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Publication date:
1997

Document Version
Publisher's PDF, also known as Version of record

Citation for published version (APA):
Middleton, C. R., & Thoft-Christensen, P. (1997). *Assessment of the Reliability of Concrete Bridges*. Dept. of Building Technology and Structural Engineering. Structural Reliability Theory Vol. R9756 No. 179

INSTITUTTET FOR BYGNINGSTEKNIK
DEPT. OF BUILDING TECHNOLOGY AND STRUCTURAL ENGINEERING
AALBORG UNIVERSITET • AAU • AALBORG • DANMARK

STRUCTURAL RELIABILITY THEORY
PAPER NO. 179

Proceedings of ESReDA Seminar on Industrial Application of Structural Reliability Theory, Paris, October 1997, pp. 21.33

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ASSESSMENT OF THE RELIABILITY OF CONCRETE BRIDGES

DECEMBER 1997 ISSN 1395-7953 R9756

Printed at Aalborg University

Assessment of the Reliability of Concrete Bridges

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1. Introduction

Although there has been a considerable amount of research into different aspects of concrete bridge reliability, it has still not been widely adopted in professional practice other than in the development and calibration of codes. This situation appears to be changing as there has been a significant shift in emphasis for highway authorities around the world away from the design of new structures over to maintaining the existing infrastructure. As a result, bridge owners are seeking improved ways of inspecting, assessing, maintaining and repairing their existing stock of bridges in the wake of ever increasing traffic loads and volumes, and an ageing population of bridges subject to various mechanisms of deterioration. Their goal is to optimise the allocation of limited resources whilst maintaining their bridges in a safe and serviceable condition. Reliability analysis is one tool being adopted to assist in achieving this goal.

Rather than review the specific research on this subject this paper examines a number of key issues related to the practical application of reliability analysis to the assessment of concrete bridges.

2. Background - The Need for Bridge Assessment Programmes

Throughout the world many highway authorities are initiating major programmes aimed at assessing the strength and condition of their bridge stock. In 1987 the Department of Transport in the U.K. launched a 15-year bridge rehabilitation programme whose aim is to strengthen and safeguard the integrity of all motorway and trunk road bridges in Britain by Ist January 1999.

There are two primary reasons why such programs are needed. Firstly, over the last 50 years legal load limits for lorries have continually been increasing. In the U.K. the current maximum vehicle load of 38 tonne is to be increased to 40 tonne and the maximum axle load increased from 10.5 tonne to 11.5 tonne from 1999. This small increase of two tonnes in gross-vehicle weight will, in itself, have relatively little effect on the number of bridges that would be considered safe. However it was realised that an inventory of bridge load capacities was required for the whole country because large numbers of the current stock of bridges were designed and built for much lower live loads than even the current 3 8 tonne limit.

It must also be remembered that it is almost inevitable that lorry loads will follow past trends and rise again in the future. The transport and trucking industry is continually lobbying governments to further increase this load. Since 1994 the British Government has allowed 6-axle articulated vehicles and drawbar-trailer combinations to operate at 44 tonnes when carrying loads to or from rail terminals (although the drive axle limit remains at 10.5 tonnes). Belgium, Denmark, Finland, Italy and Luxembourg all allow 6-axle vehicles of 44 tonnes or more on their roads. France allows axle loads of up to 13 tonnes and in the Netherlands 50 tonne vehicles are permitted (Department of Transport 1996)! Thus an effective means of evaluating the consequences of increased live loading on the safety of the bridge population is needed as strength assessment will be an ongoing part of all bridge owning organisations management strategy in the future.

The second major concern is that many bridges have deteriorated significantly and it was recognised that the management of the bridge stock would require knowledge of the overall condition of the population of bridges. In a study by a firm of consultants (Department of Transport 1996), a random sample of 200 concrete bridges was examined to evaluate their performance and maintenance requirements. Deterioration was identified in 72% of these bridges. Such a high figure raised concerns about the number of deteriorated structures in the population as a whole and the consequences for bridge safety.

There are around 160,000 bridges in the U.K. so this assessment programme is indeed a major task. The total estimated cost of assessing, strengthening and upgrading these bridges is estimated to be over £4 billion (National Audit Office 1996) (Leadbeater 1996).

By now many thousands of bridges have been assessed under this programme. Although the majority of structures have been found to be satisfactory, large numbers of bridges have "failed" their assessments. Up to the end of April 1996, 94% of the motorway and trunk road bridges in England had been assessed with 20% failing to meet the required standards. Local authorities, who maintain the majority of Britain's bridges, report an even higher percentage of bridges "failing" assessment.

These assessments have almost without exception been performed using deterministic methods of analysis. With such a massive problem facing the bridge owning authorities it is vital that engineers evaluate these current methods of assessment and seek to refine and extend these so that the most realistic and relevant methods of analysis are used.

3. Application of Reliability Analysis in Bridge Engineering

In the US, partial load and material factors in the LRFD bridge *design* code have been derived using reliability analysis (Nowak 1993). In Canada, it has been used extensively in the development of their bridge evaluation code (Buckland and Bartlett December 1992).

In the U.K. some code calibration work was undertaken when limit states codes were first introduced although this was aimed at optimising partial safety factors in codes for the *design of* new bridges rather than the *assessment* of existing bridges. The partial factors used in the limit state code for the design of steel bridges, BS5400:Part 3, were derived using reliability analysis (Flint & Neill Partnership 1980). This work was based on a methodology for calibration given in a major review of structural reliability undertaken by CIRIA in 1977 (CIRIA 1977). Subsequently an attempt was made to calibrate the partial safety factors in the concrete design code, BS5400:Part 4. This study highlighted a number of conceptual and theoretical anomalies with the calibration procedure for limit state codes, particularly with regards to prestressed concrete bridges (Ove Arup & Partners 1988).

By way of direct application in bridge design, most recent large-scale bridge design projects around the world, such as the Northumberland Straits bridge, the Second Seven Crossing and the Storebaelt bridge, have incorporated reliability analysis as an integral part of the design procedure (Buckland et al. 1996).

Another particularly important area of application of reliability analysis has been in the development of traffic load models for use in both the design and assessment of bridges (Merzenich 1994). There has also been a considerable amount of research into the use of reliability analysis in modelling deterioration and developing optimal strategies for maintenance (Thoft-Christensen and Hansen 1996).

The Highways Agency in the U.K. has lead the world in the development of separate codes of practice specifically aimed at the *assessment* of existing bridges rather than the *design* of new bridges. In the case of concrete bridges, the assessment code, BD44/95 (Department of Transport 1995), introduced a number of significant improvements. These include the adoption of *worst credible* rather than *characteristic* strength parameters, modified shear strengths and also provision for reinforcement details that are not considered to conform to current design standards but are often found in older concrete bridges. These developments have enabled engineers to undertake more realistic evaluations of structural capacity.

In addition, the Highways Agency identified the potential for applying reliability based methods for assessing existing bridges and has instigated a number of research projects to develop such methods (Das 1997). One of these projects aimed at developing bridge specific live-load models for short span bridges has already resulted in a new code for assessment loading. In many situations this results in a significant decrease in the magnitude of the live load used for assessment without compromising the level of safety associated with these structures. This new code also differentiates load intensity on the basis of traffic volume and road surface condition (Cooper 1996).

Two other projects are aimed at the development of reliability based codes of practice for the structural analysis of both steel and concrete bridges, particularly with regards to whole life performance and the long term effects of deterioration on safety (Thoft-Christensen et al. 1996).

4. The Reliability of Concrete Bridges

In this paper, some of the issues specifically related to the reliability analysis of concrete bridges and the methodology developed in the Highways Agency project for assessing the whole life performance of concrete bridges are considered. The majority of concrete bridges on the highways of Britain are of relatively short span (less than 20 m in length) and made from in-situ reinforced concrete (rather than prestressed concrete) and thus these are the bridges of most interest.

The goal was to develop a methodology with which the strength and safety of a typical short-span concrete bridge deck could be assessed realistically, taking into account the age of the structure and different levels of deterioration.

4.1 Failure Criterion

Limit States

Several key questions need to be asked. When designing a new bridge all modem design codes define a number of ultimate and serviceability limit states that should not be violated. However *what failure criterion* should be adopted for assessing the adequacy of an existing concrete bridge? In the U.K. the fundamental philosophy adopted for assessment has been to evaluate only the *ultimate limit state* as the criterion for passing or failing an existing structure. According to the code, serviceability criteria need not be checked although many bridge owners still request such checks be made. The argument given is that an existing structure that has been in-service for some time is likely to have already exhibited evidence of serviceability problems and these should have been dealt with in maintenance programs.

However some argue that serviceability limit states should be the key failure criteria. In particular, excessive cracking may lead to corrosion and/or spalling of the deck with subsequent accelerated deterioration and increased likelihood of collapse. In addition spalling of a deck slab may necessitate closure of a bridge even though there is no real risk of collapse. There is some merit in this argument although the prediction of cracking in concrete structures is a notoriously difficult and unreliable art. This is highlighted by the fact that it is possible to get predictions of crack widths for the identical structure varying by a factor of four depending on which code crack width formula is adopted. Many modem concrete design codes, such as the Australian bridge code (Austroads 1992), are in fact removing crack width formula altogether because of the scepticism over their validity and instead using detailing rules, such as a limit on the allowable spacing of reinforcing bars, to deal with crack width problems.

More recently a number of researchers have been examining fatigue failure in reinforced concrete structures and claim this is a key issue in determining the reliability of concrete bridge deck strength. Fatigue failure is effectively an ultimate limit state as it can lead to premature weakening and rupture of an affected region. Although experimental evidence indicates that this may be a problem for both the steel reinforcement and the concrete itself, the design and assessment codes in the U.K. only require that the effect on steel be considered. In the case of concrete fatigue, which may result in cracking in the deck it is very difficult to differentiate between cracks caused by fatigue and cracks caused by other possible problems in a concrete bridge deck.

It seems reasonable to argue that the highest priority for bridge owners should be to minimise the likelihood of catastrophic failure or collapse of their structures to avoid the potential loss of life, replacement costs and disruption that would inevitably ensue. Thus the methods of structural analysis employed need to be able to realistically predict this ultimate capacity. For reinforced concrete, ultimate capacity is usually verified by separate checks on *the flexural and shear* capacity of the structure.

4.2 Analysis Methods

So what methods of analysis should be adopted for determining whether or not these ultimate limit states are violated? Conventional bridge assessments are based, almost without exception, on deterministic linear elastic analysis using, for example, a grillage or finite element approach. However for many concrete bridge decks, and in particular concrete slabs, this approach will usually result in a very conservative estimate of the ultimate capacity.

In reality, concrete structures will crack under heavy loads when the strength of a region of the deck slab is exceeded. The slab stiffness will change and, provided sufficient ductility is available, loads will be redistributed elsewhere in the slab. As a result, a linear elastic analysis will not accurately model the distribution of stresses or the *actual* behaviour in the post-elastic range where non-linear effects dominate. The failure criterion for elastic methods is based on a single local failure of an element within the structure rather than global collapse which actually defines the load carrying capacity of the structure as a whole. The consequences of such an elastic 'flexural' failure are likely to be small and it may only result in some localised cracking which affects the serviceability of the structure. If one accepts that serviceability criteria do not govern and collapse is the criterion on which to base the assessment, it would seem that an elastic failure criterion is inappropriate or at least very conservative at predicting the collapse load.

There is a wealth of evidence from experiments and full-scale load tests to show that concrete bridges are often able to carry loads well in excess of the "theoretical" capacity calculated using elastic analysis techniques. This is particular evident when the bearings supporting the superstructure provide sufficient restraint for compressive membrane action to develop within the deck.

So what analytical alternatives are available to the assessing engineer? The only practical options would involve undertaking a more sophisticated analysis of the ultimate strength using either yield-line analysis or non-linear finite element methods although both of these also have their limitations.

Non-Linear Finite Element Analysis (NLFE)
Although NLFE methods have developed to a sophisticated state, their applicability is severely limited by their high cost in computing time and the advanced level of expertise required to use them. In addition, the technique is load-history dependent and very sensitive to the choice of material parameters. As a result, it is more suited to in-depth, specialised assessments of major structures or for laboratory research, and is not presently a practical option for use in assessing large numbers of existing bridges. This situation could well change in the future as computing developments continuously result in decreasing costs and greater speed with NLFE programs, although the sensitivity of results and specialised expertise required will still limit their application. The complexity of these programs also makes them impractical to incorporate within a reliability format and, although some researchers have adopted this approach, it is not currently seen as a practical solution for widespread application in bridge assessment.

Yield-Line Analysis (Plastic Collapse Analysis)

Up until the development of non-linear finite element methods in the 1970's, researchers investigating the ultimate strength of concrete slabs almost exclusively adopted this approach as the only available theoretical method for predicting *flexural* strength. By utilising the full distributed strength capacity of a structure, yield-line methods are usually less conservative than elastic methods. Some of the disadvantages of this approach are that, as an upper-bound method, there is always a degree of uncertainty that the critical failure mode has been found. A further concern is whether or not the bridge has sufficient *ductility* to justify the assumptions inherent in yield-line theory. In addition, the method only evaluates *flexural strength*. *Shear* capacity must still be checked using the conventional elastic approach since further research is required to validate the use of plastic collapse analysis for shear failure (although some very promising research has recently been completed at Cambridge in this area (Ibell et al. 1997».

In a pilot study on whole life performance of concrete bridges undertaken for the Highways Agency (Thoft-Christensen et al. 1996), a recently developed plastic collapse analysis method based on yield-line theory was adopted for evaluating the flexural load capacity of concrete bridges (Middleton 1993). This approach has been shown to model concrete slab bridges extremely well and overcomes the major difficulty faced in all structural reliability problems of finding a realistic method of analysis that can still be incorporated into a reliability format. For assessing the probability of shear failure, a conventional elastic finite element approach was adopted and incorporated into a reliability program.

It is clear that the choice of failure criterion and analysis method adopted will influence the bridge reliabilities derived. If failure is defined by element failure, derived using elastic analysis, a relatively high probability of failure will be obtained. If, on the other hand, global (or systems) failure involving collapse of the entire structure is used to define failure and the predictions are based on yield-line theory, significantly lower failure probabilities will usually be found, provided the structure is capable of some plastic load redistribution. For ranking of priorities it does not necessarily matter which approach is adopted provided the criterion used in the assessment is clearly stated and used consistently. However for optimal use of resources it is clearly important to adopt the most realistic method of analysis when deciding on whether or not a bridge reaches an acceptable level of safety so that bridges are not unnecessarily strengthened or repaired.

4.3 Acceptable Level of Safety

The next key question is how can the probabilistic calculation of risk of failure of a bridge be related to an actual level of risk acceptable to the public. This is perhaps the most difficult question in any reliability study and one which must be faced if structural reliability analysis is to be adopted in practical bridge assessment. Studies instigated by the Highways Agency examined this question in detail although the matter is still being debated (Menzies 1996). One must determine a suitable value for the acceptable probability of failure of a bridge and then see how this relates to the probability of failure determined using reliability theory. This represents a significant step forward in the application of reliability analysis to bridge engineering and a major shift in philosophy for bridge assessment codes.

Traditionally reliability methods have been used in three main areas of concrete bridge engineering. These are (i) for ranking bridges in *relative* order of risk of failure (ii) for investigating *the sensitivity* of a structure to variability in parameters such as material strength or applied loading and (iii) for *calibration* of code partial factors. To extend beyond this and use reliability analysis to define whether a bridge passes or fails a prescribed safety level requires very careful and extensive calibration.

It must be remembered that structural reliability analysis is purely a method of quantifying the *susceptibility* of a bridge to failure due to *variability* in a selection of loading and strength parameters. It does not include provision for *all* the possible causes of bridge failure. The reliability obtained is often referred to as a "notional reliability" for this reason. In practice, most bridge failures have not been due to overload or deficient material strengths, which can be modelled by reliability analysis, but due to catastrophes such as ship collisions, natural disasters or human error. Examples of these include the Tasman Bridge collapse in 1975, various pier collapses due to scouring after floods, and the Westgate Bridge collapse.

However this need not necessarily be an obstacle to the use of reliability analysis in bridge assessment. There is an implicit level of safety in existing deterministic codes which has proven to be acceptable over a long period of time. By evaluating the reliability of a sample of these structures an estimate of this "acceptable" probability of failure can be determined. The actual value itself is not particularly significant. This methodology has been adopted in the pilot study on whole life performance undertaken for the Highways Agency. This involved a probabilistic evaluation of 15 concrete bridges which were all deemed to be satisfactory using deterministic assessment techniques. These results are being used to define a reference line of reliability profiles for *ideal or acceptable* structures to be used in a proposed new assessment code (Das 1997).

The final goal is to develop a risk based assessment procedure for concrete bridges in which satisfactory structures will be defined in terms of a certain (low) probability of failure. By considering the risk of failure at different load levels a simplified assessment code suitable for general use by the profession might then be derived. Clearly such a procedure will need extensive calibration before being adopted. The next stage, which is currently underway, is to widen this calibration to include other types of concrete structures and different levels and configurations of loading.

4.4 Selection of Stochastic Variables

The choice of *stochastic or basic variables* (b.v.'s) for determining the resistance of concrete bridges is relatively straightforward. However various researchers have adopted quite different statistical models and methodologies for incorporating these parameters into a reliability analysis.

Table I lists typical basic strength and material variables adopted for the concrete bridges examined in the current Highways Agency project.

Table 1: Stochastic variables for determining the resistance of concrete bridges

No.	Name	Description	Unit	Mean	Standard deviation	Distribution type
1	fy	Steel yield strength	Mpa	289	25	Log-normal
2	fcu	Concrete cube strength	Mpa	30	6	Log-normal
3	t	Slab thickness	mm	550	10	Normal
4	c	Cover to main reinforcement	mm	60	8	Normal
5	U	Model uncertainty factor	-	1.0	0.05	Normal

A number of issues warrant discussion here. Firstly many researchers have lumped these parameters together and use a single factor, R, with its own mean, standard deviation and distribution type to represent the moment and shear capacities (or *resistances)* of a concrete member. This factor would be derived from a large number of simulations in which random combinations of each of the primary variables listed above are chosen and entered into a code formula to derive a value of moment or shear resistance. In particular, it seems this approach is commonly used in the US. In Europe it is more common to find that the basic variables are individually assigned statistical properties such as those listed above and the resistance calculated as a multi-variable function rather than as a single stochastic variable. This approach has been favoured because most assessing engineers will be familiar with the values adopted for individual parameters whereas they are unlikely to have a feel for the range of values that might be derived with a lumped resistance factor. In addition, the actual values of both mean and standard deviation for these parameters are likely to change from bridge to bridge and this is difficult to accommodate using a lumped resistance parameter without undertaking a substantial computational exercise to derive a new value in each case.

Dominant Basic Vvariable

In assessing the flexural limit state for concrete bridges, experience shows that the *yield strength* and *area of the main reinforcement* are by far the most dominant basic variables. The next most important parameters are typically *applied load* (for short span bridges where dead load does not govern), followed by the member *thickness,* which influences the lever arm of the concrete section (and also the dead load). For shear failure, *concrete strength* becomes a much more important variable.

The other variable which can have a significant influence is the so-called *model uncertainty factor, U.* This is intended to allow for uncertainty in the structural analysis strength model being employed. Unfortunately there is very little data available on which to base the statistical properties of this variable. In addition there are situations in which the influence of this variable can totally swamp that of all the other stochastic variables. As a result some researchers argue that it should not be included in the evaluation of concrete bridge reliability.

4.5 Sensitivity of the Reliability Index

It is also important to consider the sensitivity of the reliability index to the choice of basic variables. To investigate this, six separate reliability analyses of the same short span concrete bridge were undertaken, with a minor change being made to some aspect of the choice of basic variables in each case. The failure criterion was ultimate flexural capacity and the analysis was performed using a probabilistic yield-line program developed at Cambridge University (Middleton 1994).

The changes made in each of the six analyses are listed below.

1. . Fourteen independent stochastic variables for strength and load were included.

2. Only 10 stochastic variables were included. The four variables with the lowest sensitivity factors (i.e. of least importance) in analysis no. 1 were fixed, leaving 10 stochastic variables.

3. The same 14 variables as analysis no. I were used except the distribution type for each variable was made normal rather than having log-normal distributions for the strength variables and gumbel distribution for the live loading.

4. Again all the same basic variables from the first analysis were used here, except the coefficient of variation of the reinforcement yield strength was increased from 10 to 15% (i.e. an increase in the standard deviation from 26 MPa to 39 MPa).

5. The yield strength of reinforcement was decreased from 260 MPa to 230 MPa.

An uncertainty factor, U, on the resistance model, which had not been included in the first five analyses, was now added. This was assigned a mean value of 1.00 and standard deviation of 0.05 as recommended in CIRIA63 (CIRIA 1977).

The results of this sensitivity analysis are given in the table below.

Table 2. Sensitivity of reliability indeks to changes in basic variables

Analysis No	Reliability Index	Probability of failure	Comments
1	5.87	2.2e-9	14 b.v.'s used
2	6.23	2.3e-10	10 b.v.'s used
3	5.17	1.1e-7	All b.v.'s with normal distribution
4	4.36	6.5e-6	↑ C.o.V. of fy from 10 to15%
5	4.51	3.2e-6	↓ fy from 260 to 230 MPa
6	5.00	2.9e-7	Include uncertainty factor, U.

The assumptions used for each of these six analyses would seem to be quite reasonable on their own, however greatly differing failure probabilities were derived for relatively minor changes in the basic variable parameters.

These results highlight one of the most unsatisfactory elements of reliability analysis, namely the extreme sensitivity to the number and nature of the basic variables selected. The range of values obtained here emphasises the need to qualify any reference to a reliability index with a summary of the assumptions made in deriving that index. It also shows the difficulties that can arise when attempting to compare the safety levels of two different structures obtained using different basic variables.

The indices derived above are somewhat higher than those typically quoted in the literature for "satisfactory" bridge safety. In a paper on the use of reliability analysis in the derivation of the Canadian highway bridge assessment code, Allen (Allen, 1992) states *"the reliability index used to determine design safety factors is generals 3.5, based on a 50-year reference period."* However, without qualifying this figure with the full details of the analysis used to obtain this number, comparison with the results here is extremely difficult. The value of 3.5 is almost certainly based on an elemental failure criterion with the analysis being undertaken using elastic methods. Until a large body of data is available to define these basic variables more accurately, great care must be taken when interpreting the significance of reliability indices. Having said this, provided the *same* basic variables are employed in the analyses and all the assumptions are clearly stated, it is possible to investigate the safety of a variety of bridge and load configurations and failure modes, and derive information on the sensitivity of the structural reliability to variations in the different basic variables. In this way the effects on safety of increased live loading or reduced steel areas due to corrosion can be studied.

It is thus imperative that any comparative analysis adopts a consistent choice of basic variables due to the sensitivity of the reliability index to relatively minor changes in these variables.

4.6 Comparison Between Deterministic Factors of Safety and Reliability Index

In a separate exercise, the ultimate strength of six short span concrete bridges was assessed deterministically. Failure was again defined by ultimate flexural capacity which was assessed using yield-line theory. A separate reliability analysis was performed using the same failure criterion. In the reliability assessment, the total number of stochastic variables varied between the bridges depending on the complexity of the reinforcement, deck slab and loading. For example with bridge no. 6, which was comprised of a slab and three stiff beams, a large number of basic variables were chosen to allow for the many layers of reinforcement in each of the beams and various thicknesses of slab across the deck.

The reliability indices obtained for each bridge are listed in table 3. The six bridges are listed in increasing order of safety based on the calculated reliability index. The ranking obtained using deterministic and probabilistic methods is identical *except* for bridges 3 and 5 which are interchanged between the two assessment methods.

Table 3: Comparison of deterministic and probabilistic ranking of bridge safety

Bridge No.	Factor of safety (Deterministic)	Reliability Index	No.of stochastic variables
1	1.41	3.65	14 b.v.'s*
2	1.42	3.72	22 b.v.'s
3	1.54	4.21	19 b.v.'s
4	1.4.8	5.66	30 b.v.'s
5	1.74	5.87	14 b.v.'s
6	2.50	10.6	44 b.v.'s

*b.v. = basic variable

These results are re-assuring in that the overall ranking of the structures safety was consistent except for the interchange in position of bridge number 3 and 5.

It should be noted here that all six of these bridges had failed the initial deterministic elastic assessment according to the code and were scheduled for strengthening or replacement. For example, bridge no. I was originally assessed to have a load capacity of 7.5 tonnes using this approach. When a deterministic plastic analysis was performed in accordance with the code rules (and including all the required partial safety factors) a factor of safety of 0.94 was obtained corresponding to a load capacity of 38 tonnes. When this same bridge was assessed using yield-line analysis, with all partial factors set to unity, the factor of safety obtained was 1.74. When a reliability analysis was then carried out, also based on a yield-line analysis, the reliability index derived was 5.87.

4.7 Whole Life Performance - the Deterioration of Concrete Bridges

Fundamental to the study on whole-life performance is the requirement to understand and also to be able to model deterioration processes in concrete bridges. There are many possible mechanism of deterioration. Some of these affect the concrete itself (e.g. alkali aggregate reaction). However the most important form of deterioration is corrosion of the reinforcement which subsequently leads to spalling of the concrete cover, loss of bond and decreased strength.

A diffusion model for predicting the initiation time and rate of loss of steel area was developed during a recent project sponsored by the European Union. This has been adopted to model the primary mechanism of deterioration in concrete bridge decks in the Highways Agency study (Thoft-Christensen et al. 1996). It is recognised that this is a simplistic approach to the modelling of corrosion. In particular, it is likely that corrosion will not result in a uniform loss of bar area over the entire deck but will in practice be concentrated in isolated regions, possibly in the form of intense pitting corrosion. However to incorporate such an effect would add a significant level of complexity into the modelling of corrosion. One problem to be faced here is the scarcity of measurements available from bridge deck inspections which could be used to help validate and improve the current corrosion models.

In practice some model must be adopted if an allowance for deterioration is to be included and, until more and better data are available, the uniform corrosion model is probably as good as any other.

Since the goal in this particularly research project for the Highways Agency was to not only evaluate the probability of ultimate failure but also include provision for evaluating different repair strategies, serviceability criteria were also examined even though these are not usually required for assessment. Despite the problems discussed above, crack widths are still checked in the British *design* codes and thus this was chosen as one serviceability limit state. The other adopted was deflection although, unlike building codes, this is rarely a major consideration in short span concrete bridges. It is usually more relevant to prestressed concrete bridge construction where beam cambers under dead and live load must be carefully considered.

5. Conclusions

Several of the important issues that must be faced when assessing the safety of a concrete bridge using reliability analysis have been discussed here. The Highways Agency in Britain has a research project underway which aims to develop a new assessment code in which the whole life performance of concrete bridges is being modelled using probabilistic methods. The various solutions adopted to address the issues raised here are presented and some of the areas in which further development is needed are discussed.

In particular, it is still extremely difficult to accurately model the ultimate strength of concrete bridges due to the complex non-linear behaviour of this non-homogeneous material. For slabs, yield-line analysis is probably the best analysis option available, although it is limited to flexural failure modes and it is also reliant upon sufficient ductility in the slab to allow plastic redistribution of loads. Elastic methods must still be used for shear evaluation.

Clearly the choice of failure criterion is important and will affect the numerical value of the reliability index obtained. More research needs to be undertaken to understand better the sensitivity of the reliability index to the choice of failure function. Results will also be sensitive to the number and statistical properties of the basic variables used in the analysis.

The challenge now is to build up the necessary database to allow the parameters used in this bridge assessment methodology to be refined and improved. The reliability results must also be calibrated against a sufficiently large and varied sample of concrete bridges which have performed adequately in the past so that confidence in this approach can be gained.

This methodology potentially provides a means for rationally evaluating the risk of failure of concrete bridges whilst providing a tool for bridge managers to use in defining priorities for bridge repair, strengthening and replacement.

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