ISSN 1648-4142 TRANSPORT
http:/www.vtu.lt/english/editions

TRANSPORT – 2002, Vol XVII, No 5, 163–170

DAMAGE TO CONCRETE BRIDGES DUE TO REINFORCEMENT CORROSION: PART II-DESIGN CONSIDERATIONS

Zenonas Kamaitis

Dept of Bridges and Special Structures, Vilnius Gediminas Technical University, Saulėtekio al. 11, LT-2040 Vilnius, Lithuania. E-mail: tsc@ts.vtu.lt

Received 2002 05 07; accepted 2002 09 10

Abstract. The mechanisms of reinforcement corrosion in concrete are the subject of extensive research. Although reliable methods for predicting the corrosive deterioration of concrete structures do not yet exist. This paper describes the durability problem of reinforced concrete bridges based on the mechanisms of carbonation depth or chloride profile. The deterioration model considering concrete carbonation, chloride penetration and concrete cover cracking is adopted to describe the service life of concrete structures. The corrosion models include environmental conditions, concrete carbonation or chloride diffusion rates, quality of concrete cover, steel corrosion rates and many other factors that make the predicting of service life of structures extremely difficult. Finally, the author gives the details of the methods of durability verification and the proposals for its including in the national standards and practical guides.

Keywords: concrete bridges; reinforcement; corrosion; concrete cover cracking; durability; codes.

1. Introduction

Deterioration of reinforced concrete due to corrosion of steel is a worldwide problem. Corrosion of prestressed concrete has particularly serious consequences. The simultaneous action of high tensile stresses and aggressive environment can give rise to stress cracking corrosion and sudden failure of the structure without warning. Much work is being done on the mechanism of corrosion although complete knowledge of the process is still lacking. The main causes are related to insufficient knowledge of the durability problem. The mechanisms of steel corrosion in concrete are the subject of extensive research [1–5]. Reliable methods for predicting the corrosive deterioration do not yet exist. In such case there is a need to provide a good design of durability.

The corrosion of reinforcement leads to:

- the destruction of steel-concrete bonding mechanism;
- the reduction in the reinforcement cross-section;
- the increase of displacements (deformations, widening of cracks);
- the loss of ductility of members and as a result the carrying capacity or serviceability of the structures.

Reinforcement corrosion in concrete usually results in concrete cover cracking or spalling. In some circumstances pitting corrosion of steel may also occur without visible effect on the concrete surface. Our investigations show that the main reasons are: insufficient concrete cover, poor quality of concrete and ingress of aggressive salts [6, 7].

This paper describes the durability problem of reinforced concrete bridges based on the mechanisms of carbonation depth or chloride profile. The deterioration model considering concrete carbonation, chloride penetration and concrete cover cracking is adopted to describe the service life of concrete structures.

2. Design Model for Corrosion Process

The time-dependent corrosion of reinforced concrete structure can be considered schematically as a three stages process (Fig 1).

During the initiation period (t_l) steel remains passive whilst within the concrete cover the changes are taking place. Depassivation occurs when aggressive ions reach sufficient concentration to destroy the passive film locally (pitting) or generally.

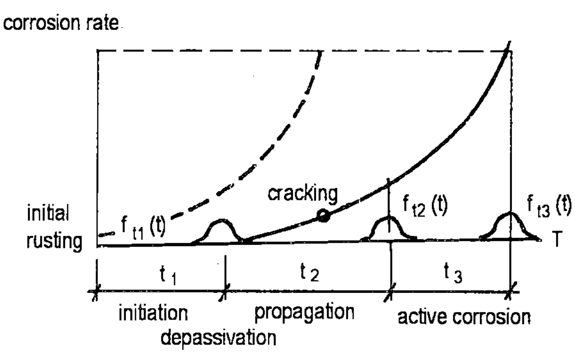

Fig 1. Simplified model of reinforcement corrosion in concrete

In the propagation period (t_2) the corrosion products of higher (about 2 to 3 times) volume than that of the steel are formed. The formation of cracks in concrete cover is observed when the stresses due to the increase in volume of corrosion products surpass the concrete tensile strength. The cracks and spalls allow the intensification of the environment attack and increase the deterioration rate of a structure.

The final state (t_3) of active corrosion is reached when the reinforcement bar exposed to the easy access of water, oxygen or chlorides loses its cross-section causing unacceptable level of safety, serviceability or appearance of a member.

The rate of carbonation, chloride penetration and corrosion of reinforcement bars is of vital importance for predicting the service life of structures.

The service life or the time before repair of bridge component:

$$T = t_1 + t_2 + t_3, \tag{1}$$

where t_1 is time for concrete carbonation or chloride penetration to reach the reinforcement and the threshold concentration as a function of concrete cover thickness and quality, as well as environmental aggressiveness; t_2 is time for corroding reinforcement causing cracking or spalling of concrete as a function of steel nature, bar size and spicing; t_3 is time for open reinforcement to cause corrosion as a function of the environmental conditions.

Time periods t_2 and t_3 will depend upon the rate of steel corrosion under concrete cover and in atmosphere correspondingly. The problem is to be able to assess t_1, t_2 and t_3 for given situations. The limit to allowable corrosion will always remain one of option. The service life of a corroding member can be identified with depassivation of steel (t_1), cracking of the cover ($t_1 + t_2$) or loss of reinforcement area ($t_1 + t_2 + t_3$) and as a result reduction of the safety factor. In most cases cracking and spalling occur in reinforced concrete structures well before the reinforcement has become significantly weakened. As it was stated in [7] the carbonation and the rebar cover depth data allow to predict the probability of the range of time t_1 when corrosion of reinforcement will be initiated. Predicting the further corrosion is more difficult.

In prestressed concrete structures the action of high tensile stresses and corrosive agents can lead to tension cracks in tendons of small diameter wires without cracking of the surrounding concrete. It seems that corrosion of reinforcement depends on the type of steel and its metallurgical structure which is influenced by composition, heat treatment, mechanical processing.

3. Depassivation of Steel in Concrete

The alkaline environment of concrete produces stable passive conditions in which the reinforcement is protected from corrosion. Ingress of carbon dioxide and chlorides

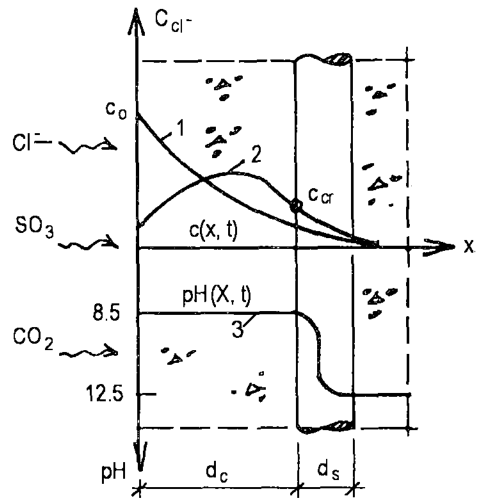

Fig 2. Scheme of chloride penetration (1, 2) and concrete carbonation depth (3) of reinforced concrete

penetrates from the outside of the cover to the steel (Fig 2).

Based on Fick's law the speed of penetration is:

$$x = A\sqrt{t}. \tag{2}$$

The rate of carbonation in atmospherically exposed bridge structures depends on the surrounding concentration of CO_2, c_0, the composition and compaction (permeability) of concrete, D, the reacting quantity with CO_2 in concrete, m_0 [8].

$$A = \sqrt{\frac{2D_{CO}c_0}{m_0}}. \tag{3}$$

The problem is much more complex especially for old structures. Many additional factors as variation of season temperatures ($t°$), wetting and drying periods (W), cracking of concrete (w_{cr}) during the service life affect carbonation rate and considerable local variation from the values calculated from (3) may occur in practice (Fig 3). Adjustment factor $k = f(W, t°, w_{cr})$ should be applied for these effects. The influence of these factors cannot be predicted in the design stage.

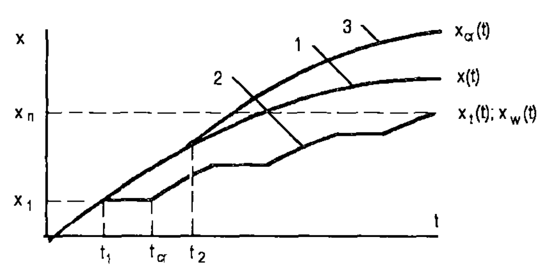

Fig 3. Scheme of concrete carbonation rate in constant climatic conditions (1), regular changing wet and dry or positive and negative temperatures (2), and cracked concrete (3)

The quality of concrete cover (composition and compaction of concrete, early cracking, cover thickness) is an issue of considerable importance. Reinforcement corrosion in concrete bridges occurs normally in an environment with changing relative humidity and temperature. It seems that diffusivity D_{CO} is different in carbonated or noncarbonated, wet or dry, cracked or uncracked concrete. In dry or water saturated concrete as well as in atmosphere with negative temperature carbonation does not occur (Fig 3, curb 2). The time with negative temperatures in formulae (1) is neglected, i.e. $t_1 = \Sigma t_i$. CO_2 penetrates at a greater rate into cracked concrete (Fig 3, curb 3).

Schiessl P. [1] introduced concrete carbonation analysis considering the effect of wet and dry periods and the influence of concrete cracks.

In the case of wet and dry periods

$$x = A\sqrt{t_1} + \sqrt{At_2 - \left(\frac{x_1}{B}\right)^2} ... + A\sqrt{t_n - \left(\frac{x_n - 1}{B}\right)^2}, \quad (4)$$

where $\sqrt{\frac{2D_w(w_0 - w)}{m_w}}$, (5)

D_w is diffusion coefficient for water vapour; $w_0 - w$ is the moisture difference between air and the evaporation front; m_w is the amount of water to evaporate from the concrete.

In the presence of cracks

$$\sqrt{D_{cr} w_{cr} \sqrt{\frac{4c_0}{D_{CO} m_0}}}, \quad (6)$$

where D_{cr} is diffusion coefficient for CO_2 within the crack; w_{cr} is width of crack.

The corrosion process in existing structures varies very widely, being effected by all the factors mentioned above. The parameters D_{CO}, D_{Cl}, m_0 should be determined indirectly by means of special tests. Therefore it is very doubtful whether it will be possible to determine an exact formula for the prediction of carbonation depth in existing structures. Different empirical formulas are proposed. The rate of the carbonation can be extrapolated in existing structures, if the age and the current depth of carbonation are determined [7]. Then all parameters are included in the constant A.

The determination of chloride penetration in the concrete of existing bridge structures is more difficult than for the prediction of carbonation depth. Chloride penetration into concrete can be described as a diffusion process according to Eq. (2), where

$$A = \sqrt{\pi D_{Cl}}\left(1 - \frac{c_x}{c_0}\right) \quad (7)$$

D_{Cl} is effective diffusion coefficient of chlorides into concrete.

When reinforced concrete is exposed to chlorides, the rate of deterioration is faster. If water with high concentration of chloride is in contact with concrete saturated with chloride-free water, the diffusion of chloride ions into the concrete will take place. If concrete saturated with chloride water is in contact with fresh water, then chloride ions will diffuse out of the concrete. Two typical curves in winter and summer seasons have to be distinguished (Fig 2). The maximum surface concentration is in winter with drop due to washing away effect in summer. It is commonly accepted that steels corrode when [Cl-]>0,6 [OH-] (Hausmann's rule), where [OH-] is the concentration of hydroxyl ions in the Portland cement.

From Eq. (2), (3) and (7) the time of concrete cover carbonation

$$t_1 = \frac{d_c^2 m_0}{2D_{CO} c_0} \quad (8)$$

and that of chloride penetration

$$t_1 = \frac{d_c^2}{\pi D_{Cl}\left(1 - \frac{c_{cr}}{c_0}\right)^2}, \quad (9)$$

where d_c is concrete cover thickness; c_{cr} is critical chloride content on the surface of reinforcement.

There is uncertainty about how much chloride is required to initiate corrosion of steel reinforcement. In various studies as well as in Codes of practice (Table) considerably varying critical chloride content has been obtained and recommended.

Spray of chloride solutions from vehicles has to be considered for parapet beams, footways, and guardrails.

Recommended limits of the amount of chloride (c_{cr}) in concrete expressed by weight of cement

Code	Concrete	Reinforced concrete	Pre-tensioned concrete	Post-tensioned concrete
AFNOR 18-011	1,0	0,65	0,1	0,2
ACI 318	1,0	0,15-0,10	0,06	0,06
BS 5400	1,0	0,35	0,06	0,06
ENV 206	1,0	0,40	0,2	0,2

For bridge structures the process is much more difficult to predict. The deterioration of bridge structures caused by chlorides is difficult to predict due to variation in moisture and chloride content (c_0) from place to place and at different depth with time which suits the periodic (annually) application of de-icing salts. Chlorides in bridges cause the localised corrosion and pitting of reinforcement.

4. Concrete Cover Cracking

Once the passivity of reinforcement has been destroyed, corrosion of steel can occur, if sufficient quantities of oxygen and moisture can reach the bar. The oxidation of steel is an electrochemical reaction that occurs at anodic region as a result of differences in the chemical composition and the state of the steel. The overall reaction is $2Fe+2H_2O+O_2 \rightarrow 2Fe(OH)_2$. When a passive bar starts to corrode its diameter decreases together with the generation of oxide of higher volume than that of the steel.

The pressure p due to rust expansion at the surface of the bar has to be equilibrated by pouching shear strength of concrete cover (Fig 4):

$$pd_s = (d_s + d_c) f_{ct}, \tag{10}$$

where f_{ct} is the tensile strength of concrete.

The depth of bar corrosion during cracking of concrete is believed to be of only 10–20 μm [2]. Loss of cross-section of a bar is too small to significantly affect its strength. According to [5], the contact pressure can be computed from:

$$p = \frac{\Delta d_s E_c}{1 + \frac{\nu_c}{2} + 1.2\nu_s \frac{E_c}{E_s}}, \tag{11}$$

where E_c, ν_c and E_s, ν_s are the modules of elasticity and the Poisson's ratio of concrete and steel, respectively.

The time period between depassivation and cover cracking:

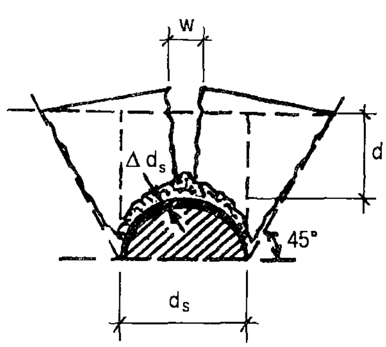

Fig 4. Cross-section of a corroding rebar with cracked concrete cover

$$t_2' = \frac{\Delta d_s}{\lambda}, \tag{12}$$

where λ is corrosion rate of steel in concrete.

Substituting equations (10) and (11) into equation (12), one obtains

$$t_2' = \frac{(d_s + d_c) f_{ct}}{\lambda d_s k}, \tag{13}$$

where $k \approx 0.5E_c$.

Calculations show that the time t_2' may be very short. The SLS might be identified with the longer time when cracks of 0.2-0.3 mm width or local spalls are reached. The time period between cover cracking ($w_{cr} \approx 0.05$ mm) and appearance of a crack of width w_{cr}:

$$t_2'' = \frac{w_{cr}\left(\frac{d_s}{d_c} + 1\right)}{2\lambda_1}, \tag{14}$$

where λ_1 is corrosion rate of reinforcement in concrete cracks ($\lambda_1 \approx \lambda$).

According to [9] the crack widths in the range of 0.3-0.5 mm occur at approximately $t_2'' \approx 30t_2'$.

From (13) and (14) the total time for reinforcement corroding causing cracking or spalling of concrete is:

$$t_2 = \frac{(d_s + d_c)(2f_{ct}d_c + kw_{cr}d_s)}{2k\lambda d_s d_c}. \tag{15}$$

Once the concrete cover has been spalled off, the intensive atmosphere corrosion of exposed reinforcement is initiated. Large cracks or spalls allow oxygen, moisture and aggressive ions to travel directly to the reinforcement and increase the rate of corrosion. The loss of bar diameter which effects the strength of a structure during its time $t_2 + t_3$:

$$\Delta d_s = \lambda_1 t_2 + \lambda_2 t_3, \tag{16}$$

where λ_2 is corrosion rate of reinforcement exposed in atmospheric conditions ($\lambda_2 > \lambda_1$). According to various studies the rate of steel corrosion is of 10–20 mg/mm^2/year. The corrosion rate λ_2 should be determined from the experimental results.

5. Limit State Definition

Verification of durability or service life of bridge structures exposed to aggressive actions should be part of the initial design or assessment of damaged concrete structures in service. Design of the durability of bridge structures is not simple. Corrosion is a stochastic process. The durability of structures is influenced by:

- the introduction of new conceptions, materials and construction techniques, requiring repeated trials and errors as well as the tendency to provide the structures at the lowest possible cost;

- very wide variation of traffic loads as well as exposure conditions during service life which depend on the situation of a bridge (over river, highway or railway, in the industrial or maritime zone) and the location of an individual member within a structure;
- significant number and rate of deterioration mechanisms and interactions which result from the combinations of the environment, concrete quality, size and configuration of structure; any global prediction of future deterioration rate is likely to be very approximate;
- manifestation of the durability problems after long time when the degradation of the existing structures is well advanced; very often the causes of degradation are obscure, the construction records or condition survey data are missing;
- different levels of maintenance (bridge maintenance is directly related to the condition state of the structures);
- gross errors during design, construction (execution) or operation.

Three paths or methods of durability verification can be considered (Fig 5).

First two methods (A1 and A2) examine mechanical limit states and durability separately. The first step is traditional calculation process associated with limit states. The second one is the design for durability.

Path A1 is based on practical detailing of durability (material specifications, structural properties), which is presented in current codes [10]. Although it is not clear how many times the measures envisaged will satisfy the durability of a structure.

Path A2 uses the time dependent corrosion models linking the environmental impacts with the properties of materials (see section 2 and 3). The durability parameters of materials obtained from laboratory tests or existing structures are needed.

A new framework for the integrated structural and durability design (path B) considering the analytical models of degradation and the inspection/maintenance plans should be established. The structures have to satisfy the performance with respect to both strength/serviceability and durability.

Very few researches combine the consequences of structure corrosion with SLS or ULS, although a number of studies has been conducted on durability as it is mentioned before. For the structures for which corrosion of the steel reinforcement is not allowed, for SLS the depassivation of steel in concrete or the cover cracking/delamination can be accepted as the end of service life. For ULS loss in reinforcement cross-section or bond strength in the partial factor format is generally accepted.

The limit states of deteriorating structures are based on the effect of actions, $S_D(t)$, compared with materials or structural time-variant resistance, $R_D(t)$. The verification can be performed in resistance or lifetime format:

$$m = R_D(t)\Theta_D - S_D(t) = R_d \varphi_R(t)\,\Theta_R - S_d \varphi_S(t)\,\Theta_S \; ; \quad (17)$$

$$m = t\,\Theta_t - t_d , \quad (18)$$

where m is the margin of safety with $m > 0$ denotes safe and $m \le 0$ denotes failure; R_d and S_d are the design values of resistance and actions effect, respectively; $\varphi_R(t)$ is degradation function; $\varphi_s(t)$ is function of action effect; Θ is uncertainty of the calculation models (resistance, durability) which takes into account simplification of the statical system, deterioration of members; t is the time of assessment; t_d is the design or target service life.

The variables S and R can be any quantities and expressed in any units. In the design of durability according to path A2, S_d can be penetration depth (X), concentration of aggressive agents (c_x), extend of concrete cracking/delamination or state of reinforcement at the moment t R_d is the resistance of a structure to corrosive actions or limiting performance criteria which can be interpreted as the actual depth of concrete cover (d_c), critical concentration of aggressive agents (c_{cr}), admissible level of concrete cracking/delamination or thickness of rebar corrosion. For example, the limit state of damage of reinforced concrete structure due to steel corrosion may be represented as follows:

$$m = 0.95A_s - 0.25\pi d_s \sum_{1}^{n} \left[d_s - 2\lambda \left(t - \frac{d_c^2}{A^2} \right) \right]. \quad (19)$$

The corrosion time of the structure is governed by the speed of steel corrosion λ and the initiation of corrosion following carbonation of concrete or chloride penetration to reinforcement (parameter A, see Eq. (2)). It is obvious that suitable examinations are necessary for each deterioration factor to fix the mean values and their dispersions.

In Eq. (17) and (18) all the parameters may be described either by deterministic values or by time dependent random variables. Environmental actions are time dependent random variables with high statistical distribution. The durability resistance R_D of concrete structures depends on several factors:

- composition and properties of the concrete;
- cover of reinforcement;
- concrete crack pattern and crack width;
- type and diameter of reinforcement (steel, prestressing steel, non-metallic);
- size, configuration and detailing of cross section;
- exposure conditions (sheltered or open to direct action of environment);
- surface protective measures (coatings, electrochemical protection).

Durability performance of a structure during its de-

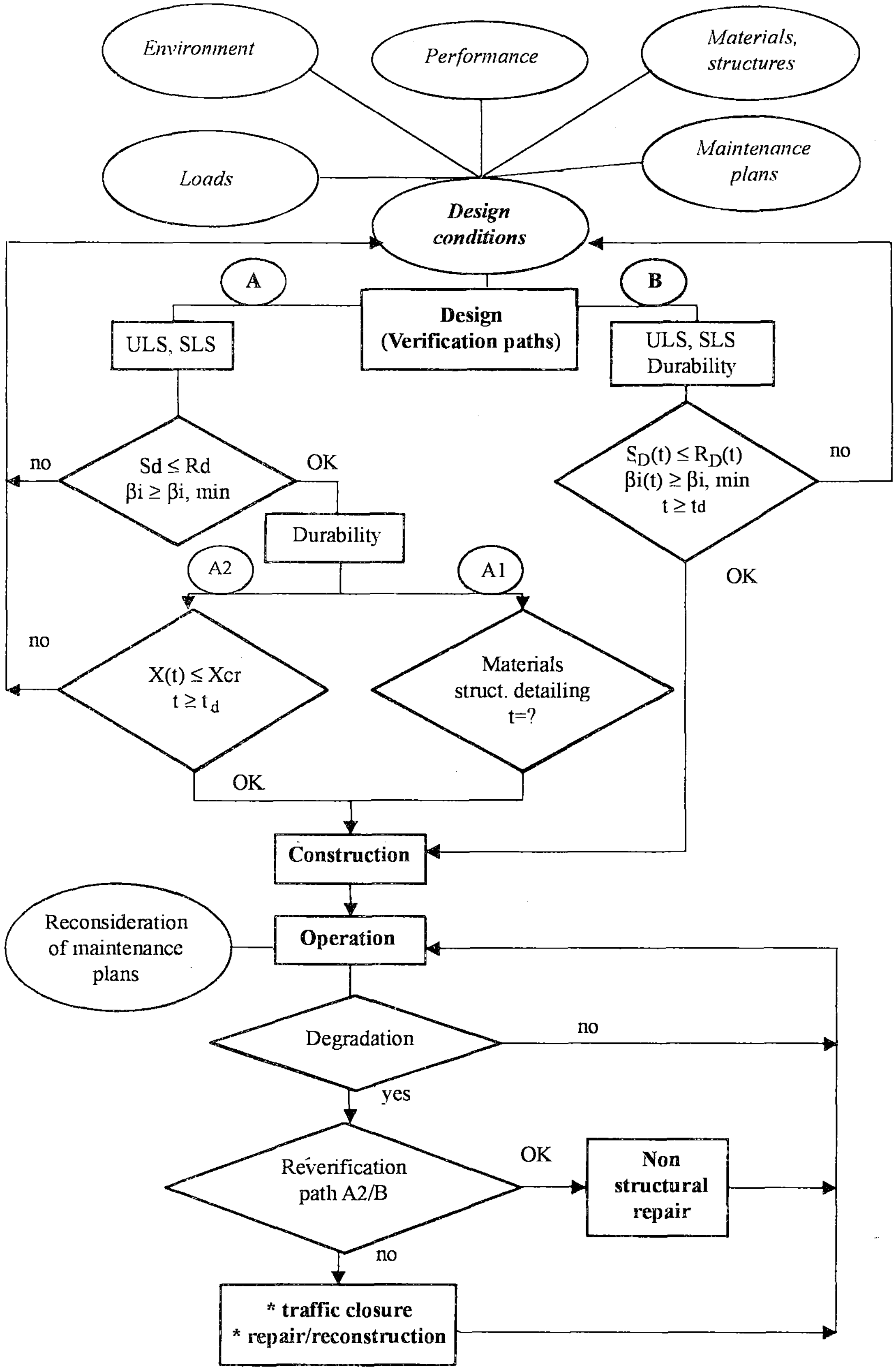

Fig 5. Flow of structural design and verification of concrete structures taking into consideration the durability requirements

sign lifetime is also modified due to maintenance level (repair, strengthening, replacement).

Since there are uncertainties in the parameter values, such as chemical and physical environmental actions and quality of construction, it is reasonable to establish the magnitude of the load and resistance factors using probabilistic calculations.

It is the author's opinion that the implementation of degradation models into a design process of new structures (path B) is not practical at present due to considerable uncertainty and variability of deterioration factors involved. Reliable and accurate *a priori* information on the environmental loads and construction quality for routine structures always is ignored. A conservative estimate of the design of new structures should be selected.

Durability limit states format can be successfully used for the evaluation of existing bridges. Based on condition surveys more realistic estimates of loading, material properties, deterioration rates of structure can be made. Predicting of long-term performance can be more easily determined than that in the original design. Time-dependent reliability assessment of degradating concrete bridges has received extensive studies lately.

6. Codes of Practice

European standards system which will replace the corresponding national standards, provides the design of structural elements for structural safety and durability. According to ENV 1991-1 concrete structures shall be designed in such a way that they maintain their required durability and performance throughout the expected lifetime having an adequate maintenance strategy. The European concrete standard EN 206 prescribes for bridge structures, which are exposed to cyclic wet and dry environment, exposure classes XC4 (for steel corrosion induced by carbonation) and XD3 (steel corrosion induced by spray containing chlorides). According to exposure classes limiting values for concrete composition, thickness of concrete cover and crack widths are prescribed. Unfortunately, these requirements are relatively short-term solutions and are not based on the true performance of concrete structures. Transport properties of diffusivity and permeability of concrete should be included in the codes.

The requirements of Codes and Standards used in the design of new structures do not cover all real situations leading to premature degradation of concrete structures. Standards or guidelines for the assessment of existing structures or for the repair of damaged structures scarcely exist. The Canadian Highway Bridge Design Code [11] covers the design of new bridges and the evaluation and rehabilitation of existing bridges in one document in the same limit state format. Bridges are evaluated when either the loads are increasing or the bridge is deteriorating. Strength losses due to corrosion or fatigue are not considered. Some countries use the guidelines or recommended practice for corrosion control or repair of concrete structures. European standards on the repair of concrete structures are in preparation (CEN TC 104 SC 8) [12].

In most countries as well as in Lithuania no official rules exist for corrosion control of reinforced concrete structures at the design stage, during construction and for a structure which is exhibiting signs of deterioration. The inclusion of the durability and the assessment of existing structures into structural life cycle design and into standards and practical guides is of vital importance. European standards are accepted and adopted in Lithuania. Although the modification or updating them for local needs and conditions is necessary. Local environmental conditions, types of materials and structures, quality of construction and maintenance are the main factors influencing the durability of structures. The author's proposals for the development of national code system for design and maintenance of road structures are presented in [13].

This also justifies the writing of a special guide: "Guide for the assessment and repair of existing concrete bridges damaged by reinforcement corrosion". The guide should include at least the following aspects:

1. Assessment of reinforcement corrosion damaged structures (mechanism of reinforcement corrosion in concrete, diagnosis, assessment of corrosion damaged structures).
2. Repair of concrete structures (repair principles, restoring protection, surface protective coatings, electrochemical protection).

7. Conclusions

1. Corrosion of steel reinforcement initiated mainly by concrete carbonation and chloride contamination is the most common type of deterioration of concrete bridges. The corrosion of reinforcement results in concrete cracking or spalling, loss of section of reinforcement bars, reduction of bond between the reinforcement and the concrete leading to a progressive loss of resistance. Reliability methods for predicting the corrosion deterioration of bridge structures do not yet exist.

2. Time-dependent empirical deterioration models can be used to predict the time of initiation of corrosion of steel in concrete (see Eq. (8), (9)), cracking or delamination of the cover concrete (see Eq. (15)) or loss of reinforcement section (see Eq. (16)). The corrosion initiation and propagation models include environmental conditions, concrete carbonation or chloride diffusion rates, concrete cover properties, steel corrosion rates. The durability parameters of materials obtained from laboratory tests or existing structures are needed. Flow chart for structural and durability limit states verification is presented (see Fig 5).

3. The performance based design of concrete bridges

taking into account both the structural and durability requirements should be introduced with consideration of types of members, inspection/maintenance levels, reference time periods (see Eq. (17) and (18)). The existing codes do not cover all design situations. The inclusion of the durability and the assessment of existing concrete bridges into structural life cycle design and into standards and practical guides is of vital importance. To achieve this the additional prenormative research is required.

References

1. Schiessl P. Corrosion of Steel in Concrete. Report for the Technical Committe 60-CSC RILEM. London/New York, 1988. 102 p.
2. Andrade C., Alonso C., Molina F. J. Cover Cracking as a Function of Bar Corrosion: Part 1 - Experimental Test. *Materials and Structures*, Vol 26, 1993, p 453-464.
3. Andrade C., Alonso C., Molina F. J. Cover Cracking as a Function of Bar Corrosion: Part 2 – Numerical model. *Materials and Structures*, Vol 26, 1993, p 532-548.
4. Gonzalez J. A., Feliu S., Rodriguez P., Ramirez E., Alonso C., Andrade C. Some Questions on the Corrosion of Steel in Concrete – Part I: when, how and how much Steel Corrodes. *Materials and Structures*, Vol 29, 1996, p 40-46.
5. Bacher P. A. M. Predictive Models for Deterioration of Concrete Structures. *Construction and Building Materials*, No 1, 1996, p 27-37.
6. Kamaitis Z. Condition State and Assessment of Concrete Bridges (Gelžbetoninių tiltų būklė ir jos vertinimas). Vilnius: Technika, 1995. 182 p (in Lithuanian).
7. Kamaitis Z. Damage to Concrete Bridges due to Reinforcement Corrosion: Part I - Site Investigations. *Transport*, Vol XVII, Nr. 4. Vilnius: Technika, 2002, p 137-142.
8. Aleksejev S.N., Rozental N.K. The corrosion resistance of reinforced concrete structures in the agressive industry environment (Коррозионная стойкость железобетонных конструкций в агрессивной промышленной среде). Moscow: Strojizdat, 1976. 205 p (in Russian).
9. Steward M. G. Spalling Risks, Durability and Life-Cycle Costs for RC Structures. Safety, Risk and Reliability – Trends in Engineering. In: Proceedings of IABSE International Conference, Malta, March 21-23, 2001, p 537-542.
10. Eurocode No 2 (EC2). Design of Concrete Structures. Brussels, Belgium, 1992. 458 p.
11. Dorton R., Reel R., Gagnon D. The new Canadian Bridge Code – Design, Evaluation, Rehabilitation. Structures for the Future - the Search for Quality. In: Proceedings of IABSE Symposium - Rio de Janeiro, August 25-27, 1999, p 681-688.
12. Draft Recommendation for Repair Strategies for Concrete Structures Damaged by Reinforcement Corrosion. *Materials and Structures*, 27, 1994, p 415-436.
13. Kamaitis Z. Standard System of Design and Maintenance of Road Structures. Modern Building Materials, Structures and Techniques. In: Abstracts of the 7th International Conference, held on May 16-18. Vilnius: Technika, 2001, p 228-229.