PROBABILISTIC EVALUATION OF THE DURABILITY OF REINFORCED CONCRETE BRIDGE DECKS EXPOSED TO THE ACTIONS OF CHLORIDES

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Acknowledgements

I dedicate this book to my wife Nice and children Victoria and Elena. I thank Nice for her understanding and patience as this book could have seen the light of day much earlier and with a totally different theme. Nice was patient because she knows me and she knows that I work at the last moment and until the last moment. I thank the children for their lovely support during mutual games during which when enjoying a lovely time together we "recharge our batteries".

This book was meant to be about probabilistic modelling of earth exchangers of heat pumps but chance or destiny wanted it and in May 2014 I was approached by Ing. Hrdlička from the research company Vladimir Fišer who was interested in modelling directly exposed reinforced concrete bridge decks. During the summer we met with ing. Fišer and agreed on mutual cooperation. In the meantime I was working on the preparation of material concerning the original theme related to a study visit at Oklahoma State University. At the end of 2014 I still had to complete the last few steps in the area of exchangers concerning random modelling of the cyclical behaviour of external temperatures and the parameters of a heat pump. I then still naively believed that I would finish the work by October. When I then learned that it is possible to get financial support from a European project with the working title "Construction Engineering" for translations or language correcting, I instantly reached for the offer.

Then the semester began and I didn't even touch my work and time flew by. October passed by, November passed by, and David Hibler from Projects now and again asked how I was doing. At the end of November I came out with the truth that I was not capable of preparing the materials. David assured me that the faculty management has a back up plan and so I forgot about it.

In the meantime I started working with my colleague Ing. Petr Lehner on an analysis of samples of high performance concrete to estimate the life span of directly exposed bridge decks in relation to the interests of Ing. Hrdlička. The work was interesting although very time consuming. During November Petr Lehner fine tuned the FEA model of 2D diffusion with a four angled element and the capability of modelling the setting of concrete. We started to count.

Before Christmas the Vice Dean for science and research Dr. Jiří Brožovský came to me with a question as to when I would have the text ready. I realised that I was in a cul-de-sac. Ahead of me research into directly exposed bridge decks, uncompleted work on the analysis of earth exchangers so a decision had to be made and I chose a theme related to my long term work in the area of probability and reliability of reinforced concrete structures under the actions of chlorides as the theme of this book.
In conclusion I would like to thank my supervisor Professor Ing. Pavel Marek, DrSc. for his guidance during my doctoral study in the area of the probabilistic assessments using the SBRA method. On the basis of his contacts with Professor Paul J. Tikalsky I was invited on a study visit in 2005 to the State College of Pennsylvania (USA) where I began to learn about durability, chlorides and bridge decks. At that time I prepared a numerical 2D model of bridge decks as well. I am also grateful to Paul Tikalsky from both a professional and a personal point of view. I learnt a lot from him and his post doctorate student David Tepke. The insurmountable openness with which Paul approaches his students and close colleagues never ceases to surprise me. I also went to Oklahoma with my family based on an invitation from Paul and paradoxically it was he who offered me the chance to try another segment i.e. heat pumps in which OSU are number one. In any case I thank Paul and Julie with my whole heart for myself and for my family for the warm welcome which we got and still get from you.

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And you Vlaďko, Lenko and Vašku, you deserve my thanks for support during the minor everyday work. Carmen, to you, thanks for the great administrative back up. In conclusion I thank Lenny for unfailing resolve when fine tuning and preparing the programming and implementation part of the numerical model of 2D diffusion. I thank you Lenny for your self-sacrifice.
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1 INTRODUCTION

The reliability of reinforced concrete bridge structures is, in many cases, predetermined by durability. Which is proven by the many structures which require premature repairs or replacement as a result of defects caused by among others the effect of environment or the long term actions of loads including chemical actions. Shortened lifespan leading to increased costs over the life cycle of the structure contributes indirectly to high costs to public budgets. The production of higher quality concrete mixes and more durable construction systems can contribute to lowering the overall costs of the structure (viz e.g. Tikalsky et al., 2007, Ghosh et al., 2014). These can be better proposed with a knowledge of the progress of the degradation process brought on by the long term actions of environment and structural loading (see e.g. Keršner et al., 1996, Šmerda et al., 1999, Teplý & Vořechovská, 2012, Sýkora et al., 2013, Novák et al., 2014).

The attention of this book is directed to the preparation of a probability model for estimating the durability of reinforced bridge decks considering corrosion caused by the effects of gritting salts. The work aims in particular on the preparation of possibilities to compare variants of directly exposed bridge decks and bridge decks protected by waterproof insulation. The reader is accompanied through the narrow specialized area of numerical models for directly exposed reinforced concrete bridge decks. The development follows on from the work (Tikalsky, 2003, Tikalsky et al., 2005, Konečný et al., 2007) enriched by introducing the effects of the setting of concrete with the aid of time varied diffusion coefficients (Lehner et al., 2014). The existing model is supplemented with a special description of the effect of cracks in the directly exposed bridge deck in the form of the application of the permeability of concrete to the penetration of aggressive agents in cracks. A novelty is an illustration from the preparation of the model of a reinforced concrete bridge deck from ordinary concrete with steel reinforcement protected by waterproof insulation under an asphalt covering.

It needs to be noted that the focus is given to replacement off steel reinforcement by glass or carbon fiber reinforcement with the aim to completely avoid corrosion, see e.g. (Kohoutková & Broukalová, 2013).

The numerical models would not be complete if the preparation of input parameters was not dealt with and mention made of the literary sources which can be used. Attention is particularly given to the preparation of the diffusion coefficients of concrete against the penetration of chlorides both with the aid of an analysis of chloride profiles and calculations using the electrical resistivity of concrete.
The aim of the author is to introduce the reader to a tool for comparing the directly exposed bridge deck in the plain concrete version and high performance concrete, with the traditional solution where the reinforced concrete slab is protected by waterproof insulation under an asphalt covering.

The text is logically divided into chapters which cover the chosen areas of analysis of the durability of reinforced concrete bridge decks subjected to the actions of chlorides. Chapter 3 decks is dedicated to the principles and possibilities of assessment of the durability of reinforced concrete bridge decks considering the actions of chlorides. A theoretical model on the basis of diffusion is described in the introduction and the method of analysis of the life span of reinforced concrete bridges is discussed. A short section describing the possibilities of protection steel reinforcement against the ingress of chlorides causing corrosion follows. A detailed study of an assessment of durability with a view to an application of a probabilistic analysis model then follows.

In chapter 4 chlorides the reader is introduced to the principles and methods related to a description of the capabilities of concrete to resist the penetration of chlorides. The methods of analysis of diffusion coefficients are mentioned and then the laboratory methods of verification with the help of electrical resistivity analysis and chloride profiles are described. A calculation of diffusion coefficients follows both for samples of high performance mixes and indicatively for ordinary concrete mixes. Due to the following probabilistic application attention is given to the description of the preparation of the distribution function of diffusion coefficients applicable to a probabilistic analysis of durability.

The following chapter 5 cracks is concerned with a description of the 2D numerical model which serves to analyse the amount of chlorides at the level of the reinforcement. It contains a description of the model and the types of tasks which can be solved and includes deterministic samples of solutions.

Chapter 6 analysis contains probabilistic estimates of the initiation of corrosion for chosen alternatives of reinforced concrete bridge decks when using the above mentioned 2D finite element model. In the conclusion a comparison of the behaviour of the individual methods for protection of steel reinforcement is carried out. This part serves as a primary study where for the verification of the functionality of the model and an example of its possibilities, certain parameters are estimated.
2 PROBABILISTIC ASSESSMENT OF RELIABILITY

A brief introduction to probabilistic assessments in the analysis of engineering structures with a concentration on the area of reinforced concrete bridge decks containing the following paragraph.

The long-term interest in probabilistic assessments of the reliability of construction structures is given by the possibilities to aptly express the random variable characteristics of the problem solved. The probabilistic approach thus allows us to capture the random interaction between often mutually contradictory parameters.

In the case of the reinforced concrete bridge decks being discussed, the random interaction between cracks in concrete, defects in epoxide coatings, and damage to waterproofing under an asphalt surface is interesting.

These probabilistic approaches (see for example Marek et al., 1995, Holický 1998, Melchers, 1999, Teplý, 1999) allowing a more credible description of the random character of the input parameters of the calculation allow analysis of mutual interaction of the random variables of the effects of loading $E$ a resistance $R$. The output of a probabilistic method is usually a qualitative description of the level of reliability whether in the form of probabilistic defects $P_t$, or the reliability index $\beta$.

2.1 Aspects of reliability assessments from the point of view of probabilistic approaches

The reliability of building structures and thus also reinforced concrete bridge decks depends in particular on two main random variable components: the reaction of the structure to loading and reference functions. These can be defined as a function of many random changes between values among which belong according to (Marek et al., 2003) in particular:

- the quality of manufacture, erection and the level of their control,
- maintenance of the structure and required inspections depending on the expected environmental effects,
- the use of the structure in accordance with the assumptions of the design,
- the choice of structural system, individual elements and structural details
- the choice of materials or combination of materials
- the requirements to bear loading, usability and durability,
- loading and its combinations
- the precision of the transformation models,
- the method chosen for the design and assessment of the structure
• special influences.

Random variable values enter the process of assessing reliability, for whose description suitable probabilistic divisions can be used, which can be parametrically defined functions or so called cut off histograms. An example of a histogram - the statistical division is a distribution function of the diffusion coefficient of concrete derived in chapter 4.6 beginning on page 54.

The divisions can be mutually independent, existentially dependent or correlated. For instance, random variable sectional characteristics of the element or parts can be correlated by the length of the element as explained in or correlated by area (viz např. Stewart, 2004, Vořechovský et al., 2008). The 2D spatial correlation for instance could be advantageously used to describe the dispersion of diffusion coefficients for a bridge deck section.

2.1.1 Loading

The change to the probabilistic approach means, among other things, the replacement of the current representation of loading (see characteristic values and loading coefficients in the partial coefficient method) by a suitable division in accordance with the essence of the probabilistic assessment. In the case of the action of chlorides it is the effect of the concentration of chlorides on the surface of the structure.

2.1.2 Transformation models and the response of the structure to loads

With the aid of a suitable transformation model the concentration on the surface - an adequately precise idealisation of the actual action of the structure - calculate the concentration of chlorides at the reinforcement level, which is the response of the structure to the chloride loading.

The choice of a transformation model should take into account the uniqueness of the concrete design situation and is thus dependent among other things on the character of the acting load, on material characteristics, on the geometry of the structure, on safety criteria, usability, durability and the importance of the structure.

In the case of reinforced concrete bridge decks it is possible to use an analytical model of an ideal structure without cracks/9/ or a numerical FEA model. The differences are then counted for an accessible range of results related to the difficulty of the calculation.

2.1.3 Reference values and the definition of defects

On the basis of the calculated response of the structure it is possible through comparison with reference values to define whether structure meets our required criteria for reliability (resistance and usability). The setting of reference values or functions can be based on:
• available statistical data,
• experiments,
• estimates,
• calibration
• agreements between users, designers, experts or responsible authorities.

2.1.4 Function - reliability

The main inputs to the assessment of reliability are the response of the structure to loading \( E \) and resistance \( R \). Their mutual interaction can be analysed by using a typical function shape reliability:

\[
RF = R - E.
\]

2.1.5 Assessment of reliability

An assessment of reliability based on a probabilistic approach and the limit state philosophy can use the probability of defects \( P_f \) (or index of reliability \( \beta \)), as a qualitative indicator of reliability, which is then compared with the design level of reliability of the structure \( P_d \) (or indicative value of the reliability index \( \beta_d \)). The condition of reliability can be expressed for example in the formula:

\[
P_f < P_d \text{ or } (\beta < \beta_d)
\]

In order to select the value of the design probability of defects \( P_d \), Addendum A of standard CSN 73 1401 (1998) served as a guide which tabulated the basic probability of defects for a design life span of the structure of 80 years. When considering other design life spans it is necessary to suitably alter the proposed probability \( P_d \) (see for instance Šejnoha&Blažek, 2005, Kmeť, 2005).

2.2 Tools for probabilistic assessments of reliability

To apply probabilistic methods of assessing reliability it is possible to use tools accessible on a number of levels, both analytic and simulation. A summary of the tools and methods can be found in domestic and foreign sources (Holický, 1998, Melchers, 1999, Teplý et al., 1999, Haldar&Mahadevan ,2000, Krejsa&Konečný, 2009) or Probabilistic Model Code (2001). Probabilistic methods can use many of the accessible tools for estimating the level of reliability (the probability of defects or the index of reliability), which are briefly discussed in the following paragraph.
2.2.1 Analytical tools

FORM (First Order Reliability Method) and (Second Order Reliability Method) are among the best known analytical approaches. The essence of this approach is a description of the probabilistic problem in the form of exact mathematical functions which are worked with in accordance with the principles of mathematical statistics. These approaches seek out within a standardised space of random variables, a so called design point lying on a defect function \( g(Y)=0 \) with the shortest distance from the beginning. This distance is the reliability index \( \beta \). A disadvantage of this approach are the difficulties in finding a design point which meets the global minimum particularly in the case of strongly nonlinear problems.

2.2.2 Simulation tools

Due to the enormous development of computer technology in the last decades of the last century, which is still ongoing, simulation approaches have gained increased significance. It concerns tools based on the principle of Monte Carlo direct simulation (Metropolis & Ulam, 1949). Direct simulation is robust but the computation costs may be rather high in case of low probability events. Thus variation reduction techniques are used in order to reduced the simulation time. These tools include for instance: Importance Sampling (Schueller, 2002, Praks, 2002, 2004), Stratified sampling a Latin Hypercube Sampling (McKey et al., 1979, Iman & Connover, 1982, Vořechovský & Novák, 2009).

These approaches can lead to a lowering of their computational difficulty which even today can be necessary for large highly nonlinear tasks. Another approach on the edge of simulation and numerical integration is the so called "Directly optimised probabilistic calculation (DOProC see Janas et al. 2010, Krejša et al., 2014).

2.2.3 SBRA probabilistic method

The Simulation-based Reliability Assessment method (SBRA, Marek & Guštar, 1988, Marek et al., 1995, 2003) will be implemented in the examples discussed later. Thus attention is driven to this method in following paragraphs.

This method is based on the principal of limit states and in particular on the application of a direct Monte Carlo simulation for the calculation of probabilistic defects \( P_f \).

SBRA uses load duration curves to express random variable values describable as cut off histograms and at the same time allows the inclusion of more components and correlated variables in the probabilistic analysis (see. Konečný, 2007).
The SBRA method allows the analysis of the interaction of random variable values of the effects of loading $E$ and resistance $R$ through a probabilistic analysis of the reliability function $RF$. The level of reliability is the probability of a defect defined as follows:

$$P_f = P(R - E < 0) = P(RF < 0)$$

The reliability of the structure is usually analysed by comparison with the probability of defects $P_f$ and the design probability of defects $P_d$. In the case of parametric studies it is also advantageous to compare the reliability of individual alternatives with the aid of numbering of the defects probability $P_f$, without a direct reliability assessment.

The essence of the method can be explained with the aid of Figure 1 where the relationship between the histogram of load effects $E$ (horizontal axis) and the histogram of load bearing $R$ (vertical axis) is shown in a 2D graph, a so called ant hill. Points on the graph represent an adequate number of pairs of $E$ and $R$. Each point corresponds to one of the simulation steps. The ratio between the number of points representing defects (under the $R$ line $/12/E = 0$ which represents the reliability criteria.) and the total number of points represents the probability of defects $P_f$. In the case of the time related durability of reinforced concrete bridge decks the lack of resistance expressed as a chloride threshold $C_{th}$ and the effect of loading in the form of chloride concentrations at the reinforcement level $C_{z,t}$ (see 22 on page ).

![Figure 1: The interaction of bearing capacity $R$ and the effect of loading $E$](image-url)
2.2.4 Precision of estimates of probabilistic defects

In every application of numerical calculations the basic issue is the precision of the calculations. In case of Monte Carlo simulation technique, a large number of steps are necessary particularly in case of tasks related to the limit state of bearing capacity. There the number of simulations can be in the millions. In the case of an analysis of durability the expected accuracy is in the range of single percentage points and the reader can see that in the case of the use of 10 thousand steps the Monte Carlo method can estimate an error with the aid of the following relationship from common statistics and deduce the dispersion of resulting probabilities:

\[
[-\varepsilon, \varepsilon] = [-t\sigma; t\sigma] = \left[-t, \frac{P_t(1-P_t)}{N_n}; t, \frac{P_t(1-P_t)}{N_n}\right]
\]

where:

- \(\varepsilon\) expected dispersion of resulting probabilities – confidence interval [-],
- \(N_n\) number of simulations[-],
- \(P_t\) probability sought[-],
- \(t\) confidence interval parameter (for 90 % reliability with \(t=1.6449\)) [-].

If the expected probability \(P_t = 1/100\) and the total number of steps is \(N\) is 10,000. Then it can be said with 90% certainty, that the result falls into the interval \(P_t = 0.01 \pm 0.0017\). In a case where 1000 simulation steps were to be used then the expected result is located in the interval \(P_t = 0.01 \pm 0.0052\). Through this approach it is possible to determine the necessary number of steps where the parameters of the task are the same.

\[
N_n = P_t(1-P_t) \left[\frac{t}{\varepsilon}\right]^2
\]

\(\text{[5]}\)
3 MODELLING THE DURABILITY OF REINFORCED CONCRETE BRIDGE DECKS

This part describes the principles and possibilities of assessments of the durability of reinforced concrete bridge decks considering the actions of chlorides. A theoretical model on the basis of diffusion is described in the introduction and the method of analysing the life span of reinforced concrete bridge decks is discussed. A short section describing the possibilities of protecting steel reinforcement against the creation of corrosion caused by chlorides follows. A detailed study of an assessment of durability with a view to an application of a probabilistic analysis model then follows.

In relation to the subject of interest it is of use to provisionally familiarise oneself with the objects of interest and thus the types of reinforced concrete bridge decks including a few sentences about their modelling.

3.1 Reinforced concrete bridge decks

The durability of reinforced concrete (RC) bridge decks can be affected by many factors, such as alkaline's, acids, repeated changes of humidity, cyclical temperature changes, carbonation, the action of chlorides, UV radiation, sulphides, fatigue and other influences including cracks. The action of gritting salt, which penetrates through the surface to the steel reinforcement, cause corrosion of the reinforcement and is one of the most important factors lowering the life span of bridge decks both in Central Europe and also in the North East of the USA. Corrosion caused by chlorides can cause a decline in the usability of the structure not only from the point of view of its use but also on its bearing capacity and in the end can lead to higher costs over the life cycle of the bridge.

Experience has shown that unprotected reinforcement made from mild steel must not only be covered in concrete but also other protection. The solution was the division of the reinforcement from the aggressive environment.
In the 1970s and 1980s North America widely adopted directly exposed bridge decks and epoxide protective layers on steel reinforcement. In Central Europe steel reinforcement tends to be separated chlorides using waterproof insulation under the asphalt layer even though here we are now experimenting with directly exposed bridge decks, (Simon et al., 2012). Both systems shown on Figure 2 allow the delaying of the onset of corrosion but even still they suffer widely from premature corrosion of reinforcement. A further step towards higher quality protection of steel reinforcement is the use of high performance concrete (HPC), allowing a lowering of the permeability of aggressive substances through concrete and also the substitution of steel reinforcement for glass or carbon fibres. It is necessary to point out that even though the steel reinforcement is protected by a covering, epoxide coating, or waterproof insulation, contact still occurs between chlorides and the steel. The contact is facilitated by either long term penetration of chlorides into the covering or defects in the means of protection.

It is appropriate to note that the principles of design of reinforced concrete structures allow for the formation of cracks of limited width. Cracks occur not only in areas of tension where the load is taken up by reinforcement but also in the compressed areas due to shrinkage processes and so on. The covering is disturbed by cracks and chlorides can easily penetrate to the reinforcement of the directly exposed bridge. For the solution most commonly used here of a reinforced concrete bridge deck protected by waterproof insulation it has however has been shown that even the insulation degrades over time and chemical substances often flow across the bridge structure due to badly carried out drainage or expansion details. Flows of antifreeze agent thus pass around the waterproof insulation and damage the bridge deck on the edges.

### 3.2 Models of the durability reinforced concrete bridge decks

One of the aids for a more advanced design and the preparation of a construction solution for a building structure can be models which estimate the progress of the degrading processes. A
suitably prepared model can tell construction Engineers how to alter the parameters of a construction solution so that it is more durable. A well-tuned model can also give answers as to whether a directly exposed bridge deck is more advantageous or a structural solution with waterproof insulation.

From experience it is clear that both construction systems have their weaknesses and typically require a reconstruction after about 25 years. A numerical model can help in comparing the behaviour of both alternatives which contains the appropriate input data and also a suitable physical verification.

A model describing a directly exposed bridge deck which allows the inclusion of the simplified effects of cracks in initiating corrosion is available (Konečný et al., 2007), and includes an evaluation of the effects of epoxide coatings. The model taking into account the effect of waterproof insulation and its defects in reliability described in this book is a new one.

It is necessary to have information on material characteristics to model the durability of bridge decks, particularly the diffusion coefficients against the penetration of chlorides, depth of cover, frequency and depth of cracks, and data about damage to the waterproof insulation or epoxide covering. Descriptions of the concrete, depth of reinforcement, corrosion threshold or spacing of cracks are reasonably accessible information. The depth of cracks and the effect of their width on the penetration of chlorides are values which are more difficult to obtain, whereas the frequency and characteristics of defects in waterproof insulation under asphalt strips are values which are almost unobtainable.

The development of a model for the analysis of the durability of reinforced concrete bridge decks protected by waterproof insulation and its comparison with the directly exposed alternative, including the evaluation of the effects of cracks in concrete, is despite the lack of all input parameters very desirable as the missing data can be obtained through surveys during future reconstructions of the existing structures.

### 3.3 Degradation caused by the action of chlorides

It is possible to use numerical models to model the durability of reinforced concrete bridge decks from the point of view of chloride action, which describe the risk of reinforcement corrosion occurring and thus the risk of the occurrence of degradation processes.

#### 3.3.1 Lifespan

If corrosion caused by the penetration of chlorides to the steel reinforcement is regarded as the
dominant parameter affecting its durability then its lifespan can be recorded according to (Tutti, 1982) as:

\[ t_{\text{service}} = t_{\text{initiation}} + t_{\text{propagation}} \]

where the time to the occurrence of corrosion is \( t_{\text{initiation}} \) and \( t_{\text{propagation}} \) which corresponds to the time in reaching an unacceptable level of corrosion in reinforced concrete reinforcement. The period of initiation of corrosion is affected in particular by diffusion characteristics, concrete cover, surface concentrations of chlorides, temperature, the level of salt saturation at reinforcement level and a concentration of salt adequate to begin corrosion. This limiting concentration can be called the chloride threshold. The means and for how long the chlorides get to the reinforcement level so that they initialize corrosion is described in the following section.

The propagation phase of corrosion which is closely related to a dramatic fall in reliability due to the gradual metamorphosis of steel reinforcement to corroded products combined with the related decline in cross sectional areas and damage to the cover lies outside the framework of this work. Basic information can be obtained in the publication Šmerda et al. (1999). Liu&Weyers (1998). for instance considered the occurrence of corrosion cracks and limiting their development. The author also tested a pilot application of an analysis of durability including a propagation phase See Konečný et. al. (2011). The work of Vořechovská&Vořechovský (2013). studies advanced numerical approaches through the application of random fields and nonlinear finite element methods for the modelling of corrosion caused by cracks.

### 3.3.2 The penetration of chlorides due to the construction solution

For instance, in the USA, directly exposed bridge decks generally do not have their pavements separated from their RC bearing structures by waterproofing. Gritting salts can freely pass through concrete to the reinforcement as distinct from the situation in the CR, for instance, where the waterproofing theoretically prevents the penetration of chlorides to the reinforcement. (see illustrative Fig). The USA thus introduced the protection of steel reinforcement against the action of chlorides using epoxide coatings at the end of the 1970s.

Under our conditions the steel reinforcement can be subject to the action of chloride ions in the case of defects in the function of the waterproof layer and the following diffusion of chlorides in the RC slab.

Steel reinforcement can be protected from chlorides causing corrosion by a surface finish or the separation of concrete from the transfer of chlorides by a waterproof membrane. Despite this
construction measure the reinforcement can still be subjected to the action of chlorides. Reinforcement treated with an epoxide coating or galvanization is commonly subject to corrosion particularly due to uncovering of the reinforcement due to damage to the epoxide coating or corrosion which begins at higher salt concentration levels. Chlorides can also pass through disturbed/degraded spots on the waterproofing. Another factor is the presence of cracks in concrete which also lowers concretes ability to protect reinforcement. It is thus necessary to consider the concentration of salt chlorides at the level of reinforcement close to cracks in concrete, under defects in the waterproofing or uncovered spots in the reinforcement (holidays).

The model described in the text concentrates on the period of initialization of corrosion $t_{initiation}$. Corrosion begins when the concentration of salts around reinforcement reaches a level adequate for the destruction of the reinforcements protective layers.

![Figure 3: Schematic section through the directly exposed bridge deck (left) and the bridge deck with separated waterproofing and an asphalt covering (right).](image)

### 3.3.3 Chloride transport model

The corrosion of steel reinforcement is primarily controlled by the diffusion of chlorides. The effect of hydraulic pressure and capillary sorption is not considered in the model described as in most cases it can be ignored. Diffusion is thus the most common way in which chloride ions are brought into contact with the reinforcement of reinforced concrete bridges decks. Diffusion occurs as a result of concentration gradients. In other words if the chloride ions are not evenly distributed in a liquid then the ions move from the place with the highest concentration to the place with a lower concentration. The process continues until the concentrations are equal.

The process of chlorides passing through concrete as a function of depth and time can then be modelled with the aid of Ficks 2nd law of diffusion (see for example Šmerda et al., 1999 or Hooton et. al., 2001),as is generally accepted.
\[ \frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial z^2} \]  
/7/,

where:
- \( C \) concentration of ions [%],
- \( z \) depth [m] (from the surface subject to chlorides),
- \( t \) time [s],
- \( D \) diffusion coefficient [m²/s].

The differential formula /7/ can be solved when applying the following marginal conditions.

\[
\begin{align*}
C(z = 0, t > 0) &= C_0 \quad \text{constant concentration of chlorides on the surface } C_0, \\
C(z > 0, t = 0) &= 0 \quad \text{the initial concentration in concrete is 0}, \\
C(z = \infty, t > 0) &= 0 \quad \text{the zero concentration is at infinity}.
\end{align*}
\]

This is called Cracks solution and was applied to the problem of diffusion in concrete in the work of (Collepardi, 1972). See /8/:

\[ C_{z,t} = C_0 \left[1 - erf\left(\frac{z}{\sqrt{4Dt}}\right)\right] \]  
/8/,

where:
- \( C_{z,t} \) the concentration of chloride ions [%] (expressed as a percentage of all materials with cementation properties) and in time \( t \) (years) depth \( z \) [m],
- \( C_0 \) concentration of chloride ions (expressed as a percentage of all materials with cementation properties) at the concrete surface [%],
- \( D_c \) the effective diffusion coefficient [m²/s],
- \( t \) time of exposure [s]

A polynomial development can help in the numerical solution to the chosen differential equation/7/.

\[
C_{z,t} = C_0 \left\{1 - \frac{2}{\sqrt{\pi}} \sum_{n=0}^{14} \frac{(-1)^n \left(\frac{z}{\sqrt{4Dt}}\right)^{2n+1}}{n!(2n+1)}\right\}  
/9/.
\]

The relationship /8/ and its solution /9/ are widely used aids for modelling the penetration of chlorides even though they do not allow the description of the time dependent changes of material properties or more complicated marginal conditions. When modelling more complicated marginal conditions it is useful to use models on the basis of numerical approaches for instance finite element
Due to the long term curing of concrete the diffusion coefficient is a time dependent parameter. Its development over time can be determined with the help of reference values and the curing coefficient (Tang & Nilsson (1992), Boddy et al. (1999):

\[ D_c(t) = D_{c,\text{ref}} \left( \frac{t_{\text{ref}}}{t} \right)^m \]  

In relationship /10/ there are:

- \( D_c(t) \) effective diffusion coefficient for the chosen age [m²/s],
- \( D_{c,\text{ref}} \) diffusion coefficient obtained from a referential old structure [m²/s],
- \( t \) curing time [years],
- \( t_{\text{ref}} \) reference period for measurement [years],
- \( m \) curing coefficient [-].

For a 1D analysis of a chloride profile during the life of the structure it is possible to use a relationship developed in relation to Cracks solution (see Magnat & Molloy, 1994):

\[ C_{z,t} = C_0 \left[ 1 - \text{erf} \left( \frac{z}{\sqrt{4 D_{c,\text{ref}} t (1-m)}} \right) \right] \]  

where:

- \( C_{z,t} \) concentration of chloride ions [%] (expressed as a percentage of all materials with cementation qualities) and in time \( t \) (years) depth \( z \) [m],
- \( C_0 \) concentration of chloride ions (expressed as a percentage of all materials with cementation qualities) at the concrete surface [%],
- \( D_{c,\text{ref}} \) diffusion coefficient obtained in a referential old structure [m²/s],
- \( t \) exposure time [s],
- \( m \) concrete curing coefficient [-].

A more detailed breakdown of the diffusion coefficient \( D_c \) is contained in chapter 4.2 Diffusion coefficient page 29.

### 3.4 Assessment of reliability

Whether the analytical or numerical model is used to determine the concentration of chlorides at the level of reinforcement or at the point of damage to the epoxide covering, the output is the...
concentration of chlorides $C_{x,t}$. Through a comparison of the chloride threshold $C_{th}$ with the actual concentration at a given time it is possible to calculate whether the corrosion has begun or not. See also the reliability function for general assessments of reliability given by the relationship /1/.

The durability of bridge decks describable by the reliability function $RF_t$ is expressed as a time dependent crossing of the corrosion threshold $C_{th}$, by the concentration of chlorides $C_{xy,t}$ which is locally dependent on the parameters of the covering of the reinforcement. The function of reliability characterizing the above described limit state is expressed as:

$$RF_t = C_{th} - C_{xy,t}$$

(Durability can be related to the initiation of corrosion which corresponds to relationship /12/ or the decrease in the area of reinforcement or the occurrence of cracks caused by corrosion due to chlorides. The period to the start of corrosion $t_i$ can be determined from the balance of concentrations of chlorides and the chloride threshold or from the reliability function $RF_t$ where the following applies:

$$RF_t = 0 \Rightarrow t_i$$

The value of $C_{th}$ depends in particular on the types and preparation of reinforced inserts and the components of the concrete. Typical values are 0.2 % of the weight of chlorides in proportion to the weight of cement according to the ACI (American Concrete Institute).

### 3.4.1 Probabilistic idea

Due to the fact that parameters with a considerable dispersion of values form part of a durability analysis it is appropriate to use a probabilistic approach to copy the behaviour of reinforced concrete bridge decks (Stewart & Rosowsky, 1998, Vu et al., 2000, Tikalsky, 2003, Tikalsky et al., 2005; Konečný et al., 2007, Vořechovská, 2010).

A probabilistic assessment of durability widens a 2D concept from Figure 1 by the time dimension so that its possible to conceive a time dependent analysis of the initiation of corrosion. The initiation of corrosion represents an intersection of the random variable realisation of time dependent functions of chloride concentration $C_{xy,t}$ and the chloride concentration threshold for the initiation of corrosion $C_{th}$. As soon as the probability of the initiation of corrosion in critical locations of reinforcement exceeds the user defined value (depending on the importance of the structure) it is assumed that corrosion has begun. The structure is then regarded as unreliable due to the ongoing propagation phase of corrosion. Figure 4 outlines the described concept of durability.
The stochastic nature of the diffusion penetration of chlorides can be described for instance by using the Simulation based Reliability Assessment method (SBRA, Marek et al., 1995, 2003), which was used in (Tikalsky, 2003) as a description of the probabilistic behaviour of the 1D diffusion of chlorides. The subject of the analysis was a directly exposed ideal bridge decks without cracks made from common reinforced concrete. The SBRA method was used in (Konečný et al., 2007) to model the 2D problems of directly exposed bridge decks considering the effect of cracks and possible protection by epoxide coverings.

The reliability function $RF_i$ is calculated in certain chosen time intervals (for various ages of structure). The probability of defects $P_{f,t}$ is obtained through an $RF_i$ analysis. The value obtained is not the real probability of defects but only the probability of exceeding the chosen limit state and can be expressed in relation to the /3/ following:

$$P_{f,t} = P(RF_i < 0) = P(C_{th} - C_{xy,t} < 0)$$

The structure meets the chosen criteria for reliability if the following criteria are met:

$$P_{f,t} < P_d$$

where $P_d$ is the proposed probability of defects. That can take on, according to (Marek et al., 2003), various values both in light of the applied limit state and also due to the importance of the structure being considered. For instance the design probability $P_d$ for limit state of bearing (safety)
is typically $P_d = 7e-5$ or $P_d = 7e-2$ for limit states of usability.

The question of design probability is debatable and is studied by for instance (Teplý et al., 2002, 2005). Despite all the possibilities of statistics, modern models and laboratory tests it is not possible to estimate the probability of the occurrence of corrosion as such. The calculated value however can serve as a value for comparing more chosen alternatives when using a model with the same simplifications and models of random input values.

### 3.4.2 The effect of cracks in reinforced concrete bridge decks.

If there is a crack in the concrete then this crack allows the simpler passage of aggressive substances to the reinforcement. Simplified modelling of the effects on cracks by placing a concentration of chlorides at the point of the crack in concrete was considered in a probabilistic 2D analysis of the durability of RC bridge decks (Konečný et al., 2007 and Konečný, 2007). A sample of this numerical model which was prepared in ANSYS environment is given -Figure 5-

![Figure 5: The concentration of chloride ions in a concrete slab with a crack, $t = 10$ years](image)

A similar method of applying marginal conditions of a concentration of chlorides directly in a crack is used by Marsavina et al., (2009). for instance. In contrast Bentz et al., (2013) model the effects of cracks in the form of changes in the material parameters in the area of the crack.

The effect of crack width on the penetration of chlorides into concrete is discussed in ACI 222 (2001). This standard identifies the effect of cracks on more rapid corrosion of steel reinforcement under cracks through referencing the contradictions around the question of the effect of crack widths on the occurrence of chlorides. As regards crack width limits it cites the working standard (Atimay & Ferguson, 1974), which gives a limiting width for cracks of 0.3mm. Cracks smaller than 0.3 mm do not have an effect on the propagation of corrosion according to this work. The other works cited in ACI standard 222 (2001) do not confirm a relationship between the width of cracks.
and the corrosion of steel reinforcement in concrete.

Other works supporting the relationship between cracks and the capability of concrete to facilitate the passage of chlorides which leads to a more rapid propagation of corrosion include (Djerbi et al., 2008, Bentz et al., 2013).

In the introduction to the work (Djerbi et al., 2008) with reference to (Francois et al., 2005) it is stated that the concentration of chlorides in a crack wider than 205µm is equal to the concentration of chlorides at the surface. Djerbi et al. (2008) carry out an analysis of the effect of the width of cracks on the penetration of chlorides with the use of a modified accelerated test of the penetration of chlorides. (AASHTO T277, ASTM C1202). These tests determine the capability of concrete to resist the penetration of chlorides. Through the passage of an electric charge it can also determine the diffusion coefficient describing its capability to prevent the penetration of chlorides into concrete (Andrade, 1993), as will be shown in section 4.2. Among the conclusions of the work (Djerbi et al., 2008) it is emphasised that a difference between the diffusion coefficient calculated on the basis of the passage of an electrical charge in an undisturbed sample and a sample with crack widths of less than 30 µm was not observed. The diffusion coefficient in a crack was equal to the diffusion coefficient in an undisturbed sample \( D_{c,0} \). For a crack width between 80 - 250 µm the diffusion coefficient calculated in a crack \( D_{c,cr} \) was roughly equal to the diffusion coefficient of the media in the crack i.e. \( D_{c,cr,max} = 14 \times 10^{-10} \) [m2/s]. In the area between 30 - 80 µm there is a linear dependence of the diffusion coefficient in the crack \( D_{c,cr} \) and the diffusion coefficient of the undisturbed sample \( D_{c,0} \). There is also a dependence on the width of the cracks and the coefficients of the media in the cracks and thus \( D_{c,cr,max} \). The calculation of the diffusion coefficient in the crack could also be written (Djerbi et al., 2008) as:

\[
D_{c,cr} = (D_{c,cr,max} - D_{c,0})/50 \times (C_{rkw} - 30) + D_{c,0}
\]

where:

\( D_{c,0} \) diffusion coefficient for an undisturbed sample [m2/s],
\( D_{c,cr,max} \) the coefficient of the media in the crack, \( D_{c,cr,max} = 14 \times 10^{-10} \) [m2/s],
\( D_{c,cr} \) the diffusion coefficient in the crack [m2/s],
\( C_{rkw} \) width of the crack in the range 30 < \( C_{rkw} < 80 \) [µm].

Bentz et al., (2013) give a comparison of a numerical model taking into account the effect of cracks in the form of changes to the diffusion coefficients in the area of cracks and close to the area damaged due to the formation of cracks. (in the process zone). The values used for modelling cracks in the study (Bentz et al., 2013) are related to cement bonding and not to concrete as a whole. The
diffusion coefficient for cement bonding in the area of the cracks was chosen in light of the comparison of the model with the experiment carried out. The authors differentiate between two values of diffusion coefficients, firstly for cracks smaller than 100µm and cracks greater than 100 µm which is in accordance with the limit of 80 µm given in (Djerbi et al., 2008).

The above mentioned considerations about the influence of cracks are backed up by experimental results describing the capability of aggressive substances to penetrate with the advantage of a disturbed area of the crack. Another possible direction is the use of the findings of quarry mechanics to model the relationship between a crack, the disturbed process zone and the diffusion coefficient. One of the attempts to model the resistance of concrete in relation to the development of cracks with the help of a lattice model is presented in the (Pacheco et al., 2011) study. (Veselý et al., 2014) follow the relationship between the resistivity of concrete and the development of cracks during sequential fracture tests with the aim to correlate the progress of fracture in processing zone with diffusion related parameter.

Figure 6 : The scheme for the section of the I99 motorway near State College in Pennsylvania, USA where an extensive survey is underway in the area of directly exposed reinforced concrete bridge decks made from high performance concrete. The reinforcement was given an epoxide coating. (Tikalsky et al., 2007).

Literature devotes quite extensive attention to the width of cracks, while on the question of frequency or the depth of cracks, the access to data is limited. A database of the spacing of cracks on new directly exposed bridge decks made from high performance concrete is contained in the survey (Tikalsky et al., 2007). Part of the survey was the preparation of high performance mixes and sample designs to solve tens of actual bridges on a newly built section of the I99 motorway in
Centre county in the state of Pennsylvania in the USA (see Figure 6). Detailed information on the localization of cracks on the bridges concerned is given in the work Chejarla (2008). Unfortunately this survey does not give information on the depth of cracks.

Saadeghvaziri&Hadidi (2002) speak about crack spacing's in their report about the analysis of cracks on bridge decks in New Jersey. The authors state that a typical crack on a bridge deck is transverse and located above the transverse reinforcement. The spacing of cracks varied from 1 - 3 m. The cracks being studied usually passed right across the whole depth of the bridge deck which is valuable information due to the fact that the depth of cracks is very important when modelling the effect of cracks. In the numerical model of a bridge deck with a crack Konečný, et al.(2007a) estimates were used for instance due to the limited data about crack depths.
4 THE RESISTANCE OF CONCRETE TO THE PENETRATION OF CHLORIDES

This chapter describes the principles and approaches related to the capabilities of concrete to resist the penetration of chlorides. The methods of analysing diffusion coefficients are mentioned and later the method of laboratory verification is described including the actual progress of the works. A calculation of the diffusion coefficient for samples of high performance mixes used in the project (Simon et al., 2012) and a description of the preparation of the distribution function of the diffusion coefficient applied in the probabilistic assessment of durability.

4.1 Introduction

The ability of concrete to resist the penetration of aggressive substances including chlorides can be described by various methods. One of the possibilities is the apparent/effective diffusion coefficients of chlorides $D_c$. The diffusion coefficient is a parameter describing the penetration of chlorides in light of Fickovs 2nd law of diffusion/(Collepardi et al., 1972, Šmerda et al., 1999, Hooton et al., 2001 and Tikalsky, 2003). The diffusion coefficient can be obtained by long term and relatively long tests of the penetration of chlorides into a concrete sample. (AASHTO T277, NT BUILD 443).

Another method of description, the resistance of concrete to the penetration of chlorides are based on the use of the electrochemical characteristics of concrete and thus the relationship between passage of an electric current and resistance to the penetration of chlorides (Rapid Chloride Penetration Test – RCPT). One of them is the accelerated test of the penetration of chlorides (see Tang&Nilsson, 1992, AASHTO T277, ASTM C1202). In the accelerated test the progress of the chlorides is supported with the help of electric current. The result of the test is the amount of electrical charge which passes through the concrete sample in the chosen time. During this relatively long term test the sample also heats up, particularly in the case of high performance concrete with a greater resistance to the passage of electrical current. The heating of the sample leads to an increased passage of ions and the experiment is thus distorted. (Betancourt&Hooton, 2004). A much faster experiment which proves a good correspondence with the results of RCPT is the measurement of the electrical resistance of concrete. Morris et al. (1996) gave a method of calculation for the measurement on concrete slabs and cylindrical samples. For a more detailed list of the possibilities of evaluating concrete against the penetration of chlorides one can look to (Hooton et al., 2001).for instance.
It is necessary to point out that the above mentioned characteristics describe concrete as a homogenous material which does not correspond to reality and therefore the diffusion coefficient is called apparent or effective. Ions progress much faster in a cement paste when compared with aggregate, or in the contact zone between aggregate and cement paste. The diffusion coefficient also changes in time in relation to the progressive curing of concrete (see Tang&Nillson, 1992, Andrade, 1993, Mangat&Molloy, 1994, Boddy et al., 1999, Andrade, 2010 and Ghosh et al., 2014). For its characterization it is thus possible to use the values from the chosen reference time or follow its change in time with the help of a curing coefficient.

4.2 Diffusion coefficient

If we chose the diffusion coefficient as a parameter which characterizes the quality of the resistance of concrete to the penetration of chlorides and thus the parameter describing the ability of concrete to resist the action of chlorides then the question is how to arrive at the diffusion coefficient $D_c$.

As a basis for the choice of a suitable method for finding out the diffusion coefficient one can use the summaries (Hooton et. al., 2001, Tang et. al., 2005) and also the work of (Kurgan, 2003 a Ghosh, 2011). The traditional possibility is an analysis of the chloride profile during the long term action of a water solution of NaCl on the surface of the sample (AASHTO T259-02 or the modification NT BUILD 443). The innovated possibilities relating to the tests of accelerated chloride penetration (AASHTO T277, ASTM C1202) and the electrical resistivity of concrete (AASHTO TP-95, Ghosh, 2011, Ghosh et al., 2011).

Due to the dispersion of values of the measured diffusion coefficients or the corresponding values of resistivity and so on (Sohanghpurwala&Scannel, 1994, Ghosh et al., 2014) it is appropriate to use descriptive statistics to describe this value and for the numerical modelling of the durability of reinforced concrete structures using the probabilistic approach (Kešner, et al. 1996, Stewart&Rosowsky, 1998), Šmerda, et al., 1999, Vu&Stewart, 2000, Tikalsky et al., 2005, Konečný et al., 2007).

4.2.1 The dispersion of random values

The dispersion of diffusion coefficients or electrical resistivity are for the purposes of probabilistic numerical modelling a suitable description of probabilistic distribution. This division can be obtained from the measurement of an adequately large population sample. For the purposes of a quality determination of the average and standard variations describing the expected dispersion when assuming a normal division of the probability of the occurrence of chosen values it is
sufficient to have 20-30 samples. The same number of samples is also adequate for the use of further accessible distributions of probability. In order to verify the suitability of distribution a test of good correspondence is used. If the distribution has a more complex shape and accessible parametric distribution does not show a good correspondence it is possible to use a frequency histogram. For a number of classes of histogram there are various empirical criteria available depending on the amount of data. Generally it can be said that sensible numbers of classes are achieved with an amount of data in the range of many tens.

The statistical parameters of high performance concrete suitable for the creation of probabilistic models of diffusion coefficients contains laboratory studies of 33 concrete mixes. The partial results describing the relationship between surface and volume resistivity and their development in time are available in (Ghosh et al., 2014). Due to the laboratory origins of this data it can be assumed that the dispersion of measured parameters will be smaller than for real structures. Despite this, it is possible to rely on this data in the case of the non-availability of a more suitable collection of data. If we dismiss the dispersion, the average value of the diffusion coefficient and the curing coefficient can be a very useful average. An interesting aspect of the statistical analysis of data (Ghosh et al., 2014) carried out by the authors in (Konečný&Lehner, 2014) is the finding that the spread of the diffusion coefficient describing standard deviations changes in proportion to the change in the average value of the diffusion coefficient in time.

The data obtained in the 1990s on tens of bridges in the North East of the United States is contained in the survey reports of (Sohanghpurwala&Scannell, 1994). The data was used to calculate the diffusion coefficient of ordinary concrete during the application of probabilistic assessments. (Tikalsky, 2003, Konečný et al., 2007). The diffusion coefficient was calculated on the basis of the concentrations of chlorides at reinforcement level and an estimate of the concentration at the surface of the bridge deck based on information on the intensity of salting of roads from the report by (Sohanghpurwala&Scannell, 1994). Due to the fact that the database also contains information about the resistivity of samples it is also possible to verify the calculation of the diffusion coefficient on the basis of measurement of resistivity by means of the method explained in the subchapter 4.4.

### 4.3 Analysis of the chloride profile

So that it is possible to obtain the diffusion coefficient it is necessary subject the concrete sample to the action of a known concentration of chloride solution for a definite period of time and then obtain the concentration of chlorides at a number of depths. Due to the fact that in order to achieve
a constant flow a long time is needed (years to tens of years) it is possible to obtain the diffusion coefficient for constant flow using Fickovs 2nd law /7/ (Hooton et al., 2001).

The measured values of concentration can be interpolated on a curve for the relevant diffusion coefficient Dc (see scheme on Figure 7), and allow the calculation of the effective diffusion coefficient.

![Diagram](image)

**Figure 7:** Schematic dependence of the concentration of chlorides Cₓ on the depth x in the time t.

A modified NordTest NT Build 443.is chosen to analyse the diffusion coefficient. Concrete samples are submerged into a saline solution for a suitable period (at least 35 days). This test of natural diffusion gives, thanks to a very high gradient of values of the diffusion coefficient D NSSD. (non-steady state diffusion coefficient), enough data for the creation of a curve of the measured chloride profile. That is then interpolated by the method of smallest squares for an equation. /9/ If a suitable tool is not available for sanding the surface including the collection of concrete dust in accordance according with NT Build 443, the approach can be modified by the taking of a chloride profile drilled out in accordance with AASHTO T259. In relation to the taking of samples by drilling however it is necessary to lengthen the period of exposure to the salt solution so that the differences between the individual chloride layers are as distinct as possible.

### 4.3.1 Preparation of samples -

- Samples of an adequate diameter are used for the test, approximately 100 mm and higher approximately 60 mm.
- Newly concreted samples are left cure for 28 days in a bath of lime water (lime-water). The bath is sealed so that it is airtight and filled up to its edge so that carbonization of the water does not occur.
• The tested surface is cut so that it is smooth and clean.
• The untested surfaces are closed with an epoxide coating.
• The samples are placed in a solution of 165 g NaCl per litre of water for a period of at least 90 days. The container containing the samples is filled with the solution up to the edge and an airtight seal is made.
• Once a week the saline solution is mixed and once every 5 weeks the solution is changed.

![Schematic drawing of the action of the saline solution on a concrete sample.](image)

• The taking of samples whether sanded or drilled out must be carried out at a suitable distance from the edge of the sample.
• Concrete dust is obtained from several layers so that it is covered by a layered concentration of chlorides over its height. NT Build 443 states at least 8.
• According to NT Build 443 it is necessary to obtain at least 5g of concrete dust from each layer.

4.3.2 Analysis of samples

The amount of chlorides can be determined using potentiometric titration (see for example ČSN EN 14629, NT Build 208, AASHTO T260).

• The chloride content can be determined according to the ČSN EN 14629 standard (Specifying the chloride content in hardened concrete) by the potentiometric titration of a measured solution of AgNO$_3$.
• The potentiometric titration curve is evaluated with the aid of a first derivative, where the equivalence point is determined which corresponds to the consumption of the titration agent AgNO$_3$.
• Then it is possible to calculate the content of Cl- in a sample as a percentage ratio to weight.
4.3.3 Calculation of diffusion coefficients

The diffusion coefficient can be determined iteratively with the aid of an analogy to the method of smallest squares. The parameters sought are the diffusion coefficient $D_c$ and the concentration of chlorides at the surface $C_0$. Through a regressive analysis the sum is minimalized:

$$ S_j = \sum_i^N \Delta C^{2(i)} = \sum_i^N \Delta(C_{m(i)} - C_{c(j,i)})^2 $$

where:

$S(j)$ sum of the squares of the $j^{th}$ step of the iteration, which is meant to minimalize $[(\% \text{ weight})^2]$,  

$N$ number of tested layers [...],  

$DC(i)$ the difference between the measured and calculated concentration of chlorides in the n-th layer [\% by weight],  

$C_{m(i)} \rightarrow$ measured concentration of chlorides in the i-th layer [\% weight],  

$C_{c(j,i)} \rightarrow$ calculated concentration of chlorides in the j-th step of the iteration in the i-th layer [\% weight].

The concentration of chlorides is calculated with the aid of the relationship /9/, whereas the diffusion coefficient $D_c$ and the surface concentration $C_0$ are chosen after the first iteration. The time $t$ corresponds to the period of exposure to the chloride solution. An alternative solution while limiting the number of points on the chloride profile is iteration on the basis of comparing neighbouring concentrations. (see for instance Olek et al., 2002 or Chejarla, 2008):

$$ \frac{C_{z1,t}}{C_{z2,t}} = \left[ 1 - \text{erf} \left( \frac{z_1}{4D_c t} \right) \right] / \left[ 1 - \text{erf} \left( \frac{z_2}{4D_c t} \right) \right] $$

where:

$C_{z1,t}$ the concentration of chloride ions [%] (expressed as a percentage of all materials with cement characteristics) and in time $t$ [years] depth $z_1$ [m] (depth $z_2$ then corresponds to the concentration $C_{z2,t}$),  

$D_c$ the effective diffusion coefficient [m$^2$/s],  

$t$ time of exposure [s]

4.4 Analysis using electrical resistivity

The diffusion coefficient can be deduced with the aid of the resistance of concrete to the passage of electrical current (electrical resistivity) $\rho$ (AASTHTO TP-95, Ghosh,2011). The determination of
this parameter is extremely fast in comparison with penetration tests (AASHTO T260, NT Built 443) and the accelerated chloride penetration tests (RCPT, AASHTO T277, ASTM C1202). Wenners probe is used for testing (See Figure 9), which uses four electrodes for measurement, spaced at approximately 5 cm. The external electrodes apply electrical current and the internal electrodes measure the difference in voltage. This method of measurement unfortunately reports a significant dispersion both as a result of the heterogeneity of the tested material but also the effect of the chosen pressure on the measuring probe.

Figure 9: Scheme of the principles of measurement of electrical resistance to the use of Wenner probes, the external electrodes maintain the current and the internal electrodes measure the change in electrical potential. If the tests are carried out on cylinders the result is analogical to the measurement of volume resistance. A corrective relationship allowing the consideration of testing resistivity on a cylindrically shaped sample is given by Morris (1996). Ghosh (2011) points to similar results obtained in an accelerated test of chloride penetration and measurement of electrical resistivity. Ghosh (2011) verifies the possibility of measuring electrical resistivity as an effective tool for the calculation of the diffusion coefficient.

Electrical resistance measured by an inverse parameter - conductivity can be more precisely verified for the total volume of the sample by a conductivity meter (bulk conductivity meter). The result measured on cylinders gives greater accuracy and is not burdened by the shape of the sample. Ghosh et al. (2014) also compare the results of the measurement of the passage of electrical current through cylinders obtained with the aid of surface resistivity and volume conductivity. The electrical characteristics expressed in the form of electrical charge passed can also be measured with the help of the above mentioned RCPT test (AASHTO T277, ASTM C1202).

4.4.1 Calculation of diffusion coefficients

For porous materials such as concrete the diffusion coefficient according to Nernst-Einstein is given in the relationship /17/which describes the relationship between electrical resistivity and the diffusion of ions (Lu, 1997):
In relationship (17) they are:

- $D$ diffusion coefficient [m$^2$/s],
- $R$ universal gas constant [J/K.mol],
- $T$ absolute temperature [K],
- $Z$ valence of ions [-],
- $F$ Faradays constant [C/mol],
- $t_i$ transport number of chloride ions [-],
- $\gamma$ activity coefficient of chloride ions [-],
- $C_i$ the concentration of chloride ions [mol/m$^3$],
- $\rho_{BR}$ volume electric resistivity [Ωm].

Molar concentration of chloride ions $C_i$ for a water solution can be determined as follows:

$$C_i = \frac{m}{n} \times 1000$$

where:

- $C_i$ molar concentration of chloride ions [mol/m$^3$]
- $m$ the weight of chlorides in 100 ml of suitable solution [m],
- $n$ molal constant [mol].

The coefficient of activity $\gamma_i$ can be taken as 1, which corresponds to one of the first measurements of the electric characteristics of concrete for calculating the diffusion coefficient (Andrade, 1993), or the diffusion coefficient deduced on the basis of the article by Ghosh et al., 2014. However Ghosh (2011) in his work also gives an approach which leads to the calculation this coefficient $\gamma$ through an iterative parameter $I$:

$$I = \frac{1}{2} \sum mZ^2$$

where:

- $Z$ charge of ions [mol/m$^3$]
- $m$ molality [-]

The interactive parameter is then used to calculate the coefficient of activity after the following relationships logarithm has been undone

$$-\log \gamma = AZ^2 \left( \frac{\sqrt{I}}{1 + \sqrt{I}} - 0.2xI \right)$$
where:
\[ A \quad \text{is an empirical constant equal at room temperature to 0.5094 [-]} \]
\[ Z \quad \text{the valence of chloride ions is equal to 1 [-].} \]

### 4.4.2 Preparation and analysis of samples:

- After the samples are concreted they cure in a water bath or in lime water.
- When a lime bath is used the container is filled with a saturated lime solution up to the edge and closed so its air tight.
- The samples are tested under saturated conditions and after drying the test surface.
- Resistance to the passage of current for each of the cylinders is tested lengthways 8x and 2x on each of the four notional sides.

### 4.5 Laboratory experiments

The aim of the experiment is the verification of the diffusion coefficient for high performance concrete used for directly exposed bridge decks as part of the TA02030164 project. Progressive coupled bridge structure with a directly exposed bridge deck. (Simon et al., 2012). Concrete marked HPC 40/50 rec.XXIII. is analysed.

In the first stage values of the diffusion coefficient are obtained based on the measurement of electrical resistivity and for concrete from core drilling. Values for the diffusion coefficient for newly concreted samples are also obtained using the same recipes. In the second phase values of the diffusion coefficient will be obtained from analysis of the chloride profile and for both samples from drilling and newly concreted samples.

#### 4.5.1 Samples

Two sets of samples are prepared for analysis. Core drills from panels are marked FV and were concreted in October 2013. The diameters of the core drills and thus the diameter/length are approximately. 93/185 mm. These core drills are the same mixes as for the directly loaded bridge decks constructed in the yard of the company FIRESTA in Brno-Modrice.
Figure 10: Samples from core drills after saturation (a) and newly concreted samples before measuring resistivity (b)

Figure 11: Core drilling with reinforcement in the lower part.

Newly concreted samples are marked FN and were concreted on 20/9/2014. Their dimensions are: diameter/length = approximately 103/204 mm. Placement in a saturated solution of Ca(OH)2.is chosen for the curing of new samples according to NT BUILD 443.

Newly concreted samples (FN - 6 pieces) and samples from core drilling (FV - 10 pieces) were divided into two groups. One group is intended for analysis of surface resistivity and the second for the analysis of chloride profiles.
4.5.2 Analysis of the chloride profile

For the group of samples intended for testing the concentration of chlorides (FN4, FN5, FN6, FV1, FV 2, FV 3, FV 6, FV 7 and FV 8) a measurement of their dimensions was carried out before they were cut up.(see Figure 12) and control measurements of resistivity. Before being measured the core drills were also placed in lime water so that they were saturated. The saturation of submerged core drills was verified by a check of the difference in weight every second day.

Figure 12: Measurement of newly concreted samples before measuring resistivity.

Samples for chloride profiles were obtained by cutting up cylinders. The upper layer was removed in a thickness of 1 cm and the lower part was further divided into two parts so that the thickness of the tested sample was at least 6 cm, and there was no reinforcement in the sample. Samples for analysis are the central parts of the cut up cylinders on Figure 13.
Samples intended for testing chloride profiles which the surface dries on before painting are shown on Figure 14.

The samples are painted on three sides with two layers of epoxide paint such that the solution of sodium chloride entered only from one side (see Figure 15, Figure 16 and Figure 17).
The surface intended for exposure to saline solution must not be dirty. Due to capillary dripping of epoxide paint into the sample the top centimetre of the sample is cut off. (Figure 18)
After removal of the soiled surface by cutting with a diamond saw on the 15.10.2014 the samples are submerged into a bath of sodium chloride solution. The container is filled up to its edge and closed with an air tight seal (see Figure 19).

Figure 19: Samples submerged in baths and painted with an epoxide coating

After removal of the soiled surface by cutting with a diamond saw on the 15.10.2014 the samples are submerged into a bath of sodium chloride solution. The container is filled up to its edge and closed with an air tight seal (see Figure 19)

Samples of chlorides in the individual layers were taken on the 13.1 and 14.1.2015 which is 90 and 91 days after the placing of the samples in a saline solution. In further analysis for simplicity, account will be taken of an exposure period of 91 days due to the negligible effects on the resulting diffusion coefficient. In the intervening period there was weekly mixing of the saline solution and on 19.11.2014 a change of the saline solution was made.

Figure 20: The drilling of a chloride profile (a) and the drilled out samples from drill core FV3 (b)

An analysis of the chloride concentration after the drilling out was carried out in the laboratory of building materials at FAST VŠB TUO. The samples were drilled out with the aid of a standing drill
in layers of 5mm. The drilling out of a concrete panel FV fixed into a standing drill and beakers and the concrete dust obtained is shown on Figure 20. During the taking of samples in the standing drill the drill bit got blunt a later a hammer drill was used before it was exchanged. When using the hammer drill the drilled out chloride profile was probably dirtied in the upper layers. The concrete dust with chlorides was then broken up and analysed after drying. The chloride content is determined according to the standard ČSN EN 14629. The chlorides in the samples were determined by potentiometric titration of a measured solution of AgNO₃. (See. Figure . 21). In the case of the potentiometric titration curve its first derivation was used to find the equivalence point which corresponded to the consumption of the titration agent AgNO₃. Then the Cl- content in the sample was calculated as a percentage ratio to weight. Chlorides are obtained from the sample taken with the help of short boiling in 100 ml of 5M HNO₃ (Standard ČSN EN 14629). The precision of the test electrodes is at a level of approximately 0.01 % of the weighted percentage of chlorides to the weight of concrete.

Figure . 21: Analysis of the concentration of chlorides - deducting the voltage corresponding to the concentration of chlorides in the solution

The resultant profiles of the concentration of chlorides for the FV core drills are given on Figure . 22, and also in Table 18 the addendum on page 104. The inexact values of samples FV1, 3, 7 and 8 are also entered into this table but are not included in the later calculation of diffusion coefficients.
Figure 22: Chloride concentration profiles from the 13.1 and 14.1. 2015 for core drills in high performance concrete FV.

In the case of one sample a verification was carried out of the concentration of chlorides without it being exposed to the solution and the result did not exceed the precision with which it can be read from the titration curve.

**Calculation of the parameters of diffusion coefficients**

From the chloride profiles two parameters are then calculated by iteration /15/, the surface concentration of chlorides $C_0$ and the diffusion coefficient $D_c$. During the calculation it is assumed that the background concentration of chlorides is zero. This assumption is based on the concentration values of the lowest profiles for most of the samples which reached the limit of measurement precision as well as the measurements in the reference samples which were not subjected to chlorides.

Due to the approximation of the chloride profile in the solution /9/ which does not take into account the curing of concrete in time the value of the diffusion coefficient is overestimated. For illustration the calculation of the diffusion coefficient for bore FV1 is presented. The calculated surface concentration $C_0$ is $1.37$ [% of concrete weight] and the diffusion coefficient $D_{e456}$, determined by iteration according to /15/, equals $3.39 \times 10^{-12}$ [ms$^{-2}$]. after 90 days of exposure to the chloride solution. The diffusion coefficient corresponds to an age of 456 days, due to the fact that the penetration test began on 15.10.2014 at an age of 366 days. Measured values are shown on Figure 23 of chloride profiles and curves approximating the chloride profile after 90 days of exposure.
Apart from the last two points which relate to values with low concentrations, a good clear correlation exists between the approximations calculated according to \(9/\). The values of the other core drills are determined in a similar way. The results including average values of \(D_{c456} = 2.47 \times 10^{-12} \text{[ms}^{-2}\text{]}\) are given in Tab. 1.

Tab. 1: The surface concentration of chlorides and the diffusion coefficient for an age of 456 days for FV core drills.

<table>
<thead>
<tr>
<th>Exposure</th>
<th>Surface concentration (t) [days]</th>
<th>Diffusion coefficient (D_{c456})</th>
</tr>
</thead>
<tbody>
<tr>
<td>FV1</td>
<td>1.38 ([% \text{ weight.}])</td>
<td>3.42 \times 10^{-12}</td>
</tr>
<tr>
<td>FV2</td>
<td>1.28 ([% \text{ weight.}])</td>
<td>2.65 \times 10^{-12}</td>
</tr>
<tr>
<td>FV3</td>
<td>0.98 ([% \text{ weight.}])</td>
<td>1.79 \times 10^{-12}*</td>
</tr>
<tr>
<td>FV6</td>
<td>1.45 ([% \text{ weight.}])</td>
<td>2.44 \times 10^{-12}</td>
</tr>
<tr>
<td>FV7</td>
<td>1.72 ([% \text{ weight.}])</td>
<td>2.60 \times 10^{-12}*</td>
</tr>
<tr>
<td>FV8</td>
<td>1.56 ([% \text{ weight.}])</td>
<td>2.05 \times 10^{-12}*</td>
</tr>
<tr>
<td>Diameter</td>
<td>2.49 \times 10^{-12}</td>
<td></td>
</tr>
</tbody>
</table>

Note: The values marked in red with a star are calculated on the basis of less than six points of the profile or show an anomaly in the measured profile (see Table 18).

The above mentioned diffusion profile is obtained for a high performance concrete where the
average value \(2.49 \times 10^{-12} \text{ m s}^{-2}\) can be considered at an age of 456 days as higher from the point of view of HPC. The values mentioned can be compared with the results obtained in the testing sets of analysis of chloride profiles carried out on ordinary concrete C30/37 XF4 prepared in a local concrete plant. The tests were carried out in the same FAST VŠB-TUO laboratory as part of conceptual development (Konečný et al., 2012).

Here it is appropriate to mention the differences between tests on high performance concrete (cylinder samples) and plain concrete. (cubes). The concentration was prepared just once and wasn't renewed for 5 weeks, the solution was not mixed. These differences lower the calculated value of the diffusion coefficient because during the testing chlorides transfer from the container to the sample and the concentration of chlorides in the solution lowers. If mixing is not carried out the solution can form layers of varying thickness and a lower value of concentration at the surface. A test was also carried out on newly concreted samples which were still in a stage of intensive curing/decline in their diffusion coefficient. Figure 24(a) shows the prepared set of cubes with a painted coating of epoxide resin. The samples were submerged on 31.8. 2012. An analysis was carried out on four cubes from this set of submerged samples and two sets of results are used for comparison due to non maintenance of the solution. Sacks of concrete dust beside the cubes are marked for work purposes as KXX-94 and shown on Figure 24(b). The number near the set represents the length of exposure to the sodium chloride solution.

![Figure 24: Samples painted with epoxide resin (a) samples of concrete dust from the profile first cube analysed (b)](image)

On Figure 25 the resulting values of the chloride profile are listed including an approximation for the sample which was subjected to the action of chlorides for 94 days. The second sample analysed was in the solution for a period of 158 days. The values of concentration profiles are given in the addenda and also in Table 19 the addenda on page 104.
The resulting diffusion coefficients obtained by the same approach as for core drills FV are given in Table -2. The values are approximately 2-3 times higher than for high performance concrete. If we consider a curing coefficient of $m = 0.2$ (Boddy et al., 1999) for ordinary concrete with portland cement then it is possible in relation to the relationship/10/ to estimate the size of the diffusion coefficient for an age of 442. In this way it will be possible to generally compare the values of ordinary and high performance concrete for the same ages. An example of the calculation of the diffusion coefficient for cube KXX/21/94 gives the following relationship Table -2. The result for the second cube is also given in:

$$D_c(t) = D_{c_{\text{ref}}} \left( \frac{t_{\text{ref}}}{t} \right)^m = 1.72 \times 10^{-12} \left( \frac{122}{456} \right)^{0.2} = 6.11 \times 10^{-12}$$

In relationship /10/ there are:

- $D_c(t)$ effective diffusion coefficient for the age chosen [m$^2$/s],
- $D_{c_{\text{ref}}}$ diffusion coefficient obtained for an age of 122 days (includes the period for curing in a water bath and the period of exposure to chlorides) [m$^2$/s],
- $t$ age [days],
- $t_{\text{ref}}$ reference period of measurement [days],
- $m$ curing coefficient [-].
Table -2: The surface concentration of chlorides and the diffusion coefficient for an age of 442 days for FV core drills.

<table>
<thead>
<tr>
<th>Exposure t [days]</th>
<th>Surface concentration $C_0$ [% weight.]</th>
<th>Diffusion coefficient $D_{ct}$ [ms$^{-2}$]</th>
<th>$D_{c,456}$ [ms$^{-2}$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>KXX-94</td>
<td>94</td>
<td>1.72</td>
<td>7.95×10$^{-12}$</td>
</tr>
<tr>
<td>KXX-158</td>
<td>158</td>
<td>1.78</td>
<td>7.17×10$^{-12}$</td>
</tr>
<tr>
<td>Diameter</td>
<td></td>
<td></td>
<td>6.05×10$^{-12}$</td>
</tr>
</tbody>
</table>

Now it is possible to compare the average value of the diffusion coefficient for high performance concrete $D_{c,442,HPC} = 2.49×10^{-12}$ [ms$^{-2}$] obtained for an age of 456 days with a value for ordinary concrete (OPC) $D_{c,456,OPC}$ equal to $6.05×10^{-12}$ [ms$^{-2}$]. From an orientational comparison of analyses carried out according to similar methodologies in one laboratory it is clear that ordinary concrete is more permeable to chlorides. This finding is proved firstly by the values of the chloride profile and also the value of the diffusion coefficients. The higher value of the diffusion coefficient is shown by concrete into which aggressive substances enter more quickly as can also be seen on the higher concentration profiles in cube KXX Figure 2694 when compared with FV samples. See the filled in graph on .

![Chloride concentration profiles - FV and KXX](image)

**Figure 26:** Chloride concentration profiles from the 13.1 and 14.1. 2015 for core drills from high performance concrete FV and control samples from ordinary concrete KXX.

**4.5.3 Analysis with the use of electrical resistivity**

The electrical surface resistivity $\rho_{SR}$ was measured on 30. 9. 2014 for all samples saturated in lime water such that it is possible to obtain a separation of the probabilistic occurrence of the given values. For certain samples (FN1-3, FV4,5,9,10) its measurement is also regularly undertaken in

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such a way that it is possible to obtain the coefficient of curing describing the development of $D_c$ in time and in accordance with the relationship /10/. After the first measurement the samples intended for determining the changes in the diffusion coefficient in time are placed in a water filled container.

Measurement of the electrical surface resistivity is carried out at the construction faculty of VŠB-TU Ostrava. on a Resi Wenner probe provided by the company Proceq. (see Figure 27).

![Figure 27: The method of measuring the surface diffusion with the aid of a Resi Wenner probe.](image)

The Resi probe uses four electrodes spaced 5 cm apart for measurement (see Figure 28). Contact between the electrodes and the concrete is ensured by black sponges which are conductive after soaking in water. Samples should be saturated for measurement and dry on the surface.

![Figure 28: Wenner probe electrodes equipped with black contact sponges.](image)

Measurements was carried out on the dates: 30. 9., 14. 10., 28. 10., 13. 11., 2. 12. 2014, 6. 1. 2015, and 10. 2. 2015.

The partial results of the measurement of resistivity are given in Addenda 8.1 V Table 20 lists the resistivity measurement for newly concreted FN samples and an age of 28 days. In Table 21 the results are given for samples of core drills with an age of 351 days. The statistics of the measured FV and FN sets are given in Tab. 4 on page 51 also because of their grouping together with the
results for the calculation of the diffusion coefficient.

**Calculation of the parameters of diffusion coefficients**

Table 3: The surface resistivity $\rho$ for FV4.core drills.

<table>
<thead>
<tr>
<th>Body</th>
<th>Surface resistivity $\rho_{SR}$ [k(\Omega)cm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder</td>
<td>30. 9. 2014 351 days</td>
</tr>
<tr>
<td>FV4</td>
<td>69 69</td>
</tr>
<tr>
<td></td>
<td>74 74</td>
</tr>
<tr>
<td></td>
<td>69 69</td>
</tr>
<tr>
<td></td>
<td>68 70</td>
</tr>
<tr>
<td>Diameter</td>
<td>70.25</td>
</tr>
</tbody>
</table>

The diffusion coefficient is calculated based on following the relationships /17/ - /22/ and the laboratory testing of resistivity as for concrete core drill sample FV4. The surface resistivity $\rho_{SR}$ is measured on 30/9/2014 which corresponds to a sample age of 351 days. The results of the measurement of resistivity are given in Table 3.

The samples from the core drilling were added to the newly concreted samples in a bath of lime water. Saturation with lime solution occurred in the bath.

The results of the measurement are altered according to AASHTO TP-95 by the correction coefficient of $K_{LW} = 1.1$ in light of the saturation with lime water as stated by Ghosh et al. (2014).

In addition, according to Morris et. al (1996), the effect of the shape of the sample and the distance between electrodes is normalized. The shape factor $K$ is equal to 3.05 for a sample diameter $d = 93$ mm, sample length $L = 186$ mm and spacing of electrodes at 50 mm. Volume resistance $\rho_{BR}$ is then:

$$\rho_{BR} = \rho_{SR} \times K_{LW} / K = 70.25 \times 1.1 / 3.05 = 25.34 \ [\text{k}\Omega\text{cm}] = 253.4 \ [\Omega\text{m}]$$

/22/.

The molal concentration of chloride ions $C_i$ is calculated below in accordance with /18/ for a three percent water solution of NACl.

$$C_i = m/n \times 1000 = 30/58.5 \times 1000 = 512.82 \ [\text{mol/m}^3]$$

/23/,

where the molal constant $n$ is 58.5 [mol] and the corresponding weight of chloride in 100 [ml] of water solution is 30 [g].

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The diffusion coefficient can thus be calculated easily after being entered into the relationship/17/. 

\[ D = \frac{RT}{Z^2F^2} \times \frac{t_i}{\gamma C, \rho R} = \frac{8.31446 \times 294}{1^2 \times 96500^2} \times \frac{1}{1 \times 512.82 \times 253.4} = 2.02 \times 10^{-12} \quad [\text{m}^2/\text{s}] /24/, \]

Whereas the parameters \( Z, t_i \) are equal to 1 for the given task, the coefficient of activity \( \gamma \) is chosen as 1 as mentioned earlier.

In cases where a coefficient of activity was considered then it is possible to deduce an interactive parameter according to /19/ as:

\[ I = \frac{1}{2} \sum mZ^2 = \frac{1}{2} x(0.5x1^2 + 0.5x(-1)^2) = 0.5 \quad /25/, \]

where:

- \( Z \) the charge of the chloride ion Na+ and Cl- are equal to +1 and -1.
- \( m \) The molality of a 3% solution of NaCl is equal to 0.5.

The concentration of sodium chloride solution is the same in relationships /23/and /25/at 3% and in accordance with the RCPT test according to ASTM C1202. For which the relationship /17/ was used originally. Later it is put into a logarithmic equation /20/:

\[ -\log \gamma = AZ^2\left[\frac{\sqrt{I}}{1+\sqrt{I}} - 0.2xI\right] = 0.5094 \times 1^2 \left[\frac{\sqrt{0.5}}{1+\sqrt{0.5}} - 0.2 \times 0.5\right] = 0.16 \quad /26/, \]

where:

- \( A \) is the empirical constant equal at room temperature to 0.5094 [-]
- \( Z \) the valence of chloride ions is equal to 1 [-].

After removing the logarithmic equation /20/ \( \gamma = 0.692 \) The calculation of the diffusion coefficient of resistivity /24/ would then be amended.

\[ D = \frac{RT}{Z^2F^2} \times \frac{t_i}{\gamma C, \rho R} = \frac{8.31446 \times 294}{1^2 \times 96500^2} \times \frac{1}{0.692 \times 512.82 \times 253.4} = 2.92 \times 10^{-12} \quad [\text{m}^2/\text{s}] /27/, \]

The diffusion coefficient with an activity coefficient \( \gamma = 1 \) has a value of 2.02 \( [10^{-12} \text{ m}^2/\text{s}] \). In the next section \( Y_i = 1 \) will be used for the calculation of the diffusion coefficient. The effect of the activity coefficient will be discussed further when comparing the results obtained on the basis of resistivity and the analysis of the chloride profiles. and due to the fact that it concerns a directly proportional relationship where the diffusion coefficient can be later amended by dividing...
by the value \( \gamma = 0.692 \).

**Results of the measurement on 30/9/2014**

The values of the diffusion coefficients for other measurement on 30.9.2014 are calculated in the same way as for sample FV4 according to the relationship /24/, \( (\gamma = 1) \) can be viewed in addenda 8.1 (Table 20:.FN - newly concreted samples, Tab: FV - samples from core drills).

The statistical analyses of data in Table 20 and Table 21 are obtained from the parameters of surface resistance \( \rho_{SR} \), the volume resistance and diffusion coefficient \( D_{c,(t)} \) calculated similarly as for FV4 (see /22/- /24/). The results are available in Tab. 4. A spreadsheet in Microsoft Excel is used for the statistical analysis. 8 measurements are obtained from each sample. For newly concreted samples then 8 values are obtained from 6 samples which give a total of 48 data points. For 10 core drills it is a total of 80 data points.

Tab. 4: Statistical parameters for electrical resistivity and diffusion coefficients as measured on 30/9/2014 for newly concreted FN samples and FV core drills.

<table>
<thead>
<tr>
<th>Body</th>
<th>30. 9. 2014</th>
<th>Surface resistivity ( \rho_{SR} ) [kΩcm]</th>
<th>Volume resistivity ( \rho_{SR} ) [kΩcm]</th>
<th>Diffusion coefficient ( D_{c} ) [ms(^{-2})]</th>
</tr>
</thead>
<tbody>
<tr>
<td>FV</td>
<td>351 days 80 samples</td>
<td>Diameter 84.48</td>
<td>30.47</td>
<td>1.70×10(^{-12})</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Standard deviation 9.80</td>
<td>3.53</td>
<td>0.206×10(^{-12})</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Variation coefficient 0.12</td>
<td>0.12</td>
<td>0.121</td>
</tr>
<tr>
<td>FN</td>
<td>28 days 48 samples</td>
<td>Diameter 33.54</td>
<td>14.07</td>
<td>3.65×10(^{-12})</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Standard deviation 2.14</td>
<td>0.90</td>
<td>0.226×10(^{-12})</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Variation coefficient 0.06</td>
<td>0.06</td>
<td>0.062</td>
</tr>
</tbody>
</table>

From Tab. 4 it is clear that the reference diffusion coefficient for the age of high performance concrete obtained from measuring surface resistance \( D_{c(28)} \) is equal to 3.65×10\(^{12}\) [ms\(^{-2}\)]. The effect of taking into account the activity of chloride ions on the diffusion coefficient is given in Table 5.
Table 5: Allowing for the effect of the activity coefficient of chloride ions on the resulting diffusion coefficient obtained with the help of electric resistivity for newly concreted FN samples and FV core drilling.

<table>
<thead>
<tr>
<th>Body</th>
<th>Date</th>
<th>Diffusion coefficient $D_c$ [$10^{-12}$·ms(^{-2})]</th>
<th>Diffusion coefficient $D_c$ [$10^{-12}$·ms(^{-2})]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\gamma = 1$</td>
<td>$\gamma = 0.692$</td>
</tr>
<tr>
<td>FV</td>
<td>30.9.2014</td>
<td>Diameter: 1.70</td>
<td>2.46</td>
</tr>
<tr>
<td></td>
<td>351 days</td>
<td>Standard deviation: 0.206</td>
<td>0.298</td>
</tr>
<tr>
<td></td>
<td>80 samples</td>
<td>Variation coefficient: 0.121</td>
<td>0.121</td>
</tr>
<tr>
<td>FN</td>
<td>28 days</td>
<td>Diameter: 3.65</td>
<td>5.28</td>
</tr>
<tr>
<td></td>
<td>48 samples</td>
<td>Standard deviation: 0.226</td>
<td>0.327</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Variation coefficient: 0.062</td>
<td>0.062</td>
</tr>
</tbody>
</table>

From Table 5 a rise in the value of the average diffusion coefficient $D_c(28)$ to $5.28 \times 10^{-12}$ [ms\(^{-2}\)], is clear, which is a value close to the usual for ordinary concrete. In the case of annual core drills the average diffusion coefficient is altered by an amount of $2.46 \times 10^{-12}$ [ms\(^{-2}\)].

**Concrete curing coefficient**

Through an analysis of the measured resistivity $\rho_{SR}$ and a calculation of the corresponding average diffusion coefficient for new FN samples in relation to the age of the samples another parameter for the modelling of time related diffusion coefficient for the concrete concerned is obtained. The average diffusion coefficients corresponding to the measuring period are given in Table 6. The curing coefficient $m = 0.388$ is obtained by smoothing the curve /10/ on the measured data with the aid of the method of smallest squares. The slight difference in the reference diffusion coefficient in Tab. 4 (3.65) and in Table 6 (3.64) is given by the differing method of preparation, where for the calculation of the m factor the value $D_c$ underwent a double averaging.

Table 6: The average diffusion coefficient $D_{c(t)}$ deduced from the surface resistivity $\rho_{SR}$ for newly concreted samples FN1-3.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Age $t$ [days]</td>
<td>28 42 57 72 91 126</td>
<td>3.64 3.27 2.66 2.40 2.22 2.20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4.5.4 Comparison of the results of the chloride profile and the measurement of resistivity.

The measured values of the diffusion coefficient from electrical resistivity as calculated for an age of 351 days can be compared with the results of an analysis of the chloride profile to aid in considering the effects of curing. The $m$ coefficient is calculated for the mix concerned at 0.388. By recalculating using the relationship /10/ Table 7 contains the given values for an age of 456 days which corresponds to the age of the core drills when ending the analysis of the chloride profile.

Table 7: Comparison of the diffusion coefficient.

<table>
<thead>
<tr>
<th></th>
<th>$D_{ct} \times 10^{12}$ ms$^{-2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\gamma = 1$</td>
</tr>
<tr>
<td>$t$ [days]</td>
<td>351</td>
</tr>
<tr>
<td>FV1</td>
<td>1.83</td>
</tr>
<tr>
<td>FV2</td>
<td>1.57</td>
</tr>
<tr>
<td>FV3</td>
<td>1.75</td>
</tr>
<tr>
<td>FV6</td>
<td>1.67</td>
</tr>
<tr>
<td>FV7</td>
<td>1.70</td>
</tr>
<tr>
<td>FV8</td>
<td>1.54</td>
</tr>
<tr>
<td>Diameter</td>
<td>1.68</td>
</tr>
</tbody>
</table>

By comparing the values in Table 7 it can be found that the resulting diffusion coefficient from electrical resistivity and the chloride profile come closer the effect of the coefficient of activity $\gamma_i$ /20/ is taken into account in the relationship /17/. It is also necessary to point out that the value of the diffusion coefficient from the chloride profile is conservative which is given by the use of the solution of calculating the concentration of chlorides with a constant diffusion coefficient over time according to /8/ and /15/. Allowing for the effect of curing on the calculation of concentrations during iterative searches for the diffusion coefficient would lead to a calculation of its lowest value which would come even closer to the results from electrical resistivity and $\gamma_i = 0.692$. In addition it is appropriate to state that the assumption of zero concentration on the concrete surface also leads to a more conservative calculation of the concrete parameter sought.

At the end of the section describing the calculation of the diffusion coefficients it is important to repeat the result of the comparison with the ordinary referential concrete. If we ignore the question of fine tuning both the tests of electrical resistivity and the analysis of the chloride profile then the most significant indicator of the quality of the high performance concrete concerned is a comparison with the test pair cubes of ordinary concrete. Here it is shown that the values of the diffusion coefficient for ordinary concrete are higher than was expected. So a clear difference can
be seen between ordinary concrete and high performance concrete both as concerns the calculated value of the diffusion coefficient and also from the point of view of salt concentration in the tested profiles. The tested concrete from the FV core drills shows a lower diffusion coefficient and lower salt concentration in the profiles than the reference cubes from ordinary concrete.

4.6 Distribution of diffusion coefficients

This section concerns the preparation of the division of diffusion coefficients from the point of view of their application in related probabilistic assessments in chapter 6. The preparation is concerned with a probabilistic description of the dispersion of diffusion coefficients in time and also deducing the division of diffusion coefficients for high performance concrete according to recipes (Simon et al., 2012). Even though when analysing the division of diffusion coefficients obtained from electrical resistivity the work is done with data with an activity coefficient of \( Y_i = 1 \) the resulting division is not burdened by errors because the aim of this section is the preparation of a non-dimensional distribution of the dispersion of the parameter sought which characterizes the permeability of concrete to chlorides for a wider range of concretes when knowing its basic statistics.

4.6.1 Timescale

During a statistical analysis of the behaviour of 32 selected high performance mixes (Ghosh et al., 2014) and one reference mix from ordinary concrete (Konečný&Lehner, 2014) it was found that the time related distribution of the diffusion coefficient could be described with the help of a variation coefficient which had a constant value for ordinary concrete regardless of the age of the sample. In the case of high performance mixes oscillations occurred in the variation coefficient around the average value and yet the use of the average variation coefficient over the measured analysis in time is found to be an advantageous method for high performance concretes as well. The statistical parameters for the three selected mixes are given in Table 23-Table 28 in the addenda on page 112. The approach for generating random values of the diffusion coefficient in time while respecting the dispersion and its reduction in time ties into relationship (10). The first step is a calculation of the nominal value of the diffusion coefficient over time \( t \):

\[
D_{c,\text{nom}}(t) = D_{c,28} \cdot \left( \frac{t_{28}}{t} \right)^{m/28},
\]

where:
\( D_{c,\text{nom}(t)} \) diffusion coefficient for a selected age \([m^2/s]\),
\( t \) curing period \([\text{years}]\),
\( t_{28} \) reference period of measurement for an age of 28 days \([\text{years}]\),
\( m \) curing coefficient \([-]\).

Later it is possible to generate your own diffusion coefficient while respecting the spread variation coefficient:

\[
D_{c,(t)} = D_{c,\text{nom}(t)} + D_{c,\text{var}} \times D_{c,\text{nom}(t)}
\]

where:
\( D_{c,\text{nom}(t)} \) diffusion coefficient for the selected age \([m^2/s]\),
\( t \) curing period \([\text{years}]\),
\( D_{c,\text{var}} \) dispersion of diffusion coefficients \([m^2/s]\).

In the relationship for generating dispersions of diffusion coefficients the spread \( D_{c,\text{var}} \) can be described as

\[
D_{c,\text{var}} = N(0,1) \times D_{c,\text{VarCoeff}}
\]

where:
\( D_{c,\text{VarCoeff}} \) average variation coefficient of the dispersion of the diffusion coefficient \([-]\),
\( N(0,1) \) normal distribution around an average of 0 and a standard deviation of 1 \([-]\).

The normal distribution can be replaced by a suitable normalized distribution \( D_{c,\text{var},\text{n}} \) describing more exactly the distribution function described. The approach in preparing the normalized distribution function is given in 4.6.4.

### 4.6.2 Ordinary concrete

The primary plan was to normalize the existing distribution prepared for the probability analysis (Tikalsky, 2003, Konečný, 2007) on the basis of a database of north American bridges (Sohanghpurwala&Scannell, 1994). The distribution was used in a probabilistic analysis of an ideal reinforced concrete bridge deck without cracks (Tikalsky et al., 2005) and also with cracks or an epoxide coating (Konečný et al., 2007). See Figure 29 for respective histogram.
After verifying the suitability of the distribution an analysis was carried out of the correlation between the measured resistivity, the theoretical amount of chlorides applied, diffusion coefficient and amount of salt at the level of reinforcement. The result is given in Table 8.

Table 8: An analysis of the correlation between selected parameters related to the calculation of diffusion coefficients for core drills on bridges in North America (Sohanghpurwala&Scannell, 1994).

<table>
<thead>
<tr>
<th></th>
<th>Resistivity $\rho$</th>
<th>Diffusion coefficient $D_c$</th>
<th>Surface concentration of salt $C_0$</th>
<th>Concentration of salt at the level of reinforcement $C_{z,t}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistivity $\rho$</td>
<td>1.00</td>
<td>-0.04</td>
<td>-0.17</td>
<td>-0.19</td>
</tr>
<tr>
<td>Diffusion coefficient $D_c$</td>
<td>1.00</td>
<td>0.16</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Surface concentration of salt $C_0$</td>
<td></td>
<td>1.00</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>Concentration of salt at reinforcement $C_{z,t}$</td>
<td></td>
<td></td>
<td>1.00</td>
<td></td>
</tr>
</tbody>
</table>

It can be assumed from the relationship between resistivity $\rho$ and the calculated diffusion coefficient $D_c$ where the correlation approaches the value 0 that these two parameters are mutually independent which they shouldn't be (AASTHTO TP-95, Ghosh, P. (2011). A certain relationship between resistivity and the concentration of salts at the level of reinforcement, although small, can be observed in the amount of -0.19. This negative correlation indicates that the larger the resistivity the more resistant the concrete is to chlorides and there are fewer chlorides in the core drills. There is a similar relationship between the diffusion coefficient and the amount of chlorides (0.16) Here it is a dependence given by the method of calculating $D_c$. The parameter directly depends proportionately on the quantity of salt on the surface and at the level of reinforcement. It can thus be deduced that the histogram listed in Figure 29 is not the most suitable for the description of ordinary concrete.
4.6.3 High performance concrete

Figure 30: Histogram of the diffusion coefficient of high performance concrete according to the recipe of SIMON, P. et al. (2012) analysed by the program HistAn (Janas et al., 2008).

The diffusion coefficient obtained from the measurement of electrical resistivity in high performance concretes prepared based on the recipe of (Simon, et al. 2012) are listed in Table 21 the addenda on page 106. On the basis of 80 measured values from 10 samples of 351 day old high performance concrete marked FV, an analysis is carried out using the program HistAn (Janas et al., 2008, 2010) of the distribution of the probability of the occurrence of the measured values whose statistics are listed in Tab. 4. The statistical parameters and drawing of a histogram from the analysis of the HistAn program can be studied in Figure 30.

The measured data are also interpolated in a selected Gumbel distribution (see Figure 31), which most closely described the measured data. The coefficient of conformity is 0.698, which is a good conformity. The value of the coefficient of conformity can vary in the range of 0-1. The value 1 represents the maximum conformity of the measured data and the distribution used. The interpolated cut off Gumbel distribution in 256 classes is shown on Figure 31.
Figure 31: The Gumbel distribution showed a greater conformity with the measured data of the diffusion coefficient for high performance concrete according to the recipes of Simon et al. (2012) with a coefficient analysed by the program (Janas et al., 2008).

This proposed Gumbel distribution prepared for samples aged 351 days is normalized for other use as will be further explained.

4.6.4 Normalized concrete distribution

The reasoning behind the normalization process is thus the creation of a non-dimensional distribution which can be used for the description of a time dependent process of a random variable diffusion coefficient according to the relationship /7/ with relation to /29/, and not only in relation to the actual question being resolved of high performance concrete for directly exposed bridge decks. This reasoning began in relation to the study of the behaviour of dispersions of the diffusion coefficient for 33 concrete mixes (Konečný & Lehner, 2014, Ghosh et al., 2014).

The histogram shown on Figure 31 describes the distribution of the diffusion coefficient for an age of 351 days and is altered by re-counting the outlying values in relation to the average and the standard deviation. The minimal value of the histogram is altered as follows:

\[ Hist_{min \text{var}, n} = \frac{(Hist_{min} - \mu_{Hist})}{\sigma_{Hist}} \]

whereas the maximal value of the histogram is modified as follows:

\[ Hist_{max \text{var}, n} = \frac{(Hist_{max} - \mu_{Hist})}{\sigma_{Hist}} \]
where:

- $Hist_{\text{min/max}}$: minimum/maximum of the original distribution [-],
- $\mu_{Hist}$: average of the original distribution [m$^2$/s],
- $s_{Hist}$: standard deviation from the original distribution [m$^2$/s],
- $Hist_{\text{min/max var,n}}$: minimum/maximum of the normalized distribution [-].

Figure 32: The normalized Gumbel distribution prepared for the description of the dispersion of the diffusion coefficient in relation to the histogram on Figure 31 and relationships /31/ and /32/.

The normalized distribution can later be modified by the real dispersion for a known variation coefficient, by multiplying the normalized distribution by the known variation coefficient. In the case being followed the data variation coefficient from core drills is given in Tab. 4. $D_{c,varCoeff} = 0.121$. The distribution used for the probability analysis can thus be obtained as follows:

$$D_{c,var} = D_{c,var,n} \times D_{c,VarCoeff}$$  /33/.

The resulting distribution looks the same as on drawing Figure 32 along with the fact that a movement in the outlying values occurs. Due to the fact that only the scale on the horizontal axis of the histogram changes, this histogram is shown only during the probability analysis on page 78 (see Figure 45). This histogram has a variation coefficient according to the measurement given in Tab. 4. It will be used in a probabilistic analysis in relation to a recalculation according to the time dependent value of the diffusion coefficient for a concrete age according to the relationship /30/ and /29/.

The parameters of the original distribution measured for an age of 351 days $D_{c,151}$, normalized histogram $D_{c,var,n}$ and diffusion coefficient dispersion $D_{c,var}$ which will be used further are given in table Table 9.
Table 9: The parameters of the histogram for a description of the dispersion of the diffusion coefficient.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Dispersion</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>The dispersion of the diffusion coefficients $D_{c,151}$ [10^{-12} m^2/s]</td>
<td>1.15-4.02</td>
<td>Gumbel distribution ($\mu$=1.7, $\sigma$=2.07), Figure 31</td>
</tr>
<tr>
<td>Normalized dispersion of the diffusion coefficient $D_{c,\text{var,n}}$ [-]</td>
<td>-2.7-11.2</td>
<td>Gumbel distribution ($\mu$=0, $\sigma$=1), Figure 32</td>
</tr>
<tr>
<td>Dispersion of the diffusion coefficient $D_{c,\text{var}}$ [10^{-12} m^2/s]</td>
<td>-0.328-1.36</td>
<td>Gumbel distribution ($\mu$=0, $\sigma$=0.121), Figure 45</td>
</tr>
</tbody>
</table>

A histogram describing the dispersion of the diffusion coefficient $D_{c,\text{var}}$ shown on Figure 45 and described in the last sentence of table 10 is also suitable for use for a description of ordinary concrete as the origin of the histogram on Figure 29 is disputable as was explained above a the laboratory origin of the mix in the study (Ghosh et al., 2014) in Table - 23 the addenda on page 112 does not guarantee a dispersion corresponding to concrete from a concrete plant.
5 2D FEA DIFFUSION MODEL TAKING INTO ACCOUNT THE EFFECTS OF CRACKS

This chapter 5 concerns a description of a 2D numerical model on the basis of the finite element method (FEA) which serves to analyse the amount of chlorides at the reinforcement level. It contains a description of the model and the types of tasks which can be solved and includes deterministic samples of solutions.

5.1 Basic description of the 2D FEA model

The applied 2D FEA model (Lehner et al., 2014) serves to solve Fickov’s second diffusion law (relationship /7/) using the computer tool compatible with the Matlab environment. The model focusses on the transport of chloride ions through a reinforced concrete bridge deck with a transverse crack and on an estimate of the concentration of chlorides at the reinforcement level or in places with damage to the epoxide coating of the reinforcement. The model allows the inclusion of cracks in the concrete and also damage to the waterproof insulation under the asphalt coating. The numerical model is extension of (Tikalsky et al., 2005, Konečný et al., 2007, Lehner, 2013).

Table 11: The construction and model parameters of the alternative selected for the deterministic analysis of the initiation of corrosion in reinforced concrete bridge decks.

<table>
<thead>
<tr>
<th></th>
<th>Cement mix</th>
<th>Crack in concrete</th>
<th>Asphalt covering and waterproof insulation</th>
<th>The effect of concrete curing</th>
<th>Protection of reinforcing</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1E</td>
<td>100TII</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Epoxy-coating</td>
</tr>
<tr>
<td>P1B</td>
<td>100TII</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Unprotected</td>
</tr>
<tr>
<td>P2E</td>
<td>100TII</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Epoxy-coating</td>
</tr>
<tr>
<td>P2B</td>
<td>100TII</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Unprotected</td>
</tr>
<tr>
<td>P3E</td>
<td>45TII-V/35G100S/20F</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Epoxy-coating</td>
</tr>
<tr>
<td>P3B</td>
<td>45TII-V/35G100S/20F</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Unprotected</td>
</tr>
</tbody>
</table>
The diffusion model based on Fickovs 2nd law is based on a four loop isoperimetric element. The theoretical derivative of the element is given in Addendum A in the work of Lehner (2013). The model is in the evaluation stage and computed time to corrosion initiation compared with theoretical solution /11/ is currently underestimated. The results are suitable for indicative comparison of selected protection strategies.

Examples of the assumed solution, the method of causing cracks in concrete, defects in epoxides or the waterproof insulation under the asphalt layer are given below Examples of deterministic solutions then follow. The variants considered by method of protection of reinforcement and the bridge decks are given in Table 11.

5.1.1 Assumptions and principles of solutions

In this section the assumptions and simplifications which the model is based on: are described:

- The diffusion of ions is the only mechanism by which chlorides move.
- The reinforced concrete bridge deck is homogenous and fully saturated across the cross section of the drawn out liquid phase.
- An "obvious" diffusion coefficient is used with the possibility of evaluating the effect of curing of concrete according to the relationship /10/.
- The diffusion coefficient is for the purposes of the calculation does not change in relation to the temperature.
- The crack is a modelled form of modification of the FEA mesh where the network of final elements also remains connected in the area of the crack.
- A crack can be modelled in two ways:
  - in the form of border conditions - the concentration on junctions in cracks is the same as on the surface, whereas the lowest junction of the crack in the FEA mesh corresponds to the depth of the crack in concrete,
  - modification of the geometry of the network (narrowing the element corresponding to the crack) and the form of change of the diffusion coefficient in the crack.
- The concentration on the surface is constant in time (it also applies for the crack if it is given in the form of a concentration on the junctions).
- The cracks form at right angles to the longitudinal reinforcement inserts.
• The slab has an unending length (adiabatic marginal conditions on the left and right margin)

• The depth of the modelled slab is equal to the depth of the bridge decks considered (adiabatic marginal conditions on the lower margin of the slab which is not infinitely deep as in Cracks solution. /7/)

• The width of the slab is increased so that it is possible to underpin the effects of the occurrence of cracks at the edge of the model.

• The concentration of chlorides around the reinforcement is regarded as the same as on the level of the top of the reinforcement.

• The concentration of chlorides in the background of the concrete is 0 percent.

• The asphalt covering is regarded as perfectly permeable - the concentration of chlorides on the surface of the asphalt is also considered at the point of waterproofing.

• Apart from the assumed areas of damage, the waterproof insulation is regarded as being perfectly waterproof.

• At the points where the waterproof insulation is damaged the concentration at the surface of the concrete is equal to the concentration at the surface of the asphalt layer.

• Apart from the assumed localities of damage the epoxide coating on the steel reinforcement is regarded as totally impermeable.

• The concentration of chlorides at the point of damage to the epoxide-coating is equal to the concentration in the concrete at the corresponding point.

• The chloride threshold for the initialization of exposed steel reinforcement at the point of damage of the covering is the same as the chloride threshold of unprotected steel reinforcement.

• In places where the epoxy-coating is not disturbed, corrosion does not occur.

5.1.2 Modelling of cracks

A crack can be introduced in the form of a marginal condition of surface concentration to a junction corresponding to the assumed positioning of the crack as for the models used in (Konečný et al., 2007, Marsavina et al., 2007). A more advanced approach following on from the work of (Bentz et al., 2013) is the second possible method of introducing a reduced diffusion coefficient taking into account the size of cracks and the modification of the FEA mesh. Originally a change of the diffusion coefficient was not considered but due to the aim of comparing a directly exposed bridge
deck with a bridge deck protected by waterproofing under an asphalt layer it was decided to prepare a model with an altered diffusion coefficient. In the following examples the approach with a change in the diffusion elements of the crack elements is used.

In the related examples of the penetration of chlorides into a crack, $D_{cr}$ is reduced in relation to the width of the cracks and the reference diffusion coefficient $D_{c,28}$ for a 28 day concrete (Djerbi et al., 2008). Where crack widths are less than 30µm the diffusion coefficient of concrete is used for the crack, whereas for a crack greater than 80 µm the diffusion coefficient of the media in the crack is used $D_{c,crack,max} = 14 \times 10^{-10}$ [m²/s] (Djerbi et al., 2008). The intermediate points of the diffusion coefficient are interpolated:

$$30\mu m < C_{crkw} < 80\mu m : D_{c,crack} = \frac{(D_{c,crack,max} - D_{c,28})}{50 \times (C_{crkw} - 30)} + D_{c,28}$$

The scheme of positioning starting cracks is shown on Drawing 33 The crack is defined by its depth and starting position $Crack_i$. The model contains overlaps on all sides so that an accumulation of concentrated chlorides does not occur when modelling the marginal cracks.

![Diagram](image)

Drawing 33: Scheme of the model of the bridge deck with cracks and unprotected steel reinforcement. Notation: Width $e$ – model edge, Width $e$ – investigated part.

The model allows the modelling of a greater number of cracks with the aid of the crack spacing parameter. If the spacing of cracks is less than the width of the model (1m) the starting position of the cracks decides the number of cracks.
The following approach is chosen for the calculation of the spacing and width of cracks related to the area of cracks on the individual bridge deck areas.

\[
P_{\text{crk}} = C_{\text{crks}}^{-1} \times C_{\text{crkw}} /1000
\]

where:

- \( P_{\text{crk}} \) - probability of the occurrence of cracks in the unit areas [m\(^{-2}\)]
- \( C_{\text{crks}} \) - crack spacing [m].
- \( C_{\text{crkw}} \) - crack width [mm].

### 5.1.3 Modelling the epoxide coating

An analysis of the concentration of chlorides when using the protection of steel reinforcement with a protective epoxide coating involves the determination of concentration of chlorides at the point of damage of the epoxide coating (holiday). A scheme is given on Figure 35 of the modelling of defects in the epoxide protection of the steel reinforcement. Damage to the epoxide protection is defined as the number of defects per metre run of reinforcement. The spacing of the damage and the position of the first defect is determined from the left hand edge of the model.
Figure 35: Scheme of the model of the bridge deck with a crack and steel reinforcement protected by an epoxide coating. Notation: Width\textsubscript{e} – model edge, Width\textsubscript{c} – investigated part. Mash\textsubscript{i} – initial position of a holiday in epoxy coating.

### 5.1.4 Modelling defects in the waterproof insulation under the asphalt layer

The prepared model assumes water permeable asphalt so it assumes a very fast exposure to the waterproof membrane by the surface concentration of chlorides. Only concrete is thus modelled from the point of view of diffusion. It is assumed that defects/cracks will occur in the waterproof insulation. These cracks run through the model, the same as in concrete, through the whole unit area of the model.

A defect in the waterproof insulation is introduced in the form of marginal conditions - the concentration at junctions on the upper surface of reinforced concrete bridge decks at the point corresponding to the assumed cracks or at the location of assumed cracks. If there are more cracks their spacing is the same. A progressive growth of the cracks with time is assumed. The scheme of the damage to the waterproof insulation is shown on Drawing 36.
Drawing 36: Scheme of the model of the bridge deck with a crack in the bridge deck and waterproof insulation under an asphalt coating. Notation: Crack_{HI,i} – počáteční pozice poruchy v hydroizolaci, - starting position of the crack in the waterproof insulation Crack_{HI} Spacing – Spacing of the cracks in the waterproof insulation

For the calculation of the spacing and width of the cracks in the waterproof insulation a similar method is used as for the calculation of the width of cracks in concrete. /35/:\

\[
\Delta P_{crck,HI} = C_{crcks,HI}^{-1} \times \Delta C_{crckw,HI} / 1000
\]

where:

\( \Delta P_{crck,HI} \) a rise in the probability of the occurrence of cracks in the individual areas in a year [m²/year]

\( C_{crcks,HI} \) spacing of cracks [m],

\( \Delta C_{crckw,HI} \) growth in the spacing of cracks per year [mm/year].

Relationship /36/ describes the dependency of the growth in the area of damaged waterproof insulation \( \Delta P_{crck,HI} \) on the growth of the width of the cracks per year \( \Delta C_{crckw,HI} \). The area of damaged waterproof insulation \( P_{crck,HI,t} \) is then given below for a given structure age \( t \)

\[
P_{crck,HI,t} = C_{crcks,HI}^{-1} \times \Delta C_{crckw,HI} / 1000 \times t
\]

The spacing of cracks is selected in the model and then the width of cracks is added. The crack is systematically placed around selected starting positions \( Crack_{HL,i} \), and grows every year. The growth of cracks is modelled symmetrically on both sides. The growth on each side is always half of \( \Delta C_{crckw,HI} \).

### 5.1.5 Types of solvable tasks

In the following subchapters a solution is shown for a selected problem for a typical requirement given in Table 11. In subchapter 5.1.6 the input parameters are described and the finite element
network is shown. In addition it is possible to study the graphic inputs of 2D concentration and curves describing the initiation of corrosion for both variation P1E/B, and also for the variation P2 E/B.

Due to the analogy between heat and diffusion problems a 4 junction isoperimetric element suitable for the solution of both heat and diffusion tasks. A slab of 0.23m is vertically divided into 23 elements with dimensions of 10 by 10 mm. The time step of the time related analysis (transient analysis) is controlled automatically on the basis of the size of the element and the diffusion coefficient.

5.1.6 An analysis with a crack in a reinforced concrete bridge deck using ordinary concrete

In the model with cracks and ordinary concrete (alternative P1B according to Table 11), the referential diffusion coefficient $D_{c,28}$ within 28 days of concreting is considered as $5.59 \times 10^{-12}$ [m$^2$/s] for ordinary concrete, which corresponds to the average value of the 100TII mix (Ghosh et al., 2014, see Table - 23 in the addenda on page 112). The aging factor is considered as $m = 0.26$. The initial concentration on the concrete surface (marginal conditions at relevant junctions) $C_0$ is chosen as 0.6 percent (weight of material with cement properties) of soluble chloride ions (Tikalsky, 2003, Kurgan, 2003). The value of the concentration at which the initiation of corrosion $C_{th}$ occurs is 0.2 (Tikalsky, 2003, ACI 222). The clear cover of concrete reinforcement above the upper layer of reinforcement is 0.05 m The steel reinforcement is unprotected. A crack is positioned at the halfway point of the width of the model which has a value of 1 m. The depth of the crack is chosen as 0.025 m and its width is 0.3 mm. Due to the modelled width of the crack of 3 mm (300 µm) the diffusion coefficient in the crack is introduced as $D_{c,crack} = 14 \times 10^{-10}$ [m$^2$/s] (see viz /34/).

![Figure 37](image)

**Figure 37:** The concentration of chloride ions in a concrete bridge deck with a crack made from ordinary concrete without taking into account the effect of the curing of the concrete. Deterministic solution - alternative 1a for an exposure period $t = 20$ years (a) shows the FEA meshed elements and concentration in the form of bands whereas (b) shows the isoline of the concentration of chlorides.
The action of cracks allowing the movement of the chloride ions both in the vertical and the horizontal direction is illustrated in graphical output from the FEA analysis (see Figure 39). Chlorides can travel to the reinforcement faster in an area with cracks. The progress of the development of chloride concentrations in time are for even decades given in addendum (see Figure 59 on page 109 and Figure 60 on page 109).

In relation to the 2D analysis of the chloride concentration in time, information is obtained for the alternative with unprotected reinforcement about the highest concentration of chlorides at reinforcement level.

Figure 38: The concentration of chlorides at the level of the reinforcement of a concrete bridge deck with a crack in ordinary concrete taking into account the curing of concrete. Deterministic solution of alternative P1B with unprotected reinforcement (Chloride – chloridy, concentration - koncentrace, time – čas)

Figure 39: The function of reliability for the analysis of durability - the initiation of corrosion in concrete bridge decks with a crack in ordinary concrete taking into account the effect of concrete curing. Deterministic solution - alternative P1B with unprotected reinforcement.
After deducting the time progress of the chloride concentration at the reinforcement level at the point with the greatest concentration $C_{zt}$ and after comparing with the value of the chloride threshold $C_{th}$, the period to the initiation of corrosion $t_i = 5.4$ years is deducted for ordinary reinforcement. See Figure 39.

5.1.7 Evaluation of durability with epoxide protection of steel reinforcement.

A model of the epoxide coating is built into the analysis in the form of the identification of defects in reinforcement. The difference when compared with unprotected reinforcement on the directly exposed bridge deck (alternative P1B) is thus in the definition of the places where the concentration of chlorides at the level of the reinforcement is deducted. For illustration a starting defect in the epoxide coating 5 cm from the left margin of the model is selected and other defects repeat every 20 cm. The concentration of chlorides in places with a defect in the epoxide reinforcement fell and is shown in comparison with the concentration on ordinary reinforcement on Figure 40.

Figure 40: Comparing the effect of protection of the steel reinforcement with the aid of a concentration of chlorides at the level of the reinforcement of the concrete bridge deck with a crack in ordinary concrete taking into account the curing of concrete. Deterministic solution - alternative P1B with unprotected reinforcement and P1E with epoxide protected steel reinforcement.

The resulting durability curve is shown and the period to the occurrence of corrosion for epoxide protected protection rose from 5.4 years to 12.6 years.
Figure 41: Comparing the effect of protection of steel reinforcement with the help of the reliability function for analysing durability - the initiation of corrosion in a concrete bridge deck with a crack in ordinary concrete taking into account the effect of concrete curing. The time from the beginning of corrosion for unprotected reinforcement $t_1 = 5.4$ years and for reinforcement with an epoxide covering $t_{1,\text{epoxy}} = 12.6$ years.

Deterministic solution - alternative P1B with unprotected reinforcement and P1E with epoxide protected steel reinforcement

5.1.8 Analysis with a defect in the waterproof insulation and a crack in a reinforced concrete bridge deck

If we add to the previous solution of the bridge deck with a crack and ordinary concrete (P1EB) an asphalt covering and waterproof insulation with a defect 40cm from the left hand margin we obtain a model corresponding to the traditional solution in central Europe. It is assumed that the area of the defect in the barrier will grow by 1 cm per year. The below mentioned output is for 20 and 40 years: the size of the defect in the waterproof insulation here corresponds to an unmeasured age and is 20 cm for 20 years or 40 cm for a 40 year age. In the concentrations of chlorides it is clear that the surface concentration representing a defect in the waterproof insulation is localised to the area of the defect and the area of the defect grows in time.
Figure 42: The concentration of chloride ions in the concrete bridge deck covered in an asphalt covering with a crack in the middle of the bridge deck and a defect in the waterproof insulation 40 cm from the left edge of the model. The output shown is for ordinary concrete when considering the effect of concrete curing. Deterministic solution - alternative P2EB for an exposure period of $t = 20$ and 40 years. (a) shows the FEA meshed elements and concentration in the form of bands whereas (b) shows the isoline of the concentration of chlorides.

The outputs for an uneven decade are given in an addendum to Figure 61 page 110 and on Figure 62 on page 111.

Through an analysis of the reliability function it was calculated that the initiation of corrosion begins for unprotected reinforcement after 28.4 years, whereas for reinforcement with an epoxide protection it will be 31.4 years. See Figure 43: The effect of epoxide protection is significant up to an age of 25 years then the area of damage to the waterproof insulation is so extensive that it corresponds to the decomposition defect on the epoxide coating. The advantage of epoxide protection is then lost.
5.1.9 Analysis of high performance concrete and with a crack in a reinforced concrete bridge deck.

If we alter the solution for a directly exposed bridge deck with a crack and ordinary concrete (P1EB) with a suitable diffusion coefficient together with a curing coefficient we then obtain a model for high performance concrete. If we then consider $D_{c,28}$ for high performance concrete equal to $2.75 \times 10^{-12}$ [m$^2$/s] and a curing coefficient $m = 0.39$ which corresponds to a mix marked as 45TII-V/35G100S/20F (Ghosh et al., 2014, see Table - 23 in the addendum on page. 112). The period to the occurrence of corrosion $t_i$ for a solution with unprotected steel reinforcement (Alternative P3E) is then 12.7 years. The period until the occurrence of corrosion $t_{i,\text{epoxy}}$ for a solution with epoxide protection of reinforcement (Alternative P3B) is 29.1 years. The durability curve for the alternative taking into account curing is given on Figure 44. The graphical 2D outputs for uneven decades are given in the addendum to Figure 63, page 111 and on Figure 64 on page 112.

Figure 43: The function of reliability for an analysis of durability - the initiation of corrosion in a concrete bridge deck covered with an asphalt layer with a crack in the middle of the bridge deck and a defect in its waterproof insulation. The output shown is for ordinary concrete when considering the effect of concrete curing. The period to the beginning of corrosion for unprotected reinforcement $t_i$ is 28.4 years and for reinforcement with an epoxide coating it is $t_{i,\text{epoxy}} = 31.4$ years. Deterministic solution - alternative P2EB.
Figure 44: Comparing the effect of protection of steel reinforcement with the help of the reliability function for an analysis of durability - the initialization of corrosion on a concrete bridge deck with a crack in high performance concrete taking into account the effect of concrete curing. The period to the beginning of corrosion for unprotected reinforcement $t_i = 23.3$ years and for reinforcement with an epoxide coating $t_{epoxy} = 63.5$ years. Deterministic solution – P3EB.

5.1.10 Discussion and summary of the deterministic solution

The outputs from the individual analyses are given in Table 12. In the results there is a clear difference in the lifespan when considering the curing of concrete. It is also clear that there is a significant difference between the alternatives with waterproof insulation and with a directly exposed bridge deck with a crack.

As expected the unprotected directly exposed bridge deck without reinforcement protection (P1B) fares the worst when considering the epoxide protection (P1E) a doubling occurs in the period to the initiation of corrosion. If the bridge deck is also protected with waterproof insulation an extension of 6 times the lifespan of the steel reinforcement occurs compared to the alternative (P1B). The epoxide coating also extends lifespan but the difference compared to unprotected reinforcement is not that significant. When using high performance concrete the initiation of corrosion in ordinary reinforcement (P3B) commences in roughly two thirds of the time to the initiation of corrosion in the case of ordinary concrete with waterproof insulation (P2B). The best protected is high performance concrete, reinforcement with an epoxide coating in a situation with
ordinary concrete and waterproof insulation. Here in alternative (P3B) the slower diffusion of chlorides through the covering proves positive and the spacing of the defects in the epoxide from the cracks in the bridge deck. Here there is also the most significant difference compared with the unprotected steel reinforcement.

Table 12: A summary of the results of the deterministic analysis of the initiation of corrosion according to the chosen alternative solution of the reinforced concrete bridge deck

<table>
<thead>
<tr>
<th>Period to the initiation of corrosion $t_i$</th>
<th>Cement mix</th>
<th>Crack in concrete</th>
<th>Asphalt covering and waterproof insulation</th>
<th>The effect of concrete curing</th>
<th>Protection of reinforcing</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_{i,\text{epoxy}}$ [years]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P1EB</td>
<td>5.4 / 12.6</td>
<td>100TII</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>P2EB</td>
<td>28.4 / 31.4</td>
<td>100TII</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>P3EB</td>
<td>23.3 / 63.5</td>
<td>45TII-V/35G100S/20F</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>
6 PROBABILISTIC ANALYSIS

A probabilistic estimate of the period to the initiation of corrosion for the selected alternatives of reinforced concrete bridge decks is the subject of this chapter. The probabilistic analysis uses the above mentioned 2D finite element model. At the conclusion a comparison is shown of the behaviour of individual methods of protection of steel reinforcement when evaluating the dispersion of input parameters. This part serves as a primary study where for the verification of the functionality of the model and an indicative example of its possibilities.

6.1 Introduction

The method of probabilistic assessments applied to the problem of the durability of reinforced concrete bridge decks considering the initiation of corrosion is illustrated for example on reinforced concrete directly exposed bridge decks from ordinary concrete in sub chapter 6.2. During the probability analysis, when modelling, only the application of the model describing the curing of concrete in time is considered. In chapter 6.2 values and a probabilistic description of the individual values are given, a description of the analysis and a breakdown of the resulting periods to the initiation of corrosion. Due to the fact that in one calculation it is possible to analyze both the durability of unprotected steel reinforcement and the durability of steel reinforcement protected by an epoxide coating, attention is paid to both sub variants. Another of the variants considered by method of protection of reinforcement and the bridge decks are given in Table 13 and in sub chapters 6.3 and 6.4. The chapters mentioned discuss the differences compared with the solutions of directly exposed bridge decks with a crack and ordinary concrete (alternative P1E/B) and also introduce the resulting parameters for the period of initiation of corrosion.

The SBRA method (Simulation-based Reliability Assessment, Marek et al., 1995, 2003) is used for the probability analysis which uses the direct Monte Carlo method to characterize the random variable cut off histograms according to (Marek et al., 1995). The period to the initiation of corrosion is calculated by a model loosely linked to a previously carried out probability analysis of directly exposed bridge decks. (Konečný et al., 2007) which was prepared in an environment compatible with Octave or Matlab. The probability model linked to the work of (Praks, 2002, 2005) starts an FEA task with a description of a concrete variant of an RC bridge deck. The solution of this macro is carried out repeatedly as part of a Monte Carlo simulation and always with randomly generated input variables according to the prescribed histogram.

The resulting probabilities are referenced to a 1x1 m square and the area of the damaged bridge
structure can then be later calculated by a simple multiplication of the probability obtained by the area of the bridge deck.

Table 13: The construction and model parameters of the alternative selected for the probabilistic analysis of the initiation of corrosion in reinforced concrete bridge decks.

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Cement mix</th>
<th>Crack in concrete</th>
<th>Asphalt covering and waterproof insulation</th>
<th>The effect of concrete curing</th>
<th>Protection of reinforcing</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1E</td>
<td>100TII</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Epoxide coating</td>
</tr>
<tr>
<td>P1B</td>
<td>100TII</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Unprotected</td>
</tr>
<tr>
<td>P2E</td>
<td>100TII</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Epoxide covering</td>
</tr>
<tr>
<td>P2B</td>
<td>100TII</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Unprotected</td>
</tr>
<tr>
<td>P3E</td>
<td>45TII - V/35G100S/20F</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Epoxide covering</td>
</tr>
<tr>
<td>P3B</td>
<td>45TII - V/35G100S/20F</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Unprotected</td>
</tr>
</tbody>
</table>

6.2 An analysis with a crack in a reinforced concrete bridge deck using ordinary concrete

The probability analysis of a directly exposed reinforced concrete bridge deck with a crack links to the deterministic solution with unprotected steel reinforcement given in chapter 5.1.6. The model also contains a module for the evaluation of durability expressed in the form of the period to the initialization of corrosion while protecting reinforcement with an epoxide coating (see sub chapter 5.1.3) and that due to the fact that both alternatives can be modelled in one simulation.

6.2.1 Input and random variables

A summary of the parameters applied in alternative 1 discussed is given in Table 14 for both random variables and deterministic variables. The basic material characteristic, the diffusion coefficient of ordinary concrete with cement 100TII is chosen based on laboratory tests (Ghosh et al., 2014, viz Table - 23). The referential diffusion coefficient is $D_{c,28} = 5.59 \times 10^{-12} \text{[m}^2/\text{s}]$ and a
coefficient of curing \( m = 0.26 \). The basis for the histogram of dispersions of diffusion coefficients of ordinary concrete is a normalised dimensionless histogram obtained by the authors from 80 measurements on 10 samples of high performance concrete. The derivation of the histogram is given in chapter 4.5.3, which starts on page 47. Against that the data of (Ghosh, 2014) containing a description of the dispersion of the diffusion coefficients in time including the variation coefficients for ordinary and high performance concretes have a dispersion of values for mixes prepared in a laboratory expressed in a variation coefficient approximately 2 times lower. The data from one source are suitable in the case of a comparison of alternative high performance concretes and ordinary concretes.

\[ D_{c,var} = D_{c,var,n} \times D_{c,var,coef} \]  

Figure 45: The histogram of the dispersion of the diffusion coefficients of concrete for a probabilistic analysis of high performance and ordinary concrete.

The data used in (Konečný, et. al., 2007) to describe ordinary concrete on the basis of 200 core drills from 40 bridges in the United States (Sohanghpurwala&Scannell,1994) was not used due to doubts about the quality of the resulting distribution of the diffusion coefficient as was discussed in section 4.6.2. The diffusion coefficient in cracks in reinforced concrete bridge decks \( D_{c,crack} \) depend on the width of the crack and the size of the referential diffusion coefficient \( D_{c,28} \). The value is calculated in accordance with relationship /34/.

The corrosion of reinforcement is related to the concentration of chlorides at the level of reinforcement. The depth of the reinforcement was selected as constant at 50 [mm]
Figure 46: Histogram of the chloride threshold $C_{th} [%]$ (Chloride threshold – chloridový prah, years – roky, Frequency - frekvence)

The distribution of the chloride threshold for unprotected steel reinforcement was obtained from the work of (Darwin et al., 2009). This distribution is also used for an analysis of the initiation of corrosion on reinforcement covered with an epoxide coating. The above mentioned chloride threshold shown on Figure 46 is used to describe the characteristics of protected reinforcement at points of the modelled damage to epoxide coatings. Further inspiration for the extent of chloride threshold is given in the work of Glass&Buenfeld, 1995.

The parameters of cracks in directly exposed reinforced concrete bridge decks are considered as follows. Let's assume that the cracks in the concrete cover 0.01% of the area. If the average length of the cracks is 3 metres, then the average width of a crack in an area of $1 \text{ m}^2$ can be determined in accordance with relationship /35/as:

$$C_{rckw} = 1000 \times C_{rcks} \times P_{rck} = 1000 \times 3 \times 0.0001 = 0.3 \text{ [mm]}$$

where:

- $P_{rck}$ the probability of the occurrence of a crack in the individual areas $[\text{m}^{-2}]$,
- $C_{rcks}$ the spacing of cracks [m],
- $C_{rckw}$ crack width [mm].

For a description of the widths and spacing of cracks in concrete a normal distribution is chosen and the standard deviation of the crack spacing is $\sigma_{C_{rcks}}=0.1 \text{ m}$, whereas the standard deviation for the width of a crack $\sigma_{C_{rckw}}$ is 0.05 mm.

The maximum crack depth chosen is equal to the depth of the bridge deck which is slightly optimistic in light of the experiences given in (Saadeghvaziri&Hadidi, 2002). The crack is further characterized by the starting position of the crack. The starting position of the crack in the bridge deck can occur with an equal probability of occurrence between the start of the model and the
Spacing of the cracks. Every other crack would be separated from the previous crack by the actual crack spacing generated. To estimate the dispersion of the crack depths an exponential distribution is chosen, where the greatest number of cracks will be at the surface and their number will lower with depth. The reinforcement cover is described in the distribution given on Figure 47 according to the study by (Sohanghpurwala & Scannell, 1994).

![Figure 47: Cover histogram [m] (Depth of reinforcement– hloubka výztuže, years – roky, Frequency - frekvence)](image)

Table 14: The random variables and deterministic input parameters for alternatives P1B and P1E (ordinary concrete with cement 100TII and a directly exposed reinforced concrete bridge deck with a crack, P1B - unprotected steel reinforcement, P1E - steel reinforcement protected by an epoxide coating.

<table>
<thead>
<tr>
<th>Parameter name</th>
<th>Dispersion</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diffusion coefficient (OPC) $D_{c,28}$ [10^{-12} m^2/s]</td>
<td>$5.59 \times 10^{-12}$</td>
<td>Constant (Ghosh, 2014)</td>
</tr>
<tr>
<td>Curing coefficient m [-]</td>
<td>0.260</td>
<td>Constant (Ghosh, 2014)</td>
</tr>
<tr>
<td>Dispersion of the diffusion coefficient $D_{c,var}$ [-]</td>
<td>$(-2.7-11.2) \times 0.043$</td>
<td>Histogram DeHPCN3.dis, Gumbel distribution ($\mu=0$, $s=1) \times 0.043$</td>
</tr>
<tr>
<td>Depth of reinforcement (cover) $R_{\text{end}}$ [m]</td>
<td>0.04-0.11</td>
<td>Histogram XDEPTH3.dis (Sohanghpurwala &amp; Scannell, 1994)</td>
</tr>
<tr>
<td>Chloride threshold for the start of corrosion $C_{th}$ [%]</td>
<td>0.09-0.51</td>
<td>Histogram thr_b.dis (Darwin, et al. 2009)</td>
</tr>
<tr>
<td>Depth of cracks $C_{\text{crktpt}}$ [m]</td>
<td>0.0-Depth</td>
<td>Histogram EXPON1.dis</td>
</tr>
<tr>
<td>Crack spacing $C_{\text{crks}}$ [m]</td>
<td>(2.4-3.6)</td>
<td>Histogram crcks.dis Normal distribution N(3,0.1) **</td>
</tr>
<tr>
<td>Crack width $C_{\text{crkw}}$ [mm]</td>
<td>(0.035-0.565)</td>
<td>Histogram crkw.dis Normal distribution, N(0.3,0.05) **</td>
</tr>
<tr>
<td>Relative spacing of the first cracks $C_{\text{crks},i}$</td>
<td>0-1</td>
<td>Even distribution</td>
</tr>
</tbody>
</table>
Table 14 (continuation): The random variables and deterministic input parameters for alternatives P1B and P1E. (ordinary concrete with cement 100TH and a directly exposed reinforced concrete bridge deck with a crack).

<table>
<thead>
<tr>
<th>Parameter name</th>
<th>Dispersion</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequency of defects in the reinforcement coating $M_{ashn}$ [m$^{-1}$]</td>
<td>0-10</td>
<td>Histogram $holi_V$.dis (Weyers, et al, 1998)</td>
</tr>
<tr>
<td>Relative spacing of the first defect in the reinforcement coating $M_{as}$</td>
<td>0-1</td>
<td>Even distribution</td>
</tr>
<tr>
<td>Concentration of chlorides at the surface $C_0$ [%]</td>
<td>0.21-1.63</td>
<td>Histogram $c_0V$.dis (Weyers, et al, 1998)</td>
</tr>
<tr>
<td>Starting concentration of chlorides in concrete $C_b$ [%]</td>
<td>0</td>
<td>Constant</td>
</tr>
<tr>
<td>Monitored life span $t$ [years]</td>
<td>100</td>
<td>Constant</td>
</tr>
<tr>
<td>Depth of RC slab $Depth$ [m]</td>
<td>0.23</td>
<td>Constant</td>
</tr>
</tbody>
</table>

** Normal distribution $N(\mu, \sigma)$ is described by 2 parameters $\mu$ is the average and $\sigma$ is the standard deviation. The distributions are approximated cut off histograms so both the minimum and maximum of the distribution used are given.

In the case of the variant of steel reinforcement with an epoxide coating the frequency of assessment is chosen as the distribution in accordance with (Weyers et al., 1998, Pyc, 1998, see Figure 48) which seems more realistic than the distribution of defects obtained on the basis of measurement of (Sohanghpurwala and Scannell, 1994). The quantity of defects in the epoxide coating according to (Pyc, 1998) works out at 10 per meter run of reinforcement. From an analysis of a study by (Sohanghpurwala and Scannell, 1994) it is possible to calculate the frequency of defects in the epoxide in the reinforcement by core drills of up to 100 m$^{-1}$.

![Figure 48: Histogram of defects in the epoxide coating $Mashn$ [m$^{-1}$] (Holidays- defects in the epoxide coating, years – roky, Frequency - frekvence)](image)
Another random variable parameter obtained from the work of (Pyc, 1998) is the concentration of chlorides on the surface of the concrete (see Figure 49)

![Figure 49: A histogram of the concentration of chlorides on the surface $C_0$ [% of the weight of material with cementation capabilities] (Surface – povrch, chloride – chloridy, concentration – koncentrace, Frequency - frekvence)](image)

### 6.2.2 FEA - transformation model

The FEA makro described in chapter 5 allows the repeated analysis of tasks in the 2D diffusion of chlorides with input parameters randomly generated by the probabilistic module. Sub chapter 5.1.6 showed a calculated model for one set of inputs - deterministic solution. In the primary study a model is used with a network 30x 32, which has 960 elements. In the probabilistic solution using the Monte Carlo method a network of finite elements is created in every simulation step. On each of the individual junctions marginal conditions are applied and the parameters of the concrete are generated. A calculation of the concentration of chlorides in the entire cross section of the bridge deck is also carried out for its studied life span.

The highest value $C_{xz,t}$ is taken from each simulation of the concentration of chlorides at the level of reinforcement and that is compared to the chloride threshold $C_{th}/12/$ through a reliability function analysis. An examination of the concentration of chlorides is carried out at the same time at points with defects in the protective coating of the epoxide coating of steel reinforcement. The highest of the chloride concentration values $C_{xz,epoxy,t}$ (usually the nearest cracks) is chosen for comparison with the chloride threshold $C_{th}$ and a function of reliability $RF_{epoxy,t}$ is created for the epoxide coating. Through an analysis of the function of reliability in time we obtain the time to defect - this is the initiation of corrosion. See /13/.
6.2.3 Probability analysis - SRBA method

The probability construct is subject to a Monte Carlo analysis with thousands of simulations. The level of reliability of unprotected steel reinforcement and reinforcement protected by epoxide coatings can be, in light of the start of corrosion, expressed with the aid of the probability of defect in time $P_{f,t}$ or as a distribution of the time to the occurrence of corrosion $t_i$ as is here. The resulting histogram of time to the initiation of corrosion $t_i$ for unprotected reinforcement is shown on Figure 50.

When analysing the histogram to the period of initiation it is found that the average value of the period to initiation of corrosion is 33.5 years. It is necessary to point out that the numerical non-stationary model was used for an analysis of the first 100 years of the life of the structure. If the initiation of corrosion was not detected, the period to the initiation is greater than 100 years and is entered into the histogram in the right hand 100 years column. Through the accumulation of larger durability's into one class on the histogram the average value is distorted downwards.

From a statistical point of view on the other hand it is interesting to analyse the years corresponding to the chosen probability of exceeding the chloride threshold and thus the initiation of corrosion. See Figure 51. The probability of the initiation of corrosion equal to 5 [%/m$^2$] corresponds to the period for the initiation of corrosion $t_i = 6.8$ years, whereas for a probability of 10 [%/m$^2$] it is 10.0 years, and for a probability of 25 [%/m$^2$] it is 28.2 years. The probability is thus related to 1 m$^2$ because the analysis is carried out on an unending slab with a cross section of 1x1 m.

![Figure 50: The period of initiation of corrosion $t_i$ for a bridge deck with a crack and unprotected steel reinforcement. Probabilistic solution alternative P1B.](image)
Figure 51: Drawing of the probability of the initiation of corrosion on a bridge deck with a crack and unprotected steel reinforcement. Probabilistic solution alternative P1B In the graph, the times corresponding to the probabilistic initiation of corrosion P=5, 10 a 25 %/m2 are divided by a vertical line. After deducting the probability of initiation the comparison with design probabilities might occur if the value of P_d for this case was determined. The value for the beginning of corrosion could be higher than the value considered for the limit states of use. 7%. See for example (Tikalsky, 2003, Teplý et al., 2005). In this case the important thing is not if it complies or does not comply with the limiting values but the comparison of behaviour the individual alternatives.

The histogram of the period to initiation of corrosion t_i,epoxy for the other alternative P1E (reinforcement protected by epoxide coatings) is shown on Figure 52. For this alternative the number of simulations, for which there was no record of the initiation of corrosion, was even greater and thus the column with the values of t_i,epoxy at a level of 100 years is dominant.

Figure 52: The period to the initiation of corrosion t_i,epoxy for a bridge deck with a crack and steel reinforcement protected with an epoxide coating. Probabilistic solution alternative P1E.
The probability of corrosion initiation can be read from Figure 53 and the probability of the initiation of corrosion \( P_i = 5\% / m^2 \) corresponds to a period to the initiation of corrosion \( t_i = 15.9 \) years, whereas for a probability of \( 10\% / m^2 \) it is 26.2 years and for a probability of \( 25\% \) it is more than 100.0 years.

![Figure 53: Picture showing the probability of not exceeding the period to the initiation of corrosion to epoxy on bridge decks with a crack and steel reinforcement protected by an epoxide coating. Probabilistic solution alternative P1E.](image)

### 6.3 Analysis with a defect in the waterproof insulation and a crack in a reinforced concrete bridge deck

By adding waterproof insulation under the asphalt covering on the surface of alternative P1B we obtain a model for a typical bridge deck in the Czech Republic (Alternative P2B). In the case of reinforcement protected by epoxide the alternative marked as P1E becomes with the addition of waterproof insulation under the asphalt coating version P2E. A description of the random variable defect parameter of waterproof insulation is given in Table 15.

For simplification here the asphalt is considered as totally permeable and the waterproof insulation can contain defects. The area of defects in the model rises linearly every year as is explained in the principles of the model applied. See sub chapter 5.1.4.

The parameters of cracks in directly exposed reinforced concrete bridge decks are considered as follows. Let us assume that the growth in defect's in the waterproof insulation is 1% of the area annually. If the average spacing of defects is 1 metre, the average defect width in an area of 1 m\(^2\) can be determined by insertion into the relationship /36/:
\[ \Delta C_{ck,HIw} = 1000 \times C_{ck,HI} \times \Delta P_{crck,HI} = 1000 \times 1 \times 0.00 = 10 \text{ [mm]} \]

where:

- \( \Delta P_{crck,HI} \) rise in the probability of the occurrence of cracks in the individual areas \([\text{m}^2/\text{year}]\)
- \( C_{ck,HI} \) spacing of cracks \([\text{m}]\),
- \( \Delta C_{ckw,HI} \) annual growth of the width of cracks \([\text{mm/year}]\).

For a description of the widths and spacing of cracks in concrete a normal distribution is chosen and the standard deviation of the crack spacing is \( \sigma_{Crcks} = 0.1 \text{ m} \), whereas the standard deviation for the width of a crack \( \sigma_{Crcksw} = 1.65 \text{ mm} \). The crack grows equally on both sides of the defined position and the starting position of the first crack is chosen with an equal probability between the start of the model and the spacing of the cracks. A selection is ensured by an equal distribution \( C_{ckshI,i} \).

Table 15: The random variables and deterministic input parameters for alternatives P2B and P2E (ordinary concrete and a reinforced concrete bridge deck with a crack under damaged waterproof insulation, P2B - unprotected steel reinforcement, P2E - steel reinforcement protected by an epoxide coating.

<table>
<thead>
<tr>
<th>Parameter name</th>
<th>Dispersion</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crack spacing in waterproof insulation ( C_{ckHI}) ([\text{m}])</td>
<td>(0.46-1.53)</td>
<td>Histogram crckHI.dis Normal distribution N(1,0.1)</td>
</tr>
<tr>
<td>Width of the cracks in the waterproof insulation ( C_{ckHIw}) ([\text{mm/year}])</td>
<td>(1.15-18.85)</td>
<td>Histogram crckHIw.dis Normal distribution N(10,1.65)</td>
</tr>
<tr>
<td>Relative spacing of the first cracks in HI ( C_{ckshI})</td>
<td>0-1</td>
<td>Even distribution</td>
</tr>
</tbody>
</table>

The probability analysis of inputs and the use of numerical models of the bridge deck with a crack protected by waterproof insulation are prepared divided into the time for the initiation of corrosion \( t_i \) for unprotected reinforcement (Figure 54) and reinforcement protected by epoxide \( t_{i,\text{epoxy}} \) (Figure 55).

The times to the initiation of corrosion for a corresponding probability listed in the pair \( t_i/t_{i,\text{epoxy}} \).

The first value represents unprotected reinforcement and the second value reinforcement protected with an epoxide coating. \( P_i = 5 \text{ [%/m2]} \) corresponds to the period of initiation of corrosion \( t_i/t_{i,\text{epoxy}} = 22.4/53.2 \text{ years} \) whereas for the probability of 10 [%/m2] it is 29.9/83.7 years and for a probability of 25 % it is more than 52.2/100.0 years. The probability curves of the initiation of corrosion are given in the collection of drawings of all variants on Figure 58 and page 90.
6.4 Analysis of high performance concrete and with a crack in a reinforced concrete bridge deck

For an analysis of directly exposed bridge decks from ordinary concrete (P1B and P1E, see sub chapter 6.2 and the summary of the input parameters in Table 14) which follows on from the analysis of similar structures made from high performance concrete. For this concrete its diffusion coefficient and curing coefficient are replaced. See Table 16.
Table 16: The random variables and deterministic input parameters for alternatives P3B and P3E (high performance concrete with cement materials 45TI-I-V/35G100S/20F and a directly exposed reinforced concrete deck with a crack, P3B - unprotected steel reinforcement, P3E - steel reinforcement protected by an epoxide coating.)

<table>
<thead>
<tr>
<th>Parameter name</th>
<th>Dispersion</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diffusion coefficient (HPC) $D_{c,28} \times 10^{-12} \text{m}^2/\text{s}$</td>
<td>$2.79 \times 10^{-12}$</td>
<td>Constant (Ghosh, 2014)</td>
</tr>
<tr>
<td>Curing coefficient $m$ [-]</td>
<td>0.390</td>
<td>Constant (Ghosh, 2014)</td>
</tr>
<tr>
<td>Dispersion of the diffusion coefficient $D_{c,\text{var}}$ [-]</td>
<td>(-2.7-11.2)×0.039</td>
<td>Histogram $D_{c,HPCN3,\text{dis}}$, Gumbel distribution ($\mu=0$, $s=1$)×0.039* See sub chapter 4.6.4.</td>
</tr>
</tbody>
</table>

*Variation coefficient according to Table - 23.

The resulting times to the initiation of corrosion $t_i$ for non protected reinforcement (Figure 56) and for reinforcement protected by epoxide $t_{i,\text{epoxy}}$ (Figure 57) for the last variant of probabilistic numerical model of the bridge deck with a crack are prepared below.

![Figure 56: The period to the initiation of corrosion $t_i$ for a bridge deck from high performance concrete with a crack and unprotected steel reinforcement. Probabilistic solution - alternative P3B.](image)

The times to the initiation of corrosion for a corresponding probability listed again in the pair $t_i/t_{i,\text{epoxy}}$. The first value represents unprotected reinforcement and the second value reinforcement protected with an epoxide coating. $P_{i}=5 \%/\text{m}2$ corresponds to the period of initiation of corrosion $t_i/t_{i,\text{epoxy}} = 11.7/91.1$ years whereas for the probability of 10 $\%/\text{m}2$ it is 37.6/100.0 years and for a probability of 25 $\%$ it is more than 100.0/100 years. The probability curves of the initiation of corrosion are again listed together with the other variations on Figure 58 on page 90.
Comparing alternatives

The prepared models serve to compare the behaviour of the individual variants and not only from the point of view of bridge structure but also from the point of view of the protection of reinforcement. At the same time variants are followed from the point of view of concrete type and the use of a waterproof insulation layer. The results of the progress of the probability initiation of corrosion after a simulated period of 100 years are shown on Figure 58.

From the analysis it is clear that the worst protected is, as expected, the steel reinforcement without an epoxide coating in the directly exposed bridge deck with a crack. When an epoxide coating was used on steel reinforcement then the model calculations lowered the risk of the occurrence of corrosion greatly. For ages greater than 80 years this variant even works out better than the directly exposed bridge deck with a crack made from high performance concrete and unprotected reinforcement. The application of waterproofing with progressively widening cracks also lowered the risk of corrosion at the beginning of its life by about 10%, but due to the progressive growth of cracks the modelled effect of waterproof insulation is lost with time and this alternative copies the variant without protection with a delay. A directly exposed bridge deck with high performance concrete shows a slower growth in the risk of the occurrence of corrosion than the variant's with unprotected steel reinforcement P1B (ordinary concrete) and variant P2B (with waterproof insulation). If epoxide covered reinforcement is combined with waterproof insulation P2E then a higher durability is obtained than for variant P3B (high performance concrete with unprotected reinforcement). The most durable version of the analysis is the directly exposed bridge deck from...
high performance concrete with epoxide protection of the reinforcement (P3E). The numerical expression of the results of the period to the initiation of corrosion for a chosen probability of the initiation of corrosion is given in Table 17.

Table 17: Period to the initiation of corrosion for a chosen probability of the initiation of corrosion for chosen variants according to type of reinforcement protection: B - unprotected reinforcement, E - reinforcement covered with epoxide; and the method of carrying out the bridge deck. 1 - ordinary concrete and a directly exposed bridge deck with a crack, 2 - ordinary concrete and a bridge deck with a crack protected by waterproof insulation under an asphalt layer. 3 high performance concrete and a directly exposed bridge deck with a crack

<table>
<thead>
<tr>
<th>FEA mesh: 30x32</th>
<th>P1 [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1B</td>
<td>6.8</td>
</tr>
<tr>
<td>P1E</td>
<td>15.9</td>
</tr>
<tr>
<td>P2B</td>
<td>22.4</td>
</tr>
<tr>
<td>P2E</td>
<td>53.2</td>
</tr>
<tr>
<td>P3B</td>
<td>11.7</td>
</tr>
<tr>
<td>P3E</td>
<td>91.1</td>
</tr>
</tbody>
</table>
6.6 Discussion of the results

The above listed results are orientational and correspond to the assumptions made. Therefore interpretation of the results is possible only with the greatest care. The prepared probabilistic numerical model allows the modelling of the required directly exposed bridge deck with a crack. The model is capable of analysing the effect of the asphalt layer and defects in the waterproof insulation. It is possible to compare the behaviour of the unprotected steel reinforcement or epoxide protection but the results reflect the assumed distribution of cracks in the bridge deck, the distribution of defects in the waterproof insulation or problems with the epoxide coating.

Results which are favourable from the point of view of improving the durability using epoxide coatings of steel reinforcement correspond to the use of the distribution of defects in epoxide according to (Weyers et al. 1998). If the distribution of the defects in the epoxide coating taken from the study (Sohanghpurwala and Scannell, 1994) had been used, where the number of detected defects in the epoxide coating in core drills was 5 to 10 times greater (Konečný, 2007), the effect of the epoxide protection would not be so significant. It is further possible to mention the crack in the bridge deck which would have to reach even deeper than just the upper reinforcement as is assumed at present. These parametric alterations could lead to changes in the results and mix up the suitability of the methods from the point of view of durability.
7 SUMMARY AND CONCLUSIONS

The text introduces the reader to the narrow area of analysis of the durability of reinforced concrete bridge decks from the point of view of the initiation of corrosion. In the examples of probabilistic assessments of the initiation of corrosion for selected alternatives of reinforced concrete bridge decks, the extent and character of the input parameters are illustrated, the possibilities of applying analyses based on the principles of finite element analysis and the method for determining the diffusion coefficient of concrete taking into account the actions of chlorides is analysed.

Due to the aim of comparing the alternatives with waterproof insulation and a directly exposed bridge deck made from high performance concrete, the implementation of a new model of cracks is introduced in the form of introducing a highly permeable area representing the crack area is introduced. This approach also allows flexible reaction to changes in the permeability of cracks while considering their size. An approach with a change of character of the diffusion coefficient is selected because the effect of cracks in concrete on the introduction of a concentration of chlorides at a junction corresponds to a crack under the waterproof insulation can not be included if the defect in the waterproof insulation is in a different position to the crack in the concrete.

The numerical model applied is at the stage of fine tuning and searching for an optimal balance between the calculation of time and the exactness of the calculation. This search is carried out at the level of the size of the FEA mesh, the shape of the element around the crack, the number of steps in the probability analysis and the length of the time stepping. The ideal number of simulations in a probability analysis of the initiation of corrosion would be between 5-10 thousand, the results presented in the text of 1000 steps with a precision of ± 0.5% are adequate anyway for an orientational visualization.

Part of the text is a description of the preparation of the resistance characteristics of the high performance concrete used against the penetration of chlorides. The method of determining the diffusion coefficient is studied through an analysis of the chloride profile deduced from the surface resistance of concrete. On the basis of measurements of resistance which were carried out a normalised distribution of the diffusion coefficient is created and the average value for a reference age of 28 days is also determined. On the basis of the calculated coefficient of curing a method of describing the development of the diffusion coefficient in time is illustrated which takes into account the numerical modelling of the durability of reinforced concrete bridge decks.

The calculation of the basic characteristics of ordinary concrete is shown in the laboratory measurement of data prepared by a concrete plant for road panels. For the use of data used in the...
analysis of the diffusion coefficient it would be useful to also carry out a further detailed test of the resistivity and chloride profile on an ordinary concrete as in the case of the analysis for high performance concrete. A description of the diffusion coefficient for ordinary concrete on a directly exposed bridge deck and bridge decks protected by an asphalt layer would then be evaluated by the same method as for high performance concrete.

Due to the fact that the development of methods for the analysis of the durability of reinforced concrete structures continues, there is a plan for widening the program by further program modules for following the durability of reinforced concrete bridge decks in the area of corrosion propagation.
LITERATURE


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KONEČNÝ, P. Simulace korelovaných neparametrických rozdělení v rámci metody SBRA. Sborník vědeckých prací Fakulty Stavební VŠB – Ostrava. 2007\(^2\), vol. VII, issue 1, s. 199-209, ISSN 1213-1962.


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\(^1\) Citation in text is given as (KONEČNY, P., 2007a).

\(^2\) Citation in text is given as (KONEČNY, P., 2007b).


KONEČNÝ, P., M. TURICOVÁ, L. ŽÍDEK, L. Měření chloridového profilu a výpočet difuzního součinitele betonu, příloha k závěrečné zprávě projektu Institucionálního rozvoje VŠ IP2283314 Veselý, V. et al. Experimentální určení vybraných parametrů betonu s ohledem na nelineární numerické modelování porušování a odolnosti vůči pronikání chloridů. 2013, Ostrava: Fakulta stavební VŠB-TUO.


LU, X. Application of the Nernst-Einstein Equation to Concrete. Cement and Concrete Research. vol. 27, issue 2, s. 293-302.


TIKALSKY, P. Chapter 20 Durability and Performance-Based design using SBRA. In: (MAREK, P. et al., 2003).


## 8 Appendix

### 8.1 Chloride profile

Table 18: The value of concentrations of chlorides according to the profile taken on 13.1. 2015 for core drills FV

<table>
<thead>
<tr>
<th>Depth [cm]</th>
<th>( C_{m(i)} ) [weight %]</th>
<th>Min</th>
<th>Max</th>
<th>Diameter</th>
<th>FV1</th>
<th>FV2</th>
<th>FV3</th>
<th>FV6</th>
<th>FV7</th>
<th>FV8</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>0.25</td>
<td>1.00709</td>
<td>0.89805</td>
<td>0.62343</td>
<td>0.99022</td>
<td>1.19385</td>
<td>1.02835</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>1</td>
<td>0.75</td>
<td>0.39269</td>
<td>0.3035</td>
<td>0.14926</td>
<td>0.32129</td>
<td>0.39985</td>
<td>0.27731</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.5</td>
<td>1.25</td>
<td>0.14322</td>
<td>0.07674</td>
<td>0.02517</td>
<td>0.06234</td>
<td>0.086</td>
<td>0.06867</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>2</td>
<td>1.75</td>
<td>0.03404</td>
<td>0.02136</td>
<td>0.02248</td>
<td>0.02647</td>
<td>0.07757*</td>
<td>0.0182</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>2.5</td>
<td>2.25</td>
<td>0.02396</td>
<td>0.02111</td>
<td>0.00897*</td>
<td>0.01971</td>
<td>0.052</td>
<td>0.02521*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>3</td>
<td>2.75</td>
<td>0.01392</td>
<td>0.01777</td>
<td>0.01066*</td>
<td>0.01885</td>
<td></td>
<td>0.02312*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>3.5</td>
<td>3.25</td>
<td>0.00818*</td>
<td>0.01572</td>
<td>0.01446</td>
<td></td>
<td>0.02701*</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: The values marked in red with stars are below the level of measurement or exhibit abnormal behaviour and are not included into the calculation of the diffusion coefficient.

Table 19: The values of chloride concentration according to the profiles taken from test cubes from ordinary concrete from KXX-94 a KXX-158. Profile obtained 3.12.2012 and 5.2. 2013.

<table>
<thead>
<tr>
<th>Depth [cm]</th>
<th>( C_{m(i)} ) [weight %]</th>
<th>Min</th>
<th>Max</th>
<th>Diameter</th>
<th>KXX-94</th>
<th>KXX-158</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>0.25</td>
<td>1.42</td>
<td>1.56</td>
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</tr>
<tr>
<td>0.5</td>
<td>1.5</td>
<td>1.00</td>
<td>0.65</td>
<td>0.77</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>2.5</td>
<td>2.00</td>
<td>0.14</td>
<td>0.33</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>3.5</td>
<td>3.00</td>
<td>0.14</td>
<td>0.33</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### 8.2 Measurement of electrical resistivity and calculation of the diffusion coefficient

Table 20: Values of electrical resistivity and diffusion coefficients according to the measurements taken on 30.9. 2014 for newly concreted samples FN

<table>
<thead>
<tr>
<th>Cylinder</th>
<th>Correction to lime water</th>
<th>1.100</th>
</tr>
</thead>
<tbody>
<tr>
<td>FN1</td>
<td>[… ]</td>
<td></td>
</tr>
<tr>
<td></td>
<td>30.09.14 28 days</td>
<td></td>
</tr>
<tr>
<td>Body</td>
<td>Corr. surface resistance</td>
<td></td>
</tr>
<tr>
<td>FN1</td>
<td>2.62</td>
<td></td>
</tr>
<tr>
<td>Surface</td>
<td></td>
<td></td>
</tr>
<tr>
<td>resistance</td>
<td>$\rho_{SR}$ [kΩcm]</td>
<td>33</td>
</tr>
<tr>
<td>Volume</td>
<td></td>
<td>31</td>
</tr>
<tr>
<td>resistivity</td>
<td>$\rho_{BR}$ [kΩcm]</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td></td>
<td>13</td>
</tr>
<tr>
<td>Diffusion</td>
<td>$D_c$ [m²/s]</td>
<td>3.70E-12</td>
</tr>
<tr>
<td>coefficient</td>
<td></td>
<td>3.70E-12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.94E-12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.70E-12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.94E-12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.81E-12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.94E-12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.83E-12</td>
</tr>
</tbody>
</table>

| FN2      | 2.62                     |       |
|          | 35  34  15  14           | 3.49E-12 |
|          | 35  35  15  15           | 3.49E-12 |
|          | 35  34  15  14           | 3.49E-12 |
|          | 36  37  15  16           | 3.39E-12 |
|          | 35.13 14.74              | 3.48E-12 |

| FN3      | 2.62                     |       |
|          | 36  34  15  14           | 3.39E-12 |
|          | 39  32  16  13           | 3.13E-12 |
|          | 33  33  14  14           | 3.70E-12 |
|          | 31  31  13  13           | 3.94E-12 |
|          | 33.63 14.11              | 3.65E-12 |

| FN4      | 2.62                     |       |
|          | 33  33  14  14           | 3.70E-12 |
|          | 31  31  13  13           | 3.94E-12 |
|          | 33  33  14  14           | 3.70E-12 |
|          | 32  31  13  13           | 3.81E-12 |
|          | 31.88 13.37              | 3.83E-12 |

| FN5      | 2.62                     |       |
|          | 35  34  15  14           | 3.49E-12 |
|          | 35  35  15  15           | 3.49E-12 |
|          | 35  34  15  14           | 3.49E-12 |
|          | 36  37  15  16           | 3.39E-12 |
|          | 35.13 14.74              | 3.48E-12 |

| FN6      | 2.62                     |       |
|          | 36  34  15  14           | 3.39E-12 |
|          | 39  32  16  13           | 3.13E-12 |
|          | 33  33  14  14           | 3.70E-12 |
|          | 31  31  13  13           | 3.94E-12 |
|          | 33.63 14.11              | 3.65E-12 |

<table>
<thead>
<tr>
<th>N</th>
<th>Surface resistivity $\rho_{SR}$ [kΩcm]</th>
<th>48</th>
<th>Volume resistivity $\rho_{BR}$ [kΩcm]</th>
<th>48</th>
<th>Diffusion coefficient $D_c$ [m²/s]</th>
<th>48</th>
</tr>
</thead>
<tbody>
<tr>
<td>FN</td>
<td>Standard Deviation $\sigma$</td>
<td>2.14</td>
<td>14.07</td>
<td>0.90</td>
<td>2.26E-13</td>
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<td>Diameter</td>
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<td>33.54</td>
<td>14.07</td>
<td>0.90</td>
<td>2.26E-13</td>
<td>3.65E-12</td>
</tr>
</tbody>
</table>

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Table 21: Values of electrical resistivity and diffusion coefficients according to the measurements taken on 30.9. 2014 for core drills FV

<table>
<thead>
<tr>
<th>Body</th>
<th>Correction</th>
<th>Surface resistivity $\rho_{SR}$ [kΩcm]</th>
<th>Volume resistivity $\rho_{VR}$ [kΩcm]</th>
<th>Diffusion coefficient $D_c$ [m²/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>surface</td>
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<td>resistance</td>
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<td></td>
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</tr>
<tr>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Cylinder</td>
<td>[...]</td>
<td>30.09.14</td>
<td>351 days</td>
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</tr>
<tr>
<td>FV4</td>
<td>3.05</td>
<td>69</td>
<td>69</td>
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</tr>
<tr>
<td></td>
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<tr>
<td></td>
<td></td>
<td>70.25</td>
<td>25.34</td>
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<tr>
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<td>89.25</td>
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<td>94.25</td>
<td>33.99</td>
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</tr>
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<td>FV10</td>
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<td></td>
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<td></td>
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<td>70</td>
<td>32</td>
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<td>FV2</td>
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<td>32.50</td>
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</tr>
<tr>
<td>FV6</td>
<td>3.05</td>
<td>85</td>
<td>86</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>71</td>
<td>93</td>
<td>26</td>
</tr>
<tr>
<td></td>
<td></td>
<td>99</td>
<td>90</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td></td>
<td>98</td>
<td>82</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>88.00</td>
<td>31.74</td>
<td></td>
</tr>
</tbody>
</table>
Table 21 (continuation): Values of electrical resistivity and diffusion coefficients according to the measurements taken on 30.9. 2014 for core drills FV

<table>
<thead>
<tr>
<th>Cylinder</th>
<th>Surface resistivity $\rho_{SR}$ [k$\Omega$cm]</th>
<th>Volume resistivity $\rho_{BR}$ [k$\Omega$cm]</th>
<th>Diffusion coefficient $D_c$ [$m^2/s$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>FV4</td>
<td>3.05</td>
<td>69</td>
<td>25</td>
</tr>
<tr>
<td>FV7</td>
<td>3.05</td>
<td>85</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>85</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>81</td>
<td>29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>90</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td></td>
<td>83.63</td>
<td>30.16</td>
</tr>
<tr>
<td>FV8</td>
<td>3.05</td>
<td>91</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>97</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>94</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td></td>
<td>84</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>92.13</td>
<td>33.23</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>FV</th>
<th>N</th>
<th>Surface resistivity $\rho_{SR}$ [k$\Omega$cm]</th>
<th>Volume resistivity $\rho_{BR}$ [k$\Omega$cm]</th>
<th>Diffusion coefficient $D_c$ [$m^2/s$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>80</td>
<td>84.48</td>
<td>30.47</td>
<td>1.70E-12</td>
</tr>
<tr>
<td>Standard Deviation</td>
<td>80</td>
<td>9.80</td>
<td>3.53</td>
<td>2.059E-13</td>
</tr>
</tbody>
</table>
Table 22: Time values dependent on electrical resistance.

<table>
<thead>
<tr>
<th>Body</th>
<th>Dimensions [mm]</th>
<th>Correction for surface resistivity</th>
<th>Correction for lime water</th>
<th>1.100</th>
<th>Correction for surface resistivity</th>
<th>Correction for lime water</th>
<th>1.000</th>
<th>Correction for surface resistivity</th>
<th>Correction for lime water</th>
<th>1.000</th>
<th>Correction for surface resistivity</th>
<th>Correction for lime water</th>
<th>1.000</th>
<th>Correction for surface resistivity</th>
<th>Correction for lime water</th>
<th>1.000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dno</td>
<td>diameter</td>
<td>height</td>
<td>Surface resistivity [Ω·cm]</td>
<td></td>
<td>Volume resistivity [Ω·cm³]</td>
<td>Surface resistivity [Ω·cm]</td>
<td></td>
<td>Volume resistivity [Ω·cm³]</td>
<td>Surface resistivity [Ω·cm]</td>
<td></td>
<td>Volume resistivity [Ω·cm³]</td>
<td>Surface resistivity [Ω·cm]</td>
<td></td>
<td>Volume resistivity [Ω·cm³]</td>
<td>Surface resistivity [Ω·cm]</td>
<td></td>
</tr>
</tbody>
</table>
|      |                |                                    | ![image](image)

FN1 2.62  
104 205  
31.33 13.37 40.13 15.40 51.13 19.50 53.38 20.36

FN2 2.62  
103.8 204  
35 34 44 46 54 52 58 57

FN3 2.62  
103.8 204  
39 32 40 43 48 49 56 58

FN4 3.05  
93.2 186  
69 69 72 86 70 78 78 79

FN5 3.05  
93 184  
74 74 86 85 84 71 70 65 89 74

FN6 3.05  
93 184.5  
89.25 32.19 84.75 27.79 74.63 24.47 87.13 28.57

FN7 3.05  
93 183  
87 95 92 99 81 92 94 96

FN8 3.05  
93 184  
78 91 83 99 80 76 90 92

FN9 3.05  
93 184  
77 76 70 72 65 77 89 96

FN10 3.05  
93 184  
89 70 69 79 64 77 75 82

108
8.3 2D outputs from the deterministic analysis of the ingress of chlorides

Figure 59: The concentration of chloride ions in a concrete bridge deck with a crack made from ordinary concrete without taking into account the effect of the curing of the concrete. Deterministic solution - alternative 1a showing the dispersion of chloride concentrations, reinforcement and the network of FEA elements (Translation: Čas – Age, roky-years).

Figure 60: The concentration of chloride ions in a concrete bridge deck with a crack made from ordinary concrete without taking into account the effect of the curing of the concrete. Deterministic solution – P1EB and showing the isoline of the concentration of chlorides and reinforcement. (Translation: Čas – Age, roky-years)
Figure 60: continuation The concentration of chloride ions in a concrete bridge deck with a crack made from ordinary concrete without taking into account the effect of the curing of the concrete. Deterministic solution – P1EB showing the isoline of the concentration of chlorides and reinforcement.

(Translation: Čas – Age, roky -years)

Figure 61: The concentration of chloride ions in a concrete bridge deck covered by an asphalt cover with a crack in the middle of the bridge deck and with a defect in its waterproof insulation 40 cm from the left hand edge of the model. The output shown is for ordinary concrete when considering the effect of concrete curing. Deterministic solution – P2EB showing the isoline of the concentration of chlorides. (Translation: Čas – Age, roky -years)
Figure 62: The concentration of chloride ions in a concrete bridge deck covered by an asphalt cover with a crack in the middle of the bridge deck and with a defect in its waterproof insulation 40 cm from the left-hand edge of the model. The output shown is for ordinary concrete when considering the effect of concrete curing. Deterministic solution – P2EB showing the isoline of the concentration of chlorides.

(Translation: Čas – Age, roky -years)

Figure 63: The concentration of chloride ions in a concrete bridge deck with a crack made from high performance concrete taking into account the effect of the curing of the concrete. Deterministic solution - alternative P3EB showing the dispersion of chloride concentrations, reinforcement and the network of FEA elements. (Translation: Čas – Age, roky -years)
Figure 64: The concentration of chloride ions in a concrete bridge deck with a crack made from high performance concrete taking into account the effect of the curing of the concrete. Deterministic solution - P3EB showing the isoline of the concentration of chlorides.

(Translation: Čas – Age, roky - years)

8.4 Statistical parameters of the diffusion coefficient 3 chosen mixes

Table - 23: Diffusion coefficient, average value (mean), standard deviation (dev) and variation coefficient (mean/dev), curing coefficient (m) and calculation error $m$ smallest squares method (SSQE) for a referential time of 7 days (Konečný&Lehner, 2014, Ghosh et al., 2014).

<table>
<thead>
<tr>
<th>days</th>
<th>7</th>
<th>100TII-V</th>
<th>80TII-V/20C</th>
<th>80TII-V/20F</th>
<th>45TII-V/35G100S/20F</th>
</tr>
</thead>
<tbody>
<tr>
<td>mean</td>
<td>7.82547E-12</td>
<td>9.66018E-12</td>
<td>1.06876E-11</td>
<td>4.92304E-12</td>
<td></td>
</tr>
<tr>
<td>dev</td>
<td>33.6E-14</td>
<td>52.1E-14</td>
<td>38.4E-14</td>
<td>23.9E-14</td>
<td></td>
</tr>
<tr>
<td>mean/dev</td>
<td>0.043</td>
<td>0.054</td>
<td>0.036</td>
<td>0.049</td>
<td></td>
</tr>
<tr>
<td>m</td>
<td>0.25964</td>
<td>0.34287</td>
<td>0.53733</td>
<td>0.40188</td>
<td></td>
</tr>
<tr>
<td>SSQE</td>
<td>0.010715</td>
<td>0.027432</td>
<td>0.003208</td>
<td>0.000679</td>
<td></td>
</tr>
</tbody>
</table>

Table. - 24: Diffusion coefficient, average value (mean), standard deviation (dev) and variation coefficient (mean/dev), curing coefficient (m) and calculation error $m$ smallest squares method (SSQE) for a referential time of 14 days (Konečný&Lehner, 2014, Ghosh et al., 2014).

<table>
<thead>
<tr>
<th>days</th>
<th>14</th>
<th>100TII-V</th>
<th>80TII-V/20C</th>
<th>80TII-V/20F</th>
<th>45TII-V/35G100S/20F</th>
</tr>
</thead>
<tbody>
<tr>
<td>mean</td>
<td>7.29824E-12</td>
<td>9.15242E-12</td>
<td>7.68156E-12</td>
<td>3.55006E-12</td>
<td></td>
</tr>
<tr>
<td>dev</td>
<td>30.5E-14</td>
<td>43.4E-14</td>
<td>21.6E-14</td>
<td>15.1E-14</td>
<td></td>
</tr>
<tr>
<td>mean/dev</td>
<td>0.042</td>
<td>0.047</td>
<td>0.028</td>
<td>0.043</td>
<td></td>
</tr>
<tr>
<td>m</td>
<td>0.30773</td>
<td>0.39551</td>
<td>0.54195</td>
<td>0.38746</td>
<td></td>
</tr>
<tr>
<td>SSQE</td>
<td>0.022491</td>
<td>0.077373</td>
<td>0.005202</td>
<td>0.001354</td>
<td></td>
</tr>
</tbody>
</table>
Table 25: Diffusion coefficient, average value (mean), standard deviation (dev) and variation coefficient (mean/dev), curing coefficient (m) and calculation error $m$ smallest squares method (SSQE) for a referential time of 28 days (Konečný&Lehner, 2014, Ghosh et al., 2014).

<table>
<thead>
<tr>
<th>days:</th>
<th>28</th>
<th>mean</th>
<th>dev</th>
<th>mean/dev</th>
<th>m</th>
<th>SSQE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100TII-V</td>
<td>5.58522E-12</td>
<td>24.1E-14</td>
<td>0.043</td>
<td>0.284</td>
<td>0.008076</td>
</tr>
<tr>
<td>2</td>
<td>80TII-V/20C</td>
<td>6.06064E-12</td>
<td>32.9E-14</td>
<td>0.054</td>
<td>0.38235</td>
<td>0.021334</td>
</tr>
<tr>
<td>3</td>
<td>80TII-V/20F</td>
<td>5.3757E-12</td>
<td>17.8E-14</td>
<td>0.033</td>
<td>0.51381</td>
<td>0.006392</td>
</tr>
<tr>
<td>24</td>
<td>45TII-V/35G100S/20F</td>
<td>2.75073E-12</td>
<td>9.7E-14</td>
<td>0.035</td>
<td>0.40356</td>
<td>0.000764</td>
</tr>
</tbody>
</table>

Table 26: Diffusion coefficient, average value (mean), standard deviation (dev) and variation coefficient (mean/dev), curing coefficient (m) and calculation error $m$ smallest squares method (SSQE) for a referential time of 56 days (Konečný&Lehner, 2014, Ghosh et al., 2014).

<table>
<thead>
<tr>
<th>days:</th>
<th>56</th>
<th>mean</th>
<th>dev</th>
<th>mean/dev</th>
<th>m</th>
<th>SSQE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100TII-V</td>
<td>4.84818E-12</td>
<td>23.6E-14</td>
<td>0.049</td>
<td>0.25579</td>
<td>0.011741</td>
</tr>
<tr>
<td>2</td>
<td>80TII-V/20C</td>
<td>4.7784E-12</td>
<td>14.4E-14</td>
<td>0.030</td>
<td>0.37087</td>
<td>0.021263</td>
</tr>
<tr>
<td>3</td>
<td>80TII-V/20F</td>
<td>3.34218E-12</td>
<td>13.4E-14</td>
<td>0.040</td>
<td>0.56855</td>
<td>0.003625</td>
</tr>
<tr>
<td>24</td>
<td>45TII-V/35G100S/20F</td>
<td>2.30575E-12</td>
<td>8.6E-14</td>
<td>0.037</td>
<td>0.35428</td>
<td>0.001726</td>
</tr>
</tbody>
</table>

Table 27: Diffusion coefficient, average value (mean), standard deviation (dev) and variation coefficient (mean/dev), curing coefficient (m) and calculation error $m$ smallest squares method (SSQE) for a referential time of 91 days (Konečný&Lehner, 2014, Ghosh et al., 2014).

<table>
<thead>
<tr>
<th>days:</th>
<th>91</th>
<th>mean</th>
<th>dev</th>
<th>mean/dev</th>
<th>m</th>
<th>SSQE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100TII-V</td>
<td>3.68105E-12</td>
<td>14.7E-14</td>
<td>0.040</td>
<td>0.31928</td>
<td>0.011084</td>
</tr>
<tr>
<td>2</td>
<td>80TII-V/20C</td>
<td>3.58426E-12</td>
<td>22.6E-14</td>
<td>0.063</td>
<td>0.41763</td>
<td>0.025756</td>
</tr>
<tr>
<td>3</td>
<td>80TII-V/20F</td>
<td>2.37643E-12</td>
<td>5.1E-14</td>
<td>0.021</td>
<td>0.59746</td>
<td>0.00669</td>
</tr>
<tr>
<td>24</td>
<td>45TII-V/35G100S/20F</td>
<td>1.71593E-12</td>
<td>5.2E-14</td>
<td>0.030</td>
<td>0.40629</td>
<td>0.000697</td>
</tr>
</tbody>
</table>

Table 28: Diffusion coefficient, average value (mean), standard deviation (dev) and variation coefficient (mean/dev), curing coefficient (m) and calculation error $m$ smallest squares method (SSQE) for a referential time of 161 days (Konečný&Lehner, 2014, Ghosh et al., 2014).

<table>
<thead>
<tr>
<th>days:</th>
<th>161</th>
<th>mean</th>
<th>dev</th>
<th>mean/dev</th>
<th>m</th>
<th>SSQE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100TII-V</td>
<td>3.09653E-12</td>
<td>13.0E-14</td>
<td>0.042</td>
<td>0.31788</td>
<td>0.010493</td>
</tr>
<tr>
<td>2</td>
<td>80TII-V/20C</td>
<td>2.84996E-12</td>
<td>11.5E-14</td>
<td>0.040</td>
<td>0.41609</td>
<td>0.025033</td>
</tr>
<tr>
<td>3</td>
<td>80TII-V/20F</td>
<td>1.91254E-12</td>
<td>9.1E-14</td>
<td>0.048</td>
<td>0.59483</td>
<td>0.002911</td>
</tr>
<tr>
<td>24</td>
<td>45TII-V/35G100S/20F</td>
<td>1.43033E-12</td>
<td>5.8E-14</td>
<td>0.041</td>
<td>0.3883</td>
<td>0.000607</td>
</tr>
</tbody>
</table>