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Reliability-based assessment of ageing bridges using risk ranking and life cycle cost decision analyses

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Abstract

Information about present and anticipated bridge reliabilities, in conjunction with decision models, provides a rational and powerful decision-making tool for the structural assessment of bridges. For assessment purposes, an updated reliability (after an inspection) may be used for comparative or relative risk purposes. This may include the prioritisation of risk management measures (risk ranking) for inspection, maintenance, repair or replacement. A life-cycle cost analysis may also be used to quantify the expected cost of a decision. The present paper will present a broad overview of the concepts, methodology and immediate applications of risk-based assessments of bridges. In particular, two practical applications of reliability-based bridge assessment are considered — risk ranking and life-cycle cost analysis. © 2001 Elsevier Science Ltd. All rights reserved.

Keywords: Structural reliability; Concrete; Safety; Bridges; Decision-analysis

1. Introduction

It is quite evident that the bridge population in Australia and elsewhere is ageing. For example, in Australia, over 60% of bridges for local roads are over 50 years old and approximately 55% of all highway bridges are over 20 years old. The incidence of structural deterioration increases with bridge age due to corrosion, fatigue, wear and tear and other forms of material property degradation. During this same period vehicle, loads and legal load limits have been steadily increasing. Ageing bridges subject to increasing legal load limits mean that existing bridges often fail to satisfy structural requirements as specified for new bridges. As such, it is estimated that US$300 million is urgently needed to strengthen or replace defective bridges in the Australian state of New South Wales (NSW) [1]. In the US 125,000 bridges are rated as structurally deficient (bridges that are restricted to light vehicles only, are closed or require immediate rehabilitation to remain open) — it has been estimated that at least US$90 billion is needed to rectify these problems [2,3]. There is obviously a strong financial incentive that existing bridges be conserved and their remaining service life extended.

Bridge performance can often be expressed in a reliability format, typically as the probability of failure (collapse or serviceability). Information about present and anticipated bridge reliabilities, in conjunction with decision models, provides a rational and powerful decision-making tool in which possible alternatives may include do nothing, repair, strengthen, replacement, reduce loads, frequency and types of inspections and inspection/testing equipment needed — this becomes an optimisation procedure. Time-dependent reliability and decision-making analyses have been proposed by a number of researchers (e.g. Refs. [4–10]).

For assessment purposes, an updated reliability (after an inspection) may be compared with some minimum acceptable reliability; from this comparison the relative 'safety' of the structure can be ascertained, a load rating assigned or a 'remaining life' estimated. However, uncertainties about the precision of bridge reliabilities means it is often more appropriate to use bridge reliabilities for comparative or relative risk purposes. This may include the prioritisation of risk management measures (risk ranking) for inspection, maintenance, repair or replacement. Moreover, a life-cycle cost or other decision analysis may be used to quantify the expected cost of a decision. Hence, bridge reliabilities may be used for bridge assessment to

1. be compared with reliability-based acceptance criteria such as a target reliability index,
2. determine relative bridge safety or prioritise bridge inspection, maintenance and repair by ranking reliabilities of different bridges, or,

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3. estimate cost-effectiveness of decisions (e.g. maintenance, replacement) using life-cycle costs.

At this point of time, comparing reliabilities with a target reliability index is extremely difficult since determining the target reliability index would require an extensive 'calibration' procedure (e.g. Ref. [11]). The target reliability index is usually based on calculated reliabilities of 'new' bridges (or existing bridges known to have performed well in-service) to provide an 'average' existing level of safety. Alternatively, for an overload permit, the target reliability index may be based on the reliability of the bridge when it is exposed to 'real' traffic. As such, a target reliability index is influenced by material, dimensional and loading probabilistic models (e.g. distribution types); type of structure and structural material; mode of failure; and remaining service life. Hence, target reliability indices vary from country to country and their derivation requires multiple reliability analyses (at least 50, preferably several hundred) — this is obviously an extensive and time-consuming process. However, items two and three are considered more suitable for immediate application and so are described in the present paper.

The present paper will present a broad overview of the concepts, methodology and immediate applications to the risk-based assessment of bridges. For *illustrative purposes* only, two practical applications of risk-based bridge assessment are considered; the case studies relate to:

1. the effect of bridge age, traffic volume and deterioration on the reliability of existing reinforced concrete (RC) bridges, and,
2. influence of durability design specifications on RC longitudinal cracking — given in terms of reliabilities, life-cycle costs and time to inspections.

Finally, the physical infrastructure of a nation consists also of other items in the public domain, such as roads, buildings and various communication links, as well as items in the private domain, such as ships, aircraft, mechanical (mining) equipment and telecommunications facilities. The present paper will focus on bridges, although the approaches described herein are relevant to other physical infrastructure also.

2. Limit states and structural reliability

Bridges and other structural load-resistance systems are substantial and expensive systems that tend to be unique or comprise of 'one-off' elements. Naturally, their reliabilities cannot be directly inferred from observation of failures or other experimental studies. In these circumstances, reliabilities need to be calculated from predictive models and probabilistic methods.

Failure of a structural element occurs when the load effect (S) exceeds the resistance (R). Reliability may then be expressed as a probability of failure (p_f) or 'reliability index' (β), defined as

$$p_f = \Phi(-\beta) = \Pr(R \le S) = \Pr(R - S \le 0) = \Pr(G(R,S) \le 0) = \int_0^\infty F_R(r) f_S(r)\,dr \quad (1)$$

where Φ is the standard normal distribution function, $G(\,)$ is the 'limit state function' [defines 'failure': in the present case this is equal to $R - S$], $f_S(r)$ is the probability density function of the load and $F_R(r)$ is the cumulative probability density function of the resistance, see Fig. 1. Structural reliability theory is easily extended to include the reliability analysis of structural systems. Limit states typically selected for reliability analysis are as follows:

- ultimate limit states — flexural failure, shear failure, collapse; and,
- serviceability limit states — cracking, durability, deflection, vibration.

See textbooks by Stewart and Melchers [12] and Melchers [11] for an introduction to structural reliability and risk assessment methods.

3. Why a risk-based approach?

In practice, uncertainties arise in predicting service or lifetime loads; when using non-exact structural behaviour (prediction) models; in considering the variability in material properties, workmanship, element dimensions, environmental conditions, inspection and test data and maintenance; in predicting fatigue, corrosion or other deterioration processes; and so on. Consequently, decisions related to bridge assessment are based on uncertain or incomplete information.

For obvious reasons deterministic approaches are not efficient for decisions taken under uncertainty since such decisions tend to be overly conservative and based on 'prudent pessimism' or 'worst case' scenarios. Conventional approaches also often fail to address what 'safety' actually is; such as how safe is safe enough? and how to

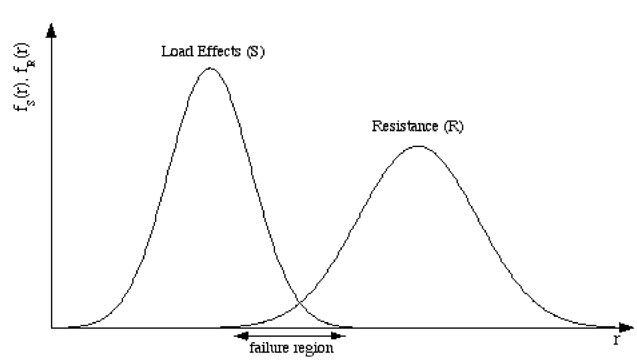

Fig. 1. Distributions of load effects and structural resistance.

measure 'safety' or performance. Risk-based approaches clearly define 'safety' and provide a measure (i.e. risk) by which safety, cost-effectiveness and other bridge management considerations can be measured or compared against. It is for these reasons that the Highways Agency (UK), amongst others, is developing reliability-based management systems for highway structures [13].

4. Risk-based bridge assessment

Generally, bridge assessment is conducted to determine a load rating, overload permit, relative safety for present bridge conditions or current or future inspection, maintenance or repair needs (life-cycle costs or asset management). Long-term predictions of structural performance are inherently inaccurate since it is very difficult to predict with any degree of confidence long-term deterioration processes. Hence, bridge assessments of this type are based on a limited reference period (normally two to five years) and at the end of this reference period, the bridge should normally be re-assessed since after this period traffic conditions and structural capacity are likely to have changed.

Many existing bridges do not satisfy structural requirements as specified for new bridges. However, decisions must often be made regarding the 'safety' of these bridges. For bridge design, it is acceptable to be conservative because the additional material costs are marginal; however, a conservative bridge assessment (i.e. a satisfactory bridge which fails an assessment) will result in unnecessary and costly bridge strengthening or repairs, traffic disruption, etc. There is therefore a need for bridge assessment criteria to be as precise as possible.

4.1. Site-specific data collection and updating of structural reliability

The initial stage of a risk-based bridge assessment is to estimate the reliability of the existing bridge, such a reliability analysis should consider the following uncertainties:

- representation of real structures by idealised prediction models (model errors);
- inherent variability of material properties;
- variability in workmanship, element dimensions and environmental conditions;
- spatial variations in material and other properties;
- collection and analysis of site-specific test data (measurement and calibration errors);
- prior and current variability of service loading; and,
- assessment of current fatigue, corrosion or other deterioration processes.

For bridge assessment, reliabilities should be calculated using updated probabilistic load and resistance models developed from data which are representative of known site characteristics. These are now described.

4.1.1. Updating load models

A complete probabilistic model for traffic loading can readily be developed from on-site collection of traffic load and volume data. However, probabilistic load models can also be developed for specific bridges without the need for large amounts of data and associated detailed statistical analyses. For example, the statistical parameters of single heavy vehicle loads are readily available from the literature. Information about the volume of heavy vehicle traffic can then be used to estimate the probability distribution of the simultaneous presence of overloaded heavy vehicles. For a low traffic volume bridge the probability of the simultaneous presence of overloaded heavy vehicles is thus reduced from that assumed in design load models (e.g. Ref. [14]); resulting in an increased updated reliability. The influence of anticipated increases in traffic volume and loads over the expected life of a bridge can be considered also in a reliability analysis.

4.1.2. Updating resistance models

Information on parameters controlling bridge resistance can be updated by carrying out on-site inspections of the bridge. Inspections and testing can be used to determine:

- material properties (e.g. core testing, the rebound hammer, ultrasonic pulse velocity to estimate on-site compressive strength of concrete),
- element dimensions (e.g. electromagnetic covermeters to locate and measure cover of reinforcing bars), and,
- evidence of defects or deterioration (e.g. impact-echo techniques, half-potential and resistivity measurements to predict the likelihood and rate of corrosion, chloride concentration measurements from core testing).

Predictive probabilistic models for the bridge resistance can then be updated with collected site-specific data using Bayes' theorem (e.g. Ref. [15]), which is a general theorem applicable to any situation in which existing probabilistic knowledge (prior distribution) is updated with new evidence. In such cases, 'existing knowledge' are probability distributions of variables that represent generic bridges that reflect a variety of construction practices, environmental conditions, etc. (i.e. obtained from a large population of structures). Such information is often available direct from the literature. The 'new evidence' is site-specific data collected from the particular bridge of interest. For example, Fig. 2 shows how the number of core tests (n) influences the probability distribution of in situ concrete compressive strength, for a prior distribution obtained from Canadian and US sources [5,16]. The results demonstrate the positive effect of increasing the number of samples from 3 to 10 since this reduces variable uncertainty and will result in increased reliability [15]. The mean value may

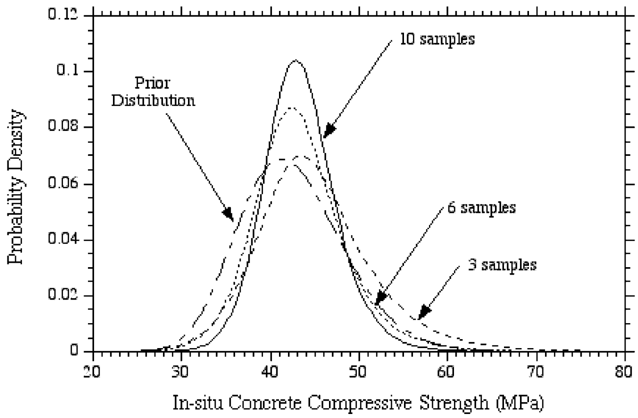

Fig. 2. Influence of sample size on updated distributions of concrete strength (adapted from Ref. [15]).

change also. Clearly, developing prior and updated (predictive) probabilistic models does not necessarily require extensive or expensive data collection exercises.

A number of risk-based procedures for bridge assessment have been proposed (e.g. Ref. [17]). However, it is expected that the risk-based approaches most suitable for immediate implementation are:

- life-cycle costs, and
- risk ranking.

These two decision analyses should be viewed as complementary and are now described.

4.2. Life-cycle costs

A bridge management or assessment decision may be based on a comparison of risks (costs) against benefits. An optimal solution, chosen from multiple options, can then be found by minimising life-cycle costs. Such a decision analysis can be formulated in a number of ways, consider various costs or benefits, and can be referred to also as a whole life costing, cost-benefit or cost-benefit-risk analysis. Life-cycle costs may be used to assess the 'cost-effectiveness' of

- design decisions such as optimal durability requirements (cover, protective coverings),
- construction quality, or,
- inspection, maintenance and repair strategies.

If the benefits of each alternative are the same, then life-cycle costs up to time t_N may be represented as

$$\mathrm{LCC}_k(t_N) = C_\mathrm{I} + C_\mathrm{QA} + \sum_{i=1}^{t_N} \frac{C_\mathrm{IN}(t_i) + C_\mathrm{M}(t_i) + C_\mathrm{R}(t_i) + \sum_{\mathrm{LS}=1}^{M} p_{f_\mathrm{LS}}(t_i) C_{f_\mathrm{LS}}}{(1 + r^{t_i})} \tag{2}$$

where k is the bridge being considered; C_I is the design and construction cost; C_QA is the cost of quality assurance/control; $C_\mathrm{IN}(t)$ is the expected cost of inspections; $C_M(t)$ is expected maintenance costs; $C_\mathrm{R}(t)$ is expected repair costs; M is the number of failure limit states (flexure, shear, cracking); $p_{f_\mathrm{LS}}(t)$ is the annual probability of failure for each limit state (i.e. probability that failure will occur during the ith time interval conditional on updated loads or resistances obtained from most recent inspection, maintenance or repair — see Section 4.1) and C_{f_LS} is the failure cost (damages, loss of life, injury, etc.) associated with the occurrence of each limit state. The failure modes and costs may also be correlated and the expected inspection, maintenance and repair costs should consider the probability of survival at the time of the inspection or other management strategy. Costs and benefits may occur at different times so it is necessary for all costs and benefits (income) to be discounted to a present value using a discount rate r. Note that a high discount rate favours a short service life whereas a low discount rate encourages longer service lives.The above representation of life-cycle costs fails to account for the advantage of designing or maintaining structures to have longer service lives. Alternatively, it may be more meaningful to compare costs on an annual-equivalent basis by distributing life-cycle costs over the lifetime of the structure by an annuity factor — this produces annuity (or annual) costs. For example, the average annuity cost C_A during the design life of the structure (n years) is

$$C_\mathrm{A}(t_n) = \sum_{j=1}^{n} \frac{p_\mathrm{f}(t_j) r [C_\mathrm{I} + C_\mathrm{QA} + C_\mathrm{IN}(t_j) + C_\mathrm{M}(t_j) + C_\mathrm{R}(t_j)]}{1 - (1 + r)^{-t_j}}$$

$$p_\mathrm{f}(t_n) = 1 - \sum_{j=1}^{n-1} p_\mathrm{f}(t_j) \tag{3}$$

where C_I, C_QA, C_IN, C_M and C_R are all present value costs, r is the discount rate and $p_\mathrm{f}(t_j)$ is the probability that the bridge will 'fail' in year j. This approach is particularly powerful when comparing structures with different service lives. Annuity costs more realistically represent economic considerations of higher investment costs (say on durability specification) and associated benefits such as longer service lives.

Life cycle cost formulations may consider other costs and benefits such as traffic delays or reduced travelling time; the efficiency of inspections, maintenance and repair strategies (i.e. probability of 'defect' detection); etc. Evidently, the decision analysis should be subject to a sensitivity analysis to ensure that decisions are not unduly influenced by uncertainties in bridge reliabilities and damage, construction or other costs.

4.3. Risk ranking

Since limited funds are available for bridge inspection, maintenance and repair then a useful and complementary decision-making tool is risk ranking. Such an approach is

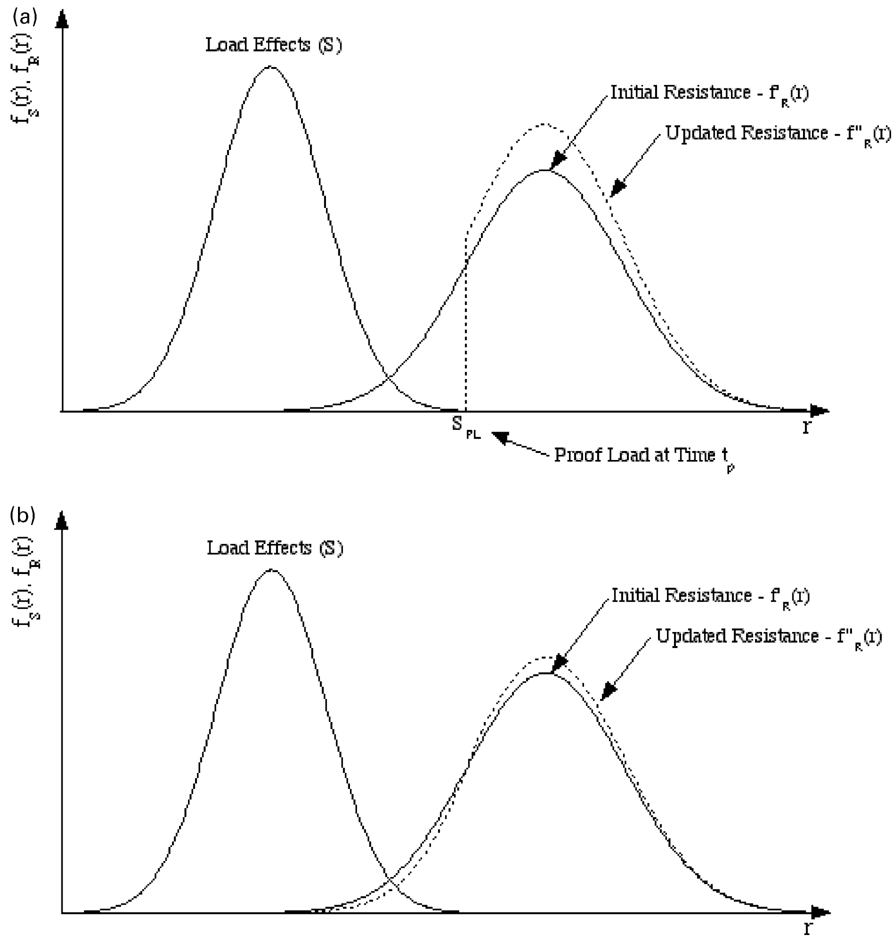

Fig. 3. Effect of (a) Proof load and (b) Prior service loads on structural resistance.

suitable also for assessing the relative safety of bridges. In this approach, reliabilities for existing bridges are ranked from highest to lowest risk and so provides for the prioritisation of maintenance and repair strategies or other risk management measures.

Note that reliabilities are not simply probabilities of collapse. The risk of collapse is generally much lower than risks from element failure, (corrosion-induced) cracking or delamination, spalling or other forms of damage that need maintenance or repair since user delay costs are often much greater than direct repair and maintenance costs. Hence, 'the risk of disruption to functional use is very real' and 'structures management must therefore be aimed at eliminating as far as practicable, not only the risk to life safety, but also the risk of element failures which will result in traffic disruption and other costs' [13]. Hence, if a management decision relates to inspection frequency, it may be more meaningful that reliabilities be calculated for time to corrosion initiation or time to cracking and spalling (e.g. Ref. [9]).

The ranking of bridge reliabilities is appropriate only if the consequences of bridge failure are similar for all bridges considered. However, it is more likely that the consequences of bridge failure will not be similar for all bridges since delay and disruption costs would be higher for high volume highway bridges. As such, a more meaningful risk ranking measure is expected cost of failure, defined as

$$E_{c_k} = \sum_{i=1}^{M} p_{f_i} C_{f_i} \tag{4}$$

where p_{f_i} is the probability of failure for each limit state and C_{f_i} is the failure cost (damages, loss of life, injury, etc.) associated with the occurrence of each limit state. Failure costs may also be discounted to a present value.

5. Illustrative example 1: deterioration and relative bridge safety

For the present study, it is proposed to use a time-dependent reliability analysis to estimate the relative safety of bridges using a risk ranking (expected cost) analysis.

5.1. Time-dependent structural reliability

Current bridge reliability analyses ignore the influence of

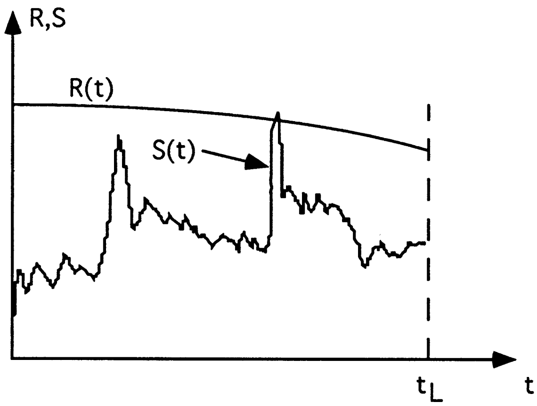

Fig. 4. Typical realisations of load effect $S(t)$ and resistance $R(t)$.

service loads on bridge resistance. This is quite conservative. For bridge assessment, the observation that the bridge has actually survived is in itself a proof load and provides the assessor with some information about its structural capacity and safety (e.g. Ref. [18]). In other words, this enables information about structural resistance to be updated. As such, a revised or updated distribution of structural resistance can be truncated at a known proof load (see Fig. 3a). Service loads may be treated as a proof load with uncertainty (see Fig. 3b) — resulting in the increase of bridge reliability for service proven (older) bridges (i.e. as survival age T increases). A significant increase in bridge reliability occurs even if only normal or design service loads (plus dead load) are used to update bridge reliability; further increases will occur if 'abnormal' events (e.g. grossly overloaded vehicles) are known.

In general, if it is assumed that n load events S_j occur within the time interval $(0, t_L)$ at times t_j $(j = 1, 2, \ldots, n)$, then the cumulative probability of failure of service proven bridges (strength limit state) anytime during this time interval is

$$p_f(t_L) = 1 - \Pr[R(t_1) > S_1 \cap R(t_2) > S_2 \cap \ldots \cap R(t_n) > S_n]$$
$$t_1 < t_2 < \cdots < t_n \leq t_L \qquad (5)$$

where $R(t_1)$ represents the initial distribution of resistance and $R(t_2), R(t_3), \ldots, R(t_n)$ represent the structural resistances at time t_j updated on survival of the previous load events, see Fig. 4. It is evident that the updated structural resistances are influenced by the load history $S_1, S_2, \ldots, S_n$ as well as time-dependent changes in material properties. Thus, the cumulative probability of failure is dependent upon the prior and updated load and resistance histories. Monte-Carlo simulation analysis is used as the computational procedure [19].

The updated probability that a bridge will fail in t subsequent years given that it has survived T years of loads is referred to herein as $p_f(t|T)$ and is expressed as

$$p_f(t|T) = \frac{p_f(T + t) - p_f(T)}{1 - p_f(T)} \qquad (6)$$

where $p_f(T + t)$ and $p_f(T)$ are defined by Eq. (5). This conditional probability may also be referred to as a 'hazard function' or 'hazard rate'. For assessment purposes, it is often more convenient to compare probabilities of failure for a fixed reference period. If a reference period of one year is selected then this is referred to as an 'updated annual probability of failure'.

As is evident from Eqs. (5) and (6) bridge reliability is not constant over the lifetime of the structure. It may decrease with time due to increases in traffic loads and volume, material deterioration (corrosion, fatigue, cracking) and other wear and tear processes. Fig. 5 shows an example of

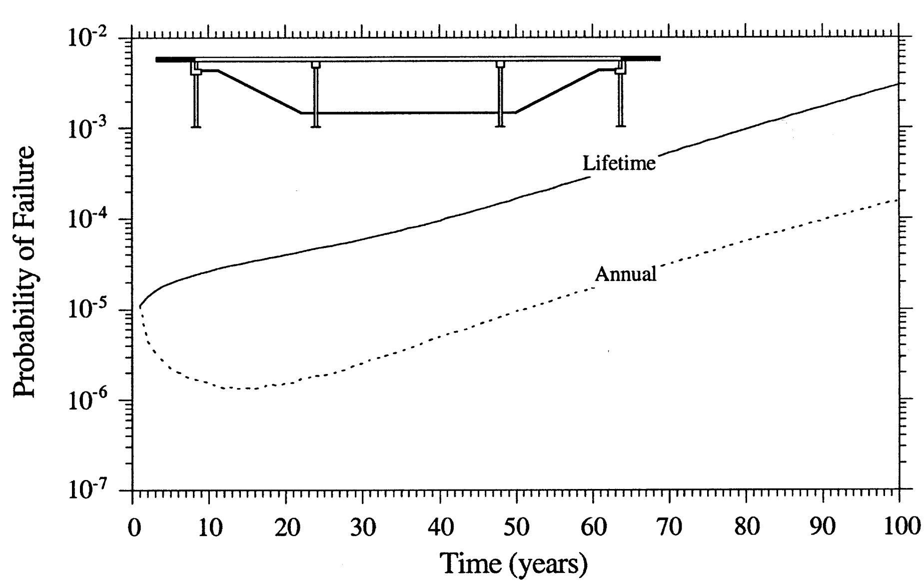

Fig. 5. Annual and cumulative time-dependent probabilities of failure, for a deteriorating bridge [9].

Table 1
Statistical parameters for resistance and loading variables

Parameter	Mean	Coefficient of variation	Distribution
Depth	D_{nom} + 0.8 mm	$\sigma = 3.6$ mm	Normal
C_{bottom} (cover)	C_{bnom} + 8.6 mm	$\sigma = 14.7$ mm	Normal
f'_{cyl}	F'_c + 7.5 MPa	$\sigma = 6$ MPa	Lognormal
k_w (workmanship factor)	0.87	0.06	Normal
f'_{ct} ($f'_c = k_w f'_{cyl}$)	$0.69\sqrt{f'_c}$	0.20	Normal
E_c	$4600\sqrt{f'_c}$	0.12	Normal
E_s	$1.005E_{snom}$	0.033	Normal
f_{sy}	465 Mpa	0.10	Normal
Model error (flexure)	1.01	0.046	Normal
Dead load	1.05G	0.10	Normal
Asphalt load	90 mm	0.25	Normal
Single heavily loaded truck	275.0 kN	0.41	Normal

updated annual and cumulative reliabilities considering a flexure limit state (collapse) for a bridge exposed to de-icing salts. The updated annual probability of failure is the probability of failure in year T, whilst a cumulative (or lifetime) probability of failure is the probability that the structure will have failed anytime up to year T. Note that initially the updated annual probability of failure reduces (due to beneficial effect of service proven performance) but then increases at later ages due to deterioration.

5.2. Results

The following example will help illustrate the reliability-based assessment of existing RC bridges of different ages subject to varying degrees of deterioration. The bridges considered in this example are of the same structural configuration; namely, simply supported RC slab bridges with a span length of 10.7 m and a width of 14.2 m. The bridges were designed according to the AASHTO LRFD Bridge Design Specifications [20] for a HL-93 live load. In the reliability analysis 'failure' is deemed to occur if the bending moment exceeds the structural resistance at mid-span. It has been found that for multiple-lane bridges, the critical load effect occurs when heavily loaded trucks are side-by-side and have fully correlated weights [21]. Statistical parameters for dimensions, material properties and loads appropriate for these RC bridges are given in Table 1.

The statistical parameters shown in Table 1 were developed from Australian, US and Canadian sources. These distributions are essentially prior (generic) distributions obtained from the literature — these distributions can be updated for a specific bridge by using collected site-specific data and Bayes' Theorem (e.g. see Fig. 3). The effect of low and high deterioration on the time-dependent structural resistance of a RC element is given by Enright and Frangopol [5].

5.2.1. Reliabilities

If the extent of existing deterioration is estimated from a

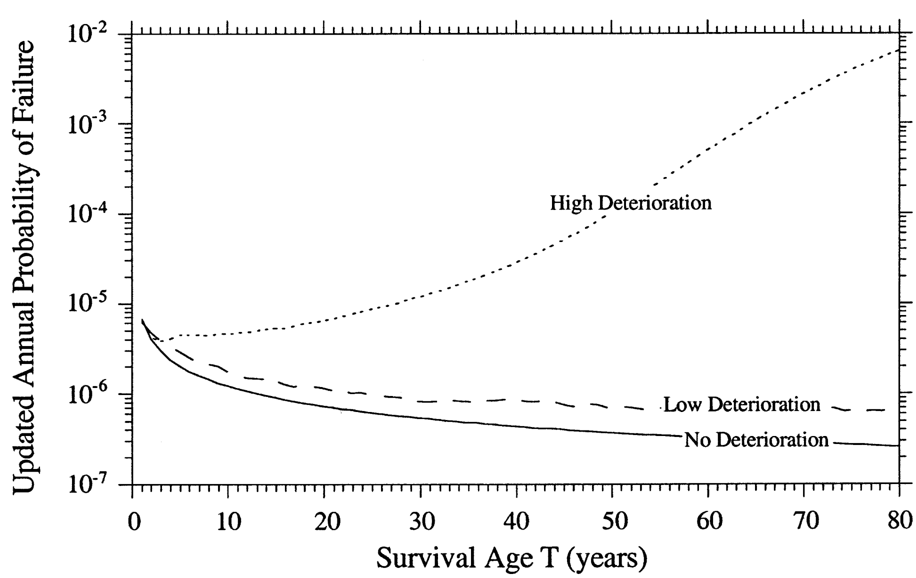

Fig. 6. Comparison of updated annual probabilities of failure [19].

field-based bridge assessment at time T, then Fig. 6 shows the updated (service proven) annual probabilities of failure as a function of survival age for a bridge with no deterioration and two levels of deterioration. The analysis is based on an annual traffic flow of 300,000 vehicles per year and assumes the absence of inspections and repairs; hence, 'failure' will occur whereas in practice signs of structural distress may indicate a need for repair.

Fig. 6 shows that the updated annual probabilities of failure reduce with time for the no deterioration and low deterioration cases. This exhibits the advantage of updating the structural resistance as survival age increases since the reliability will increase for service proven structures — assuming little or no deterioration. In other words, service loads are treated as proof loads with uncertainty; resulting in a truncation of the lower tail of the distribution of structural resistance. For more serious rates of deterioration, it is evident that the detrimental effects of loss of structural capacity outweigh the beneficial influence of considering prior service loads in the estimation of structural reliabilities. More details of this analysis are described by Stewart and Val [19].

5.2.2. Risk ranking and acceptable risk

This example will consider four 'hypothetical' bridges of similar structural configuration (span length of 10.7 m and width of 14.2 m) but of varying age and where inspections reveal evidence of varying degrees of deterioration. A decision needs to be made regarding the relative safety of these bridges.

It is assumed that the load rating is not changed (i.e. no load restrictions) and reliabilities are based on a five year reference period (probability that bridge will fail within the next five years — $p_f(5|T)$). Bridge reliabilities considering extent of deterioration and traffic volume were calculated from the method developed by Stewart and Val [19] for the four bridges, see Table 2.

Table 2 compares the risks for the four bridges. As expected, Bridge 3 exhibiting medium deterioration has a much higher risk of failure. Somewhat surprisingly, Bridge 1 with no deterioration is not the bridge with the lowest risk. Bridges 2 and 4 have lower risks because these bridges are either older (service proven) or subject to lower traffic volume (or loads). As such, risk assessment is not based on a condition assessment alone, but considers these other factors influencing bridge performance.

Table 2
Ranking of bridge reliabilities (risks) for a reference period of five years

Bridge	Age (T years)	Traffic volume (per year)	Deterioration	Risk $p_f(5\|T)$
1	10	300,000	None	7.3×10^{-6}
2	25	300,000	Low	4.6×10^{-6}
3	20	300,000	Medium	3.5×10^{-5}
4	10	150,000	Low	5.8×10^{-6}

An acceptable risk may also be defined. A bridge that shows no evidence of deterioration or construction error and is only 10 years old (Bridge 1) is more likely to be perceived as 'acceptable'. It follows that bridges with lower risks are also acceptable; suggesting that Bridges 2 and 4, although exhibiting minor deterioration, will require no maintenance or repairs during the next five years. This suggests that Bridge 3 is the only bridge whose safety is not acceptable and so the bridge may need repair or strengthening, a reduced load rating, a proof load test, etc. — risks or life-cycle costs may be used to determine optimal maintenance and/or repair strategies.

Finally, a comparison of reliabilities calculated (updated) in five years time may well result in different recommendations since the detrimental effects of deterioration will increase and possibly outweigh the benefits of increased survival age or lower traffic volume.

6. Illustrative example 2: inspections and durability specifications

To date, reliability analyses have concentrated on the loss of flexural strength being the main measure of structural performance. However, serviceability considerations such as cracking and spalling is a more common problem than 'collapse', that in some circumstances constitutes a life-safety hazard since falling concrete has been known to cause death and damage to vehicles. Serviceability limit states such as, longitudinal cracking (coincident cracks following the line of the reinforcement caused by reinforcement corrosion), delamination and spalling of the concrete cover may often be precursors to more critical and dangerous strength limit state problems.

6.1. Longitudinal cracking

The observance of longitudinal cracking or spalling indicates that 'something' needs to be done — such as an assessment of existing safety, repair or rehabilitation, or the need for more frequent inspections to monitor further deterioration. All these cases will require the allocation of additional financial resources. To be sure, serviceability limit states, is a more appropriate criteria when optimising durability requirements or inspection and maintenance schedules.

The serviceability limit state considered in this example is the onset of longitudinal cracking. The probability of longitudinal cracking (p_s) is defined herein as

$$p_s(t) = \Pr(t > T_i + T_{cr1}) \tag{7}$$

where T_i is the time to corrosion initiation and T_{cr1} is the time to longitudinal cracking. The time to longitudinal cracking is influenced by a large range of time-dependent and strength dependent variables as well as time to initiation

[22]. This model is subject to considerable uncertainty, considers only time to first cracking and so may well be over-conservative. An improved corrosion rate model considering the effect of durability specifications on corrosion rate and time-variant corrosion rates is included in the analyses [23]. Work is continuing to better model crack growth and time to spalling and delamination [10].

6.2. Durability design specifications: life-cycle costs

Reliability analyses were conducted to assess the influence of cover and specified concrete compressive strength on cracking failure probabilities. Increasing cover or reducing water–cement ratio (*w/c*) impedes the diffusion of chlorides from de-icing salts, although existing crack width models suggest increasing cover also leads to wider surface transverse cracks which may reduce time to corrosion initiation. Fig. 7 shows the probability of cracking for a RC bridge subject to repeated applications of de-icing salts, again obtained from Monte-Carlo simulation analysis.

With reference to Fig. 7 the baseline (design) case is 50 mm cover and water–cement ratio of 0.4. Clearly, reduced cover and/or increased water–cement ratio, leads to a significant increase in the probability of cracking. If cover is reduced to 25 mm cover (and $w/c = 0.4$) the mean time to cracking is approximately 25 years. However, cracking can occur much earlier; for example, there is a 15% chance that cracking will occur within the first five years of service. On the other hand, for a 50 mm cover there is only a 0.1% of cracking within the first five years of service.

Life-cycle costs may be used to assess the effectiveness of durability design specifications. Costs of quality (C_{QA}) are given in terms of initial construction cost of the baseline case (C_I); namely, it is assumed that $C_{QA} = -0.055C_I$ for 25 mm cover and $w/c = 0.40$ (i.e. lower costs), $C_{QA} = 0.075C_I$ for 50 mm cover and $w/c = 0.35$ (i.e. higher costs), etc. where costs are estimated from the literature. For convenience, inspection, maintenance and repair costs are ignored in this analysis. If it is assumed that service life is ended when the bridge cracks and spalls (severe deterioration) then life-cycle costs (see Eq. (3) using reliabilities given in Fig. 7) based on a 4% discount rate and a design life of 100 years are shown in Fig. 8.

It is evident that reducing cover to 25 mm results in relatively high life-cycle costs. On the contrary, a 75 mm cover and $w/c = 0.45$ or 50 mm cover and $w/c = 0.40$ both appear to produce near minimum life-cycle costs and so may be viewed as optimal durability design specifications. These life-cycle cost calculations are based on relatively simple costing data and a convenient assumption of failure to help illustrate the potential of life-cycle cost analyses only. More detailed and accurate analyses may be conducted that consider inspection and maintenance costs, a more realistic definition of 'failure' and more accurate costing data. Life-cycle costs may also be calculated using Eq. (2) or other risk–cost-benefit formulations.

6.3. Prioritisation of inspection strategies

Information contained in Fig. 7 may be used also to help prioritise bridge inspection strategies. For instance, a bridge with a known 75 mm cover (say measured from a covermeter) will need little, if any, inspections in the first 20 years since Fig. 7 suggests that it is highly unlikely that inspections before this period will reveal any evidence of damage or deterioration. However, a bridge with a known cover of 25 mm will require a different inspection strategy. For this bridge, inspections should be conducted more frequently (say every 2–5 years) since during this period there is a relatively high probability that damage or deterioration will be detected.

The influence of inspection data on updating of cracking

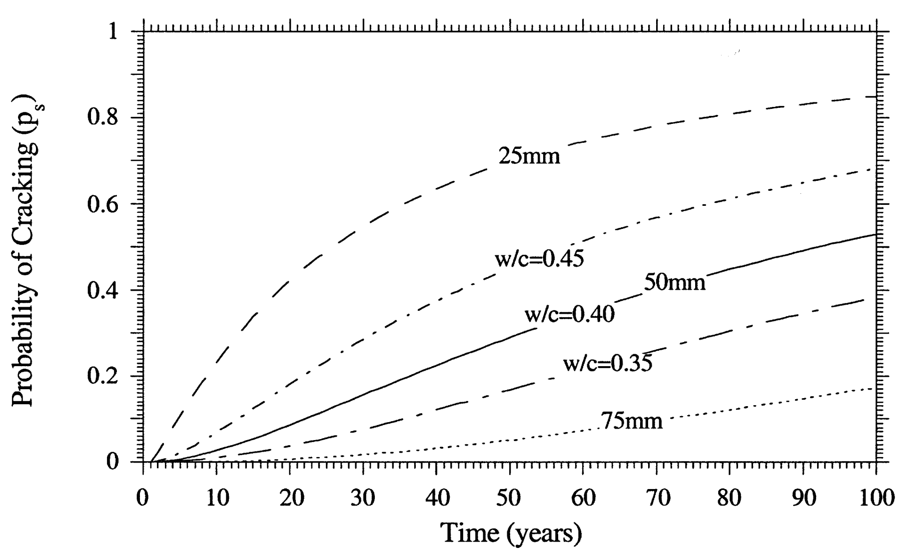

Fig. 7. Influence of cover and concrete compressive strength on probability of cracking.

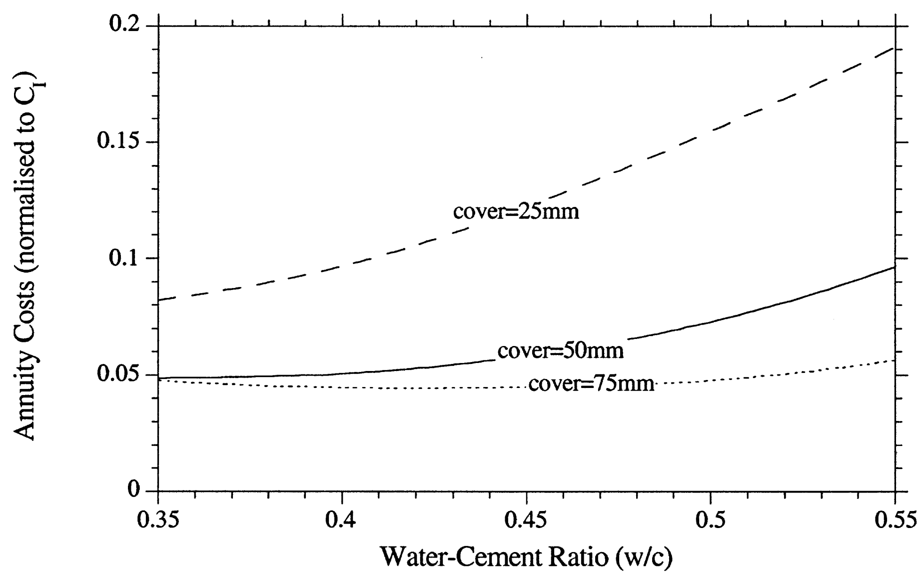

Fig. 8. Annuity costs for different covers and water-cement ratios.

probabilities is illustrated by the following scenarios. An acid-soluble chloride content analysis is conducted from cores taken from the upper surface of bridge decks when the bridges are say 25 years old and show no signs of cracking. The measured chloride contents are 0.2 (no corrosion), 0.6 and 1.0 kg/m^3 (corrosion likely). It is assumed that there is no test or measurement uncertainty. The critical threshold chloride concentration tends to lie within the range of 0.6–1.2 kg/m^3. Fig. 9 shows the conditional failure probabilities for these scenarios and a scenario assuming no updating (using information available at time of construction only), for the baseline case of 50 mm cover and $w/c = 0.4$. In this example only the probability of cracking of the upper deck is considered. If the measured chloride content at 25 years is 0.2 kg/m^3 (i.e. $y(25) = 0.2$ kg/m^3) then the updated probabilities of cracking are significantly lower than that predicted when the bridge was new. For example, there is only a 0.01% chance of cracking within the next 15 years of service. However, if the measured chloride content is 0.6 or 1.0 kg/m^3 then these high chloride contents imply that

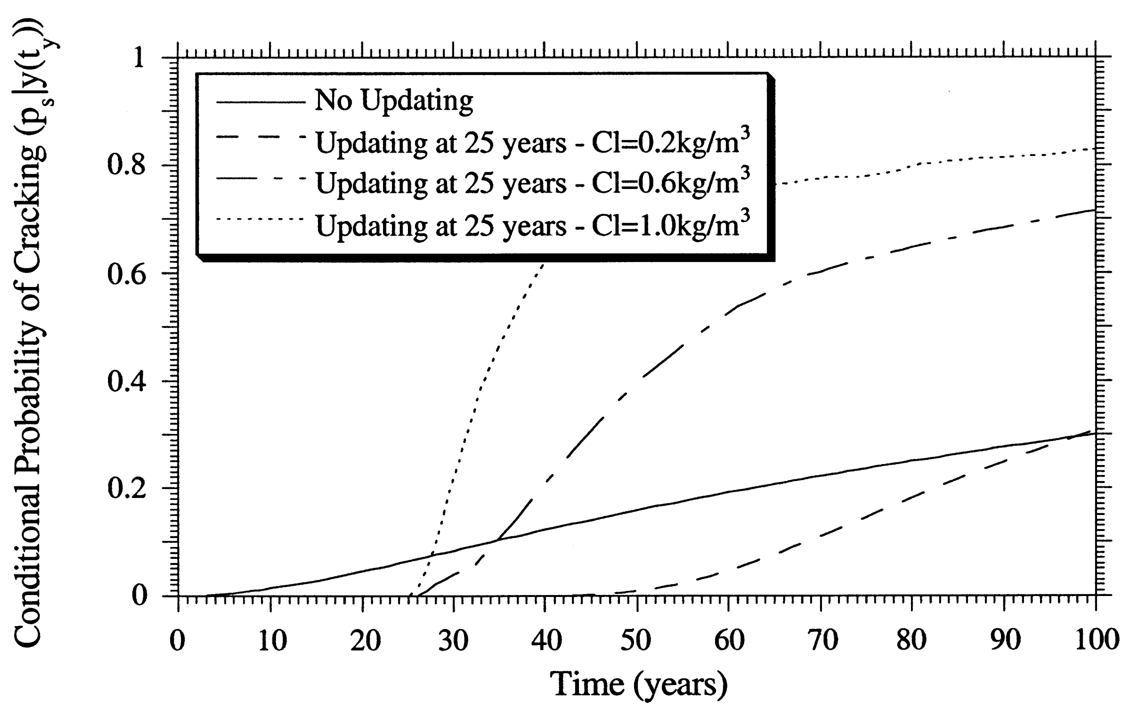

Fig. 9. Influence of inspection data updating (at 25 years) on probability of cracking.

corrosion is likely to have initiated and so it is not surprising that the updated probabilities of cracking are much higher than that predicted when the bridge was new. Clearly, inspections would be needed more frequently in order to monitor the earlier expected onset of cracking. The approach described here (and perhaps then coupled to a life-cycle costs analysis) may thus be used to help optimise inspection strategies by focusing inspection resources on those bridges most likely to experience damage or deterioration.

7. Conclusions

Risk-based approaches to bridge assessment for present conditions provides a meaningful measure of bridge performance that can be used for prioritisation of risk management measures (risk ranking) for maintenance, repair or replacement. Also, a life-cycle cost or other decision analysis may be used to quantify the expected cost of a decision. The present paper presented a broad overview of the concepts, methodology, immediate applications and the potential of risk-based assessment of bridges. Two applications of risk-based bridge assessment were considered for illustrative purposes.

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