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A CRITICAL DISCUSSION ON THE USEFULNESS AND RELIABILITY OF MATHEMATICAL MODELING FOR SERVICE LIFE DESIGN OF INFRASTRUCTURE

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Abstract. In view of the increasing age of existing structures asset managers are becoming more and more interested to have a clearer picture on the actual condition of the complete stock of existing infrastructure as to anticipate on possible maintenance regarding planning and allocation of the financial resources. Consequently, a clear need is emerging for prediction of the condition level over time using mathematical models. Regarding the design of new structures, the current codes are based on traditional options and thus give ample possibilities for alternative options. For instance, at present the significantly different performance of binders is not taken into account. Therefore it is not surprising that in recent years a clear trend can be observed towards the application of mathematical modelling using a probabilistic approach for durability, e.g. the fib Model Code on Service Life Design. In order to allow for prediction of the condition of a structural component over time or to demonstrate equal performance of design solutions, widely accepted mathematical models that describe degradation processes are required. Ideally, such models should be mathematically and physically sound, provide logical and realistic results, understandable and usable for practitioners, and thus to be to a considerable extent foolproof. However, most models include significant pitfalls and limitations which are either not mentioned or not known even to the developer. In addition, in most cases the quantification of the input parameters is not addressed which will undoubtedly result in ‘shopping’. In this respect the use of input values based on expert opinion should be treated with serious caution. In addition it has to be noted that most models have been calibrated on results obtained for laboratory experiments that have been performed under ideal conditions not reflecting situations encountered in practice. Experience has also shown that probabilistic approaches are frequently misused as to support a wrong decision or an execution error (shallow cover depths).

1 INTRODUCTION

Through regional authorities Rijkswaterstaat acts as the asset manager of the Dutch national road network and waterways system. In this role Rijkswaterstaat is responsible for a wide range of structures, e.g. bridges, viaducts, overpasses, tunnels, sluices, and sea defense structures. Given the high economical importance of the national network it is logical that highways agencies will adopt a more conservative approach in the design of new structures in order to minimize major maintenance activities and thus traffic disruption during the full operational service life, see Figure 1. This implies that any changes in design solutions from the common approach are normally treated with caution as this imposes a potential and unknown risk for the future availability of the network system.

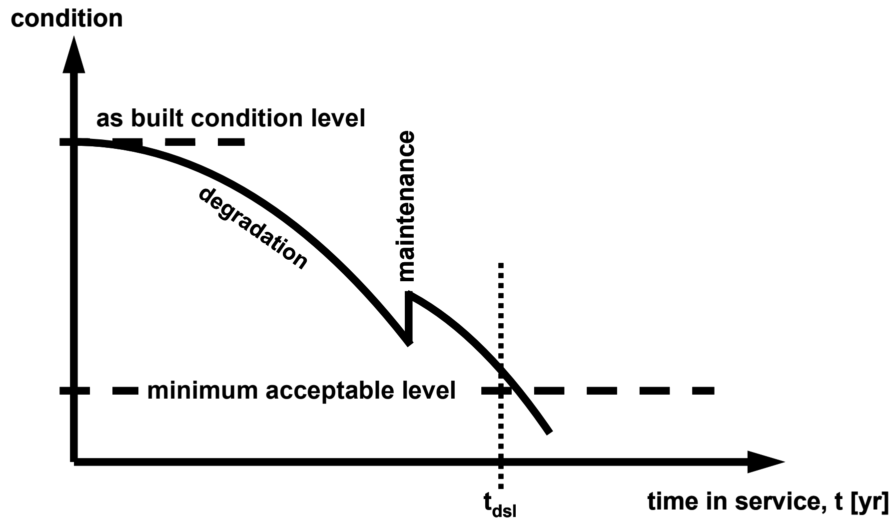

Figure 1: Conceptual model for condition development of infrastructural facilities aimed at a minimum of maintenance during the operational service life.

For most infrastructural facilities it is common to adopt a design service life of 100 years, i.e. much longer than the experience obtained until now based on the stock of existing structures. In practice, defect joints in bridges and viaducts clearly impose most of the maintenance problems necessitating frequent repairs at intervals of 10 to 20 years. Generally, reinforced and pre-stressed concrete are considered the most versatile and economical solution for most infrastructural facilities, resulting in a largely maintenance free service life. However, in literature it is often argued that a significant number of concrete structures suffer from premature corrosion of the embedded reinforcing steel, posing a major threat for serviceability and necessitating vast amounts of money for repair. In the Netherlands, however, until now damage due to corroding reinforcement is very limited although a significant number of structures have reached an age in excess of 50 years. With respect to the infrastructure facilities owned and managed by the Ministry of Infrastructure this is attributed to a combination of conservative design, good quality control during construction, inspections and adequate preventive maintenance at regular time intervals, relative moderate exposure conditions, and the use of concrete based on blast furnace slag cement with a minimum slag content of 50% or fly ash cement having a minimum fly ash content of 25%. In situations where damage has occurred this is limited to specific areas and most often due to a lower cover depth than the design cover and poor execution practices (compaction and curing).

In view of the increasing age of existing structures asset managers are becoming more and more interested to have a clearer picture on the actual condition of the complete stock of existing infrastructure as to anticipate on possible maintenance regarding planning and allocation of the financial resources. Consequently, there is also a clear need for prediction of the condition level over time. Regarding the design of new structures, the current codes are based on traditional options and thus give ample possibilities for alternative options. For instance, at present the significantly different performance of binders is not taken into account. Therefore it is not surprising that in recent years a clear trend can be observed towards the application of mathematical modelling using a probabilistic approach for durability, e.g. the fib Model Code on Service Life Design [1]. In the Netherlands a guideline for durability design was published providing performance criteria regarding chloride induced corrosion based on the model used in the European DuraCrete project [2, 3].

This paper will critically discuss issues related to probabilistic approaches for service life design of new concrete structures based on mathematical modelling of relevant degradation mechanisms, i.e. reinforcement corrosion resulting from carbonation and chloride ingress.

2 RELEVANT REQUIREMENTS FOR MATHEMATICAL MODELS

In order to allow for prediction of the condition of a structural component over time or to demonstrate equal performance of design solutions, widely accepted mathematical models that describe relevant degradation processes are required. Ideally, such models should be mathematically and physically sound, provide logical, meaningful and realistic results, understandable and usable for practitioners, and thus should be to a considerable extent foolproof.

In the literature, a significant number of mathematical models on single degradation mechanisms can be found, ranging from purely empirical, e.g. by employing a simple polynomial approach taking into account parameters considered to be relevant, to models demonstrating a high academic level incorporating mathematical functions and parameters that are unknown or at least rare to most practitioners. It is clear that for a sound use in practice the latter level of models will surely be inappropriate as the associated mathematical expression will hardly be properly understood.

Most often models include significant pitfalls, constraints and limitations which are either not mentioned or not known even to the developer of the model. Consequently, models are frequently used in situations for which the model is not applicable. In addition, in most cases the quantification of the input parameters is not addressed which will undoubtedly result in ‘shopping’ or quantification by expert opinion, engineering judgement, wishful thinking or through random guesswork. In this respect the use of input values that is not supported by factual evidence, e.g. measurement results or other data, should be treated with serious caution. Frequently the qualification ‘expert opinion’ or ‘state-of-the-art’ is often misleading. It has to be noted that most models used for practice have been calibrated on results obtained from laboratory experiments that have been performed under ideal conditions not reflecting relevant situations encountered in practice.

3 MODELS FOR INITIATION OF REINFORCEMENT CORROSION

As corrosion of embedded reinforcing steel is identified as the major problem with respect to serviceability and maintenance of concrete structures, during the last decades most attention has been focused on the development of mathematical expressions for carbonation and chloride ingress. For new structures initiation of reinforcement corrosion is commonly accepted as the end of design service life. This situation is considered to be achieved either when the concrete cover has become carbonated or when the chloride content at the level of the reinforcement has reached a critical content, C_{crit}.

The expression most commonly used in practice to predict carbonation depth, x_c, is based on a simple square root of time relationship, according to:

$$x_c(t) = A\sqrt{t} \tag{1}$$

The empirical constant A is derived from carbonation depths measured on real structures. This expression will usually yield a conservative estimate of the carbonation rate as the ingress of carbon dioxide is not only dependent on the ambient relative humidity but also on the frequency of rain events.

These environmental effects have been implemented in the model used in the fib Model Code for Service Life Design [1] resulting in an extended version in which the development of the carbonation depth, x_c, over time is predicted by:

$$x_c(t) = \sqrt{2 k_e k_c (k_t R_{ACC,0}^{-1} + \varepsilon_t) C_S} \sqrt{t}\, W(t) \tag{2}$$

At first glance this mathematical expression seems to contain 8 parameters, however, in the elaborated expression 19 basic parameters are identified, including ambient CO_2 concentration, Cs, relative humidity of the prevailing exposure environment, k_e, rain events, W(t), and curing time, k_c. For concrete surfaces exposed to rain the development of carbonation depth over time will follow a power law relationship, according to t^n, with $n \leq 0.50$, see Figure 2.

In Eq. (2) basically 7 parameters are imposed either by a constant value or by fixed statistical characteristics. Overall 6 parameters are empirical in nature as these have been quantified by regression analysis comparing measured and predicted performance. As a conclusion, the model proposed by the fib Model Code [1] has to be qualified as being highly empirical. This implies that this model including its quantified parameters may be valid only for those situations that have been used for calibration.

In fact, for given environmental exposure conditions, only the inverse effective carbonation resistance of concrete, $R_{ACC,0}^{-1}$, as determined by an accelerated test can be influenced by the user through modifying the concrete composition, i.e. type of cement and water to cement ratio. In [1] a table is provided giving indicative values for $R_{ACC,0}^{-1}$ for a range of water to cement ratios and binder combinations. These indicative values have been introduced in Eq. (2) as to evaluate the potential performance of these binders for concrete exposed to identical exposure conditions (atmospheric carbon dioxide concentration, w/c ratio, relative humidity, and rain events), see Figure 3.

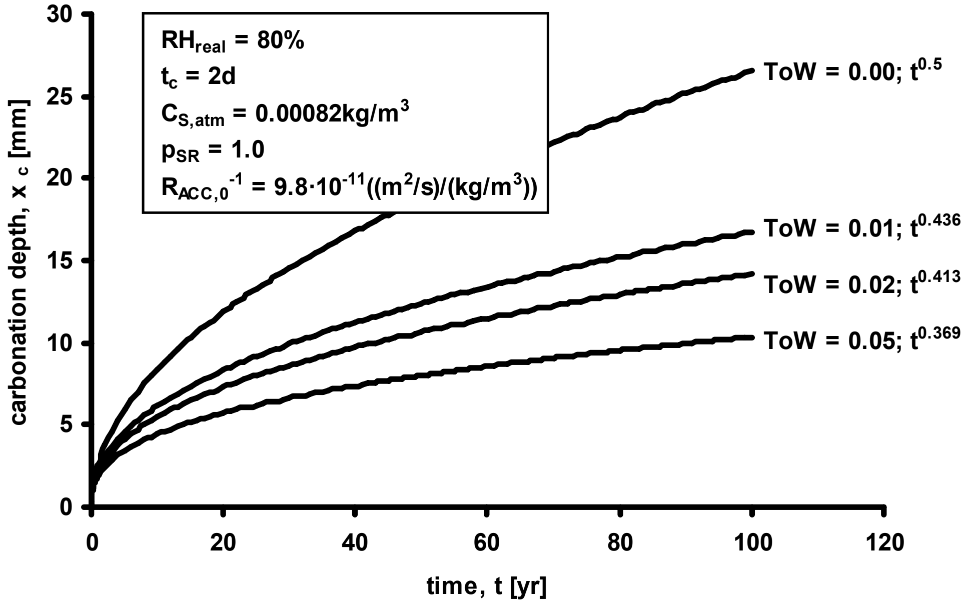

Figure 2: Development of carbonation depth over time according to Eq. (2) demonstrating the influence of ToW (Time of Wetness, i.e. the relative number of days per year with a daily rainfall in excess of 2.5mm).

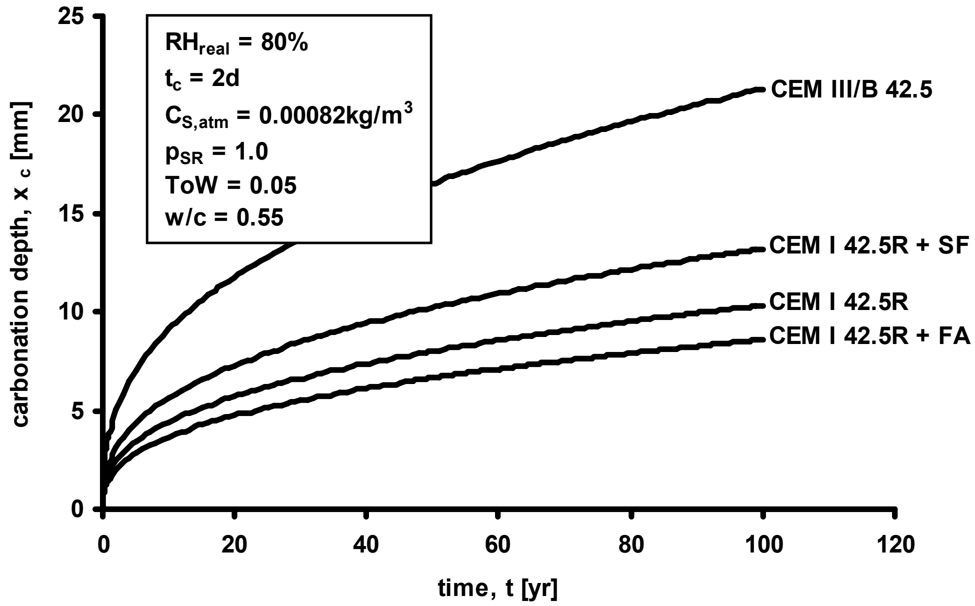

Figure 3: Development of carbonation depth over time according to Eq. (2) demonstrating the influence of type of cement (CEM I reflects ordinary Portland cement; CEM III/B contains more than 65% of blast furnace slag; FA = Fly Ash; SF = Silica Fume).

For Rijkswaterstaat the pronounced different results obtained for blast furnace slag cement (CEM III/B) seem highly unrealistic as experience in practice has clearly demonstrated that concrete made with CEM III/B has an equal performance to CEM I regarding carbonation. Consequently, the model, the input values, and the test method are seriously questioned.

For chloride ingress a modification of the mathematical solution to Fick's 2nd law of diffusion is often used. Both DuraCrete [3] and the fib Model Code for Service Life Design [1] use a similar basic format, adopting a so-called factorial approach, with the expression employed in DuraCrete [1] given by:

$$C(x,t) = C_s - (C_s - C_i)\, erf\left(\frac{x}{2\sqrt{k_e\, k_c\, D_{nssm,0} \left(\frac{t_0}{t}\right)^n t}} \right) \tag{3}$$

With the help of Eq. (3) the development of chloride content over distance to the exposed concrete surface, x, can be calculated at any time, representing the so-called chloride profile, or the development over time of the chloride content at any depth, see Figures 4 and 5, respectively.

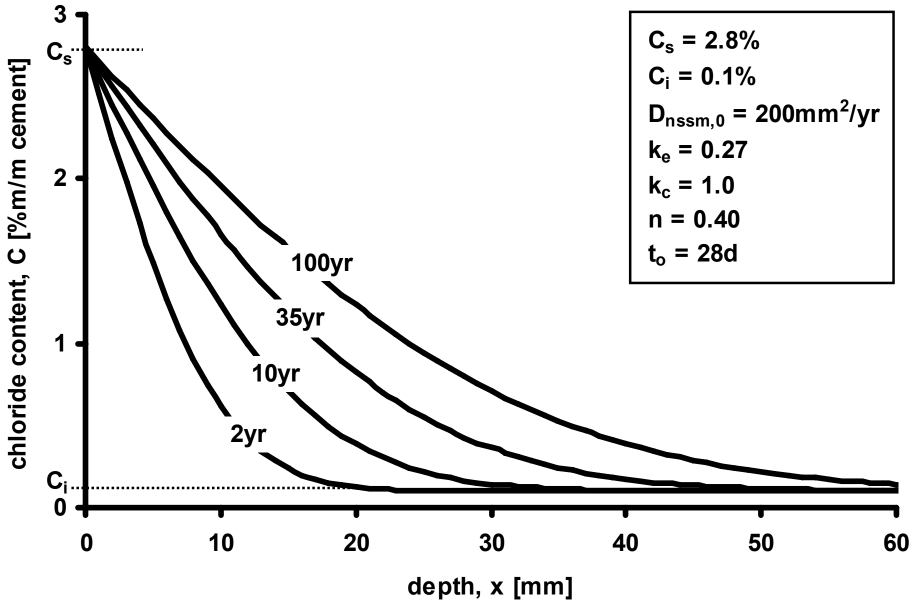

Figure 4: Examples of chloride profiles demonstrating the development of chloride ingress over time according to Eq. (3).

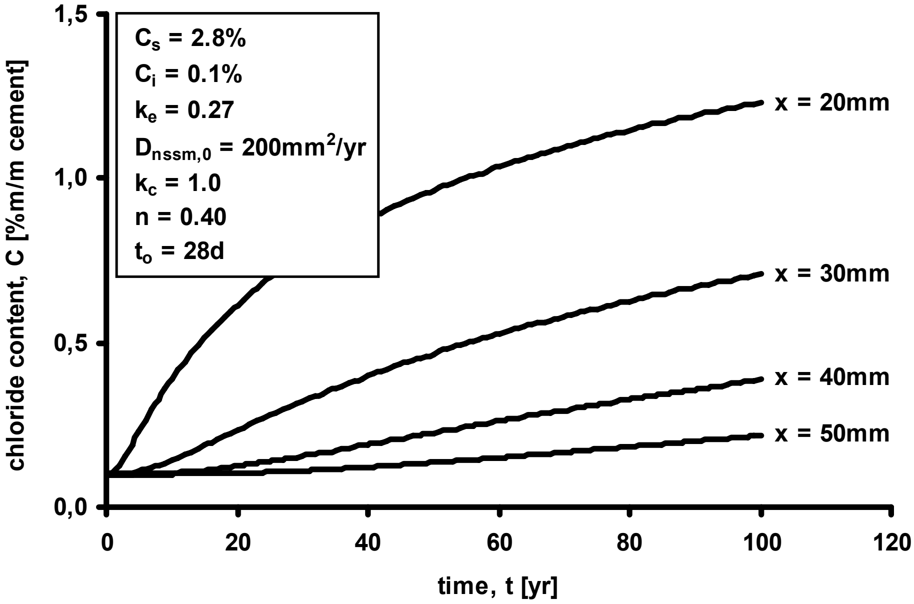

Figure 5: Development of chloride content over time at specific depths according to Eq. (3).

However, Eq. (3) does not satisfy the underlying differential equation. Consequently, numerical modeling based on a sequence of small time steps may yield significantly different results. Although the correct mathematical solution has been known for more than 20 years still no action is undertaken to modify the expression, see e.g. [4]. In addition, Eq. (3) is valid for 1-dimensional situations of chloride ingress for an infinitely thick homogeneous (porous) medium. Thus, as an example Eq. (3) cannot be applied to certain geometries, e.g. cylindrical columns, or to thin concrete elements exposed on both sides to chlorides. However, these limitations are always forgotten or unknown to the consulting engineer.

In contrast to carbonation, for chloride ingress there is no clear depassivation front as the resulting chloride profile shows a gradual decrease with depth, see Figure 4, and due to the fact that the chloride content resulting in the onset of corrosion, C_{crit}, remains largely uncertain, demonstrating a high scatter (dependent amongst others on the test method being used). In practice it is often left to the 'expert's' discretion what value for C_{crit} will be adopted. In addition a wide range of values for the so-called ageing exponent, n, can be found in the literature used to express the improvement of concrete quality over time. As the ageing exponent has a dramatic influence on the overall result, it may be clear that shopping around is not advocated, at least from the owner's point of view. In addition, the implicit assumption is being made that this ageing effect will continue until infinite. However, to the logical thinker this beneficial effect seems highly unlikely to occur over the long term as the increase in resistance against chloride ingress is attributed to cement hydration and drying-out. Both contributions will gradually come to an end within at most 50 years and it would be very unwise to anticipate on these assumed long-term improvements. Therefore the chloride ingress model should be used with serious caution for a design service life in excess of 50 years and forbidden to be used for a design service life in excess of 100 years. Despite this the Dutch durability guideline [2] contains tables for 80, 100 and even 200 years, yielding unrealistically optimistic results regarding cover depth, see Figure 6. According to the prevailing European standard [5] for a design service life t_{dsl} = 100 year the nominal cover depth has to be increased by 10mm relative to the nominal cover depth required for t_{dsl} = 50yr. Thus it has to be concluded that according to [2] the European standard has to be regarded much too conservative: an extra of 1mm to 5mm at the most will already be sufficient to achieve an extension to 100 years design service life. It will be evident that any respectable and knowledgeable asset manager will never accept such a result, even if this result is said to be the calculated outcome of the mathematical model employed in DuraCrete.

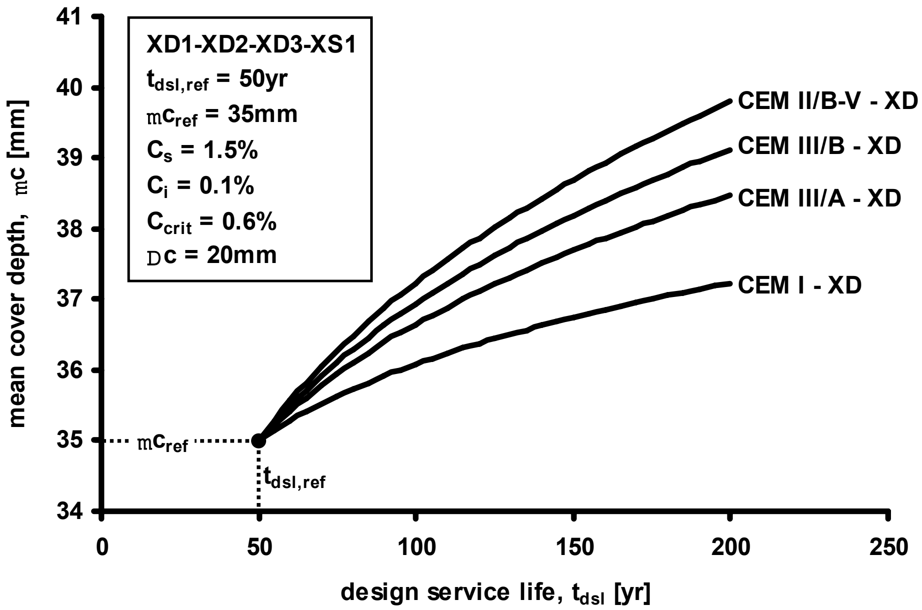

Figure 6: Cover depth according to Polder et al. [2,10] for t_{dsl} = 100yr, using $_mc$ = 35mm for t_{dsl} = 50yr as a reference situation.

4 PROBABILISTIC APPROACHES

4.1 General

Given the high variability of all influencing factors, e.g. cover depth and exposure conditions; it is logical that a probabilistic approach similar to that adopted for structural design is often considered to be more appropriate. The first steps for a probabilistic approach for service life design were made in the European project DuraCrete, resulting in an output of 17 reports, the Final Technical Report [3] being the most quoted and used in literature. The main focus was on reinforcement corrosion induced by chloride ingress and carbonation. The mathematical expressions and input values proposed by DuraCrete constitute the basis for both the fib Model Code for Service Life Design [1] and the Dutch CUR Guideline 1 [2]. In the following the semi-probabilistic approach for chloride induced corrosion adopted by the CUR Guideline will be discussed in more detail.

4.2 CUR Guideline 1

The major task of the Dutch CUR VC81 committee was to develop performance criteria for concrete exposed to chlorides (comprising exposure classes XD and XS), thus allowing more freedom for the contractor to choose the most economical solution. In line with this performance criteria related to concrete quality were made dependent on thickness of concrete cover and type of cement. As a full probabilistic approach was considered too laborious and tricky to be used in practice a simplification was introduced. This resulted in a 'semi-probabilistic' approach [2,6], which was based on a graph presented by Rostam [7]. In essence, the underlying reasoning was that the use of mean values for all model parameters would result in an unacceptable probability of depassivation, P_{dep} = 0.50, but through the introduction of an allowance for (mean) cover depth Dc = 20mm, it was thought that P_{dep} would be reduced to a level of 0.10, being the proposed criterion for normal reinforcing steel. For pre-stressing steel Dc =- 30mm was chosen considered to correspond to P_{dep} = 0.05, see Figure 7. It should be noted that originally Dc = 15mm (reinforcing steel) and Dc = 25mm (pre-stressing steel) were adopted [6].

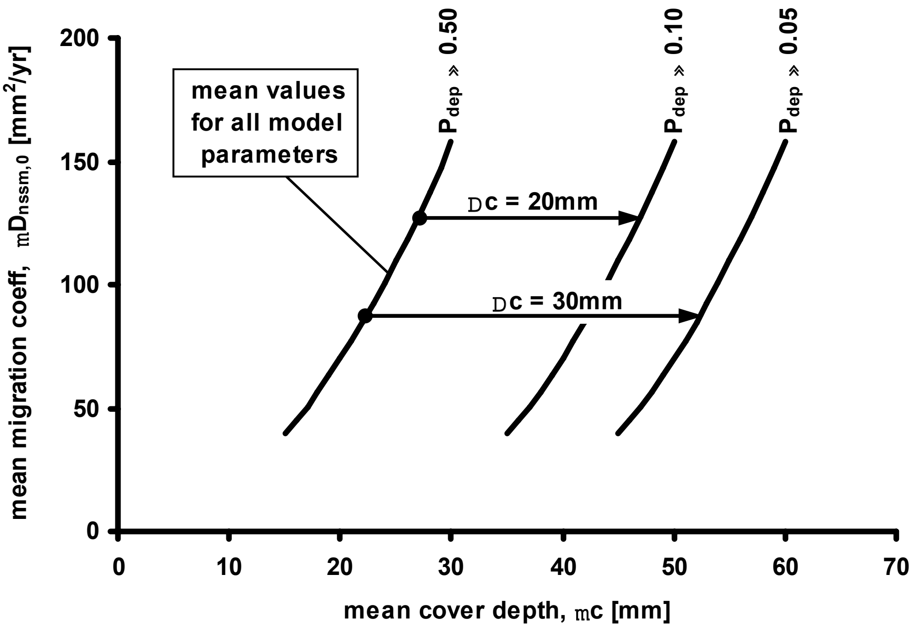

Figure 7: Basic principle of the semi-probabilistic approach suggested by Polder et al. [2,10] for achieving P_{dep} = 0.10 (reinforcing steel) and P_{dep} = 0.05 (pre-stressing steel).

The non-steady state migration coefficient $D_{nssm,0}$ is determined at a concrete age t_0 = 28d employing a laboratory test with chloride ingress being accelerated through an applied electrical voltage applied over a concrete specimen which results in an electrical field of typically 600 V/m[1] [8,9].

In the final CUR Guideline it is written that: 'with this (semi-probabilistic) approach it is expected that the probability of corrosion initiation at the end of the intended design service life will be less than approximately 10% for reinforcing steel and less than approximately 5% for pre-stressing steel'. Gulikers [10] performed detailed full probabilistic calculations for all combinations of cover depth and migration coefficient, $D_{nssm,0}$, that are allowed for reinforcing steel according to CUR Guideline 1 as to establish the corresponding levels of P_{dep} see Figure 8.

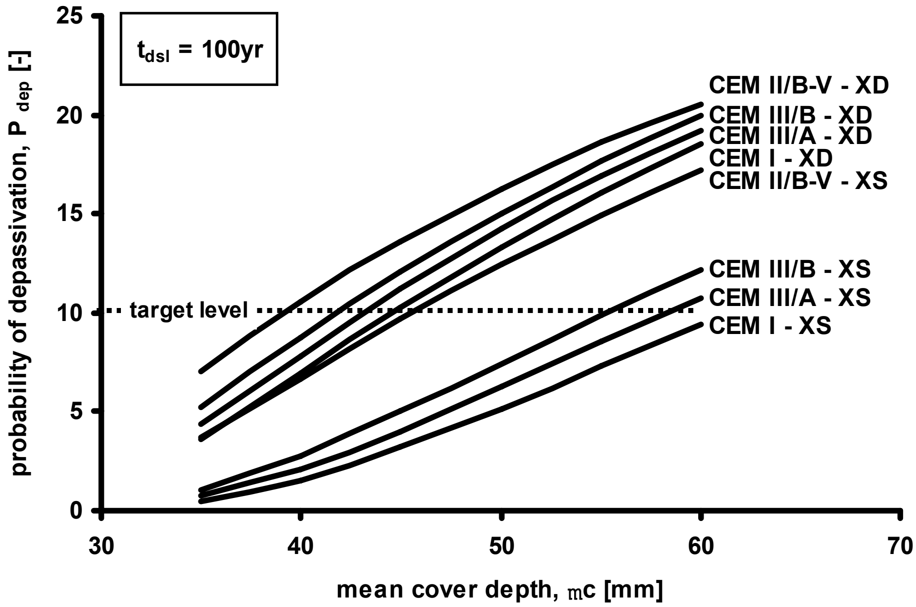

Figure 8: Relationship between acceptable combinations mc-$D_{nssm,0}$ according to the semi-probabilistic approach suggested by Polder et al. [2,10] and probability of depassivation for reinforcing steel according to a full probabilistic approach for $_t$dsl = 100yr.

From Figure 8 it can be deduced that for a design service life t_{dsl} = 100yr the actual values of P_{dep} range from 0.05 for mc = 35mm (CEM I-XS) to P_{dep} = 0.21 for mc = 60mm (CEM II/B-V-XD), whereas P_{dep} = 0.10 was mentioned as the target level. This would imply that an assumingly knowledgeable and reliable principal, like Rijkswaterstaat, by accepting CUR Guideline 1would consider design solutions corresponding to P_{dep} = 0.21 as equal as solutions with those corresponding to P_{dep} = 0.05. For use in practice this large range of P_{dep} claimed to result in equal durability performance is considered highly undesirable in real life as this would undoubtedly result in misuse by the contractor or concrete producer. Thus the conclusion is that this so-called semi-probabilistic approach is incorrect and misleading and that it is merely presented as a selling argument to convince owners, contractors and concrete producers rather than a sound and acceptable simplification. Such a mathematical simplification as used in [2] has therefore to be considered a rule of thumb at the best.

It should be noted that CUR Guideline 1 [2] presents tables for allowed combinations of concrete cover depth and concrete quality ($D_{nssm,0}$), however this document does not contain any graphs demonstrating the principle of this 'semi-probabilistic' approach, as well as the graphical relationship between mc and $D_{nssm,0}$, and the actual probability of depassivation for each combination mc-$D_{nssm,0}$ that is allowed.

Although the main authors have indicated to be fully aware of the dramatic errors in the approach, the unrealistic nature of the calculation results, the strong indications that the theoretical basis of the migration test is questionable, and the serious problems that will be encountered in practice for the concrete producers and contractors, after publication of CUR Guideline 1 they have authored several articles on CUR Guideline 1, presenting it as 'the Dutch approach', see e.g. [11]. However, Rijkswaterstaat has officially rejected this guideline to be used for their projects [12], and according to [13] in the Netherlands hardly any projects can be found in which this approach has been applied.

5 CONCLUSIONS

- Mathematical models for service life design should be mathematically and physically sound, provide logical and realistic results, understandable and usable for practitioners, and thus to be to a considerable extent foolproof. However, most models available at present do not obey these essential requirements.
- Through the introduction of calibration factors most models are basically empirical in nature and thus the output will only be valid for the conditions employed for calibration. However, such conditions may not prevail in practice for real structures.
- Due to a lack of data input parameters are often quantified based on 'expert opinion'. However, generally this is not explicitly mentioned. Since calibration factors have been quantified using such input parameters, any change of expert opinion in the course of time should be accompanied a re-quantification of the associated calibration factors.
- Qualifications like expert opinion, state-of-the-art and semi-probabilistic approach have to be regarded with serious caution as these qualifications are frequently misused to conceal lack of information, knowledge, and responsibility.
- Full and semi probabilistic approaches for service life design are prone to manipulation of input values as to achieve the desired result. Neither the principal, asset manager, consulting engineer, concrete producer nor contractor have sufficient knowledge and expertise to validate the input and the output of probabilistic calculations.
- A critical evaluation of the reality level of the output generated by models through graphs is a basic requirement.
- The limitations, constraints and assumptions of any mathematical model should be clearly identified and explicitly indicated.
- The actual physical meaning of probability of depassivation of reinforcing steel, remains largely unknown. In literature this issue is hardly addressed as the main focus is on the calculation exercise. However, for the asset manager probabilities are less important as he is interested in the associated consequences and thus in his risk. This aspect is often overlooked.

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