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Service life prediction of concrete structures by reliability analysis

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This paper describes the development and application of a reliability-based system for the service life prediction and management of maintenance and repair procedures of reinforced concrete structures. By using statistical databases and probability theory, this system is capable of establishing levels of reliability in satisfying various governing conditions (also known as limit states) as determined by performance requirements or financial considerations. By using the reliability as a measure of performance requirements, inspection, preventive maintenance, repair and major rehabilitation decisions can be made based on economic analysis over the life-cycle of a structure. Therefore, this system will assist in making rational management decisions on a scientific basis. The paper details the derivation of governing conditions of reinforced concrete structural slabs and demonstrates the possible applications of this method in managing these components. Following similar procedures, reliability-based life-cycle management systems can also be developed for other concrete structural components.

Keywords: service life; concrete structures; reliability analysis

For any given building or structural component, there is a distinct anticipated service life which is directly dependent upon its environmental conditions, material properties, as well as operations and maintenance practices. This also holds true for any repairs which may be effected upon these components.

Individual structural components and subsystems have typical service life ranges that do not necessarily coincide with one another. Yet these subsystems are expected to perform satisfactorily throughout the anticipated service life of the structure. In an effort to make components last to the desired service life it is invariably required to repair or reinstate the components' capabilities to function as intended.

Figure 1 illustrates the life-cycle performance of a typical structural component. The upward slope, labelled *P1*, represents the period shortly after installation, when the system is expected to experience an increase in performance to an optimum operating level of *P2*. From this point onward in the service life, the performance is expected to decline. By conducting repairs and altering operations or maintenance practices it is possible to effect minor increases in performance (the intermediate peaks) as well as reduce the rate of degradation. Eventually, the system will have deteriorated to a point that it will no longer function to the required level, *P3*, thus marking the end of its service life, *Tn*. Should the actual in-service life fail to meet specific expectations, such as the design life, the system is said to have deteriorated prematurely, i.e. a shortened service life of *To*.

The ability to evaluate performance accurately throughout an asset's life-cycle and to manage that asset to alter future performance would be very useful. A thorough examination of the relative efficiency of repair and replacement and the long and short term return on investment is crucial to the decision making process. An ongoing refinement in the prediction of system service lives would be helpful in preparing and managing investment plans. The relative impacts of various repair and maintenance options would also be

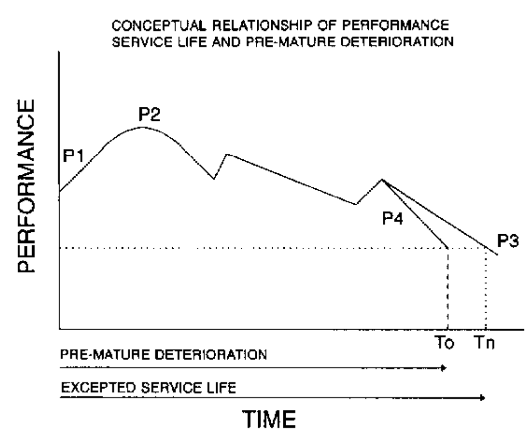

Figure 1 Performance, service life and premature deterioration

used as a guide for annual operations and maintenance allocations.

This paper describes the philosophy, basic development and potential applications of a reliability-based system for the service life prediction and management of maintenance and repair procedures of reinforced concrete structures. The intent of this undertaking is to develop a tool to rationally estimate the performance of reinforced concrete structures and thereby permit scientifically based assessments of the most appropriate, likely and cost-effective maintenance, repair and major rehabilitation schedules.

A significant number of governing conditions has been considerd in the development of this reliability-based predictive system. However, in order to illustrate clearly the techniques applied, only the most critical governing conditions for structural reinforced concrete slabs are presented. Five limit state functions are presented in this paper. They are: flexural strength; punching shear; deflection; delamination; and wearing surface deterioration. Other limit state functions, addressing performance issues or specific maintenance needs, can be readily incorporated into the assessment framework without modification to the computer program.

Service life prediction – a literature review

The service environment for many concrete structures results in very harsh exposure conditions. Defects and deteriorations in both Portland cement and bituminous concrete components are common. The causes and processes of premature deteriorations are very intricate, relating directly to many physical and environmental parameters. The prediction of both initial service life and the effects of maintenance and rehabilitation options presents a considerable challenge. The decision making process must consider the impact of any repair and maintenance options upon the future performance of the structure. This is a very complex and indeterministic process.

Thoft-Christensen and Sørensen[1] proposed a reliability-based methodology to optimize inspection, maintenance and repair of structural systems. Key design variables considered are inspection quality and frequency. The model minimizes total inspection and repair costs while maintaining system reliability to an acceptable level.

A service life prediction model was developed for bridge superstructures by Jiang and Sinha[2] using the Markov chain technique and a third-order polynomial performance function. The performance function related the bridge age to an average condition rating. The model is based upon a subjective condition rating and does not permit an enhanced assessment as a result of repair and rehabilitation.

Funahashi[3] examined chloride ion attack in prestressed concrete parking garage decks and used a computerized finite difference method to predict the movement of the chloride concentration profile with respect to time.

Attwood *et al.*[4] proposed the application of first-order reliability methods to monitor and maintain the degree of delamination in parking garages. The rate of delamination is affected by the type of protective treatment applied and ranges from 3.14% per year with no treatment to 1.57% per year with an absorptive form liner.

The ultimate flexural capacity of typical pretensioned AASHTO bridge girders was evaluated using reliability analysis by Tabsh[5]. A parametric study of structural safety was conducted using Monte Carlo simulation to evaluate the ultimate moment limit state for bridge girders.

Godson[6] outlined the major processes of concrete deterioration, investigative techniques as well as service life prediction of the concrete elements. The primary focus was on building facades.

Mori and Ellingwood[7] evaluated the time-dependent reliability of reinforced concrete structures. The sensitivity of structural reliability to three degradation models was evaluated. Linear, square-root and parabolic functions were used to represent corrosion, sulphate attack and diffusion-controlled deterioration mechanisms respectively. The focus of the work is upon nuclear power plants and the authors conclude that similar approaches could be used in the evaluation of other civil structures where safety *versus* time is of concern.

Performance reliability theory

The need for probabilistic approach to assess service lives resulted from the often uncertain nature of loadings and the performance aspects of reinforced structural concrete. Since it cannot take the natural variation of the physical parameters into account, a deterministic approach, using fixed or arbitrary values for pertinent variables, should not be used to assess performance. These uncertainties can be dealt with effectively by using probabilistic methods in which the safety and service/performance requirements are measured by their reliabilities (defined as the probability of survival, p_s).

History of reliability theory

In the past, reliability theory has most often been identified with the military, aerospace and electronics fields. The importance of reliability theory in the area of civil engineering has been increasingly realized over the past number of years. The real beginning of structural reliability as a topic for academic research may be traced back to Freudanthal[8]. Only over the past decade has the area of reliability in civil engineering applications received the attention necessary to ensure the optimal performance and safety of structures.

The importance of civil engineering reliability theory stems from the very nature of the various approaches to structural design. Most often in the past, a deterministic approach has been taken in civil engineering design where the design parameters usually consisted of selected factors of safety multiplied by expected service loads. However, these service loads are rarely known with certainty. As a result these loads should actually be treated as random variables. This different approach calls for the implementation of probabilistic and statistical techniques. These analytical tools have traditionally formed the basis of reliability theory and more recently provide the basic framework for the study of civil engineering related reliability problems.

The history of reliability and maintainability management is much shorter than that of the technical reliability. Only in the last ten years have researchers started to look into the applications of reliability theory to repair, maintenance and management of important structures such as nuclear power stations[9–11]. The analysis of time-dependent changes in system reliability has only been considered in the recent past[7].

Structural reliability

The reliability of a structure or component is defined as its probability of survival, p_s, which is related to the probability of failure, p_f by:

$$p_s = 1 - p_f \tag{1}$$

Considering the uncertainties associated with load effects on structural resistance, the probability of failure (or reliability) provides a meaningful measure of the adequacy of a structure or member. Failure is defined in relation to different possible failure modes, commonly referred to as limit states. For example, ultimate limit states represent the inability of the structure to resist the imposed load effects and can be associated with large inelastic displacements and, for bridge decks, large cracks or punching shear failure. On the other hand serviceability limit states are defined as the inability of the structure to meet its normal use or durability requirements. Examples are excessive delaminations or deformations.

The performance of a structure in relation to a certain limit state can be described as a function of a set of basic parameters (X_i, $i = 1, \ldots, n$). For example, the deformation of a structural component, d, can be expressed as a function of the load effects, material properties and geometric parameters:

$$d = d(X_1, X_2, \ldots, X_n) \tag{2}$$

Assuming that the maximum allowable deformation for a certain serviceability condition is d_0, the boundary between failure and survival can, in this case, be described by $d = d_0$ or:

$$d - d_0 = 0 \tag{3}$$

Failure occurs if $(d - d_0) > 0$ while the structure is safe for $(d - d_0) < 0$. Substituting Equation (1) into Equation (3) one can write:

$$f(X_1, X_2, \ldots, X_n) = 0 \tag{4}$$

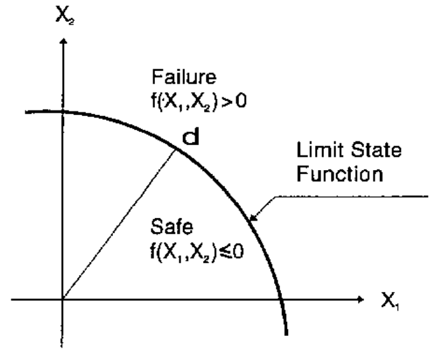

Figure 2 Illustration of reliability problem in two-dimensional space

in which d_0 was incorporated in the function f, such that failure occurs if $f > 0$ and no failure occurs if $f < 0$. The function f is called the limit state function, and it separates the failure region ($f > 0$) and the safe region ($f \leq 0$).

This concept is illustrated in *Figure 2*, where the problem is simplified by assuming that the limit state function depends only on two basic parameters, $f(X_1, X_2)$. This simplification is convenient because it allows the visual respresentation in *Figure 2*, but the concept is equally appliable to an n-dimensional space. The limit state function can then be plotted in the (X_1, X_2) plane, and the failure and safe regions are as depicted in *Figure 2*. If one envisages a perpendicular axis to the (X_1, X_2) plane on which a joint probability density function is defined, the probability of failure is represented by the volume under the density function and over the failure region. The calculation of the failure probability in the general n-dimensional case is a complex task. Much research has been devoted to this problem, and solutions with different levels of detail and accuracy are available[12,13].

The above discussion concentrates on the reliability of a single component. Methods are available to obtain reliability estimates (or bounds) for systems composed of several components.

The reliability-based maintenance and management concept and the predictive techniques described within this paper are based on the above theory. In order to meet the operational requirements, the probability of exceedance for all limit states during the entire life span, or the remainder life span, must be kept within a set of predetermined performance limits. These limits define the performance requirements for a component or an entire structural system. They have significant implications on operation and maintenance costs as well as the degree or risk of failures that may occur.

Methodology

The methodology hinges upon the identification and quantification of the limit states which govern the performance of reinforced concrete components. The functional expressions of these limit states relate various physical parameters with performance characteristics of the system being investigated. More significantly, the methodology should be capable of reflecting changes in these physical and chemical conditions via adequate consideration of repair, inspection and maintenance information.

The performance of any particular component, subsystem or system of an asset is dependent upon numerous variables. Only in rare instances would the values of any of these variables or their interactions be known categorically. In most instances the values and nature of these variables are in accordance with specific distribution functions. The exact distributions are evasive; the records of performance from various sources are used to indicate tendencies in observed conditions which reflect physical conditions.

Using figures generated from the physical data, the reliability-based framework calculates the probability of exceeding designated acceptable limits. Reliability is considered to be the probability that these limits will not be exceeded and is equal to the probability of survival ($R = p_s = 1 - p_f$). The predicted service life is defined as the age when reliability falls below a permissible level.

Output from the analysis will be available to modify the output of, or input to, performance prediction databases. By scrutinizing the condition of any given asset, the merits of various construction and maintenance options may be determined on an economic as well as scientific basis. Since the intent of the analysis is to examine the performance indicators at any point within the life span of the asset, it is crucial to the methodology that information gathered be useful for on-going supervision of operations and maintenance on an annual basis. Reassessment of the service lives and economic factors affecting any given facility must be conducted frequently enough to provide an accurate reflection of the asset performance and to allow for modification of the operations and maintenance practices if (and as) deemed necessary.

Governing conditions and primary deterioration mechanism

The primary design concerns for most structures are the provision of strength and the fulfilment of given serviceability criteria. In most instances the initial strength requirements will be adequately addressed by following standard design procedures. The variations in load, material properties, construction practice and mechanical resistance are incorporated in recognized design practices. However, the physical and chemical changes that occur during the life of structural members and the impact upon structural performance are usually not considered at the design stage. Similarly, serviceability aspects are considered as factors of the conditions existing at the time of design and construction.

The rate of degradation of the initial conditions and the nature of those changes drastically affect the long term performance and functional life of a structure. Therefore, by evaluating the progression of deterioration in any given member, it is possible to assess that component's capability to provide the initial design function at any time during the service life.

In order to more adequately demonstrate the concept of service life prediction via reliability-based techniques, this paper does not attempt to be exhaustive in its examination of concrete structural components. For this reason further discussions are limited to the conditions that govern the performance of conventionally reinforced concrete slabs.

Typically, reinforced concrete slabs must be able to: withstand all imposed loads; accommodate deformations; and provide a suitable wearing surface. Emphasis is usually placed upon the ultimate limit states of flexural resistance and punching shear capacity yet final designs are generally controlled by the serviceability limit states such as cracking and deflection.

Limit states functions, considered to be representative of the conditions most likely to govern performance of concrete slabs, have been developed and are presented later. They are: (a) flexural strength; (b) punching shear; (c) deflection; (d) delamination; and (e) wearing surface serviceability.

Changes in slab performance are directly related to the degradation taking place. The effect of the deterioration will have varying significance, depending upon initial design and construction conditions as well as the functional requirements under consideration. To properly assess these limit states over the life of the structure, it is essential to understand the deterioration mechanism(s) at work.

The bulk of deterioration of reinforced concrete members results from corrosion of reinforcing steel or other imbedded ferrous elements. The corrosion affects the integrity of reinforced concrete in two distinct ways. The most evident factor is the loss of reinforcing steel cross-section, thereby lessening structural capacity. Secondly, the products of corrosion occupy considerably more volume than the reinforcing steel had initially and cause delamination between concrete cover, the remaining reinforcement and the remainder of the member.

Sound concrete usually provides an environment of relatively stable high pH surrounding the reinforcing steel and corrosion would not be expected if all conditions at the depth of steel remained stable. Standard safeguard measures such as decreasing the water to cementing materials ratio, increasing concrete cover as well as the application of membranes and sealants are all aimed at maintaining the conditions around the rebar at close to their original states.

Carbonation

Immediately after the exposure of concrete to air, the concrete paste begins to form carbonates. These carbonates reduce the pH of the concrete and can damage the protective oxide film bound around the reinforcing steel. If the carbonates are found in conjunction with moisture and oxygen the reinforcing steel may corrode. The rate of carbonation is a function of the concrete quality, the relative humidity and the concentration of carbon dioxide.

The rate of carbonation is generally considered to be inversely proportional to the square root of time in accordance with Fick's law[14].

Recent surveys in Canadian buildings by Canada Mortgage and Housing Corporation[15] indicate that carbonation related corrosion is not of major concern. The observed occurrence of carbonation was considerably higher on vertical surfaces and it was concluded that this may have been partially due to inadequate concrete cover or poor concrete placement and consolidation. In addition, poorly ventilated areas such as interior parking structures were slightly more susceptible due to elevated carbon dioxide levels.

Since corrosion due to carbonation does not yet appear to be a major deterioration mechanism in Canada, the effects of carbonation will continue to be monitored and the development and implementation of deterioration model(s) for use with this predictive system will proceed as warranted.

Chloride ion attack

In North America, the majority of the deterioration to concrete structures results from the corrosion of reinforcing steel induced by the diffusion of chloride ions (Cl) throughout the concrete matrix. Chloride-induced deterioration is considerably more evident in coastal areas and in regions where de-icing chemicals are routinely applied to thoroughfares. The affected members are primarily slabs and beams located in structures that are exposed to vehicle traffic and the direct winter application of calcium chloride and sodium chloride.

The profile of chloride ion penetration is typically determined by extracting samples of concrete by coring or drilling. High chloride ion concentration at the level of reinforcing steel correlates with the presence of active corrosion. Investigations conducted by the National Research Council[16] indicated that the threshold acid-soluble chloride ion level required for the depassivation of reinforcing steel and the initiation of corrosion is in the range of 0.028% to 0.041% by mass of concrete. Several test procedures are available for the determination of chloride ion content in concrete; most commonly cited are those described in AASHTO-T260[17] and ASTM C114 Section 19[18].

According to Clear[19] an average ratio of water-soluble to acid-soluble results is between 0.75 and 0.80. For chloride ions which had permeated hardened cement pastes, Berman[20] observed a ratio of 0.87. A conversion factor of 0.75 is widely used, and yields an upper value for the water-soluble threshold of 0.03% by mass of concrete.

The threshold level and the progression of corrosion may be significantly affected by the cement composition, pore structure, the alkalinity of pore water and concrete, chloride cation type, sorptivity and porosity[21–29].

Care must be taken in the interpretation of the chloride ion results. In certain regions of Canada the regularly used aggregate has very high levels of chloride ions. These aggregates, however, do not produce concrete which is inherently more prone to corrosion. The chloride content of the aggregate is apparently 'locked-in' and relatively insoluble under normal circumstances. The chloride ions from these aggregates are made available with standard acid soluble and water soluble tests due to the practice of aggregate pulverization. In some circumstances a more discriminating test method, such as the Soxhlet extraction technique suggested by Berman, should be used to determine the 'available' chloride and reduce the potentially misleading assessment of field conditions.

Clear[30] proposed that the time prior to chloride induced corrosion may be empirically modelled as follows:

$$RT = 129(S_i/25.4)^{1.22} / [K^{0.42}(w/c)] \tag{5}$$

where RT is the time (years) to onset of corrosion, S_i is the concrete cover (mm), K is the chloride concentration of water deposited on slab (ppm), and w/c is the water-to-cement ratio.

Funahashi applied Fick's diffusion law to evaluate the time when chloride concentration level exceeds the threshold limit to initiate corrosion on prestressing strands. A computerized finite difference method was used to develop a method of predicting the movement of the chloride concentration profile with respect to time.

Rigorous mathematical and computer models for the prediction of chloride ion penetration in marine concrete were developed by Grace[31]. The model incorporates the effects of both diffusion, via Fick's Second Law, and dispersion occurring as a result of wetting and drying via capillarity and evaporation respectively. The successful application of the model to predict laboratory results and to study the sensitivity of chloride penetration to various concrete parameters reaffirms the importance of sorptivity and moisture movement in the transport process. Grace recommended that further work be conducted to describe more accurately the moisture transport mechanisms which will control chloride penetration in unsaturated concrete.

Framework for reliability-based analysis

The reliability-based assessment follows the methodology described earlier.

Statistical information provides base input of initial design data as well as all relevant inspection and maintenance records or tendencies. Each of the limit state function variables is assigned a probability density function (pdf) that reflects the background statistical information. The analysis program designates values for each of the parameters used by the limit state functions (in accordance with their assigned pdf) and calculates the reliability using either Monte Carlo Simulation or Second Order Reliability Methods. Results are presented in both graphical or textual report format as well as being electronically retained for calibration purposes.

The predictive technique employed relies upon the definitions of the deterioration mechanism and limit state functions to describe the performance. As the knowledge of these items changes so may the models used by the computer program. The reliability-based framework provides the capability to input, edit, record and test limit state functions without requiring computer programming knowledge. By continually updating the statistical databases on inspection and maintenance, the distributions assigned will be reflective of historical tendencies as well as current conditions, thereby providing a more realistic basis for prediction.

Limit state functions

While the empirical model presented by Clear (Equation (5)) fails to explicitly consider many of the factors recognized as having significant impact upon chloride ion diffusion (porosity, sorptivity, chemical composition of cementitous materials, changes in concrete quality and alkalinity with time), it does permit assessment of the commencement of corrosion in terms routinely available to investigators and designers; namely concrete cover and water-to-cement ratio. Its principal advantage is simplicity. This model has been implemented, with minor modification, as the deterioration mechanism for the reliability-based assessments described herein.

The exposure conditions examined by Clear[30], i.e. constant daily saline exposure, are considerably different from those likely to be encountered in most structures. This fact coupled with corrosion information gathered from parking garage and bridge data[32–36] leads to the modification of the constant of 129 in Equation (5) to an empirical factor, F, to consider the slabs' exposure level, frequency of washdown, slope and adequacy of drainage. The time to corrosion as implemented in the program is as follows:

$$RT = F\,(S_l/25.4)^{1.22} / [K^{0.42}(w/c)] \tag{6}$$

As discussed above, the water soluble chloride ion threshold level required to initiate corrosion is approximately 0.03% mass of concrete for ordinary Portland cement concrete. It is assumed that the migration rate is constant and that at time RT the chloride ion concentration is expected to reach the threshold level. If chloride ion inspection information is available, a comparison between the expected and actual levels of Cl^- may be used to determine the 'apparent' age of the slab. The apparent age is a reflection of the in-service condition relative to previous (initial) prediction and may be expressed as:

$$Age_a = Age_r - (T_i - RT^*(Cl_i^-/0.03)) \tag{7}$$

where Age_a is apparent age (years), Age_r is the real-time age (years), T_i is the age at inspection (years), Cl_i^- is the water soluble chloride ion content at rebar embedment at T_i.

If the apparent age of the slab (Age_a) is greater than the calculated time-to-corrosion (RT), the cross-sectional area of the slab reinforcement A_s (mm^2/m) is calculated as:

$$A_s = \pi\,[R_d - 2C_r\,(Age_a - RT)]^2 / 4b \tag{8}$$

where b is unit slab width (m), R_d is initial rebar diameter (mm) and C_r is the rate of reinforcing corrosion (mm/yr).

All five of the limit state functions described in the following text, and as employed by the system, incorporate the model for deterioration by using the modified value of A_s, as per Equation (8), in their formulation. These limit states are, therefore, time-dependent.

Flexural strength – one-way bending

If a linear strain distribution is assumed for concrete slabs under one-way bending, resistance to tensile forces across the section may be considered to be provided solely by the reinforcing steel and the ultimate bending moment per unit width, M_u (MNm/m), in the slab can be expressed as

$$M_u = A_s\,df_y\,(1. - 0.59\,A_s f_y / (df'_c)) \tag{9}$$

where d is the distance from extreme compression fibre to centroid of steel (mm), f_y is the yield strength of steel (MPa) and f'_c is the compressive strength of concrete (MPa).

Equation (9) assumes a rectangular stress distribution at ultimate load[37], and that collapse will occur as a result of progressive corrosion of the reinforcing steel. This corrosion will lead to tension failure which is characterized by yielding of the steel, followed by large strains, concrete crushing and, finally, collapse.

The limit state function for flexural strength as used in the program is expressed as follows:

$$M = M_u - M_d \tag{10}$$

where M_d is the applied service moment (MNm/m).

Punching shear

The expression evaluated for allowable punching shear stress, v_c (MPa), by the program is:

$$v_c = 0.2\,(1 + 2/\beta_c)\sqrt{f'_c} \tag{11}$$

where β_c is the aspect ratio of the load area.

This expression corresponds to the CSA A23.3-94 standard for structural concrete design[38], and is of the same basic form yet slightly less conservative than the ACI[39] formula for computation of shear strength under two-way action.

The punching shear stress, v_u (MPa), resulting from a load area is expressed as follows:

$$v_u = V_u / b_0 d \tag{12}$$

where V_u is the applied load (MN) and b_0 is the perimeter of the critical shear zone at a distance $d/2$ from the perimeter of the load area (mm).

The function used by the program for determining the likelihood of failure due to punching shear takes the following form:

$$V_p = v_c - v_u \tag{13}$$

This limit state assumes that the slab is not reinforced for shear resistance.

Deflection – one-way bending

The program evaluates the total deflection of concrete slabs, δ (m), by using the end-moment coefficient method as calculated with the following equation:

$$\delta = 5\ell^2[+M_d - 0.1\ (M_1 + M_2)] / 48E_cI_e \tag{14}$$

where ℓ is the span under consideration (clear span between supports for parking garages slabs and girder or stringer spacing for bridge decks) (mm), M_1 and M_2 are the end moments as determined by moment coefficient method (randomly generated) (MNm/m), E_c is the modulus of elasticity of concrete (MPa), and I_e is the effective moment of inertia of the unit width of slab (mm^4/m).

The limit state function for deflection of slabs as implemented by the program is as follows:

$$\text{Deflect} = \delta_{\text{allow.}} - \delta \tag{15}$$

where $\delta_{\text{allow.}}$ is the prescribed allowable deflection (m).

Delamination

A model for the percentage of surface delamination occurring in concrete slabs was generated and presented by Attwood *et al.* This limit state function couples the time-to-corrosion of Equation (5) with consideration of temperature effects, crack width and slab protection systems. The program uses this model in the following form:

$$D = N_t\ N_{crw}R_{del}\ (Age_a - RT) \tag{16}$$

where D is the percentage delamination, N_t is the temperature adjustment factor, N_{crw} is the crack width adjustment factor and R_{del} is the rate of delamination (%/year).

The limit state function for delamination is defined as:

$$Del = D_{cr} - D \tag{17}$$

where D_{cr} is the critical level of delamination (%).

Wearing surface serviceability

Pavement wearing surface performance indicators were developed by the Highway Research Board in the late 1950's and early 1960's in order to quantify the 'rideability' of roadway surfaces. The data gathering in the AASHO Road Tests as well as the development of these Present Serviceability Indices is detailed by Carey and Irick[40].

The serviceability index was not intended for use on bridge decks. These are significant differences in the behaviour of paved surfaces when applied on bridges and approaches as compared to typical road surfaces. The framework for assessment of pavement serviceability that these indices establishes is, however, very beneficial and only minor 'stiffening' of certain variable distributions was required to produce results in keeping with the observed performance of wearing surfaces on concrete bridge decks in Canada.

The Highway Research Board had developed a Present Serviceability Index for both rigid and flexible pavements. These indices have been adopted, with minor modification, and are presented below:

$$psi_{\text{rigid}} = 5.41 - 1.80\log(1 + SV) - 0.09\sqrt{(10D)} \tag{18}$$

$$psi_{\text{flex}} = 5.03 - 1.91\log(1 + SV) - 1.38R_D^2 - 0.01\sqrt{(10D)} \tag{19}$$

where SV is the slope variance in wheel path, R_D is the rut depth, and D is the % of delamination as determined in Equation (16).

The limit state for wearing surface performance takes the following form:

$$Surf = psi - psi_{\text{allow}} \tag{20}$$

where psi_{allow} is the minimum allowable serviceability index, and psi is the present serviceability index as calculated by either Equation (18) or (19).

Performance databases

Material selections and maintenance options chosen over the life of a facility undoubtedly influence both the annual cost of operating an asset and its service life. Thorough examination of the relative efficiencies of various design options, repair and replacement scenarios, as well as the long and short term returns on investment is crucial to the decision making process. Such assessment can only be accomplished if adequate and suitable data exists.

Inspection data and the records of performance, obtained from various sources, are used to indicate tendencies in observed conditions which reflect physical conditions. This link provides essential input to the reliability-based analysis. It is believed that, through extensive collection of field performance data, and its implementation within the methodology, the confidence in the distributions being chosen and hence in the methodology itself will steadily increase. Output from the analysis will, similarly, be available to modify the output of, or input to, the performance database.

The ability to manage annual operations and maintenance activities will be greatly increased once the desired service life of an asset has been determined and translated into annual operations budgets. The relative impacts of various operations and maintenance scenarios on asset performance will become more evident as the experience/knowledge base grows.

By regularly collecting data which reflect performance, the knowledge of the costs associated with owning the structure as well as of the effect of the maintenance procedures upon the service life will be continually improved.

As our knowledge of the controlling conditions grows so will the understanding of the maintenance levels required to provide functional continuity at an acceptable level of reliability. The costs associated with various levels of maintenance may then be used to manage the structures in a fiscally prudent manner so as to optimize financial performance.

Applications

The limit states described above are time-dependent functions. This characteristic permits the examination of any of these limit states, or systems that they may define, at varying times within the life-cycle of a concrete slab.

The analysis framework provides great flexibility. The ability to define, record and modify limit state functions, without computer programming, permits the rapid evaluation of the significance of the functions, parametric sensitivity analysis, as well as the capability to develop and test limit states for specific purposes.

The computer program developed permits the examination of limit states as independent functions (a component basis) or in systems as defined by the user and the particular configuration under consideration. Systems, graphically depicted in *Figure 3*, can be defined as a number of components combined in parallel or in series. In series systems, if one component fails so does the system. All components of a parallel system must fail for system failure to occur. More complex systems may be modelled by combining parallel and series subsystems.

Assessing service life and performance comparison

For this methodology and program to be of value it must be able to readily predict and graphically display the service life. The service life is defined as the age of the slab when the reliability of the particular system modelled falls below an acceptable level. The acceptable level of performance, or risk acceptance level, is chosen by the user and may be influenced by economic considerations or a desire to harmonize the life-cycles of other components.

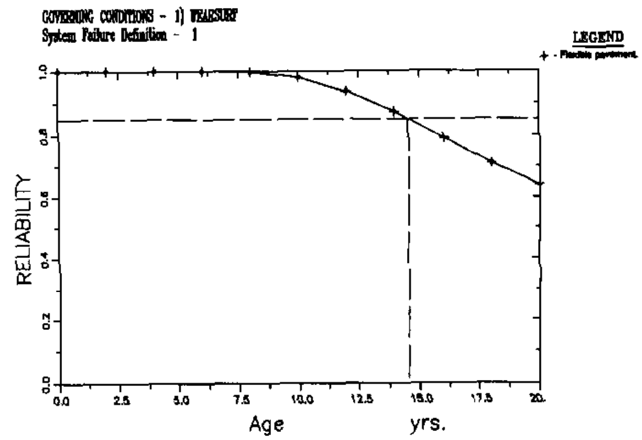

Figure 4 Service life of flexible wearing surface

The predictive techniques, deterioration mechanism and limit state functions, particularly the delamination model, described within this paper, have been very successfully applied for condition assessment and the prediction of remaining service life of parking garage structures[41]. Ongoing studies apply these techniques to the maintenance and management of bridge decks[42].

The performance of a flexible pavement wearing surface is depicted in *Figure 4*. The acceptable reliability (risk acceptance) has been set as 85% and the service life has been determined to be roughly 14 years.

At the conceptual stage, various design choices may be evaluated to determine the impact that their implementation would have upon the service lives. *Figure 5* presents the performance expectations for two options being considered for a new parking garage slab. Option one includes 40 mm of concrete cover coupled with a protective membrane and Option two is an untreated slab with 50 mm concrete cover. These design alternatives follow the recommendations of CSA S413 - 94[43]. With the critical level of delamination, D_{cr}, set at 10% and a permissible total deflection, $\delta_{allow.}$, of $\ell/240$, the

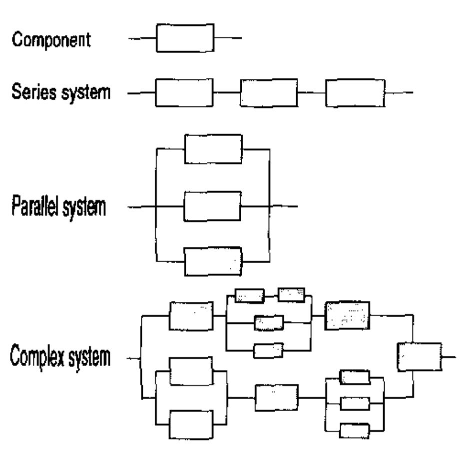

Figure 3 System definitions

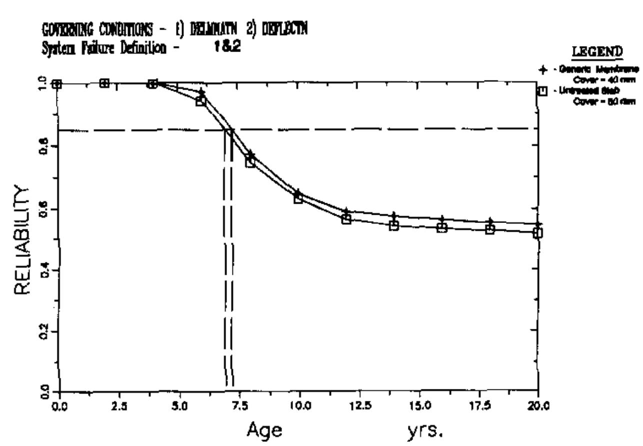

Figure 5 Effect of membrane *vs* additional concrete corer

service life expectations are approximately 7 years and 7¼ years respectively. In assessing the economic merits of these alternatives, the total costs, initial capital as well as operations and maintenance requirements, must be considered.

The determination of the service life of any given aspect of reinforced concrete slabs at any time in its life permits rational analysis of the economic factors involved during the life-cycle. Existing practice only permits analysis of these factors either prior to the service life (speculative) or at conclusion of the life-cycle (historical value only).

Operational decision making tool

The program provides the user with the ability to examine the consequences of potential action or inaction relative to operational and maintenance procedures. *Figure 6* depicts the example of two bridge decks of comparable configurations and located in the same city. The critical level of delamination was set at 5%. The only appreciable historical difference between these two slabs was the increased frequency of washdown and cleaning of drains and expansion joints for the well maintained slab. By evaluating the service lives and results presented in *Figure 6*, relative to delamination and chloride ion levels, the benefits associated with

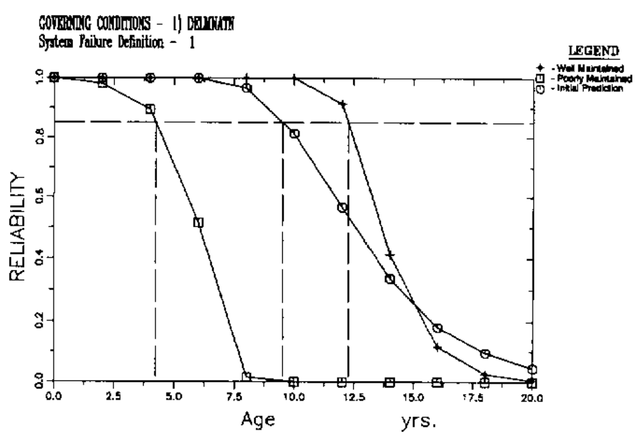

Figure 6 Effect of operations and maintenance practices on service life

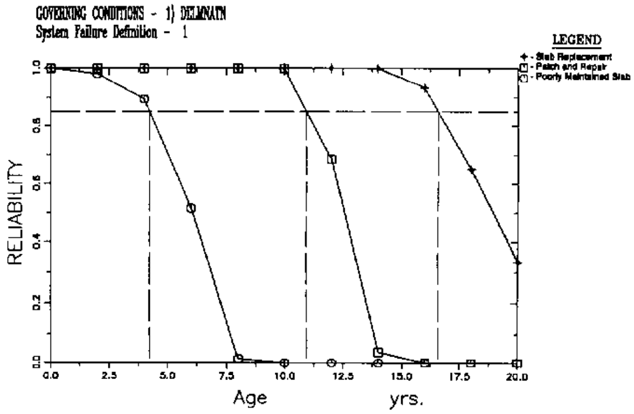

Figure 7 Comparison of slab repair and replacement

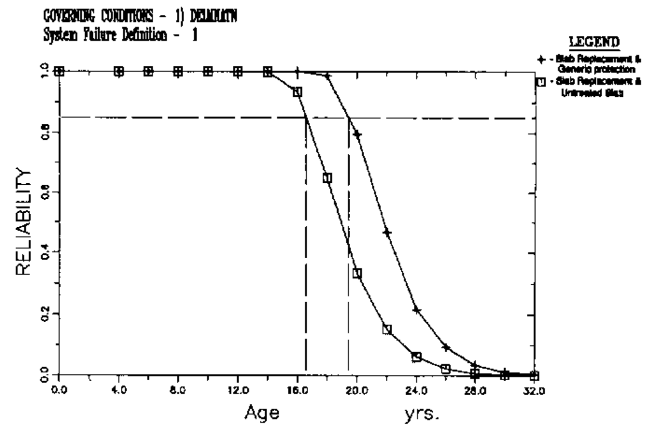

Figure 8 Further extension of service life via generic protection system

cleaning become evident. The central curve, represented by O's, indicates the predicted performance of the slabs; an acceptable reliability for approximately 9.5 years. The actual slab performances, as reflected by inspection information, yielded service lives of approximately 12.5 years for the well maintained slab and 4 years for the poorly preserved deck.

If the decision is made to retrofit the badly delaminated slab (i.e., that with a service life of 4 years, indicated with □'s in *Figure 6*) numerous options may be available. The possible choices include (a) the removal and patching of the delaminated areas and (b) the complete slab replacement.

By evaluating the performance of the slab for each of these options it is possible to assess their relative benefits, in terms of prolonged life and economic factors. *Figure 7* illustrates the predicted performance for options (a) and (b); extending the service life to 11 years or 16.5 years respectively, from the original prediction of 4 years.

If the concrete slab is completely replaced there are numerous preventative measures that may be implemented to further extend the slab life. *Figure 8* depicts the potential benefit, a further prolongation of approximately 3 years, to be realized through the application of one such system. The illustration represents the performance curves for the new untreated deck, yielding a service life of 16.5 years, and that for a slab with a generic protection system that produces a service life of 19.5 years. Decisions as to the final protection system, if any, are usually driven by economic considerations.

Conclusions

This paper has demonstrated that reliability theory can be efficiently applied to the maintenance, repair and management of reinforced concrete slabs. Levels of reliability in satisfying various limit states can be established based on performance requirements or economic considerations. This approach provides the necessary flexibility to owners or engineers in meeting their specific performance requirements and financial constraints.

By using reliability as a measure of performance, decisions concerning inspection, preventive maintenance, repair and major rehabilitation can be made based on economic analysis throughout the life-cycle of a structure. Therefore, the methodology presented in this paper has great potential for various types of applications in maintenance and management of existing structures, i.e., assessing service life; performance life-cycle analysis; evaluating impact of operations and maintenance practices; analysing maintenance, repair and renovation alternatives; as well as the optimization of material selection and operation and maintenance schedules.

Various levels of complexity of deterioration mechanisms and limit state functions can be readily incorporated into the reliability-based framework. Specific needs and levels of accuracy of the life-cycle predictions can be achieved by using different limit state functions.

This methodology may be used as the basis for development of reliability-based systems for a broad range of structural concrete components such as columns, shear walls, concrete dams, bridge piers and abutments.

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