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CHAPTER 134

THE FUTURE OF LIFE-CYCLE COST BRIDGE MANGEMENT¹

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1. INTRODUCTION

Obtaining and maintaining advanced infrastructure systems plays an important role in modern societies. Developed countries have in general well established infrastructure systems but most non-developed countries are characterized by having bad or no effective infrastructure systems. Therefore, in the transition from a non-developed country to a well developed country construction of effective infrastructure systems plays an important role. However, it is a fact that construction of new infrastructure systems as well as maintaining existing systems requires great investments so a careful planning of all details in the system is essential for the effectiveness of the system from an operational but also economical point of view.

Most of the infrastructure systems (highways, bridges, harbours, railways etc.) built in Europe in the past seventy years was designed on the basis of a general belief among engineers that the durability of the materials used could be taken for granted. Although a vast majority of infrastructure systems have performed satisfactorily during their service life, numerous instances of distress and deterioration have been observed in recent years. The causes of deterioration of e.g. reinforced concrete bridges, piers etc. are often related to durability problems of the composite material. One of the most important deterioration processes which may occur in reinforced concrete bridges is reinforcement corrosion, caused by chlorides present in de-icing salts and/or carbonation of the concrete cover zone.

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2. BRIDGE MANAGEMENT SYSTEMS

In this paper bridge management systems are discussed with special emphasis on management systems for reinforced concrete bridges. Management systems for prestressed concrete bridges, steel bridges, or composite bridges can be developed in a similar way.

Present bridge management systems are in most cases based on a deterministic approach and the assessment of the reliability or the safety is therefore in general based on subjective statements. In future bridge management systems we will see a change to stochastically based systems with rational assessment procedures. Future management systems will be computerized and different types of knowledge based systems will be used. Further, recent developments in non-linear optimization techniques will make it possible to produce a much better decision tool regarding inspection and repair.

It is beyond the scope of this paper to give a complete presentation of existing bridge management systems. Most existing management systems are presented in detail in the literature. In this paper a number of changes which are expected in future management systems will be discussed. A survey of existing systems is given by Casas [1], Chase [2], Das [3], and Roberts [4].

For many years it has been accepted that steel bridges must be maintained due to the risk of corrosion of steel girders etc. The situation is a little different for reinforced concrete bridges. Reinforced concrete bridges built in Europe in the past seventy years were designed on the basis of a general belief among engineers that the durability of the composite material could be taken for granted. Although a vast majority of reinforced concrete bridges have performed satisfactorily during their service life, numerous instances of distress and deterioration have been observed in such structures in recent years. The causes of deterioration of reinforced concrete bridges are often related to durability problems of the composite material. One of the most important deterioration processes which may occur in reinforced concrete bridges is reinforcement corrosion, caused by chlorides present in de-icing salts and/or carbonation of the concrete cover zone.

This paper is mainly based on Thoft-Christensen [5], Thoft-Christensen et al. [6], and de Brito et al. [7].

3. FUTURE BRIDGE MANAGEMENT SYSTEMS

Future advanced bridge management systems will be based on simple models for predicting the residual strength of structural elements. Improved stochastic modelling of the deterioration is needed to be able to formulate optimal strategies for inspection and maintenance of deteriorated reinforced concrete bridges. However, such strategies will only be useful if they are also combined with expert knowledge. It is not possible to formulate all expert experience in mathematical terms. Therefore, it is believed that future management systems will be expert systems or at least knowledge-based systems.

Methods and computer programs for determining rational inspection and maintenance strategies for concrete bridges must be developed. The optimal decision should be based on the expected benefits and total cost of inspection, repair, maintenance and complete or partial failure of the bridge. Further, the reliability has to be acceptable during the expected lifetime.

The first major research on combining stochastic modelling, expert systems and optimal strategies for maintenance of reinforced concretes structures was sponsored by

EU in 1990 to 1993. The research project is entitled “Assessment of Performance and Optimal Strategies for inspection and Maintenance of Concrete Structures Using Reliability Based Expert systems”. The results are presented in several reports and papers; see e.g. Thoft-Christensen [5], [8] and de Brito et al. [7]. The methodology used in the project is analytic with traditional numerical analysis and rather advanced stochastic modelling.

Monte Carlo simulation has been used in decades to analyse complex engineering structures in many areas, e.g. in nuclear engineering. In modelling reliability profiles for reinforced concrete bridges Monte Carlo simulation seems to be used for the first time in December 1995 in the Highways Agency project “Revision of the Bridge Assessment Rules based on Whole Life Performance: Concrete” (1995-1996, Contract: DPU 9/3/44, Project Officer: P.C. Das). The project is strongly inspired of the above-mentioned EU-project. The methodology used is presented in detail in the final project report by Thoft-Christensen & Jensen [9].

In the Highways Agency project “Optimum Maintenance Strategies for Different Bridge Types” (1998-2000, Contract: 3/179, Project Officer: N. Haneef) the simulation approach was extended in December 1998 by P. Thoft-Christensen [10], [11] to include stochastic modelling of rehabilitation distributions and preventive and essential maintenance for reinforced concrete bridges. A similar approach is used in the project by D.M. Frangopol [12] on steel/concrete composite bridges.

In a recent project “Preventive Maintenance Strategies for Bridge Groups (2001-2003, Contact 3/344 (A+B), Project Officer V. Hogg) the simulation technique is extended further to modelling of condition profiles, and the interaction between reliability profiles and condition profiles for reinforced concrete bridges, and the whole life costs. The simulation results are detailed presented by Frangopol [13], Thoft-Christensen & Frier [14], and Thoft-Christensen [15].

Many authors have published a large number of reports and papers on this subject in the last decade. A number of improvements, additions and modifications are described in this literature. However, The Highways Agency projects have played a major role in this development.

4. ESTIMATION OF LIFE-CYCLE COST (LCC)

During the last 20 years important progress has been made in Life-Cycle Cost (LCC) analysis of structures, especially offshore platforms, bridges and nuclear installations. Due to the large uncertainties related to the deterioration and maintenance of such structures, analysis based on stochastic modelling of significant parameters seems to be the only relevant modelling. However, a great number of difficulties are involved in this modelling, but also in the practical implementation of the models developed. The main purpose of this section is to discuss these problems from a social point of view.

LCC analysis may be used not only in the design of new bridges, but also in designing maintenance strategies for individual structures as well as groups of bridges. Therefore, several potential applications are obvious. However, it is a fact that only a few real applications of LCC in bridge engineering are reported in the literature; see Thoft-Christensen [16].

To understand why LCC is seldom used in bridge engineering, it is necessary to look at the modelling techniques used. In planning maintenance budgets for e.g. highway agencies the total expected costs for a group of bridges must be estimated and minimized. There are several models available in the literature, but most of them are

similar to the modelling presented in section 5. The situation is quite different and more complicated if only a single bridge is considered whether LCC design of a new bridge or maintenance of an existing bridge is considered. The most complete modelling seems to be the modelling presented in section 7.

Why is LCC not used in bridge engineering? There are many reasons, but the main reason seems to be that the bridge engineers do not at all understand the probabilistic concepts behind LCC. It is certainly not enough to have taken a course on probability theory or in structural reliability theory. What is needed is first of all a deep understanding of the advantages on using LCC.

It is very hard to convince an experienced structural engineer that a stochastic approach to safety is more relevant than a deterministic approach to modelling uncertainties. Even to-day many structural engineers feel more confident with a traditional approach. Also notice that modern codes using partial safety coefficients are deterministic although the calibration is often based on stochastic modelling of the relevant parameters.

5. MODELLING-OF LIFE CYCLE COST (LCC)

A large number of models for LCC of groups of structures have been proposed in recent years. These models are usually based on an estimate of the LCC where the expected initial costs InC, the expected failure costs FC, the expected inspection costs IC and the expected repair costs RC are simply added

$$LCC = InC + FC + IC + RC \quad (1)$$

The single terms in this equation have been discussed by numerous researchers, and more and more sophisticated models have been developed. The state of the art is now so advanced that one would believe that it is straightforward to use these models in the future. However, it seems fair to say that LCC design has until now been used in few cases only in bridge engineering.

A bridge management system consists of a large number of different types of bridges. The objective of a bridge maintenance strategy is to minimize the cost of maintaining such a group of bridges in the service life of the bridge stock. Estimation of the service life costs is very uncertain so that a stochastic modelling is clearly needed. Let the number of bridges in the considered bridge stock be m . The expected total cost for the bridge stock can then be written; see Thoft-Christensen [15]

$$E[LCC] = \sum_{m \text{ bridges}} \left[\sum_{\text{life-time } T, \text{ bridge } m} \left[\sum_{\text{year } i, \text{ bridge } m} \text{costs} \right] \right] \quad (2)$$

$$= \sum_{j=1}^m \sum_{t=1}^T \left\{ (1 + \gamma)^{-1} \left[E[C_{Mi}(t)]P(M_{it}) + E[C_{Ui}(t)]P(U_{it}) + E[C_{Fi}(t)]P(F_{it}(t)) \right] \right\}$$

where

$E[C]$ is the expected total cost in the service life of the bridge stock,

γ is the discount rate (factor), e.g. 6 %,

$E[C_{Mi}(t)]$ is the expected maintenance cost for bridge i in year t ,

$E[C_{Ui}(t)]$ is the expected user cost for bridge i in year t ,

$E[C_{Fi}(t)]$ is the expected failure cost for bridge i in year t ,

$P(M_{it})$ is the probability of the event "maintenance is necessary" for bridge i in

- year t ,
 $P(U_{it})$ is the probability of the event “maintenance is necessary” for bridge i in
 year t ,
 $P(F_{it})$ is the probability of the event “maintenance is necessary” for bridge i in
 year t ,
 T is the remaining service life or reference period (in years).

The modelling presented in this section will be extended in section 7 by including user the costs introduced in section 6.

6. USER COSTS

It is a fact that user costs are usually not included when optimal maintenance strategies and decisions are made, although authors often mention that user costs ought to be included. The life-cycle costs are minimized for the considered structure without considering the often significant costs for the users of the bridge and even without considering the long-term effects of the decision. Unfortunately, the maintenance decisions are often political decisions which are not easy to accept for the community. There is clearly a need to convince the decision-makers that user costs should be considered when major decisions are made; see Thoft-Christensen [19].

Life-Cycle Cost (LCC) analysis is in reality based only on the direct costs such as inspection and repair (preventive and essential) costs. Therefore, user costs are generally not included in an LCC analysis. However, Life-Cycle Cost-Benefit (LCCB) analysis is an extended LCC analysis where all kinds of indirect costs such as user costs are included.

To illustrate the importance on including user costs in an LCCB bridge management system, a brief review of a few reports is presented in this section. Notice, that in these reports user costs are modelled deterministically although user costs are always very uncertain. Therefore, user costs should in the future be modelled by stochastic variables or stochastic processes to obtain a rational modelling. However, a deterministic modelling based on statistic documentation is a good starting point for a stochastic modelling of user costs.

The following excerpts are taken from the Highway Bridge section of a technical report entitled “Corrosion Cost and Preventive Strategies in the United States”, see Koch et al. [20]. The project is sponsored by the Federal Highway Administration.

“There are 583,000 bridges in the United States (1998). Of this total, 200,000 bridges are steel, 235,000 are conventional reinforced concrete, 108,000 bridges are constructed using prestressed concrete, and the balance is made using other materials of construction. Approximately 15 percent of the bridges are structurally deficient, primarily due to corrosion of steel and steel reinforcement. The annual direct cost of corrosion for highway bridges is estimated to be \$8.3 billion, consisting of \$3.8 billion to replace structurally deficient bridges over the next ten years, \$2.0 billion for maintenance and cost of capital for concrete bridge decks, \$2.0 billion for maintenance and cost of capital for concrete substructures (minus decks), and \$0.5 billion for maintenance painting of steel bridges. Life-cycle analysis estimates indirect costs to the user due to traffic delays and lost productivity at more than ten times the direct cost of corrosion maintenance, repair, and rehabilitation.”

“Overall, approximately 15 percent of all bridges are structurally deficient, with the primary cause being deterioration due to corrosion. The mechanism is one of chloride induced corrosion of the steel members, with the chlorides coming from deicing salts and marine exposure.”

It is interesting to notice that Koch et al. [20] estimate the user costs due to traffic delays and lost productivity to be more than ten times the direct cost of maintenance, repair, and rehabilitation. User costs are here estimated as the product of additional travel time and the value of time.

Next consider some excerpts from a research report of a project entitled “Strategic review of bridge maintenance costs”; see Maunsell [21]. The report is produced by Maunsell Ltd., UK for the Highways Agency, London, UK.

“A strategic review has been undertaken of annual maintenance costs of the Highways Agency’s structures. ... The object of the exercise was to predict the annual expenditure on essential and preventive maintenance which will be required in each of the next forty years on the Highways Agency’ bridge stock”.

“Road user delay costs due to maintenance were also estimated. These ranged from relatively small amounts to over ten times the direct maintenance costs, depending on the work being done and the type of road. However, the results are very sensitive to the assumptions used and only give a broad indication of likely delay costs”.

“If essential maintenance were underfunded, bridges would, in time, need to be closed or restricted while awaiting repair. The main effect would be road user delay costs of the order of £4.6 million a year for each £1 million of essential maintenance not undertaken. The review showed that the cumulated effects of such under funding would soon become unacceptable due to the disruption ...”.

On May 26, 2002 a barge slammed into the bridge on Interstate 10 over the Arkansas River near Webbers Falls, Oklahoma, USA; see Federal Highway Administration [22]. Four of the bridge’s approach spans collapsed and fourteen people were killed. The bridge is the states most important east-west transportation link, so the collapse had a major influence on the economy. The cost of repair of the bridge was about \$ 15 million and the total user cost was estimated to \$430,000 per day for every day the bridge was closed. It was therefore essential to accelerate the repair which was completed in about 2 month. \$12 million were spent on upgrading of the detour highways. The detours were used by approximately 17,000 vehicles every day the bridge was not open.

Replacement of the Holcombe Flowage structure and the Fisher River structure on STH 27 in the Town of Lake Holcombe, WI, USA with two new concrete bridges is estimated to cost approximately \$ 2.43 million; see Schmidt [23]. The detour will be approximately 16 miles long. With a fuel cost of \$ 1.90 per gallon and a traffic volume of 4,500 cars per day, the fuel cost will be about \$ 2 million for a 6 – 8 month period.

The importance of including user costs is clear from these studies. Therefore, a cost-benefit analysis is needed when life-cycle analysis of maintenance (including inspection cost, repair cost, and user cost) of bridges is performed. This conclusion is based on an extensive study of documents on maintenance costs. They are related to estimation of the importance of estimating user costs when repair of bridges are planned and when optimized strategies are formulated. These studies clearly show that

user costs in most cases completely dominate the total costs. In some cases, the user costs are even more than ten times higher than the repair costs. Therefore an LCCB analysis is more reasonable to use.

There is an enormous amount of work on user costs in bridge engineering in the literature. However, much more research is needed before an LCCB analysis in the bridge area can be made in a satisfactory way. Much of the work done until now is limited to narrow models without a wide area of application. A reliable life-cycle based tool must include direct as well as indirect cost. The bridge owners must learn to listen to the public when decisions regarding repair or replacement of structures are taken.

7. MODELLING OF LIFE-CYCLE COST BENEFIT (LCCB)

For individual bridges LCCB may be used in designing a new bridge, but it is also very useful in connection with decision problems regarding e.g. repair after an inspection has taken place.

After a structural assessment at the time T_0 a difficult problem is to decide if the bridge should be repaired and if so, how and when should it be repaired. After each structural assessment the total expected benefits minus expected repair and failure costs in the remaining lifetime of the bridge are maximized. This model can be used in an adaptive way if the stochastic model is updated after each structural assessment or repair and a new optimal repair decision is taken. Therefore, it is mainly the time of the first repair after a structural assessment which is of importance.

In order to decide which type of repair is optimal after a structural assessment, the following optimization problem is considered for each repair technique; see Thoft-Christensen [5] and de Brito et al. [7]

$$\begin{aligned} \max W &= B(T_R, N_R) - C_R(T_R, N_R) - C_F(T_R, N_R) \\ \text{s.t. } \beta^U(T_L, T_R, N_R) &\geq \beta^{\min} \end{aligned} \quad (3)$$

where the optimization variables are the expected number of repair N_R in the remaining lifetime and the time T_R of the first repair. W is the total expected benefit B minus the repair costs C_R capitalized to the time $t = 0$ and minus the expected failure costs C_F capitalized to the time $t = 0$ in the remaining lifetime of the bridge. T_L is the expected lifetime of the bridge. β^U is the updated reliability index. β^{\min} is the minimum reliability index for the bridge.

The benefits may be modelled by

$$B(T_R, N_R) = \sum_{i=[T_0]+1}^{[T_L]} B_i (1+r)^{T_0-T_{ref}} \frac{1}{(1+r)^{T_i-T_0}} \quad (4)$$

where $[T]$ signifies the integer part of T measured in years and B_i are the benefits in year i . T_i is the time from the construction of the bridge. The i^{th} term in (4) represents the benefits from T_{i-1} to T_i . The benefits in year i may be modelled by

$$B_i = k_0 V(T_i) \quad (5)$$

where k_0 is a factor modelling the average benefits for one vehicle passing the bridge. It can be estimated as the price of rental of an average vehicle/km times the average detour length. The reference year for k_0 is T_{ref} . It is assumed that bridges are considered in isolation. Therefore, the benefits are considered as marginal benefits by

having a bridge (with the alternative that there is no bridge, but other nearby routes for traffic). V is the traffic volume per year estimated by

$$V(T) = V_0 + V_1(T - T_{ref}) \quad (6)$$

where V is the traffic volume per year at the time of construction, V_1 is the increase in traffic volume per year, and T is the actual time (in years).

The expected repair costs capitalized to time $t = 0$ are modelled by

$$C_R(T_R, N_R) = \sum_{i=1}^{N_R} (1 - P_F^U(T_{R_i})) C_{R_0}(T_{R_i}) \frac{1}{(1+r)^{T_{R_i} - T_0}} \quad (7)$$

$P_F^U(T_R)$ is the updated probability of failure in the time interval $[T_0, T_R]$. The factor $(1 - P_F^U(T_{R_i}))$ models the probability that the bridge has not failed at the time of repair. r is the discount rate. $C_{R_0}(T_{R_i})$ is the cost of repair and consists of the three terms, namely the functional repair costs, the fixed repair costs, and the unit dependent repair costs, respectively.

The capitalized expected costs due to failure are determined by

$$C_F(T_R, N_R) = \sum_{i=1}^{N_R+1} C_F(T_{R_i})(P_F^U(T_{R_i}) - P_F^U(T_{i-1})) \frac{1}{(1+r)^{T_{R_i}}} \quad (8)$$

The i^{th} term in (8) represents the expected failure costs in the time interval $[T_{R_{i-1}}, T_{R_i}]$.

$C_F(T)$ is the cost of failure at the time T .

8. STOCHASTIC MODELLING OF DETERIORATION

A realistic modelling of the deterioration of reinforced concrete bridges is essential since deterioration is the main reason for the need of a bridge management system. In this section is as an example presented a simple modelling of the deterioration of reinforced concrete structures due to corrosion of the reinforcement; see Thoft-Christensen [24]. The corrosion process is very complex and the modelling is often based on observations or speculations rather than a clear understanding of the physical and chemical processes.

Corrosion initiation period refers to the time during which the passivation of steel is destroyed and the reinforcement starts corroding actively. In this paper Fick's law of diffusion is used to model the rate of chloride penetration into concrete as a function of depth x from the concrete surface and as a function of time t

$$\frac{\partial C(x,t)}{\partial t} = D_c \frac{\partial^2 C(x,t)}{\partial x^2} \quad (9)$$

where $C(x,t)$ is the chloride ion concentration, as % by weight of cement, at a distance of x cm from the concrete surface after t seconds of exposure to the chloride source. D_c is the chloride diffusion coefficient expressed in cm^2/sec . The solution of the equation (9) is

$$C(x,t) = C_0 \left\{ 1 - \text{erf} \left(\frac{x}{2\sqrt{D_c \cdot t}} \right) \right\} \quad (10)$$

where C_0 is the equilibrium chloride concentration on the concrete surface, as % by weight of cement, erf is the error function. More sophisticated models, which e.g. take

into account variation of D_c regard to x or the spatial or time variation of C_0 have also been formulated.

Let C_{cr} be a critical chloride corrosion threshold and d the thickness of concrete cover, then the corrosion initiation period T_{corr} can easily be calculated from equation (10)

$$T_{corr} = \frac{d^2}{4D_c} \left(\operatorname{erf}^{-1} \left(\frac{C_{cr} - C_0}{C_i - C_0} \right) \right)^{-2} \quad (11)$$

It follows from (11) that the time to corrosion imitation is inversely proportional in D_c . It is therefore of great interest to get a good estimate of D_c . The diffusion coefficient D_c is not a real physical constant for a given concrete structure since it depends of a number of factors such as the water/cement ratio and he temperature.

It is complicated to model the evolution of corrosion after corrosion initiation at time T_{corr} . A linear relation between the diameters $D(t)$ of the reinforcement bars at the time t is therefore often used

$$D(t) = D_0 - c_{corr} i_{corr} (t - T_{corr}), t \geq T_{corr} \quad (12)$$

D_0 is the initial diameter. c_{corr} is a corrosion coefficient, and i_{corr} is the rate of corrosion.

After corrosion initiation $D(t) = t \geq T_{corr}$ the rust products will initially fill the porous zone and then result in an expansion of the concrete near the reinforcement. As a result of this, tensile stresses are initiated in the concrete. With increasing corrosion the tensile stresses will reach a critical value and cracks will be developed.

During this process the corrosion products at initial cracking of the concrete will occupy three volumes, namely the porous zone, the expansion of the concrete due to rust pressure, and the space of the corroded steel. The corresponding total amount of critical rust products W_{crit} to fill these volumes is

$$W_{crit} = W_{porous} + W_{expan} + W_{steel} \quad (13)$$

where W_{porous} is the volume of the porous zone, W_{expan} is the amount of corrosion products needed to fill in the space due to the expansion of the concrete around the reinforcement, and W_{steel} is the amount of corrosion products at time T_{crack} of cracking.

Let the expansion of the concrete around the reinforcement have the thickness t_{expan} at time T_{crack} , then W_{expan} can be written

$$W_{expan} = t_{crit} \times \rho_{rust} \pi (D + 2t_{por}) \quad (14)$$

where t_{crit} is the thickness of the expansion at crack initiation.

Liu & Weyers [25] have estimated t_{crit} by assuming that the concrete near the reinforcement bars is a homogeneous elastic material and can be approximated by a thick-walled concrete cylinder with inner radius $a = (D + 2t_{por}) / 2$ and outer radius $b = c + (D + 2t_{por}) / 2$ where c is the cover depth.. Then the approximate value of the critical expansion t_{cr} is

$$t_{crit} = \frac{cf'_t}{E_{ef}} \left(\frac{a^2 + b^2}{b^2 - a^2} + \nu_c \right) \quad (15)$$

where E_{ef} is the effective elastic modulus of the concrete and f'_t is the tensile strength

of the concrete. ν_c is Poisson's ratio of the concrete.

Finally

$$W_{steel} = \frac{\rho_{rust}}{\rho_{steel}} M_{steel} \quad (16)$$

where ρ_{steel} is the density of steel, and M_{steel} is the mass of the corroded steel that is proportional to W_{crit} . Liu & Weyers [25] have calculated the factor of proportionality for two kinds of corrosion products as 0.523 and 0.622. For simplicity, it is here assumed that $M_{steel} = 0.57 \times W_{crit}$. Then

$$\begin{aligned} W_{crit} &= W_{porous} + W_{expan} + W_{steel} = W_{porous} + W_{expan} + 0.57 \frac{\rho_{rust}}{\rho_{steel}} W_{crit} \\ &= \frac{\rho_{steel}}{\rho_{steel} - 0.57 \rho_{rust}} (W_{porous} + W_{expan}) \end{aligned} \quad (17)$$

The rate of rust production as a function of time (years) from corrosion initiation can Liu & Weyers [25] be written

$$\frac{dW_{rust}(t)}{dt} = k_{rust}(t) \frac{I}{W_{rust}(t)} \quad (18)$$

i.e. the rate of corrosion is inversely proportional to the amount of rust products W_{rust} (kg/m). The factor $k_{rust}(t)$ (kg²/m²year) is assumed to be proportional to the annual mean corrosion rate $i_{cor}(t)$ (μ A/cm²) and the diameter D (m) of the reinforcement.

The proportionality factor depends on the types of rust products, but is here taken as $0.383e-3$.

$$k_{rust}(t) = 0.383 \times 10^{-3} D i_{cor}(t) \quad (19)$$

By integration

$$W_{rust}^2(t) = 2 \int_0^t k_{rust}(t) dt \quad (20)$$

Let $i_{cor}(t)$ be modeled by a time-independent normally distributed stochastic variable $N(3 ; 0.3)$ (μ A/cm²) then the time from corrosion initiation to cracking Δt_{crack} can be estimated by (20) by setting $W_{rust}(\Delta t_{crack}) = W_{crit}$.

$$\Delta t_{crack} = \frac{W_{crit}^2}{2k_{rust}} = \frac{W_{crit}^2}{2 \times 0.383 \times 10^{-3} D i_{cor}} \quad (21)$$

The time to initial cracking is then given by

$$T_{crack} = T_{corr} + \Delta t_{crack} = \frac{d^2}{4D_c} \left(\text{erf}^{-1} \left(\frac{C_{cr} - C_0}{C_i - C_0} \right) \right)^{-2} + \frac{W_{crit}^2}{2 \times 0.383 \times 10^{-3} D i_{cor}} \quad (22)$$

Let the initial crack width be w_0 at time T_{crack} . The crack width will increase for $t > T_{crack}$ when the production of corrosion products is increased. It has not yet been possible to find measurements on real structures, which can indicate how the corrosion crack width increase with time.

Andrade, Alonso & Molina [26] have investigated experimentally the evolution of corrosion cracks in reinforced concrete beams. After formation of the initial crack

the rebar cross-section is further reduced due to the continued corrosion, and the width of the crack is increased. In the paper four simple test specimens have been investigated. An impressed current artificially corrodes the beams. The loss of bar sections is monitored and the corresponding crack evolution is measured by the use of strain gauges attached to the surface of the beams. In all four experiments the function between the reduction of the rebar diameter and the maximum crack width measured in the surface of the concrete specimen can be approximated by a linear function.

Above is presented a brief review of modern modelling of the reliability profiles for reinforced concrete structures. Deterministic models for the different steps in the deterioration process are discussed. Several of the parameters used in the modelling are so uncertain that a stochastic modelling is natural. By crude Monte Carlo simulation predictions for time to initial corrosion, time to initial cracking, and time to a given crack width may be obtained.

9. OPTIMAL MANAGEMENT SYSTEMS

After a structural assessment of the reliability of a reinforced concrete bridge deck at the time T_0 the problem is to decide if the bridge deck should be repaired and, if so, how and when should it be repaired? Solution of this optimisation problem requires that all future inspections and repairs are taken into account. In a decision model proposed in the European research project BREU 3091 some approximations are introduced. After each structural assessment the total expected benefits minus expected repair and failure costs in the residual lifetime of the bridge are maximized considering only the repair events in the residual service life of the bridge see section 7.

10. EXPERT SYSTEMS

Expert systems technology is nowadays being considered as a powerful mechanism for helping human experts in their everyday decision tasks. Being able to represent in the computer system the knowledge structures and reasoning strategies that the human expert follows when approaching a problem, enables other users to share this knowledge and the expert system thus constructed establishes a common decision criterion for the prospective users of the system.

The objective of using expert system technology in bridge management is to produce a software tool to assist bridge inspectors as well as engineering experts in their tasks of assessing and improving the reliability of concrete bridges; see de Brito et al. [7] and Thoft-Christensen [17], [18].

The first step is to identify the various software subsystems and the relations between them i.e. the software architecture that will set the basis for the development of the expert systems. It is natural in bridge management to develop two different modules aimed at different goals. The first should provide technical support to the inspector during the inspection process at the bridge site. The second should assist the engineer in the analysis of the safety of bridges as well as in the selection of maintenance and repair methods.

A number of software modules will interact with the expert systems through specifically designed data files:

- Updating analysis: Based on inspection information and other new information the reliability estimates and the data in the databases must be updated.

- Reliability analysis: The reliability of the bridge must be evaluated as a function of time.
- Structural analysis: The system should be open so that the user is able to use his own finite element software.
- Inspection program: Based on the data in the databases and the reliability estimates the optimal time for the next inspection is calculated using the updating module.

The next step is to identify the representation schemes and inference mechanisms best suited for the implementation of the expert systems, as well as the evaluation and selection of the most promising expert system shells available that would guarantee that the representation and inference requirements identified are fulfilled. The functional interrelations between the expert modules and the analysis programs must be defined.

In bridge management it is convenient to have at least two systems, namely one to be used in the inspection phase and one to be used during maintenance and for repair decisions. In such a case the first system will be highly based on “correlation matrices”. Correlation matrices must be defined for: defects/diagnostic methods, defects/causes and defects/repair methods. A pseudo-quantitative classification of the type no correlation, low and high correlation is useful. Correlation between defects as well as diagnostic and repair methods is also needed. Each matrix must e.g. be organized so that each line represents a defect and each column a possible diagnosis method, cause or repair method. At the intersection of each line and column a number representing the correlation between defect and possible element of reference is to be introduced.

It is important for the applicability of the expert system that it gives all the information needed during and after inspections. Such information could be: general information about the bridge, related diagnostic methods, probable causes, associated defects and provisional defect report.

A crucial task in the development of expert systems is the definition of the databases. An exhaustive study of the data collected for concrete bridges, both at the design stage and after it has been constructed must be provided. At relevant moments of the bridge's service life (usually after construction and after important rehabilitation work is performed), its real situation must be thoroughly described so that future inspections have something to relate to. When the database definition is completed then the set of parameters required for the reliability estimation, the cost optimization, additional bridge parameters dealing with the bridge repair cost and corrosion descriptive parameters are added. Most existing bridge management databases are insufficient for e.g. reliability assessment and for implementing modern decision making tools.

A number of expert modules are needed to define the architecture of the expert system: database module, inspection module and a decision module. The decision module will in general be divided into a number of sub-modules such as: a maintenance/small repair submodule, an inspection strategy submodule and a repair/upgrading/replacement submodule.

In the expert systems a number of strategies must be implemented, such as: Should technical knowledge regarding the need to perform a structural assessment be incorporated into the system and should it also be used to double check when the reliability index estimates that the condition of the bridge is good?

The inspector must be able to perform activities like: Review all the information contained in the database of the bridges. Different types of data are recorded for each bridge: identification and bridge site information, design information, budget

information, traffic information, strength information, load information, deterioration information, factors that model the costs and data for the cross-sections defined for the bridge.

The inspection engineer must at his office be able to e.g. view the inspection results recorded at any previous inspection performed in any of the bridges of the database.

11. DISCUSSION

Designing a new bridge or a bridge maintenance strategy based on LCCB will in general result in an apparently increased initial cost, so it is not attractive for Highways Agencies. This recognition in connection with the conservative tradition of only looking at the initial costs makes it unattractive to use LCCB.

A modern LCCB design is based on a probabilistic approach. Some of the terms in the cost equations are based on probabilistic distributions, expected values, etc. A bridge engineer not familiar with probability theory will be less prepared to accept designs based on a stochastic modelling. This is true not only for design of a bridge, but also for design of bridge maintenance strategies.

Bridge engineers often believe that the design of a new bridge or the repair of an existing bridge is 100% safe in the remaining service life of the bridge. Likewise, if you inform politicians that there is a failure probability of say 10^{-6} you will often be asked whether failure could take place to-morrow. Your answer will probably be yes, it is possible but, unlikely. His reply could then easily be that he does not want the suggested design, but a 100% safe bridge. The conclusion is that we need to educate the general citizen but especially the decision-makers.

The public will is low, since designing a structure based on LCCB will result in an increased initial cost and could therefore give budget and re-election problems for the politicians.

The mathematical modelling is not complete, since there are relevant factors for the LCCB which may not be included in the model. Some minor repairs are often needed even if they are not directly important for the safety of the bridge. It may not always be possible to estimate the condition of the bridge in a rational way. Therefore, for some bridge engineers the concepts behind LCCB is not always acceptable. They feel that the modelling is in some way too complicated and detailed, but at the same time not complete.

It is obvious that using LCCB in bridge engineering will require a lot of reliable data which in many cases are not available. This is especially true when a single bridge is considered. In the case of a single bridge very good and comprehensive data regarding the condition of the bridge is needed. Using LCCB in such a case requires a bridge engineer not only familiar with probabilistic thinking, but also with a lot of experience.

The situation is perhaps a little easier for groups of bridges, since only average data is needed. Such data may to some extent be available in Highways Agency databases. For groups of bridges LCCB based strategies at level 1 may be the way ahead. However, the output of a level 1 modelling should not stand alone – it must be followed up by the knowledge of experienced bridge engineers.

In most countries user costs will be the dominating term in the modelling of LCCB, but they are not usually included in the modelling. The reason is that modelling

user costs are problematic and difficult. However, this is not a reasonable argument for not taking user costs into consideration

Some of the terms in the above-mentioned modelling of LCCB are strongly dependent on the discount rate. A high discount rate will make LCCB design less important than a low discount rate. There is a clear tendency in most countries to use an unrealistically high discount rate. If this is so then using LCCB may be meaningless.

12. CONCLUSIONS

In the future we will see more and more applications of reliability based LCCB bridge management systems. Benefits (user cost) will play an important role in all future systems. Likewise, expert knowledge will be integrated in the systems.

Initially such advanced bridge management systems will be used in a small scale on a limited stock (perhaps only few) of bridges or on new bridges. The experience learned from such studies will be useful in defining areas where more research and data is needed.

A serious problem is that many bridge engineers do not appreciate the probabilistic concepts behind LCCB. The only solution to this problem seems to be to introduce the probabilistic concepts in the university courses in bridge engineering. There is also a great need for statistical data related to inspection and repair of reinforced concrete structures. Therefore, the national bridge databases should be modified to make them useful for designing and using modern bridge management systems.

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