STRUCTURAL SAFETY

Structural Safety 23 (2001) 397–405

www.elsevier.com/locate/strusafe

# Reliability-based bridge assessment using risk-ranking decision analysis

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**Abstract**

Information about present and anticipated bridge reliabilities can be used in conjunction with decision models to provide a rational decision-making tool for the assessment of bridges and other structural systems. The present paper presents a broad overview of reliability-based assessment methods and will then focus on decision-making applications using updated time-dependent estimates of bridge reliabilities considering a risk-ranking decision analysis. A practical application of reliability-based safety assessment is illustrated herein which relates the effects of bridge age, current and future (increasing) traffic volume and loads, and deterioration on the reliability and safety of ageing RC bridges. 

*Keywords:* Reliability; Bridges; Structures; Asset management; Risk; Decision analysis

## 1. Introduction

Time-dependent reliability analyses such as those by Mori and Ellingwood [1], Thoft-Christensen [2], Enright and Frangopol [3], Stewart and Rosowsky [4] and others can be used as decision-making tools, or provide additional information on which to base inspection, maintenance and repair strategies. Risk-cost-benefit and other probabilistic decision-making tools are increasingly being used for structural safety assessment as well as optimising inspection, maintenance and repair strategies [5–8]. However, the present paper will focus on structural safety assessment where bridge reliabilities may be used to:

- develop load/resistance/partial factors for use in reliability-based assessment codes,
- compare with reliability-based acceptance criteria such as a target reliability index,

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PII: S0167-4730(02)00010-3

- estimate cost-effectiveness of decisions using life-cycle costs, or
- determine relative bridge safety by ranking reliabilities of different bridges.

These assessment procedures will be reviewed and discussed. However, given uncertainties about the precision of bridge reliabilities, particularly when predicting the long-term effects of deterioration, and what constitutes "safety" it is often more appropriate to use bridge reliabilities for comparative or risk-ranking purposes. The present paper will focus on a decision-making application considering risk-ranking decision analysis. For illustrative purposes, a practical application of reliability-based safety assessment is considered which relates the effects of bridge age, current and future (increasing) traffic volume and loads, and deterioration on the reliability of ageing RC bridges.

## 2. Review of reliability-based bridge assessments

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. Bridge assessments of this type are based on a limited reference period (normally 2–5 years) and at the end of this period the bridge should normally be re-assessed since traffic conditions and structural capacity are likely to have changed.

### *2.1. Reliability-based assessment codes*

An approach to assessment based on the use of structural codes looks like an attractive option because of its simplicity and familiarity to practicing engineers. Unfortunately, current structural codes, which have been explicitly developed for new design, are not applicable to assessment of existing bridges because of significant differences between design and assessment situations [9]. First, there is the different nature of uncertainties associated with design and assessment. In design, uncertainties arise from the prediction a priori of load and resistance parameters of a new bridge. In assessment an existing bridge can be inspected/tested so that these parameters can be measured on-site. However, other uncertainties arise from on-site inspection/testing, including the possibility of undetected damage in bridge elements caused by deterioration. Hence, the load/resistance/partial factors introduced in current structural codes to cover uncertainties associated with design need to be re-assessed to reflect correctly different uncertainties associated with assessment. Second, satisfactory past performance of an existing bridge reduces uncertainty about its physical properties, especially the possibility of gross errors in its design/construction, since the bridge has been subject to and survived prior service loads. And third, there are different economic consequences of overconservative design and assessment. Overconservative design usually results in a marginal increase in the bridge cost, while overconservative assessment may result in unnecessary and costly repairs or replacement.

As far as the authors are aware, no reliability-based codes for bridge assessment are currently available. However, provisions for bridge assessment can be found in bridge design codes [10], or have been issued separately as guidelines and recommendations [11]. Work is now in progress to develop reliability-based (or probabilistic) code type documents for structural assessment [12,13], which can be extended for bridges.

It is desirable that assessment codes be compatible with current design codes such as the load and resistance factor design (LRFD) format adopted in Australia, New Zealand and North America and the partial factor format used in Europe. Generally, values of the safety factors are determined by calibration. For design codes the calibration procedure is well established and its essential steps have been described in several books and papers [14]. Calibration of assessment codes can follow similar steps; though there are important differences between these two procedures. These are described below.

First, assessment usually involves inspection/testing of an existing bridge that provides new information about its basic random variables (i.e. loads and resistance parameters which govern the performance of the bridge). Using this new information (including its uncertainty) statistical properties of the basic random variables are then updated. This can be done using a Bayesian statistical approach. Depending on the source of the information (e.g., an on-site inspection, proof load testing or past performance of a bridge) different formulae to evaluate updated distribution functions of the basic random variables can be derived [15]. Updated distribution functions (often referred to as "predictive") are more complex than those normally used in calibration of design codes (e.g. normal, lognormal, Gumbel) and depend on a number of parameters related to prior information and inspection/testing data such as (e.g.) the number of tests, level of proof load, etc. Thus, prior information and inspection/testing data may result in very different predictive distribution functions. This raises the possibility of inconsistency in the calibration process.

Second, the target reliability index ($\beta_T$) for existing bridges may differ from that used in design (see following section). An approach to the selection of $\beta_T$ employed in the calibration of design codes is based on the use of existing codes or practices. In this approach, reliability indices are first calculated for typical design situations (often referred to as "calibration points") for bridges designed according to existing codes. A weighted-average of the calculated indices is then selected as the target reliability index for a new code. It may be difficult to use the same approach for assessment, since many assessment situations are currently not covered by codes and assessment practices are often inconsistent.

Finally, as it has been noted above, the cost issue is much more important for assessment situations than for design. In this context code-based assessment will hardly be efficient, since a limited number of safety factors have to cover all possible assessment situations; hence, in many situations the values of the safety factors will be too conservative, particularly for non-standard bridges. And while a conservative design does not result in a significant increase in structural cost, a conservative assessment may result in unnecessary and costly repairs or replacement.

### *2.2. Target reliabilities*

It is an extremely attractive proposition that a reliability-based safety assessment can "pass" or "fail" a structure based on comparing a structural reliability with a target reliability index ($\beta_T$). The difficulty arises mainly in selecting $\beta_T$. For design, the target reliability indices have been selected from an extensive "calibration" procedure [14], which tends to consider elastic behaviour and single structural elements or components. In such cases $\beta_T$ is typically in the range of 3.5–4.5. The target reliability index is increased if the consequences of failure are high, the structure is non-redundant or the failure mode is non-ductile. For design, reliability indices calculated for calibration purposes are ''notional'' because reliabilities are calculated using relatively simple structural models, statistical parameters represent high variability associated with populations of structures and human error is not considered.

The target reliability index for assessment may be influenced by consequences of failure, reference period (eg. time between assessments), remaining service life, relative cost of safety (upgrading) measures, importance of structure and so on. For example, ''tentative' target reliability indices have been derived from cost optimisation procedures that consider the relative cost of safety measures and the consequences of failure [16]. In this case, the target reliability indices for a one year reference period vary from 3.1 (large relative cost of safety measure, minor consequences of failure) to 4.7. (small relative cost of safety measure, large consequences of failure). It is noted in JCSS [16] that for existing structures the cost of achieving a high reliability level is higher than for design and so the target reliability index for assessment should be lower than that used in design. There are a number of other important issues to be confronted when selecting and using target reliability indices; some of these are now discussed.

Since structural assessment is structure specific, the reliability analysis should consider site-specific material, dimensional and loading variables. The reliability analysis is based on the ''as-built'' structure, and assuming that sufficient site sampling is conducted, most design or construction errors will be detected and their effect included in the reliability analysis. Loading is more controlled so user errors are unlikely. This suggests that these updated reliabilities better represent reality and so target reliability indices more closely linked to acceptable societal risks may be more appropriate.

Clearly, the use of a target reliability index is only appropriate if the calculated structural reliability and calibrated target reliability index are obtained from similar limit states and probability models. The following case study will show the influence of limit state selection on structural reliability. In an assessment of a concrete and masonry arch bridge, Casas [17] used linear-elastic behaviour models to calculate a reliability index of 0.24 for a critical cross-section over the piers. By considering a nonlinear model and failure criteria based on system behaviour the calculated reliability index increased to 10.4. This increase in reliability indices is considerable. Casas [17] then concluded that the bridge is ''sufficiently reliable''. In this assessment a target reliability index was not defined, although it was implied that it must be somewhat below 10.4. However, if new designs were recalibrated considering nonlinear behaviour and system effects then reliability indices (and hence target reliability indices) would be much higher than 3.5–4.5 currently used for design.

### 2.3. *Optimisation of life-cycle costs*

Probabilistic time-dependent analyses can be used to evaluate and optimise life-cycle costs associated with bridges. This is often referred to as risk-cost-benefit analysis or whole-life costing. By incorporating risk information into the cost-benefit analysis, a bridge management or assessment decision can be made on the basis of a comparison or risks (which can include cost information such as cost of failure) against benefits. The optimal solution, chosen from multiple options, can then be found by minimising life-cycle costs.

Life-cycle costs may be represented as cumulative costs over time or on an annual-equivalent basis by distributing life-cycle costs over the lifetime of the system by an annuity factor—this produces annuity (or annual) costs. A number of other measures may be used to measure life-cycle cost; these include net benefits, net savings, benefit-to-cost ratio, savings to investment ratio, etc. [18]. There is no internationally used method and no measure of life-cycle costs is suitable for all applications.

For bridge assessment, life-cycle cost analyses can be performed considering different construction procedures (and materials), inspection strategies, maintenance strategies, and repair methods. Stewart [19] and Stewart and Val [15] present a number of illustrative examples in which time-dependent reliability analysis is used in probabilistic risk-cost-benefit analysis of concrete bridges subject to deterioration, for collapse and serviceability (severe cracking and spalling) limit states. In another study [8], the effect of repair strategies on life-cycle costs is examined assuming the bounding cases that, once spalling is detected and repaired, (i) spalling will not re-occur during the remaining life of the structure, and (ii) spalling may re-occur during the remaining life of the structure. Life-cycle costing can also be used to evaluate the effectiveness of:

- Design decisions (e.g. durability requirements- cover, protective coverings);
- Construction materials, procedures and quality;
- Inspection, maintenance and repair strategies.

### 2.4. Risk ranking

Consideration of the relative risk associated with a range of inspection, maintenance and/or repair options can provide valuable information, particularly since decisions about specific bridge management activities are often made under the constraint of limited funds. Risk-ranking can be used to evaluate various alternatives by comparing their relative risks (taking into account deterioration rates, relative frequency of overload, costs of failure, costs and efficiency of repair strategies, etc.). The focus of much of the most recent work in this area has been on the probability of corrosion initiation and/or corrosion effects (cracking, spalling, delamination) rather than probabilities of collapse [4]. Risk-ranking is appropriate only if the consequences of failure are similar for all bridges considered. Since delay and disruption costs associated with bridge repairs vary depending on traffic volume, a more meaningful measure is the expected cost of failure during the time period $[t_1,t_2]$:

$$E_c(t) = \sum_{i=1}^{M} \int_{t_1}^{t_2} \frac{f_T(t) C_F}{(1+r)^t} \mathrm{d}t \tag{1}$$

where $r$ is the discount rate, $f_T(t)$ is the probability density function of time to failure $t$, and $C_F$ is the failure cost associated with the occurrence of each limit state $i$. The precise definition of the term $f_T(t)$ is dependent on the context and scope of the decision analysis since in some circumstances it may be more appropriate to replace $f_T(t)$ with a conditional probability (such as a "hazard function"). The failure costs can include both direct and indirect costs.

Since the expected performance (point-in-time probability of failure, expected residual life, etc.) of a bridge is assumed to vary with time, results from a risk-ranking procedure cannot be viewed as stationary. Recommendations based on a comparison of relative risks may well change in time (e.g., as the bridge inventory ages and as management strategies are implemented). Stewart [19] suggests, for example, that predictions of the effect of deterioration processes on bridge performance can only be viewed as being accurate for periods of 5–10 years. For that reason, it is suggested that risk-ranking only be performed for reference periods of this length or shorter.

Both optimal life-cycle costing and risk-ranking offer significant improvements over the more deterministic approaches forming the basis for many traditional bridge management systems. By coupling mechanics-based deterioration models with statistical models of loads and material properties in a probabilistic time-dependent analysis, important information about expected performance and relative risk, both as functions of time, can be obtained. This information can be used to make informed decisions about the inspection, maintenance and repair of the existing bridge inventory as well as about the design and construction of new bridges.

## 3. Illustrative example: deterioration and relative bridge safety

The following example will help illustrate the reliability-based safety 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 [20] Bridge Design Specifications for an HL-93 live load. Reinforcing steel is 400 MPa, specified concrete strength ($F'_c$) is 30 MPa, specified bottom cover is 25 mm and there is a 75 mm thick bituminous overlay. The bridge design required a 550 mm thick slab with a reinforcement ratio of 0.8%.

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 effect of deterioration on the time-dependent structural resistance of a RC element is given by Enright and Frangopol [3].

### 3.1. Time-dependent structural reliability

Stewart and Val [22] have described the calculation of an updated (or conditional) probability that a bridge will fail in $t$ subsequent years given that it has survived $T$ years of loads$-p_f(t|T)$. This conditional probability may also be referred to as a "hazard function" or "hazard rate". For a reference period of one year this is an "updated annual probability of failure". For example, if the extent of existing deterioration is estimated from a field-based bridge assessment at time $T$, then Fig. 1 shows the updated annual probabilities of failure as a function of survival age. This analysis includes the time-dependent effects of deterioration and resistance updating based on past satisfactory performance and assumes the absence of inspections and repairs. Fig. 1 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.

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) | $C_{b,nom} + 8.6$ 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 |
| $E_s$ | 1.005 $E_{s,nom}$ | 0.033 | Normal |
| $f_{sy}$ | 465 MPa | 0.10 | Normal |
| Model error (flexure) | 1.01 | 0.046 | Normal |
| Dead load | 1.05 G | 0.10 | Normal |
| Asphalt load | 90 mm | 0.25 | Normal |
| Single heavily loaded truck | 275.0 kN | 0.41 | Normal |

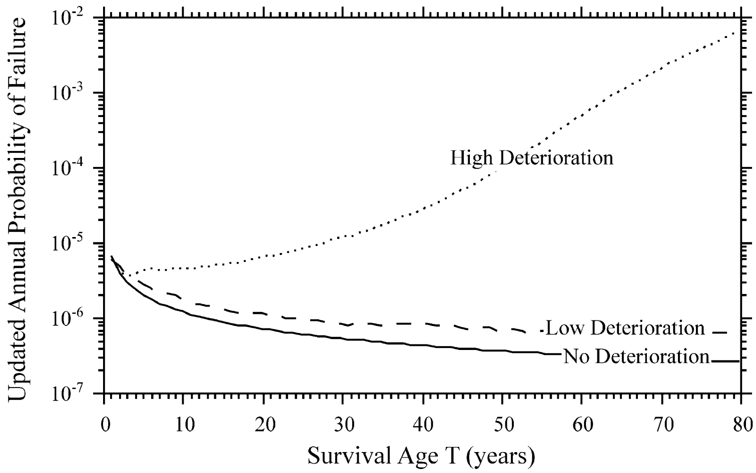


Fig. 1. Comparison of updated annual probabilities of failure.

### 3.2. Risk ranking

This example will consider four “hypothetical” bridges of similar structural configuration but of varying age and where inspections reveal evidence of varying degrees of deterioration. If the consequences of collapse for all bridges are all similar then a ranking of reliabilities is the same as a ranking of expected costs. It is assumed that the load rating is not changed (i.e. no load restrictions) and reliabilities are based on a 5 year reference period [probability that bridge will fail within the next 5 years—$p_f(5|T)$]. Bridge reliabilities for the four bridges are compared in Table 2. Somewhat surprisingly, Bridge 1 with no deterioration is not the bridge with the lowest

Table 2
Bridge reliabilities for a reference period of 5 years

| Bridge | Age ($T$ years) | Traffic volume (per year) | Deterioration | Risk $p_f(5\|T)$ |
|---|---|---|---|---|
| 1 | 10 | 300,000 | None | $7.3\times10^{-6}$ |
| 2 | 25 | 300,000 | Low | $4.6\times10^{-6}$ |
| 3 | 20 | 300,000 | Medium | $3.5\times10^{-5}$ |
| 4 | 10 | 150,000 | Low | $5.8\times10^{-6}$ |

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.

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 repairs or upgrading during the next 5 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.—life-cycle costing can be used to determine optimal maintenance and/or repair strategies. Finally, a comparison of reliabilities calculated (updated) in another 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.

It should be recognised that the calculated reliabilities may be sensitive to modelling assumptions. For example, changing concrete compressive strength from a normal to a lognormal distribution changes the probability of failure for a typical RC bridge [23] from $2\times10^{-14}$ to $4\times10^{-20}$, The known effect of tail sensitivity on calculated structural reliabilities means that such sensitivities are considerably higher at higher reliabilities. This observation helps demonstrate a significant difficulty in using risk-based criteria for decision making since absolute values of risk may be sensitive to modelling assumptions and so are inherently uncertain. Hence, it may be more appropriate to use bridge reliabilities for comparative or relative risk purposes. This also raises the question how risks of $10^{-10}$, $10^{-15}$ or $10^{-20}$ be interpreted for decision-making since such values are perhaps too low to be meaningful.

## 4. Conclusions

Risk-based approaches to bridge safety assessment for present conditions provides a meaningful measure of bridge performance that can be used for prioritisation of risk management measures for maintenance, repair or replacement. The present paper presented a broad overview of the concepts, methodology, immediate applications and the potential of risk-based safety assessment of bridges. An application of risk-ranking was considered for illustrative purposes where it was shown that risk assessment should not be based on a condition assessment alone.

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