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NEW APPROACH TO SERVICE LIFE DESIGN OF CONCRETE STRUCTURE

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ABSTRACT

Developed in a European Brite-Euram research project, a new service life design concept for reinforced concrete structures has been established. This new concept allows the design of reinforced concrete structures for a defined lifetime related to limit state formulations. The result of the durability design is a limit state-based failure probability of the structure. This new approach has been used for the design against reinforcement corrosion in uncracked concrete regions for various new structures as well as for the redesign and estimation of remaining service life of existing structures. These practical applications demonstrated the appropriateness of the approach. Within this publication the application of the approach to a bored tunnel construction in the Netherlands (Western-Scheldt project) will be presented. The construction consists of two different elements, the ramp and the bored tunnel itself (reinforced concrete and steel members). The tunnel has two tubes with an external diameter of 11.0 m and a length of 6.5 km. All elements have been designed according to the new performance based durability design procedure.

Keywords: durability, probabilistic based service life design, concrete structures, tunnel

1. INTRODUCTION

At present, design codes and guidelines include prescriptive requirements, to ensure sufficient durability of reinforced concrete structures. Prescriptive rules relating to environmental factors are given (maximum water/cement (w/c) ratio, minimum binder content, nominal concrete cover, etc.). Further rules (e.g. concerning curing, and air entrainment to avoid frost and frost-deicing-salt-deterioration) complete this type of durability design. An objective comparison between various options to improve durability as well as a limit state-related lifetime design is not possible.

As formulated by Bamforth [1], a structural engineer would consider a code allowing only some few loading regimes, each of which additionally being based on minimum dimensions, minimum concrete strength and minimum volume of steel, to be wholly

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inadequate.

Although the described prescriptive design approach would be unacceptable to the structural engineer, this type of approach is accepted for durability problems. Growing durability-related problems and damages to reinforced concrete structures in the past highlight the necessity of establishing not only a new performance-based durability design approach but also the need to integrate such a new approach into the standard procedures of structural design.

The following sections will clarify the new overall durability design concept for the quantitative determination of the lifetime of a structure. Based on this concept, a durability design example carried out within the Westerschelde project is presented.

2. DESCRIPTION OF THE NEW DESIGN CONCEPT

2.1 General

The general concept for durability design, which was introduced in its main features by an international expert group [2], is sketched in Figure 1. Following the definition of the minimum required performance of the structure to be built, usually formulated by the investor, the supervising authority and the designer, the introduced concept allows various design strategies (basic defence strategies).

One option of the basic defence strategy is the total avoidance of the deterioration mechanism (strategy A). Strategy A can be subdivided into different possibilities. For example:

A1Change the environment (loading), e.g. by linings, membranes and coatings.

A2Select non-reactive materials (infinite resistance), e.g. stainless steel or coated steel.

A3Inhibit the reaction, e.g. cathodic protection or cathodic prevention.

Generally, a detailed design is necessary for the various options of strategy A. Strategy A can be successfully used to prevent frost attack (by providing an appropriate air void system) and sulphate attack (by using sulfate resistant binders).

This paper will only deal with strategy B. Strategy B minimizes deterioration by optimal design and choice of materials. This strategy is used to prevent attack causing reinforcement corrosion. For structures or parts of a structure exposed to very aggressive environmental conditions, multi-barrier protection strategies are recommended. Such a strategy consists of:

- excellent concrete quality plus
- increased cover plus
- extra protection for the concrete or the reinforcement and/or
- provisions for later extra protection should it become necessary.

These measures may be completed by monitoring of the structure e.g. by installing sensors into the most sensitive parts of the structure.

The design of concrete quality and concrete cover may be done on different levels. For the majority of structures the "macro-level" is sufficient. What is meant by "macro-level" is the application of the prescriptive rules currently employed (specification of w/c ratio, cover thickness etc.), though based on probabilistic models (refinement of these rules may be necessary).

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For structures or parts of a structure exposed to aggressive environmental conditions and built for long target service life (100 years or more), a "meso-level" design is recommended. Based on simplified deterioration models, this level of design requires an adequate identification of the environmental loading, an adequate modelling of the prevailing transport mechanisms as well as performance tests, characterizing the material resistance to deterioration. In the following the durability design concept at the meso-level will be introduced. Finally, the basic information that has to be obtained in order to perform a complete probabilistic based durability design will be listed.

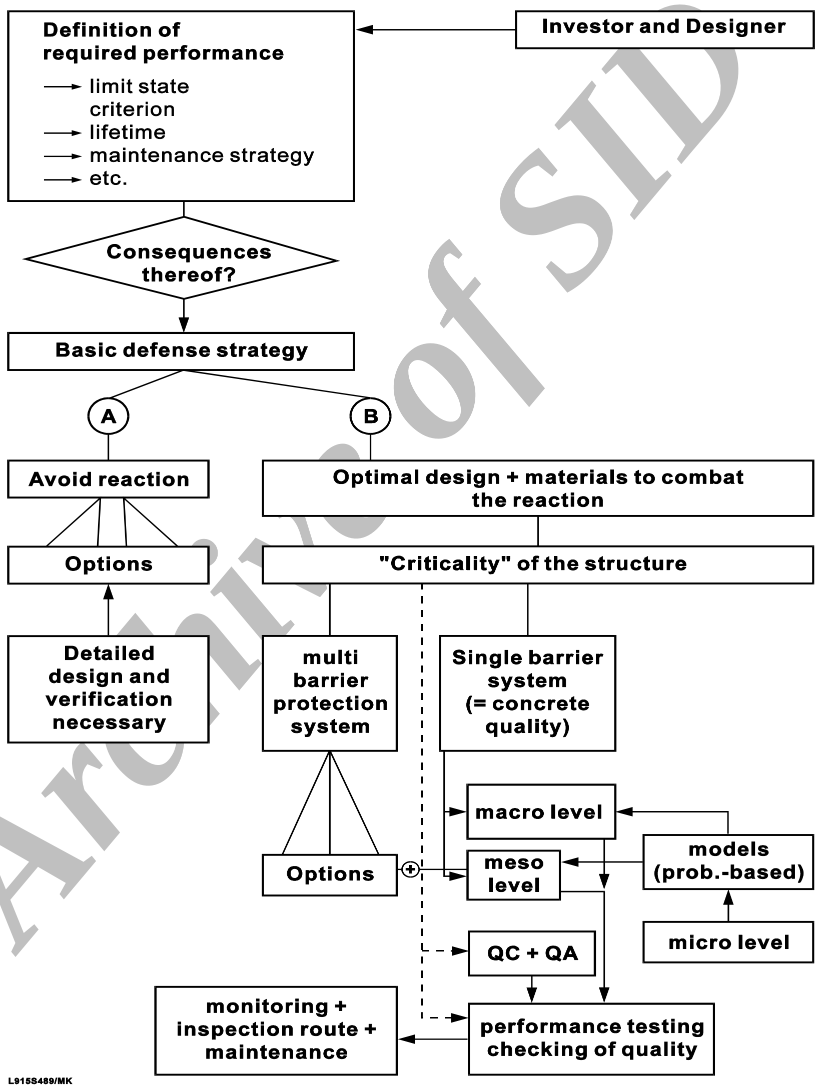

Figure 1. Format of a model code for durability design [2]

2.2 Safety Concept of a Probabilistic Design

Generally design processes are based on the comparison of the resistance of the structure

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(the R variable) with the action or load (often called the S variable). Failure occurs when the resistance is lower than the load. Since the load on a construction as well as the resistance sometimes highly variable, S and R cannot be compared in a deterministic way. The decision has to be based on maximum acceptable failure probabilities. The probability of failure, denoted p_f, describes the case when a variable resistance R is lower than a variable load S. This probability is required to be lower than the target probability of failure p_{Target} (Equation (1)).

$$p\{\text{Failure}\} = p_f = p\ \{R - S < 0\} < p_{Target} \qquad (1)$$

with:

p_f: failure probability

p_{Target}: target failure probability

For that kind of design problems, it is necessary to calculate the relevant probability of failure. Here the procedure of Basler [3] using the notation of Cornell [4] is advised. This procedure starts with the limit state function Z = R – S, and introduces the variables R and S to the equations including their average and standard deviation. Z is the reliability of the construction. Assuming that the variables S and R are normally distributed, the reliability Z, the difference between the variables R and S, is a variable which itself is normally distributed. Averages and standard deviations of the parameters can be calculated according to the relationship given in Figure 2. The so-called reliability index β is also introduced in this figure. This index is often used for design purposes. The reliability index is calculated using the average and the standard deviation of Z: μ_Z and σ_Z respectively. The reliability index is the difference between the mean values of R and S divided by the standard deviation of the variable Z or, alternatively, the mean value of Z divided by the standard deviation of Z.

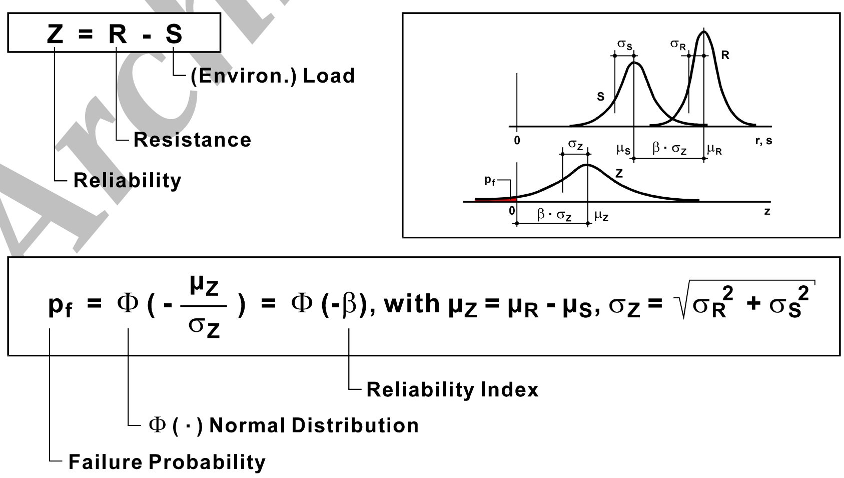

Figure 2. Probabilistic performance based durability design, terms, safety concept

For a normally distributed reliability function the relation between the failure probability p_f and the reliability index β is given in Table 1.

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Table 1. Interrelation between failure probability and reliability index for a normally distributed reliability function

Reliability Index β	**Failure Probability p_f in %**
1.5	6.6807000
1.8	3.5930000
2.0	2.2750000
3.0	0.1349900
3.6	0.0159110
3.8	0.0072348

The proceeding according to Basler [3] can easily be transformed into a design equation. The design equation can be expressed as follows: $\beta \geq \beta_0$, where β_0 is the required safety level (reliability index).

2.3 Extension of the Safety Concept on Probabilistic Basis

The general safety concept described here can rarely be applied in the aforementioned simple form. Extensions of the safety concept concern the transition from two variables to numerous variables, from linear functions to non-linear functions and from normal distributed variables to arbitrarily distributed variables. Extensions of these types do not allow manual calculations, so problems can only be solved by the use of computer programs (for example STRUREL [5], which has been used for the design described in this paper).

2.4 Requirements for a Lifetime Design

To carry out a lifetime design the following information is required:

a) a global design model in combination with relevant deterioration models is necessary to describe the time dependent development of the resistance R of the structure and the environmental loading S
b) sensible and operational limit states have to be set up
c) the investor and the supervising authority have to define a maximum admissible failure probability (reliability index) related to the limit states formulated earlier
d) a target service life should be defined by the investor.

3. DESIGN EXAMPLE: THE WESTERN SCHELDT TUNNEL

3.1 Construction and Environmental Loading

The cross-section and a longitudinal cut of the bored tunnel are sketched in Figure 3. The bored tunnel, being located in chloride contaminated soil, must be considered to be under chloride attack. The internal walls of the tunnel are subjected to the influence of carbonation

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and to chloride-contaminated salt fog and splash environment (from road traffic). Leaking of joints will lead to a chloride attack within the joints and, especially in points deep in the tunnel, at the internal surfaces of the lower half circle elements. The service life design must be executed considering the environmental loading described above in order to prevent reinforcement corrosion.

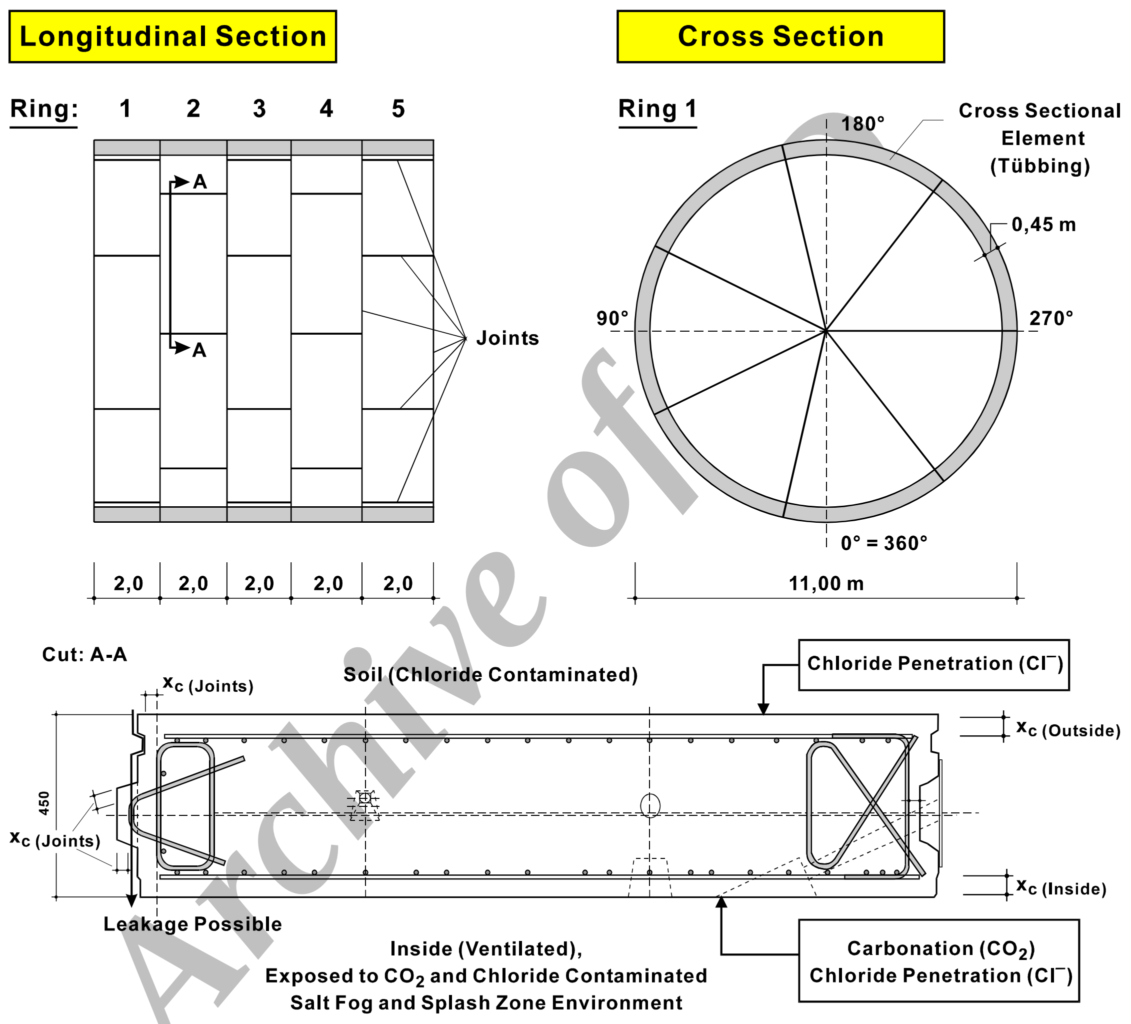

Figure 3. Geometry of the bored tunnel under the Western Scheldt

3.2 Specification of Performances and Reliability Levels

The service life of reinforced concrete structures depend on the length of two time periods:

a) the initiation period

b) the propagation period.

The initiation period is defined as the time until the reinforcement is depassivated either by carbonation or by penetrating chlorides exceeding a critical chloride content. After depassivation, reinforcement corrosion becomes possible. Under certain circumstances (e.g.

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availability of sufficient oxygen and moisture), reinforcement corrosion can take place, which will lead to a reduction of the reinforcement cross-section. Additionally, the formation of corrosion products may lead to a bursting pressure, causing visible cracks at the concrete surface and spalling of the concrete cover. This period is defined as propagation period.

For each event, so-called limit states are applied (onset of corrosion, cracking, spalling, etc.). If a limit state-related failure leads only to economical consequences, serviceability limit states (SLSs) are applied (e.g. onset of corrosion). If failure leads to severe consequences (e.g. total loss of the structure or victims) ultimate limit states (ULSs) are applied (collapse due to overloading or due to excessive material degradation). Other intermediate stages may be defined by the investor, mainly in order to reduce operating costs or repair (e.g. restricted leakage).

In Table 2 some values for the reliability index are given. As presented by Siemes and Rostam [6] these reliability indexes can also be used if lack of durability leads to an event resulting in an unacceptably loss of serviceability or to a loss of structural safety.

Table 2. Set of Operational Limit States

Limit State	**Event**	**Reliability Index β_0**
SLS	Onset of corrosion	1.5 – 1.8 (EC 1, NEN 6700, respectively)
SLS	Corrosion induced spalling and corresponding failure in water tightness	2.0 – 3.0 Proposal
ULS	Collapse of the structure	3.6 – 3.8 (NEN 6700, EC 1, respectively)

This paper will only present a SLS design. The design target is to restrict the probability of the event 'onset of corrosion'. In this paper only internal surfaces of the tunnel segments are considered, giving an answer to the following question:

"What is the required material resistance against chloride penetration in combination with which concrete cover x_c will fulfil the investors' requirement: $t_{\text{Service Life}} = 100$ year with a minimum reliability index of $\beta_{SLS,0} \geq 1.50 - 1.80$?"

3.3 Deterioration Models

The internal surface of the lower half-circle elements at points deep in the tunnel is identified being the most critical detail. Internal surfaces are exposed to frequently changing chloride-contaminated solutions from various sources (leakage of joints, de-icing salts), due to the assumed high humidity. The rate of carbonation is assumed to be negligibly low. Consequently, chloride-induced corrosion is identified to be the relevant deterioration mechanism. To describe the initiation period until onset of corrosion, a deterioration model is required.

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The following deterioration model describes the time-dependent diffusion-controlled penetration of chlorides, allowing the time to the onset of reinforcement corrosion to be calculated. The suggested model for predicting of the initiation period in the case of chloride-induced reinforcement corrosion has been identified as an operational model by Alisa et al. [7] from an intensive literature research carried out within a Brite-Euram-Project DuraCrete. This is defined by

$$x(t) = 2 \cdot C_{(Crit)} \cdot \sqrt{k_t \cdot D_{RCM,0} \cdot k_e \cdot k_c \cdot \left(\frac{t_0}{t}\right)^n \cdot t} \quad (2)$$

where:

$$k_t \cdot D_{RCM,0} = D_0 \quad (3)$$

$$C_{(Crit)} = \mathrm{erf}^{-1}\left(1 - \frac{C_{Crit}}{C_{SN}}\right) \quad (4)$$

where x_c is the concrete cover [mm]; D_0 is the effective chloride diffusion coefficient under defined compaction, curing and environmental conditions, measured at the time t_0 [m^2/s]; $D_{RCM,0}$ is the chloride migration coefficient under defined compaction, curing and environmental conditions, measured at the time t_0 [m^2/s]; C_{Crit} is the chloride threshold level [wt % Cl^-/binder]; n is a factor which takes the influence of age on the measured material property into account [-]; k_t is a constant which transfers the measured chloride migration coefficient $D_{RCM,0}$ into a chloride diffusion coefficient D_0 [-]; k_e is a constant which considers the influence of environment on D_0 [-]; k_c is a constant which considers the influence of curing on D_0 [-]; erf^{-1} is the inverse of the error function; C_{SN} is the surface chloride level in [wt % Cl^-/binder]; t is the exposure period [years]; and t_0 is the reference period [years], in this case t_0 = 28 days

3.4 Statistical Quantities

As mentioned before statistical information is required to perform the probability-based service life design. The corresponding statistical quantities of the variables are given in Table 3. The variables to be considered are obtained from Equation (2) and Equation (4):

In the following it will be shown how each distribution function was obtained for the first three listed stochastic variables (concrete cover, x_c; chloride migration coefficient, $D_{RCM,0}$; critical chloride content C_{Crit}).

Concrete cover, x_c: During production of the circular tunnel segments (precast elements), precise installation of the concrete cover is possible. The reinforcement should be embedded with a nominal value of x_c = 50 mm. The geometrical variable concrete cover is introduced as a Beta distribution (in order to exclude negative covers and covers larger than the half of the elements thickness) with a relatively low standard deviation (precise production conditions).

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Table 3. List of stochastic variables influencing the duration of the initiation period

Variable No	Parameter	Dimension	μ	σ	Distr. Type
1	x_c – Concrete Cover	[mm]	50	5	Beta Distr.[1)]
2	$D_{RCM,0}$ 5 Cl^- Migration Coef.	$[10^{-12} m^2/s]$	4.75	0.71	Normal Distr.
3	C_{Crit} – Critical Chloride Content	[wt %/binder]	0.70	0.10	Normal Distr.
4	n – Age Exponent	[–]	0.60	0.07	Normal Distr.
5	k_t – Factor Test	[–]	0.85	0.20	Normal Distr.
6	k_e – Factor Environment	[–]	1.00	0.10	Normal Distr.
7	k_c – Factor Execution	[–]	1.00	0.10	Normal Distr.
8	C_{SN} – $c(Cl^-)$ – Concrete Surface	[wt %/binder]	4.00	0.50	Normal Distr.
9	t_0 – Reference Time	[years]	0.0767	–	Deterministic

1) $a = 0$ mm $\leq x_c \leq 225$ mm $= b$

Chloride migration coefficient, $D_{RCM,0}$: For design purposes an important starting measure is the material resistance, when focusing on the expected deterioration processes. Suitable results may be drawn from literature to be used as starting parameters in a service life design calculation. When working with special concrete mixes with very low water/binder ratios and high contents of plasticiser, quantitative results from literature are not available. Therefore, it is essential to determine the efficiency of the materials to be used through basic tests, e.g. in order to identify the suitability of the designed concrete mix. In this context the decisive material resistance is the chloride diffusion coefficient.

The determination of chloride diffusion coefficients is a rather complex and time-consuming procedure. In practice, chloride penetration into concrete structures follows a non-steady state process. For this process, conventional methods are to immerse the concrete specimens in chloride contaminated solutions with constant chloride concentration. After immersion the chloride profile is determined by taking samples from the exposed surface and by analysing the total chloride content in each sample. The chloride diffusion coefficient can be found from the Fick's second law by curve fitting. This procedure is very time-consuming. In order to prepare a meso-level durability design, the measured material performance should be known as soon as possible in the design phase. Among different rapid test methods, the rapid chloride migration method (RCM) revealed to be theoretically

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the clearest, experimentally the most simple and related to precision (repeatability) the most promising tool. [8] Diffusivity data measured with conventional immersion methods correlate well to data determined with the rapid chloride migration method. [8,9] The measurement procedure is described by Gehlen and Ludwig [8]. For this project, different concrete mixes were tested under laboratory conditions at the reference time t_0, and the determined properties of the chosen concrete are defined by the average and standard deviation in Table 3 (variable 2).

In addition to the measured performance at the time t_0, substantial differences in the time-dependent development of the resistance of the material have to be considered. [10,11] The increase in the resistance of the material, expressed by the reduction of the chloride diffusion coefficient, is indicated by the age exponent n (variable 4 in Table 3). Typical quantities of n for different concretes are given by Siemes et al. [10] and Bamforth [11].

Critical corrosion inducing chloride content, C_{crit}: From the Design Guide of the Comité Euro-International du Béton (CEB) for durable reinforced concrete structures of 1989 the relationship between the critical chloride content depending on the moisture content of concrete and the quality of the concrete cover is as illustrated in Figure 4, Ref. [12].

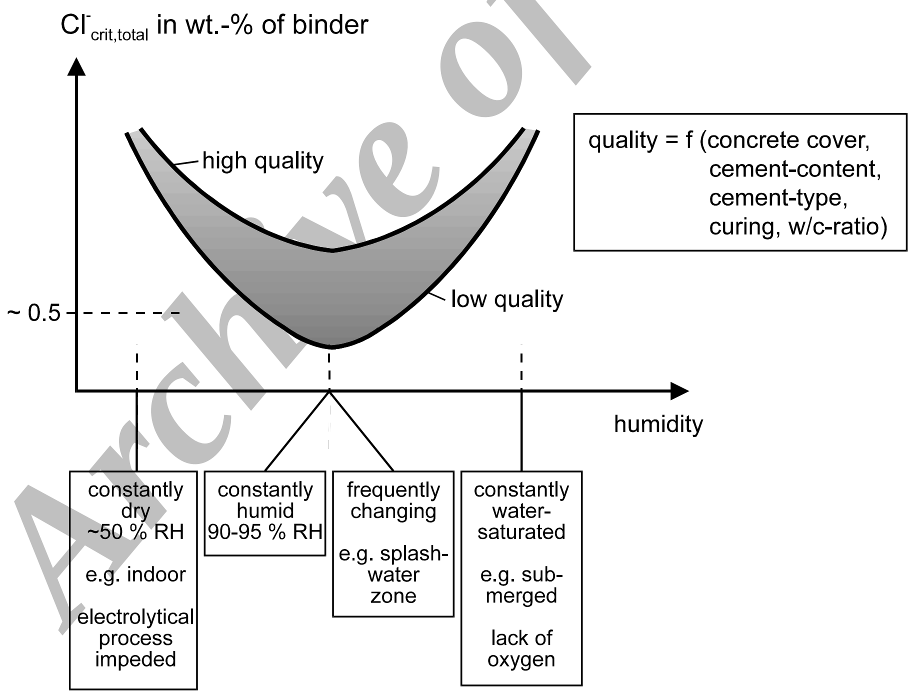

Figure 4. Relationship between the critical chloride content to environmental conditions and the quality of concrete cover [12]

Since for permanently dry or permanent water-saturated concrete practically no reinforcement corrosion is possible, higher values for the critical chloride content can be allowed under these conditions. The lowest value for the critical chloride content appears

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under permanently moist or alternating environmental conditions. In the case of large concrete cover and good concrete quality, which will be basically obtained by a low w/b ratio and a suitable curing of the concrete, higher critical chloride contents can be permitted compared with low or insufficient quality of the concrete cover.

The required statistical quantities for C_{Crit} can be taken from the results of the investigations of Breit[13]. In this investigation, critical threshold values for the chloride content were determined for various mortar mixtures. Considering the results of the various mixes as a whole, the surveys of Breit suggest that the introduction of the critical corrosion-inducing chloride content can be taken as a normal distributed stochastic variable ($\mu = 0.48$, $\sigma = 0.15$).

Making use of only one mixture, which will cause a decrease in variation, and taking into account higher concrete covers, one can estimate higher averages, due to increased concrete cover, and lower standard deviations (Table 3, variable 3).

3.5 Results

With the evaluated data it is possible to calculate whether the stochastic variable x_c, which was applied in the design (Table 3, variable 1), is sufficient to ensure a serviceability limit state-related minimum service life of $t_{Service\ Life} = 100$ years. The results of the evaluation are shown in Figure 5.

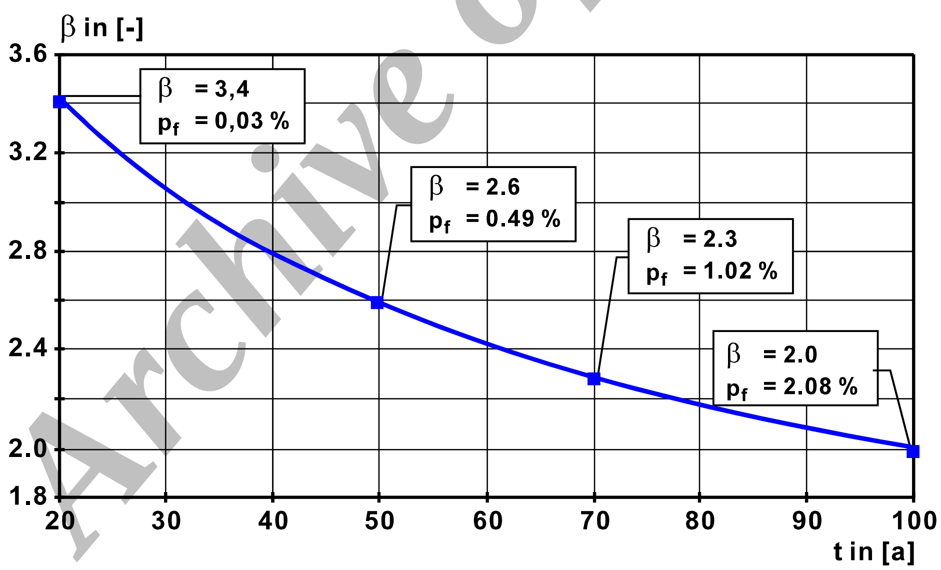

Figure 5. Reliability index versus time of exposure (SLS: depassivation of the reinforcing steel)

With increasing time the failure probability p_f increases, which corresponds with a decrease in β. Nevertheless, Figure 5 shows that the tested material in combination with the planned concrete cover is able to achieve a durable structure concerning chloride induced steel corrosion with the required safety according to the serviceability limit state fixed earlier. The obtained reliability index at time $t_{Service\ Life} = 100$ a is $\beta_{SLS} = 2.0$ and is above the

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required range of $\beta_{SLS,0}$ = 1.5 - 1.8.

3.6 Quality Control

The quality level achieved in the laboratory must be reached on the construction site as well, since the quality of the built-in material determines the service life of the construction. Criteria for the final examination (acceptance criteria) of the tested material resistances can be determined following EC1, appendix D. The continuous examination of the material variable $D_{RCM,0}$ cannot be performed on the construction site as this method is too complicated to perform in the field. Extensive surveys at the Institute for Building Materials at the Technical University of Aachen (ibac) have shown a good correlation between the chloride migration coefficient $D_{RCM,0}$ and the electrolytic resistance of concrete $\rho_{WER,0}$ determined with the so-called WENNER-probe (Figure 6). Use of the WENNER method is described in detail by Gehlen and Ludwig [8]. Considering this, the material resistance to chloride penetration was verified on concrete cubes stored on the construction site (immersed in tap water, T = 20 °C) by indirect examination of electrolytic resistance of the concrete.

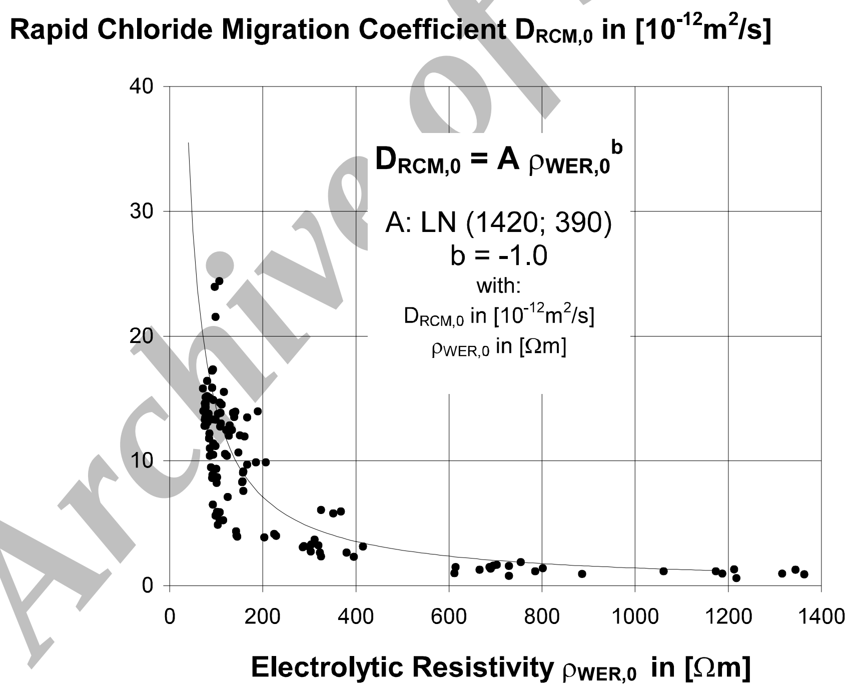

Figure 6. Relationship between $D_{RCM,0}$ and $\rho_{WER,0}$

In addition to the material control, a permanent measurement of the concrete cover is required to confirm that the measured performance with regard to x_c is equal to the required performance of the geometrical variable x_c. In the Figure 7 a distribution plot of measured concrete covers (internal surface) of one single concrete element is given.

The determined statistical quantities are as follows: normal distribution (μ; σ) = ND

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(50.98; 4.23)

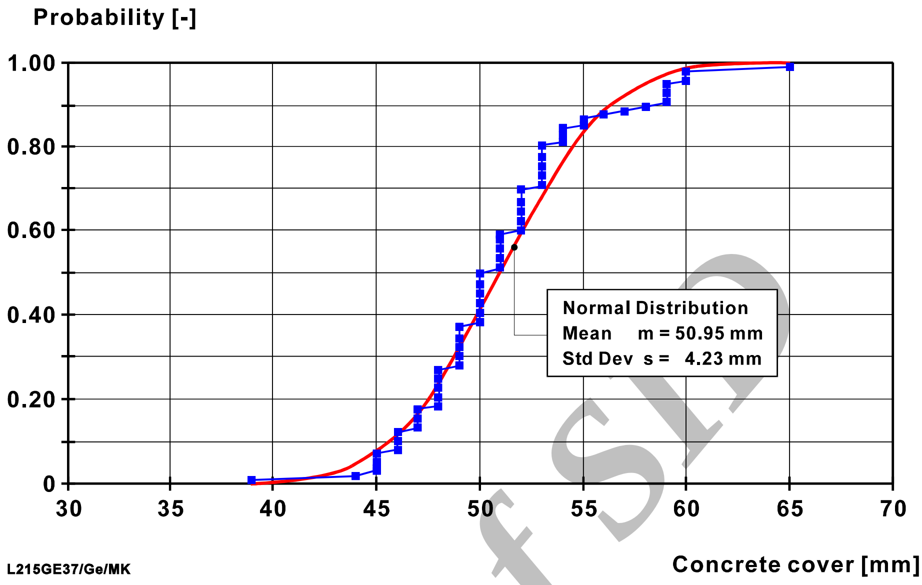

Figure 7. Distribution plot of measured concrete cover

Taking this new information into account, the updated reliability index at time $t_{Service\ Life} = 100$ is $\beta_{SLS} = 2.2$. This value is higher than the calculated one of the original design ($\beta_{SLS} = 2.0$) and is clearly above the required range of $\beta_{0,SLS} = 1.5$ - 1.8.

More detailed information about the applied quality control procedures and the related measurements are published by Breitenbücher [14].

4. REFINEMENT OF SERVICE LIFE PREDICTION

The probabilistic service life design method does not only allow to design a structure in the planning phase for a given target service life design, it can be advantageously applied also for existing structures to estimate the remaining service life or by repeated application in certain time intervals to improve the precision of the service life prediction until a given safety index will be reached. In this way it gives the contractor/operator/owner the possibility not only to target the durability of new structures but also to assess the performance of already built structures with regard to durability.

As demonstrated in chapter 3.6 the precision of the prediction of the real safety index to be expected at the end of the target service life can be improved by a second calculation when the structure has been built by using material resistances (diffusion resistance, concrete cover) achieved in the structures and statistically quantified by appropriate measurements.

A further improvement of the prediction is possible after a certain period in use, e. g.

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5 years. In this case the real interaction between environment and structure can be taken into account by taking chloride profiles or measuring carbonation depths at the relevant concrete surfaces. This procedure is especially advisable if a sufficient durability of the structure is in doubt and protective measures may eventually be necessary. An example of such a procedure is given by Schiessl, Gehlen and Pabsch [15].

5. CONCLUDING REMARKS

In the last decade much effort has been spent on the service life design of structures and especially of concrete structures. This has finally led to a situation where it was possible to make a fully performance- and reliability-based service life design for real structures.

This paper has summarized the design framework and has identified the information which is required to perform a probability based service life design. One design example has been described in detail: the concrete mix in conjunction with the concrete cover of the tube of the Western Scheldt Tunnel was designed for the expected deterioration mechanism (environmental loading). Further measures to complete the durability design (quality control) were briefly presented.

The Western Scheldt Tunnel project and other projects have shown that the outlined performance-based design concept is working not only within the design phase but also during construction and use. Within the design phase the ideal material resistance in conjunction with the corresponding ideal concrete cover can be identified to guarantee the required durability of the structure. During construction these two variables (x_c and $D_{RCM,0}$ measured indirectly by $\rho_{WER,0}$) at least can be checked continuously. The measured values can be taken to update the original service life design. If the original designed performances are not reached on the construction site (this can also be an output from the quality control procedure), additional measures can be taken and considered statistically to improve the structure (e.g. coating), or checked again after a certain period of use when the interaction between environment and structure can be taken into account in the service life design.

REFERENCES

1. Bamforth, P., Double Standards in Design, Concrete (submitted for publication).
2. International Workshop of Service Life Design of Concrete Structures, ACI-CEB-RILEM, Copenhagen, 1994.
3. Basler, E., Untersuchungen über den Sicherheitsbegriff von Bauwerken. Schweizer Archiv für angewandte Wissenschaft und Technik, Heft 4, 1961.
4. Cornell, A. C., A Probability Based Structural Code. *ACI Journal,* No. 12, Proceedings **66**(1969).
5. RCP GmbH: Strurel, A Structural Reliability Analysis Program System, RCP Consulting GmbH München, 1995.
6. Siemes, A.J.M. and Rostam, S., Durable Safety and Serviceability - A Performance Based Design Format, IABSE Report 74: Proceedings IABSE Colloquium "Basis of

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Design and Actions on Structures - Background and Application of Eurocode 1", Delft, pp. 41-50.
7. Alisa, M., Andrade, C., Gehlen, C., Rodriguez, J. and Vogels, R., Modelling of Degradation. Brussels: Brite-Euram, 1998. Project No. BE95-1347, R 4/5, pp. 174.
8. Gehlen, C. and Ludwig, H.M., Compliance Testing for Probabilistic Design Purposes: Brussels: Brite-Euram, 1999. Project No. BE95-1347, R 8, pp. 114.
9. Tang, L., Chloride Transport in Concrete - Measurement and Prediction, Gothenburg, Chalmers University of Technology, 1996.
10. Siemes, A.J.M., Gehlen, C., Lindvall, A., Arteaga, A. and Ludwig, H.M.: Statistical Quantification of the Variables in the Limit State Functions. Summary: Brussels: Brite-Euram, 1999. Project No. BE95-1347, Draft R 9, pp. 106.
11. Bamforth, P., Spreadsheet Model for Reinforcement Corrosion in Structures Exposed to Chlorides. London: E & FN Spon, 1998. - In: Proceedings of the Second International Conference on Concrete Under Severe Conditions, Tromso, Norway, 1998, Vol. I, (Gjorv, O.E.; Sakai, K.; Banthia, N. (Ed.)), pp. 64-75.
12. Ceb; Comité Euro-International du Béton: Durable Concrete Structures CEB Design Guide Second Edition 1989. Lausanne: Comité Euro-International du Béton -CEB- In: Bulletin d'Information, No. 182, 1989.
13. Breit, W., Untersuchungen zum kritischen korrosionsauslösenden Chloridgehalt für Stahl in Beton. In: Schriftenreihe Aachener Beiträge zur Bauforschung, Institut für Bauforschung der Rwth Aachen, Nr. 8, Aachen, 1997.
14. Breitenbücher; R., Probabilistische Lebensdauerbemessung – Umsetzung am Beispiel des Westerscheldetunnels, Festschrift zum 60. Geburtstag von Prof. Peter Schießl, Schriftenreihe Baustoffe der TUM, cbm, Heft 2/2003.
15. Schiessl, P., Gehlen, C. and Pabsch, G., Prediction of service Life of existing Structures by Applying Probabilistic Service Life Design (SLD) Procedures, International Conference Advances in Concrete and Structures (ICACS 2003), Xuxhou, Jiangsu, China, 2004.

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