between the critical chloride content \( C_{\text{crit}} \) and the residual chloride content \( (C_{\text{crit}} - C_r) \) is reached at the surface of the reinforcement due to the ingress of external chloride ions. The gradient of the residual chloride profile and its redistribution are not taken into account. The procedure described above is illustrated in figure 4.

The limit state equation for the reliability analysis is, by analogy to the 1-layer system (eq. (3)), as follows:

\[
g(C_{\text{crit}}, C_r, C_{\text{remain}} (a_{\text{remain}}, a_{\text{new}}, t)) = (C_{\text{crit}} - C_r) - C_{\text{remain}} (a_{\text{remain}}, a_{\text{new}}, t) < 0 \tag{10}
\]

The results of the reliability analyses performed as shown above for a typical 2-layer system comprising a 40 mm thick layer of repair material made of CEM III/B concrete and a 20 mm thick layer of remaining concrete made of CEM I concrete are shown in figure 5. The residual chloride content at the surface of the reinforcement, \( C_r \), was varied, being taken to be 0, 0.1 or 0.2 wt.-%\( /c\). By analogy to the 1-layer system [1], a beta distribution with a mean value of 0.6 wt.-%\( /c\) was assumed for the critical chloride content. The variation of the residual chloride content results in different service lives of approximately 31, 22 and 18 years, for a target reliability index of 1.5.

By way of comparison, a calculation was performed without taking account of the layer of remaining concrete. In this 1-layer system, the initiation phase was calculated on the basis of
the point at which a critical chloride content of 0.6 wt.-%/c was reached at the interface with the layer of remaining concrete (x = 40 mm). In this case, a service life of around 6 years was determined. In another calculation, the thicknesses of the layers of repair material and remaining concrete were exchanged (with the overall thickness still being 60 mm). The service life of 12 years determined in this calculation was considerably shorter than for the comparable case with a service life of 22 years (40/20/0.1).

Figure 5: Estimated corrosion initiation time after application of a repair material

The results of the analyses indicate that both the residual chloride content at the surface of the reinforcement and the thickness of the layer of repair material are of great importance for the residual service life of the structural element. The extent to which disregarding the gradient of the residual chlorides and their redistribution affects the results has not yet been investigated. The question of whether the assumptions result in a conservative calculation of the residual service life or the opposite also remains unanswered.

The reliability analyses were performed numerically using the STRUREL program [8]. In the following section, the redistribution of the residual chlorides is investigated numerically using the Finite Element Method.

5. NUMERICAL INVESTIGATIONS OF CHLORIDE PENETRATION IN A 2-LAYER SYSTEM

The chloride ingress and redistribution in a 2-layer system was investigated using the COMSOL Multiphysics® software [9]. In this case, the chloride penetration was considered to be by diffusion only and described using Fick’s law. The mechanisms are modeled in the program by means of differential equations and solved using the Finite Element Method (FEM). The calculations in this chapter are due to a mean value approach, i.e. β = 0.

The results of a FEM simulation are presented in figure 6. The example shows a structural element made of CEM I concrete which was repaired after 10 years of exposure in a tidal zone (XS3 exposure class). The structural element was repaired by partially removing the 60 mm thick concrete cover (mean cover depth for XS3 according to the German technical guideline „ZTV-W LB 215“ [10]) and replacing it with a CEM III/B concrete. The following two pragmatic criteria were considered when removing the concrete cover in order to avoid the risk of corrosion by the residual chlorides:
1. The distance between the critical chloride content and the surface of the reinforcement must be at least 10 mm.

2. The maximum residual chloride content must not exceed 2.0 wt.-%/c.

In many standards and directives, such as RiLiSIB [11], the mean critical chloride content ($C_{\text{crit}}$) is taken to be 0.5 wt.-%/c. In order to comply with these criteria, the concrete cover was removed to a depth of 27 mm. Thus the thickness of the remaining concrete cover was 33 mm, with a 27 mm thick layer of repair material being applied.

In figure 6, left, the chloride profiles 1, 5, 10, and 50 years after repair and external chloride exposure are compared with the residual chloride profile immediately before repair. The figure shows an unmistakable back-diffusion of the residual chloride ions into the layer of repair material and redistribution in the layer of remaining concrete. The critical chloride content is exceeded at the surface of the reinforcement after a period of around 50 years.

6. EXPOSURE TESTS

While laboratory tests form the basis for developing durability models, such models are verified, calibrated and validated by studying structures and by field tests. Thus the models are checked against the conditions existing in actual practice and modified as required.

In 1991, the Federal Waterways Engineering and Research Institute (BAW) and the Institute of Building Materials Research (ibac) produced large concrete slabs (2.45 x 1.40 x 0.12 m³) coated with a variety of repair materials as part of a research project to study the durability of repair materials for use in marine environments [12]. The slabs were exposed in four different locations, including on marine structures on the North Sea and Baltic Sea coasts. Figure 7 shows the test location on the Baltic Sea coast. The slabs cover three exposure zones: submerged (XS2), tidal and splash (XS3) zones. The exposed slabs correspond to the case no. 2 in chapter 3.
The studies focused on investigating all the characteristics of relevance to the durability of the eight selected repair systems, such as frost resistance, resistance to water penetration, bond strength, etc. Two sprayed mortars proved to be suitable repair materials. Their resistances to chloride ingress behave in a similar way to those concretes made with CEM III/B: these were the sprayed mortar “H” modified with microsilica and the polymer-modified sprayed mortar (SPCC) “I”. Chloride profiles for these materials after 3, 6 and 21 years are available. They were obtained by determining the total chloride content of ground drill cores taken from the slabs. The concrete is a CEM III/B concrete with a water/cement ratio of 0.50. The concrete layer and the mortar layer are 120 mm and 20 mm thick respectively. Figure 8 shows, by way of an example, the chloride profiles for the concrete slabs coated with the sprayed mortars and those for uncoated slabs in the tidal zone.

It can be clearly seen that after 21 years’ exposure the chloride contents at the surface in the case of sprayed mortar are considerably higher than for concrete. However, the initially high values decrease to such an extent over the profile that the chloride contents in the concrete of the coated slabs are either lower or the same as those of the uncoated slabs. A stagnation of the chloride transport in the outer zone (convection zone) of the slabs without mortar which does not correspond to a diffusion process can be seen. This is presumably due to the removal (washing off / back-diffusion) of the chloride ions in the outer zone. Thus the sprayed mortar and the concrete possess different long-term behavior due to the chloride

Figure 7: Exposed slabs on the Baltic Sea coast

Figure 8: Chloride profiles of concrete slabs coated with mortars H and I after 3, 6 and 21 years’ exposure on the North Sea (left) and Baltic Sea coast (right)
ingress, which could not be observed from the early investigations after 3 and 6 years’ exposure. It is also possible that an interfacial resistance caused by pore blocking leads to the high chloride concentration in the sprayed mortar layer (see chapter 3, case no. 2). Further investigations are required on this effect.

7. CURRENT LABORATORY INVESTIGATIONS

Laboratory investigations are currently being conducted to observe and model chloride transport mechanisms in a 2-layer system. Composite specimens comprising a concrete layer with chloride gradients and a layer of repair material, see figure 9, were prepared for the investigations. The ingress, back-diffusion and redistribution mechanisms for chloride ions in the layer of repair material and the layer of concrete are being studied in long-term laboratory storage tests. The storage tests with the different types of specimen and the storage conditions are summarized in table 1. Three different types of concrete (OPC with different w/c ratios) and a single repair material (PCC: Polymer modified Cement Concrete) are being investigated.

Figure 9: Laboratory storage tests with composite specimens (width: 200 mm)

Table 1: laboratory storage tests

<table>
<thead>
<tr>
<th>test series</th>
<th>specimen</th>
<th>storage</th>
<th>studying mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>composite specimen with chloride gradients in concrete</td>
<td>Cl – Cl</td>
<td>D, B, R</td>
</tr>
<tr>
<td>1b</td>
<td>composite specimen with chloride gradients in concrete</td>
<td>Cl – Iodide</td>
<td>D, B, R</td>
</tr>
<tr>
<td>2</td>
<td>composite specimen with chloride gradients in concrete</td>
<td>Cl – water</td>
<td>B, R</td>
</tr>
<tr>
<td>3</td>
<td>composite specimen</td>
<td>20 °C/65 % RH – Cl</td>
<td>D</td>
</tr>
<tr>
<td>4</td>
<td>concrete specimen with chloride gradients</td>
<td>Cl – 20 °C/65 % RH</td>
<td>R</td>
</tr>
</tbody>
</table>

D: Diffusion (ingress) of external chloride ions in repair layer and further in concrete layer
B: Back diffusion of residual chloride ions from concrete to repair layer
R: Redistribution of residual chloride ions in concrete layer

The long-term resistance of repair materials to chloride ingress is the subject of further investigations. As already described in section 2, the apparent chloride diffusion coefficient,
D_{app,C} decreases over time. This time-dependence is modeled using the ageing exponent, $\alpha$, which has a major influence on the service life of structural elements. Little is known about the long-term behavior of repair materials and their composition is also unknown. However, comparative tests with concrete with known ageing exponents are being conducted in an attempt to estimate the ageing exponents of the repair materials in spite of this.

8. SUMMARY

The methodology of determining the residual service life of repaired concrete structural elements based on an existing concept for calculating the service life of new concrete marine structures has been described in this paper. The process by which diffusion of chloride ions takes place in a 2-layer system and the mathematical modelling of that process have been discussed. An approximate calculation of the residual service life of repaired structural elements has been presented. The redistribution of the residual chloride ions in the remaining layer of concrete has been demonstrated by performing sample FEM numerical calculations. The effects of the content and gradient of the residual chloride ions on the residual service life of structural elements have been illustrated by examples.

Laboratory investigations are needed to study the mechanisms by which chloride ions are transported in a 2-layer system and field investigations are essential to enable the results of the laboratory investigations to be assessed for their relevance in practical applications.

REFERENCES