Management of highway structures

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Preface

The 20th century has seen major expansions of the transport networks in many countries of the world accompanied by the construction of most of the bridges that exist today. The majority of the bridges are relatively new, constructed in the last 35 years or so. As such, until recently, the overall maintenance needs for the bridges have been modest. In the early days, up to the 70s, it was sufficient to carry out the work on an *ad hoc* manner, that is as and when considered necessary by the engineers responsible. However, as the bridge stocks have grown collectively older and signs of deterioration have become more and more evident, the maintenance needs have rapidly grown. In the absence of any rational methods for prioritising or planning, backlogs of essential work on bridges deemed to be at risk have built up alarmingly.

The bridge engineering profession has taken positive steps in recent years to develop, initially, inventory and inspection databases and subsequently more comprehensive bridge management systems (BMSs). These are now being implemented in a number of countries and the details have been publicised at various international gatherings.

Following on from these initial works, a new technology is emerging in the interface between bridge engineering and financial planning. The impetus for this has come from the need to prioritise funds with credible justification in the face of competition from multitudes of other urgent socio-economic demands, and from the need to plan future resource demands using tools such as whole life costing and asset valuation.

This book is intended to acknowledge this emerging technology on a formal basis and introduce the latest developments to engineers, accountants and financial decision makers who are, or may in future be, involved in the related activities. The book contains papers written by representatives of bridge authorities and internationally renowned experts covering the important political and socio-economic needs which are relevant to bridge maintenance and the engineering and financial tools and procedures which are being developed either as BMSs or in other ways. Most of the papers are based on the presentations made at the international symposium, *The management of highway structures*, held in London, United Kingdom, in June 1998. The book, however, is not strictly the proceedings of the conference as many of the papers have since been enhanced and new ones added to fill perceived gaps.

Part 1, following an introduction by the Chief Highway Engineer of the Highways Agency, contains a number of papers providing an overview of the needs and objectives of bridge management as seen by various highway authorities and operators from the UK and abroad. In Part 2, a number of bridge management systems and methodologies currently available or under development in many countries are discussed, together with the important socio-economic issues that lie behind these developments.

Parts 3, 4 and 5 contain ten papers which describe the recent procedures and computer systems being developed for the Highways Agency. These cover the procedural standards for whole life assessment, for determining the appropriate interim safety measures for structures assessed to be substandard and for whole life costing (life cycle costing) of highway structures. Also described in these parts are the new segmental inspection and reliability based assessment procedures as well as the computer based maintenance bid assessment and structures database systems.

The recent developments in bridge management methodology exemplified by the papers in this book have been the result of extensive research efforts involving the application of advanced techniques, for example time-dependent reliability analysis, which are still in progress. It is therefore considered appropriate to conclude the book with a number of papers detailing current research projects from different countries relating to various aspects of bridge management.

The administrative arrangements for maintaining bridge stocks differ from authority to authority. Hence, any methodology or computer systems developed for this purpose need to be user-specific in their details. However, the overall methods and the underlying principles are applicable not only to bridge stocks generally but also to the maintenance of any civil infrastructure composed of large groups of similar elements. As such, this book should be of interest to all those who are involved in infrastructure maintenance generally.

The new procedures and tools described in these papers represent probably the first applications anywhere of some of the state-of-the-art developments of today in bridge management methodology. These developments have only been possible through extensive co-operative efforts by research workers and representatives of bridge authorities in many countries, many of whom are contributors to this book. I should like to take this opportunity to thank all the authors for their whole-hearted co-operation in producing this book. Of those that do not feature as authors here, thanks are particularly due to Mr Alan Pickett, Divisional Director (Civil Engineering) of the Highways Agency, whose progressive approach and enthusiasm has been central to the advances made within the Agency in the area of bridge management and to Professor Palle Thoft-Christensen, Head of the Department of Civil Engineering, Aalborg University, Denmark, whose imaginative theoretical guidance has underpinned many of these developments.

Finally, I should like to register my, and the publishers', sincere thanks to Mr Lawrie Haynes, Chief Executive of the Highways Agency, for giving permission to publish the papers relating to the Agency's projects and also for kindly allowing me to act as editor.

Parag C. Das, Project Director Bridge Management, Highways Agency

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Part 1. Overview

Introduction

J. A. Kerman, *Chief Highway Engineer, Highways Agency*

Background

The Highways Agency is responsible for managing the trunk road network in England. That means maintenance and improvement of the existing network of which bridges form an integral part. The importance of the bridges comes not only from their functional presence but also from the substantial resources required for their maintenance.

Since 1988, capital expenditure on the trunk road bridge maintenance programme has been in excess of £1000 million. The main driver for most of this expenditure has been the need to bring up to standard the present stock of bridges, particularly in respect of the European international transport vehicles. The objective is to ensure that all structures carrying trunk roads, and other important routes over trunk roads, can be used *safely* by these heavier lorries from 1 January 1999.

Trunk road bridges have assumed a rather prominent place for the Agency Management Board in recent years while the bridge programme was being investigated in succession by the National Audit Office, the Parliamentary Public Accounts Committee and the Transport Select Committee. This had the effect of focusing the Agency's attention to its structures even more than usual.

Clearly, the 15 year rehabilitation programme for trunk road bridges represents a major endeavour by the Agency aimed at improving the bridge stock so that it continues to perform adequately under the very onerous traffic conditions of today. Local authorities and other bridge owners are also carrying out similar exercises involving their own bridges.

Much has been learnt from this exercise. One clear message is that a comprehensive bridge management system is urgently needed. Many bridges without any sign of distress, for instance, have failed the assessments, so it is important to know whether the work is necessary to ensure public safety, or is being justified purely in terms of whole life cost considerations. This is where clear engineering judgement is required.

The benefits derived from any money spent on management activities are not so clear. The benefits of any remedial or preventative work are very complex. If there appears to be an immediate risk of collapse or failure, obviously, the work would be essential. But, if the benefits lie only in terms of reducing future costs, these are likely to be apparent only in the long term.

The Highways Agency has undertaken a major revision of its current somewhat outdated management database and other procedures through a number of projects involving experts from both here and abroad. The teams are examining the fundamental principles of the current bridge management procedures. The aim is to produce an effective regime which will provide the answers.

The Agency's procedures will affect the activities of not only its own maintenance agents and contractors, but also of other bridge authorities in the UK and elsewhere who may wish to adopt these methods in due course. Hence, it has to be emphasised that any conclusions reached on these matters will need to be based on a wide consensus. It is hoped that this book will significantly contribute to the discussion on these important subjects.

Worldwide issue

As a member of the Executive Committee of the World Road Association (PIARC), the author is only too aware that bridge maintenance is now a matter of great concern for many countries around the world. In recent decades, the growing demands of road transport have meant a rapid expansion of the road networks in almost every country. Large numbers of bridges were built within a relatively short span of time. Many of these structures have since developed major problems within a few years of construction. Maintenance needs have grown rapidly from the 60s to almost unmanageable proportions in many countries.

By the time the authorities woke up to the problem, major backlogs had grown up. On the trunk road network in England, during the current rehabilitation programme, it has been found that 20% of the structures are sub-standard. It is believed that nearly 30% of the existing bridges in the USA are expected to be inadequate. Last year, Lawrie Haynes, the Agency's Chief Executive and the author were shown how the elevated structures of the strategic Hanshin Expressway near Osaka in Japan were in a state of extensive deterioration after only 30 years of use. These situations represent major financial and logistical problems for the authorities concerned.

But solutions have to be found. We must all pull together and develop the technical capability to deal with the problems, whether they are the usual deterioration led problems or new ones affecting concrete such as alkali-silica reaction or Thaumasite. The profession must explore new innovative methods and materials such as the use of fibre reinforced plastics, non-destructive evaluation and smart technology. The Highways Agency is fully committed to encouraging innovation in every sphere of its activity. It spends something like £5 million annually for R&D in the structures area.

Objectives

However, in the world we live in today, mere pursuit of engineering solutions will not be enough; society demands that whatever we do as managers and technologists, other objectives also have to be kept in mind. The first and foremost is, of course, safety. Structures must be kept in a safe state, and if any serious risk is perceived at any time, adequate safety measures have to be put in place immediately.

Next comes customer needs and user perception. The customer requires that traffic disruptions should be kept to the minimum. As far as structures are concerned, this can be a double edged sword — if a failure occurs you get disruption, and if you try to prevent potential failure with repairs, you cause disruption.

The road is there to serve the road user. Keeping roads open for use must, therefore, be a primary objective. Maintenance work has to be designed to minimise any traffic disruption caused by it. Disruptions in traffic flow are not only inconvenient to the road user, but also at slow speeds, pollution from exhaust fumes is at its worst. Minimising the harmful effects on the environment in whatever we do is also therefore an important consideration in how we manage our activities.

The public and the taxpayers expect us to ensure value for money in whatever we spend. In determining value for money, we must look beyond the immediate or short term returns. Whole life performance in terms of future maintenance costs must be taken into account.

One must not forget sustainability. The major backlog of rehabilitation occurring today is a clear indication that we have failed in the past. We must try to plan ahead and warn our masters, in good time, of the funding and logistical requirements of the future. Preventative measures need to be devised so that they are effective and not wasteful of scarce resources. More durable materials and methods are therefore desirable as a matter of principle. At the same time we must not overlook the need for easy replaceability of parts.

Whatever engineers and others plan or implement, all these objectives have to be carefully considered, and the results must reflect a satisfactory balance between them.

New challenge

It is well understood that technically it is a very difficult task to determine the extent to which any engineering work meets these core objectives. It is particularly difficult because long term estimates are not only less reliable, but they are also affected by issues such as the extent to which real capital expenditure can now be compared with, for example, future traffic delay costs.

However, this is a challenge that the bridge engineering community now faces. Engineers have to be aware of the wider implications of their work, not only in the immediate term, but also for the future generations. They must consider, as far as practicable, all the options available and the risks associated with each of them, and recommend the best possible course of action.

The future

Partnership and co-operation is the key to a better future. The Agency is putting partnership alongside innovation to ensure that we really do get the most out of our existing and future roads and bridges.

As the result of the research being carried out around the world, it is now possible to develop much more sophisticated methods for considering risks and options in bridge engineering. The impetus for this work has come from the issues faced by highway authorities everywhere, the main problem being how to prioritise scarce funds among the multitude of urgently needed tasks.

It is very encouraging that the work presented in this book is the result of extensive co-operation by international experts, and is aimed at pulling together state-of the-art knowledge from many countries. It is to be sincerely hoped that such international co-operation continues to flourish.

Finally, it is very creditable that the projects involved are not just academic exercises, but are producing techniques and criteria that benefit the practising engineer, and which will have real impact on the strategy and economics of managing the road networks. The new whole life assessment procedures are already being used for the structures on the Midlands Links and the M4 elevated sections in London, two of the largest bridge maintenance projects on the UK trunk road network. Such innovative practical applications must continue.

The maintenance of the Highways Agency's structures

David Baker, *Highways Agency, UK*

Introduction

The Highways Agency was established as an Executive Agency of the UK Department of Transport on 1 April 1994, with responsibility for delivering an efficient, safe and environmentally acceptable motorway and trunk road network in England. Through efficient traffic management and maintenance, the Highways Agency aims to make best use of the existing 6500 mile network. This network includes some 9500 bridges and 5100 other structures such as retaining walls, culverts and sign gantries. Some 80% of the bridges are made predominantly from concrete, 15% are mainly steel structures and 5% are masonry arches. They range from the Severn Bridge with a 988 metre span to small culverts with a span of less than one metre. The majority of the bridges were built in the last 30 years, but some are over 100 years old.

Bridge rehabilitation programme

In November 1987, the Department of Transport launched a 15 year programme to restore to a good condition motorway and trunk road bridges and other structures (the Bridge Programme). This programme was in response to the rapid growth in the volume of traffic, an increase in permissible vehicle weight, weathering and evidence of durability problems with concrete bridges and other structures caused mainly by de-icing salt.

The Highways Agency has operated the bridge programme alongside their road construction and maintenance programmes. Maintenance Agents, both firms of engineering consultants and local authorities, implement the bridge programme on behalf of the Highways Agency. Their main objectives for motorway and trunk road capital maintenance are

> *to preserve past investment in the road system at optimum whole life cost and with minimum disruption to road traffic, and to ensure that the roads are safe and reliable, offering an acceptable quality of ride (Highways Agency Business Plan, 1995–96).*[1]

More specifically for bridge maintenance, the Highways Agency's objective is *to maintain the structural integrity and safety by preserving and where necessary upgrading bridges and other structures to adequate standards at minimum cost over the life of the structure (Department of Transport's Road Business Plan, 1988–89).*[2]

The Highways Agency develops and issues standards and advice notes which all engineers are expected to follow when designing, strengthening, upgrading and maintaining motorway and trunk road bridges and other structures. The standards cover the specific requirements of the bridge programme, and are used by the Highways Agency to define the nature, quantity and quality of work on the structures. These standards are updated as the Highways Agency's knowledge of the behaviour of bridges and structures increases. The Highways Agency funds a regular cycle of inspections to ensure that structures are kept to the required standard and to identify necessary remedial, upgrading and maintenance work.

In addition to the Highways Agency's bridges there are about 100 000 bridges on the local road network. The majority of these belong to local authorities although some 10% are owned by bodies such as Railtrack and British Waterways. These bridge owners are responsible for the maintenance and upkeep of their own bridges. The Highways Agency's standards and advice notes are widely used by these other owners to ensure the reliability and safety of their bridges.

Origins of the programme

In May 1983, the maximum weight limit for lorries was raised from 32·5 tonnes to 38 tonnes. This, together with an increase in the number of heavy lorries, led the Department of Transport to introduce a new code for assessing the load carrying capacity of structures. In December 1984, the European Council of Ministers adopted a Directive establishing a gross vehicle weight limit of 40 tonnes. The United Kingdom negotiated a derogation from this Directive until 1 January 1999.

Between 1984 and 1986, the Department chaired a working party which carried out a census of publicly owned, non-trunk road, bridges to determine the number of older structures, mainly pre-1922, likely to be affected by the new assessment code. The working party assessed a sample of 550 older short span bridges (under 50 m in length) to find out how many would fail the assessment code. The results indicated that 22% of this type of bridge did not meet the standard laid down in the new code. The Department subsequently estimated that 2000 of their own older, short span motorway and trunk road bridges would need to be assessed, of which 1000 might need strengthening or replacement.

Two further stages to this work were also identified by the Department. The first related to the need to check the load carrying capacity of all reinforced concrete bridges designed prior to 1973, when new design rules for concrete structures were introduced. As a result of this, a further 3000 trunk road bridges were estimated to need assessment, of which it was estimated 1000 could require strengthening or replacement. The second related to potential loading caused by queues of heavy vehicles on long span structures. This added an estimated extra 150 trunk road bridges to the assessment programme. No estimate was made at that stage of how many of these bridges might need strengthening. As the assessment and strengthening sub-programme has developed over the years these early estimates have had to be substantially revised.

During the early 1980s, the Department also had to address a separate problem with bridges built of reinforced or prestressed concrete. The Department expected that these bridges would be durable and require little or no maintenance but, during the late 1970s and early 1980s, they began showing signs of deterioration due mainly to the increased use of de-icing salt. In July 1986, the Department appointed consulting engineers, G. Maunsell and Partners, to carry out a study of their concrete bridges and to forecast possible maintenance requirements. The Maunsell report, *The performance of concrete bridges*, was published in April 1989.[3] The report did not reveal any new problems but showed that known deterioration was more widespread than anticipated. Deterioration was identified in 144 out of 200 randomly sampled bridges. By extrapolating these results, the Department estimated that some 4250 motorway and trunk road bridges might be affected (some of these bridges would also feature in the assessment and strengthening sub-programme).

The programme

The bridge programme comprised three main sub-programmes

- *Assessment and strengthening* aimed to ensure that bridges and other structures could continue to carry safely the current permissible vehicle weight of 38 tonnes and, in the future, 40 tonne vehicles. The initial assessment used mathematical modelling to test the load carrying capacity of the structure. If the structure failed this initial test, then more rigorous analyses and site investigations were undertaken. If a structure still failed, then a range of measures were put in place to safeguard it.
- *Upgrading* aimed to bring specific elements up to current requirements, particularly safety and durability standards. It included waterproofing unprotected bridge decks, replacement of sub-standard parapets and reinforcing piers to withstand greater impact.
- *Steady state maintenance* covered the capital repair and replacement of elements which had deteriorated or been damaged with time and use. It

was additional to routine maintenance work, such as clearing blocked drains on bridges and lubricating bearings, which was funded from the Highways Agency's current expenditure budget. It was envisaged that steady state maintenance work would continue, albeit at a reduced level, beyond the 15 years of the bridge programme.

Maintenance Agents

Although the Highways Agency has overall management responsibility for the bridge programme, the inspections, assessments, strengthenings, upgradings and other works are normally managed by Maintenance Agents. The Highways Agency inherited a long-term arrangement whereby the majority of Maintenance Agents were local authorities operating under agency agreements. However, in order to introduce more competition in a structured way, taking account of the needs of trunk road users and obtaining value for money, the Highways Agency is in the process of adopting arrangements whereby both consultants and local authorities can compete for Agencies for fixed terms. The first tranche of Agents operating under these new arrangements started work on 1 April 1997 and all Agencies will be operating on this basis from 1 April 1999. The number of agencies will have reduced from 91 in 1996 to 24 after 1 April 1999, the size and coverage of the new agencies being more strategically based.

DBFOs

Currently, design build finance and operate companies (DBFOs) have 30 year concessions to improve, maintain, operate and manage some 5% of the network. The infrastructure is required to be in a condition no worse than defined in the contract at the end of the concession. These firms too are required to comply with Highways Agency's standards and advice notes in carrying out their works. Payment is based on 'shadow tolls', except on the Dartford and Severn River Crossings where the concessionaires receive the actual tolls.

Inspections and bids for funds

Currently, General Inspections of structures, which comprise a visual inspection of representative parts, are normally carried out at two yearly intervals, and Principal Inspections, which involve a close examination of all inspectable parts, are normally carried out every six years. In addition, Special Inspections of particular areas or defects causing concern are carried out as necessary. A review of the inspection regime is currently underway aimed at targetting vulnerable features.

In the period up to the end of July each year, Agents are required to enter bids for work in the next financial year to the Agency's national structures database (NATS). The bids would include maintenance or repair work, for which the need has been identified by an inspection, plus assessments, strengthening identified from assessments and upgrading items.

They would be broken down to the relevant part of the structure (e.g. piers, main beams, parapets), work type (e.g. concrete repairs, replacement of sub-standard parapets, strengthening) and priority category (e.g. committed expenditure, assessment, work which would cost more than 10% more in real terms if delayed for a year). Bids are also submitted for inspections and routine maintenance. A new system is currently being trialled which permits up to four different maintenance strategies for each element of a structure to be submitted with estimated costs over a 30 year period.

Reasons for change

Progress with the programme

Principally as a result of a shortfall in funding in the early years, only about a third of the estimated £2·2 billion total cost had been spent in the first half of the programme. An in-depth review of the programme was therefore carried out in 1996–97 as a result of which the following decisions were made

- □ although all structures carrying trunk roads and important routes over trunk roads should be either permanently strengthened or safeguarded to carry 40 tonne vehicles by 1 January 1999, the remainder of the permanent strengthening would be programmed for completion by 1 April 2001
- □ the remaining upgrading work would be programmed for completion by 1 April 2006
- □ steady state maintenance would continue for the foreseeable future at an annual cost of around £120 million.

NAO study

In 1995–96 the National Audit Office (NAO) carried out a study of the Highways Agency's management of the bridge programme for motorways and trunk roads. Specific NAO recommendations were as follows (with subsequent Highways Agency action shown in italics)

- □ On managing the work of Maintenance Agents
  - — take steps to ensure that the frequency of inspections by Maintenance Agents meet the Highways Agency's requirements.

As a result of a review of the existing inspection regime aimed at achieving better value for money, the Agency will be targeting vulnerable features for more frequent inspections whilst increasing inspection intervals for non-vulnerable parts. Also the more stringent requirements in the new agency agreements should help to achieve better compliance on inspection frequency.

— enhance quality control by periodically validating a small sample of inspections to help ensure that Maintenance Agents' inspectors are rating structures in a consistent manner. *Audits of a sample of bridge inspections are now being carried out. Also the Agency's Bridge inspection guide is being reviewed with a view, among other things, to providing advice aimed at achieving more consistent inspection ratings. When the updated guide is available, training courses for inspectors will be set up.*

— monitor the funding of Maintenance Agents' bids to be fully satisfied that urgent safety related work is always undertaken promptly. *A specific category for urgent safety work introduced in the bid system in 1996 will enable a central check that work bid in this category is in fact carried out.*

— identify the reasons for any significant variations in the costs of Maintenance Agents' remedial work on bridges and other structures, both within and between regions, in order to seek to reduce the cost of work where appropriate. *Benchmarking the cost of certain specific work items, risk analysis, value engineering and value management were techniques introduced by the Agency in 1996 with a view to obtaining better value for money. Sample auditing of maintenance works is also under way.*

— introduce a system of technical audits to periodically verify the quality of a small sample of Maintenance Agents' work on bridges and other structures. *Sample audits have now been introduced.*

☐ On management information

— use information on the condition of structures to enhance the allocation of funding. *One of the Agency's basic requirements for the bridge management system which it is developing is the ability to assess the relative value for money of alternative maintenance and repair strategies for every bridge, or type of bridge, taking into account condition and rate of deterioration, and to use this information to indicate relative priorities for funding.*

— establish data validation processes to ensure operational information on structures is accurate and complete. *These will be introduced as part of an update of the Agency's national structures database.*

— obtain details of the management information needed by users and ensure that the computerised database is capable of routinely producing reports which provide this information. *This is planned* *to be part of the update of NATS linked to development of a bridge management system.*

- On measuring performance
  - use a range of performance indicators to determine whether the bridge programme as a whole is being delivered to time cost and quality, including an annual assessment of the change in the condition of the bridges. *Condition indices have now been established.*
  - assess the proportion of work completed to time and budget. *Pending development of the new bridge management system, assessment continues to be carried out locally through Agents' business plans.*
  - assess the annual number of unplanned interventions. *This is planned to be introduced in the new bridge management system.*

Conclusion

Since 1987, the Highways Agency, and the Department of Transport before it, have been operating a comprehensive programme for the rehabilitation of motorway and trunk road structures. Inadequate financial support for that programme, and the results of an in-depth study by the National Audit Office, have led to a thorough review of the way in which these structures are maintained, and the systems required to ensure maximum effectiveness and value for money. These systems are described in detail in subsequent papers.

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The bridge maintenance programme of the United States Federal Highway Administration

Steven B. Chase, *Federal Highway Administration, USA*

Introduction

The United States has one of the largest and most efficient highway networks in the world. There are more than 6·3 million kilometres of roadway and more than 581 000 highway bridges greater than six metres in length on the public road network in the USA. Each year this network provides more than 3·7 trillion vehicle kilometres of service and each day there are more than 3 billion bridge crossings. The efficiency, safety and integrity of this network is essential for the continued economic health of the country. Highway bridges are particularly important elements of the nation's transportation system. To illustrate this point, consider the national highway system (NHS) which was designated in November 1995 as the backbone of the nation's transportation network. It includes the interstate highway system as well as other roads considered important to the nation's economy, defense and mobility. While the 260 000 kilometres on the NHS represent only 4% of all highways, these roads carry almost half of all traffic and the vast majority of all truck traffic. Only 4% of the mileage of the nation's roadways are on the NHS, but over 122 000 bridges, or 20% of the nation's highway bridges are on the NHS. Clearly, the continued safety, reliability, health and serviceability of highway bridges is essential and good management of this critical national asset is essential.

The Federal Highway Administration (FHWA) of the United States Department of Transportation is the federal agency responsible for administration and oversight of federal programmes relating to the highway component of the surface transportation system in the United States. The FHWA's most significant role is the administration of the Federal-aid Highway Program which annually disperses approximately $20 000 000 000 to the States for the planning, design, construction, reconstruction, rehabilitation and improvement of highways and bridges. This programme is funded almost entirely by the Federal-aid Highway Trust Fund which is funded primarily by federal gasoline taxes, currently at 14·5 cents per gallon. This $20 billion includes the $2·8 billion Highway Bridge Replacement and Rehabilitation Program (HBRRP) which is intended to accelerate the replacement or rehabilitation of deficient highway bridges.

This paper presents an overview and summary of the federal highway administration programmes relating to the management of highway structures in the United States emphasising history, funding, current issues, programme administration and strategic research and development.

Background

The Federal Highway Administration (formerly known as the Bureau of Public Roads) celebrated its centennial in 1993. There has been a continuous federal emphasis and support for good roads for over 100 years. However, there was little particular emphasis on highway bridges. That changed on 15 December, 1967 when one of the I-bars in a chain link suspension bridge over the Ohio River experienced a brittle fracture. The resulting collapse resulted in 46 fatalities.

This tragedy, occurring so close to the Christmas holiday, focused the nation's attention on the critical issue of the safety of highway bridges. The FHWA issued the national bridge inspection standards (NBIS) in 1971 which for the first time required that bridges greater than 20 feet in length on the federal-aid system were to be inventoried and, in addition, they were to be inspected by a qualified inspector at least once every two years. This requirement was extended to all bridges on all public roads in 1978. Also in 1978, the United States Congress initiated the Highway Bridge Replacement and Rehabilitation Program which provided significantly increased funding solely for the replacement and rehabilitation of deficient highway bridges. That programme has continued and grown over the years and has just been reauthorised in the new Transportation Equity Act which provided significantly increased funding for federal-aid to the States through to the year 2003. The total funds provided for the HBRRP since its inception in 1979 and the proposed funding under the new legislation are shown in Fig. 1. It should be noted that the funding for the Highway Bridge Replacement and Rehabilitation Program is limited to bridge replacement and rehabilitation of deficient bridges. Other federal funds can also be used to construct new bridges as elements of new highways funded under other federal-aid programmes. In addition, the individual States are required to provide up to 25% of the cost of these projects as a condition for federal-aid. Many States also build bridges using their own funds, without federal-aid. When all sources of funds are considered for bridge construction and reconstruction in the United States at least \$5 000 000 000 per year are spent annually for capital improvements for highway bridges. Although separate reporting is not provided on bridge and non-bridge related administration, operational and maintenance expenditures (AO&M), overall AO&M expenditures for highways in the United States in 1995 was \$51·6 billion. This compares to total capital improvement funds dispersements of \$46·5

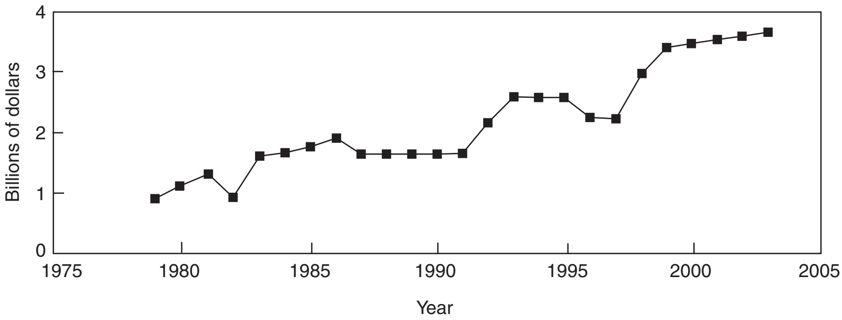

Fig. 1

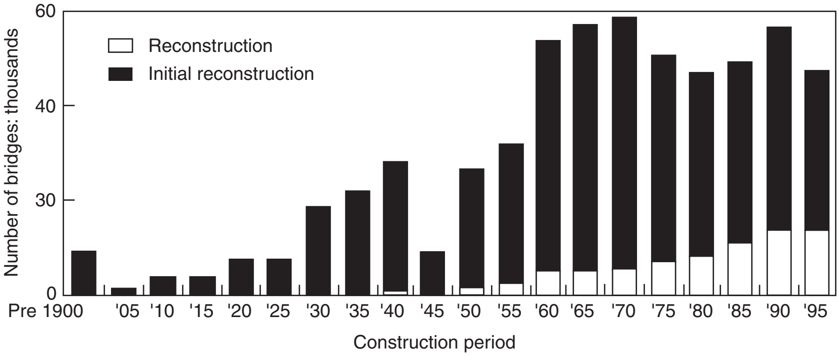

Fig. 2

billion. In other words, 47·4% of all funds expended on highways went towards capital improvements and 51·6% went to AO&M. It is therefore reasonable to estimate that approximately $5 billion are being expended annually for maintenance and operation of highway bridges. It is important to note that with minor exceptions, the Federal-aid Highway Program does not provide funds which are used for maintenance and operation. By and large these functions are the responsibilities of the highway owners. The Federal Highway Administration does not own any highways or bridges in the United States.

The significant federal funding is reflected in the bridge construction activity profile shown in Fig. 2. This figure shows the number of bridges initially constructed or reconstructed over the last 100 years in five year increments. Current annual bridge construction activity is summarised at approximately 7000 new bridges being constructed and approximately 2000 bridges are being reconstructed each year. However, these 9000 bridges represent only 1·5% of the bridge population. It is estimated that approximately 1% or 5900 bridges become deficient each year. The number of deficient bridges in the United States is slowly being reduced. The number of deficient bridges has been reduced from 251 000 in 1982 to 182 726 in 1996. The increased funding projected through 2003 should further reduce these numbers. In fact this is a specific goal in the Federal Highway Administration's strategic plan.

Overview of bridge population and identification of strategic goals

As stated, there are more than 590 000 large highway bridges and culverts on the public roads network in the United States. The number of bridges by date of initial construction and superstructure material are shown in Fig. 3. Several significant features of the American bridge population are captured in this figure. The bridge building boom brought on by the Great Depression in the late 1930s and the initiation of the construction of the interstate system in the 1960s are evident. The average age of the bridges in the United States is 37 years but the majority of bridges have been built in the last 30 years. The predominant material is reinforced concrete but this is skewed somewhat due to the inclusion of large culverts in the population. Over 90% of large culverts are in reinforced concrete. If culverts are excluded, the predominant bridge building material prior to 1970 was structural steel. Today, the predominant bridge building material is prestressed concrete. If culverts are excluded, then the proportions by material type for all bridges are as shown in the upper pie chart in Fig. 4. As can be seen about 40% of all bridges are structural steel, followed by concrete, prestressed concrete, timber and other materials such as masonry, wrought iron and aluminium. Because of the predominance of steel as a bridge building material, especially prior to 1970, the older, deficient bridges are predominantly steel. As shown in the lower pie chart in Fig. 4, roughly 60% of those bridges classified as deficient are constructed from steel, followed somewhat surprisingly by timber, and then concrete,

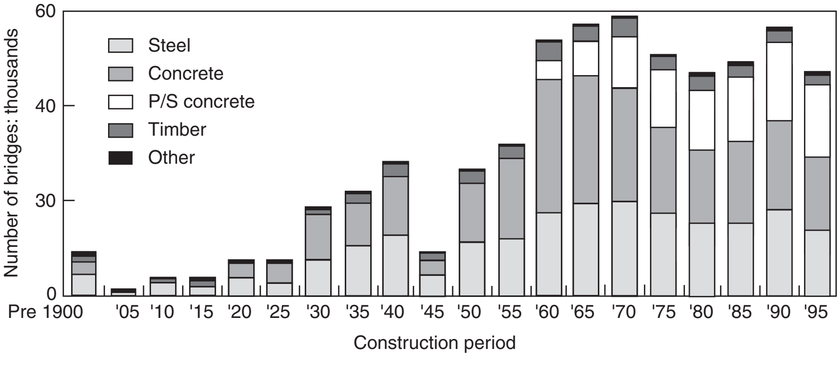

Fig. 3

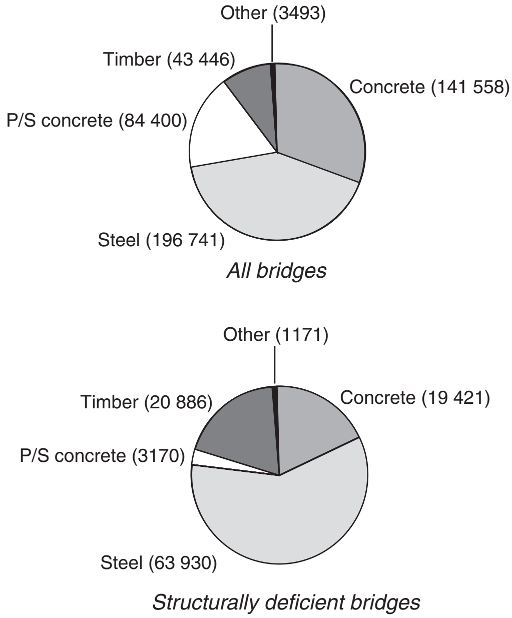

Fig. 4

prestressed concrete and other. The predominance of deficient steel bridges has an influence on the FHWA's strategic plan and goals.

In order for a bridge to be classified as deficient, it must be determined to be either structurally deficient or functionally obsolete. These classifications are determined by the characteristics of the bridge as recorded in the national bridge inventory database. The FHWA maintains a database of records for each bridge greater than 6·1 m in length on public roads. In each record are details on the bridge such as location, owner, material, traffic and, most importantly, condition. Over 100 pieces of information are stored on each bridge. When a bridge is inspected, usually every two years, a condition rating is assigned to elements of the bridge. These ratings are aggregated into an overall condition rating for the superstructure, an overall rating for the substructure, and an overall rating for the deck. These condition ratings range from 0, for an essentially intolerable and dangerous condition, to 9 for a pristine condition. Several appraisal ratings are also calculated for the bridge ranging from 0 (closed) to 9 (excellent). The first, the structural evaluation appraisal, is based upon the bridge's load rating and the average daily traffic. The second, the waterway appraisal, is based upon the frequency the bridge is out of service due to flooding. The third is, deck geometry, based upon curb-to-curb width, number of lanes on the bridge, traffic direction, the functional classification of the highway system the bridge is on and average daily traffic. The fourth, and last, is underclearance. The underclearance appraisal rating is determined by the vertical or horizontal clearances under the bridge, the functional classification of the roadway, the direction of traffic and the presence or absence of a railroad.

A bridge can be either structurally deficient or functionally obsolete, with structural deficiency being cardinal. In order for a bridge to be classified as structurally deficient, it must have a condition rating for the deck, superstructure, or substructure of 4 (poor) or less, a structural evaluation appraisal of 2 (intolerable, high priority for replacement) or less, or a waterway evaluation appraisal of 2 or less. In order for a bridge to be classified as functionally obsolete, the bridge must have a deck geometry evaluation appraisal of 3 (intolerable, high priority for corrective action) or less, an underclearance appraisal rating of 3 or less, a structural evaluation appraisal of 3 or a waterway appraisal rating of 3. Using these criteria, in 1996, there were 182 726 deficient bridges with 101 518 being structurally deficient and 81 208 being functionally obsolete. The single most common reason for structural deficiency is a low load rating, and the single most common reason for functional obsolescence is a bridge that is too narrow.

The importance of highway bridges on the national highway system has been established. Currently, 32 920 or 25·8% of the 127 736 NHS bridges are deficient (9690 are structurally deficient and 23 230 are functionally obsolete). One of the FHWA's strategic goals is to reduce the percentage of deficient bridges on the national highway system to 20% over the next ten years. A second goal is to improve the condition of all bridges so that no more than 25% of all bridges are deficient in ten years. This will require the improvement of over 40 000 bridges in the next decade.

Administration

As noted, the Federal Highway Administration does not own the highway network in the United States. The highway and bridges are owned by the states, counties and local municipalities. Consequently, the Federal-aid Highway Program, including the bridge programme, is administered in partnership with the States. The FHWA appropriates funds to 50 different States, the District of Columbia and Puerto Rico. The FHWA maintains a local Division Office in each of these locations and works very closely with the State Departments of Transportation to administer a number of different federal-aid programmes. The amount of money each State receives is based upon complicated allocation formulas that consider many factors including population, highway network size and classification, amount of federal land, the contribution to the Federal-aid Trust Fund and infrastructure needs. The allocation of Highway Bridge Replacement and Rehabilitation Funds to each State, under the new legislation, is based upon percentages contained in the legislation. The administration of a national bridge programme is very different from the administration of a bridge programme at the State or local level.

The FHWA works with the individual States to ensure that the national bridge inspection standards (NBIS) are complied with and provides technical and administrative assistance to the States. The NBIS prescribe the qualifications of inspectors and requirements for bridge inspections. In general, inspections are to be conducted in accordance with guidance provided in the American Association of State Highway and Transportation Officials (AASHTO) *Manual for bridge maintenance* and the AASHTO *Manual for condition assessment of bridges*.[1,2] The individual States determine which bridges will be replaced or rehabilitated using procedures and policies they have developed which have been approved by the Federal Highway Administration. Each State administers its programme differently.

Each State is free to develop its own procedures, but in the bridge management area the FHWA has worked with a group of States to develop and promote a standardised approach. The FHWA initiative in bridge management began in 1985 with the development of a project which documented different approaches to bridge management that individual States had developed. Through this demonstration project, it was recognised that data collection and prioritisation procedures which were suitable for administration of a bridge programme at the national level, designed for equitable distribution to States, was not necessarily useful for individual States for selecting individual projects or policies. The condition and appraisal ratings which are suitable for national needs assessments generally lack sufficient detail to prove useful for project level and even network level decision support. Specific maintenance actions, improvement needs and costs were not identifiable. An outgrowth of this demonstration project was a project to develop a bridge management system better suited to the needs of the bridge owners. The Pontis Bridge Management System was developed, with FHWA funding and technical support under Demonstration Project 71. The FHWA co-operated with the California, Minnesota, North Carolina, Tennessee, Vermont and Washington Departments of Transportation to award a contract to Cambridge Systematics and Optima Corporation for the design and development of Pontis.

Pontis was designed to be a network level bridge management system. It utilises dynamic integer programming optimisation methods, probabilistic condition state deterioration models, and a detailed bridge database to predict bridge maintenance and improvement needs, recommends optimal policies, and schedule projects within budget and policy constraints. It was intended to better meet the needs of State Departments of Transportation. Pontis has been fully developed and is now owned by the American Association of State and Highway Transportation Officials which is currently licencing the software to 40 State Departments of Transportation.

Most States are using or are planning to use Pontis but several States continue to use systems that they have developed on their own which they feel are adequate for their needs. Despite the widespread adoption and use of Pontis in the United States there are still areas where additional work and improvements are needed. These are the topics of current research and development.

Strategic research and development

The Federal Highway Administration is currently focusing structures related research and development activity into three broad areas. The first is bridge asset management which deals with improvements for bridge inspection, nondestructive evaluation and bridge management systems. The second is high performance materials, which currently encompasses research in high performance steel, high performance concrete, fibre reinforced polymers and aluminium. The third area, named engineering applications, is something of a catch-all grouping for high priority research activities which do not fall under the two previous focus areas. These activities include improved design methods and technology, seismic issues, corrosion, hydraulics and geotechnology.

The objective of the bridge asset management focus area is to improve the quality of the nation's bridges through the effective use of improved technology. Some specific projects which are underway or will be initiated include

- incorporate element level inspection and condition state assessment, and load and resistance factor concepts into the AASHTO condition evaluation manual
- research and evaluate methods to utilise element level inspection data as the basis for national needs assessments, performance measurement and funds allocation procedures
- research improved rapid, quantitative deck inspection technologies
- research improved fatigue crack detection and evaluation technologies
- research improved methods for rapid and quantitative load rating of highway bridges
- research improved methods for the evaluation of unknown foundations
- construct and operate a validation centre for highway application of nondestructive evaluation technologies
- sponsor an international conference on nondestructive evaluation technologies for highways
- conduct probability of detection study for visual inspection on highway bridges
- research improved methods for bridge management decision support systems including optimisation methods, database design, data modelling and improved spatial data modelling

- develop improved life cycle cost methodologies
- sponsor an international conference on bridge management.

The objective of the high performance materials focus area is to promote and advance more durable and stronger materials in bridge construction and rehabilitation to extend the service life of highway bridges. Some specific projects which are underway or will be initiated are to

- initiate and provide technical assistance for high performance materials test and evaluation projects in multiple States
- conduct large scale material characterisation study of high performance steel
- update and improve material specifications for all high performance materials
- develop a design guide and workshop for fibre reinforced polymer (FRP) materials
- develop a database on FRP projects including material properties
- develop technical advisories on high performance materials.

The objective in the engineering application area is to bring about innovations in bridge design and construction and to maintain and enhance technical competency in core technical areas. Some specific projects which are underway or will be initiated are

- in cooperation with AASHTO, to develop the Virtis and Opis software suites for improved load rating and design of highway bridges
- to sponsor an international seismic conference
- to develop second generation seismic design specifications
- to develop a demonstration project on scour countermeasures.

Conclusion

The Federal Highway Administration has developed and implemented a workable bridge management programme for administration of a multi-billion dollar programme to ensure the safety and improve the condition of several hundred thousand highway bridges in the United States. The key to this system is partnering with the State Departments of Transportation to develop flexible and yet consistent procedures. These procedures have evolved over more than 25 years and efforts are continually underway to improve them. New bridge management systems have been developed and implemented, new philosophies have replaced old ones, new technologies have been developed and continue to be invented, and new priorities and business practices will continue to evolve. The only constant has been change. The experience of the FHWA in dealing with bridge management at the national level within an environment of continual change has been that change is usually for the better, especially if all parties work toward a common clearly defined goal within a framework of partnership and mutual respect.

Acknowledgement

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The Scottish Office bridge management procedures

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Introduction

A high quality and well managed system of trunk roads is essential to the well-being of the Scottish economy. Bridges are vital links in this network and these valuable assets have to be protected through adequate maintenance to ensure they perform satisfactorily throughout their service lives. The National Roads Directorate (NRD) is responsible for the maintenance of this network, including its structures.

Up until about 20 years ago bridge maintenance was the poor relation of new construction. It was often carried out in a haphazard manner, usually only when some serious defect had become apparent. This late intervention approach meant that remedial works were often costlier and more disruptive than perhaps would have been necessary had action been taken sooner. However, the importance of maintenance has been growing in recent years mainly due to concerns about durability and the live load effects of modern vehicles and traffic patterns, particularly on bridges not designed to current standards. As a result greater attention has been directed to developing a systematic approach to bridge management with a greater emphasis on regular and preventative maintenance.

Bridge maintenance consists of periodic inspection, condition assessments, upgrading including strengthening, planned maintenance, the repair of accidental damage and eventual replacement. Maintenance works have been defined as the action taken to prolong the useful life of a bridge at minimum cost with least interference to its operational function.[1]

Key objectives of the National Roads Directorate's bridge management strategy are

- ☐ to maintain the integrity of the structures' stock and to ensure public safety
- ☐ to make the best use of available funding ensuring value for money
- ☐ to minimise disruption to road users.

Bridge maintenance is a complex activity and effective use must be made of available management and financial resources by efficient planning and prioritisation of expenditure to ensure that these objectives are achieved in a cost-effective way.

This topic poses many important questions such as: What has to be repaired? When is the optimum time to intervene? What level of funding should be allocated in future years? The subject requires consideration of a wide range of often conflicting technical, operational and economic issues to arrive at informed decisions.

This paper describes the procedures and tools that are used currently for the management of trunk road structures in Scotland.

Structures management

Management of programme

The Secretary of State for Scotland is responsible for some 1860 bridges with spans greater than three metres on the trunk road network ranging from simple single-span structures to major estuarial crossings. They are constructed predominantly from concrete, although 9% are masonry arches. More than 70% of these bridges have been constructed since 1960. Additionally, there are approximately 3700 other structures such as retaining walls, culverts and sign-signal gantries. The bridges section of the National Roads Directorate is responsible for the overall management of the structure maintenance programmes. New contract-based arrangements for the management and maintenance of trunk roads were introduced in Scotland in April 1996. These specify services and the standards required for both works and management functions.

The Scottish trunk road network is divided into eight units consistent with the Directorate's route management approach. These consist of three Premium Units dealing with the motorways and major inter-urban dual carriageway routes and five All Purpose Units responsible primarily for single carriageway routes. For each unit a Management Organisation (MO), defined as the Operating Company for a Premium Unit or the Maintenance Agent for an All Purpose Unit, is employed on a term contract basis and is responsible for carrying out the inspection and maintenance of trunk road structures.

A Performance Audit Group has been established to ensure that the specified service levels are achieved and that the best possible value for money is being secured.

Despite increased spending on bridge maintenance, demand is always likely to exceed supply. It is therefore important that available financial resources are allocated in a rational way. In recent years there has been a move away from the subjective approach to maintenance programming towards a systematic one which establishes a relationship between priorities and available funding. In 1996 a system for prioritising maintenance works and allocating maintenance funds for trunk road structures was introduced and this will be described later.

Like the other UK trunk road overseeing organisations, a major programme of bridge rehabilitation was initiated by the Directorate in 1988 to assess its bridge stock against current standards, and to carry out strengthening and other upgrading and remedial works where this is considered desirable. The annual budget for inspection, maintenance, assessment and rehabilitation of trunk road structures in Scotland is in the order of £13 million a year.

Computerised bridge management system

An effective maintenance management system needs good relevant data on the bridge stock, materials, costs and performance to aid decision-making. A computerised trunk road bridges database (TRBDB) has been developed by the Directorate to hold information and programs for the management of trunk road structures. TRBDB currently provides the following modules with input, querying and reporting facilities

- inventories for all structures
- cyclic programme of Principal Inspections
- monitoring of Principal Inspections and structural assessment programmes
- maintenance works prioritisation
- expenditure and works records
- weather resistant steel bridge monitoring
- abnormal vehicle movements
- parapet priority ranking
- technical approval of structures.

Design, development, support and training of users has been carried out in a manner which encourages and ensures effective use of the system by the Directorate and Management Organisations.

This system provides the basis for measuring the whole life performance of structures including ancillary components such as joints, bearings, waterproofing systems, etc. Each Management Organisation can gain access to TRBDB through a wide area network linked to the host computer at The Scottish Office in Edinburgh.

Departmental documents

The Directorate's inspection and maintenance requirements are generally contained in the *Design manual for roads and bridges*[2] and in the *Manual of contract documents for highway works*.[3] Additionally, a Guidance Note[4] has been issued to those responsible for the inspection and maintenance of trunk road structures explaining how Principal Inspection (PI) reports are to be prepared so that their results can be prioritised for maintenance works funding.

Management procedures

Inspection

Periodic inspections are an essential part of the bridge maintenance process. Apart from being necessary to safeguard the public they provide a record of the condition of structures and identify defects. The Directorate's general requirements for inspecting and reporting are set out in Standard BD 63[5] and Advice Note BA 63.[6] There are four categories of inspection: Superficial Inspection (cursory check for obvious defects which might lead to accidents or high maintenance costs); General Inspection (made at intervals not exceeding two years); Principal Inspection (made at intervals not exceeding six years) and Special Inspection (as required).

Specific requirements for Principal Inspection reporting are set out in the Guidance Note: *Trunk road structures: principal inspections for maintenance works prioritisation*.[4] Identification of defective main elements and parts of a structure must be based on a comprehensive location system. The methods to be used are described in an associated document entitled *Location system: principal inspections: trunk road structures*.[7]

Maintenance work prioritisation

Of fundamental importance to the bridge management process is the determination of priorities for maintenance works, from which resources can be allocated and programmes planned. TRBDB records all current trunk road structures which are eligible for PI and the years in which they are to be inspected within a six year cyclic programme. Maintenance funds are allocated primarily on the basis of the preceding year's PI reports. Although the PI system for maintenance works prioritisation applies only to routine works of varying degrees of urgency, the system is flexible enough to allow for unexpected events, budget changes and the rescheduling of priorities: for example, delaying bridge maintenance so that it can be combined with other roadworks to minimise disruption to traffic.

PI reports are prepared to a standardised format, including colour photographs to illustrate reports on serious defects in main elements, and must contain prioritisation rankings for main elements and estimates to provide a justification for and means of allocating future maintenance funds. These reports are analysed by NRD bridges section in the autumn of each year using TRBDB to generate the maintenance programme for the following year. This analysis automatically re-examines priorities for any maintenance works for which funds were not allocated in previous years.

Defective main elements of a structure are ranked according to a scale of 1 to 4 ranging from insignificant to severe. A prioritisation ranking of 4 would be assigned to defects which either pose a risk to safety or the cost of repair is likely to escalate rapidly, and these are likely to generate a high priority for remedial works in the following year's programme. Conversely, defects allocated a 1 or 2 are unlikely to generate an early response and these are only likely to be monitored during General Inspections until prioritisation rankings are reviewed at the next PI six years later.

A PI report must end with a recommendation for action. Before making a recommendation, the results of the PI and any supplementary investigation should be taken into account. TRBDB offers the following eight actions from which the PI engineer must make a selection

1. no maintenance works required; no defective main elements with maintenance prioritisation ranking >2
2. maintenance works should proceed as soon as possible; defective main elements having prioritisation ranking >2 (General Inspections to monitor if repair delayed)
3. special investigation required next financial year to determine the nature and extent of works required
4. await programmed strengthening or other upgrading and carry out maintenance concurrently with these works
5. where an improvement scheme and detrunking are involved, postpone maintenance works until after opening of new trunk road to minimise traffic disruption
6. postpone maintenance works so that they can be phased with other future works to be carried out on the route or with land acquisition
7. demolition planned as part of trunk road scheme; structure can safely be neglected. (Inspections to monitor until demolition take place)
8. beyond economical repair; replace.

PI reports must, depending on the action selected, contain an estimate of costs for a practical package of maintenance works or alternatively, an estimate of costs for a special investigation. These estimates will be used as bids for funding in the next financial year. This estimate should include, as appropriate, but is not limited to

- ☐ scheme preparation
- ☐ site supervision
- ☐ preliminaries
- ☐ list of defective elements and their repair/remedial costs
- ☐ contingencies
- ☐ traffic management
- ☐ statutory undertakers' costs
- ☐ testing services.

Choice of a repair option should also take into account whole life costs including future maintenance and traffic delay costs as well as initial costs. Information on performance and life cycle costs is needed to assist decision-making on maintenance and repair strategies Unfortunately, little is known of the long term performance of repairs and it can be very difficult to predict their life with confidence.

Until recently bridge records were often incomplete in this respect and such information as was available on repairs was seldom in a user friendly form. Consequently, life cycle costing was difficult. However, data is now being gathered through TRBDB which should enable better forecasting of maintenance costs and give a better understanding of the effectiveness of different repair materials and methods.

Conclusion

We are confident that the computerised system which has been developed for the management of trunk road structures in Scotland will enable maintenance decisions to be made on a rational basis thereby ensuring that remedial works are funded in the most cost-effective way.

Acknowledgements

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Bridge management: the local authority perspective

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Introduction

The bridges in the United Kingdom are managed by a number of bridge owners including the Department of Transport, Railtrack, British Rail Property Board, British Waterways Board and the local authorities of the United Kingdom. Close working co-ordination is achieved by way of the CSS Bridges Group. There are 203 local authorities (Table 1) charged with responsibilities for bridges under the various enactments of statute.

Table 1. The distribution of local authorities

Distribution	
County Councils	34
Unitary Authorities	46
Metropolitan Authorities	36
London Boroughs	33
Scottish Authorities	32
Welsh Authorities	22
Total	203

While no accurate figures for the total number of bridges in the United Kingdom are available, the Department of the Environment, Transport and the Regions estimated, in 1997, that there were a total of 44 000 bridges in England which needed to be assessed in preparation for 1 January 1999, when 40 tonne lorries will be permitted onto UK roads and when derogation expires. If one placed a value on each bridge of, say, £100 000, the bridges being assessed represent an asset value of £4·4 billion. Each bridge serves a different environment, some industrial and some rural, and if a hypothetical value was placed on the infrastructure supported by each bridge of, say, £1 million, then one could at least be influencing £40–50 billion worth of infrastructure. Such a hypothesis is uncertain and no doubt wildly inaccurate, but nevertheless the influence of the bridge stock to the community should not be underestimated.

Table 2. Bridge allocations

Year	Allocation (£)*
1995–96	122 million
1996–97	103 million
1997–98	112 million
1998–99	100 million

* These figures are for England.

Local authorities manage the majority of bridges in the United Kingdom and are therefore at the ‘business’ end of policy implementation. One cannot simply categorise the importance of a bridge solely by virtue of its route designation (trunk road, primary route, A, B, C and U); each bridge serves an individual community.

Funding for bridge management in the United Kingdom is from Central Government sources and the bridge allocations through the TPP settlement over the last four years were as in Table 2.

This paper seeks to identify the major issues that confront local authorities in the management of their statutory duty.

Of the allocation in 1998–99 of £100 million, £64 million was allocated to strengthening and structural maintenance. Local authorities identified over £200 million of strengthening works alone in their bids for 1998–99, and as the rate of progress on assessments quicken in preparation for 1 January 1999, the need for interim works, strengthening, weight restrictions and reconstruction no doubt increased.

Statutory duty

Section 41 of the Highways Act imposes a duty upon Highway Authorities to maintain the highway, not surprisingly. There have been many disputes as to what is a highway and to what is the standard of maintenance, and Section 58 of the Highways Act 1980 sets down a special defence against actions for nonrepair, i.e.

> *such care as in all circumstances is reasonably required to secure that the highway was not dangerous for traffic.*

In defining what standard of maintenance is applicable, in the case of Sharpness, New Docks and Gloucester and Birmingham Navigation Company versus Attorney General, Lord Atkinson said

> *in the argument of this appeal many authorities were cited to establish that it is the duty of road authorities to keep their public highways in a state fit to accommodate the ordinary traffic which passes, or may be expected to pass, along them. As the ordinary traffic expands or changes in character, so must*

the nature of the maintenance and repair of the highway alter to suit the change. No person really can contest that principle.[1]

The point surely is that each location is individual and must be dealt with on its merits — and that is the problem for many local authorities in that each bridge is unique in its surroundings and the infrastructure that it supports and its use, and therefore can demand as much attention under a statutory duty as the major routes. Unfortunately, most of our bridges lie on the B, C and U roads of the country.

Current bridge management issues

Having said that local authorities are under statutory duties, what are the major issues that confront those delegated with the task of managing those responsibilities? A short list could be as follows

- Managing assessments and implementing weight restriction — managing the network
- Managing and prioritising reconstruction and strengthening
- Managing maintenance — cyclic, preventative and remedial
- Managing records, public utilities, abnormal loads, retaining walls and cellars

This list is by no means exhaustive but it probably represents the major issues that test the metal of bridge managers.

Managing assessments and implementing weight restrictions

David Lynn, in his paper to the Surveyors' Conference in 1998,[2] indicated that most local authority bridges will have been assessed by 1 January 1999; nevertheless, there will be bridges which have not been checked for the 40 tonne vehicle by the assessment procedures. Not only will there be these residual assessments but also there will be a substantial number of bridges, already assessed, where more refined assessments will need to be carried out. Therefore, it is essential that funds for residual assessments and refinement of existing assessments are made available in forthcoming years to satisfy these needs. Advice Note BA 75/98 identifies further levels of more refined assessment which bridge owners may have to consider, either to maintain temporarily structures in use, or to relieve the need for any measures at all.[3]

For any local authority which has not completed its assessment programme it would be advisable to carry out a careful inspection and 'strength appraisal', as recommended by Brian Swan[4] who suggested that such a special inspection and strength appraisal would be the minimum sensible precaution.

Managing sub-standard bridges has troubled all bridge owners and the new BA 75/98 goes a long way to providing a structured approach to their management and the identification of sub-standard bridges is in some sense a philosophical leap. The concept of monitoring appropriate bridges may help in some specific circumstances; nevertheless, many bridges which have failed their assessment will not be regarded as 'monitoring-appropriate' and will therefore require weight restrictions or other protective measures. Where a bridge is regarded as being monitoring-appropriate the BA identifies the importance of managing that monitoring.

As local authorities have been given funding to complete the assessment programme by the end of 1998, there is an unavoidable consequence in that all the problems will be known at one time. Again, an unavoidable outcome is that weight restrictions and interim measures will proliferate as local authorities seek to complete their programmes. Given the level of available funding for strengthening or replacement, the pragmatic risk assessments of the past will have to be replaced by weight restrictions.

Turning to weight restrictions, it is easy to regard them as statistics. However, weight restrictions affect communities and communities require consultation. It is necessary to identify the problems to industry and agriculture, and ameliorate them wherever possible. Diversion routes require agreement both locally and with other local authorities and they require signing. The cost, both in time and money, is high and local authorities would do well not to underestimate the cost of each individual weight restriction in their bid for funding.

Managing and prioritising reconstruction and strengthening

Given infinite resources, all needs would be attended to immediately. Local authorities are well familiar with the conflicts between resources and demand. However, resources for strengthening and reconstruction have been curtailed in order to fund the assessment programme. An unavoidable consequence is that the list of schemes grows longer for each local authority and irrespective of prioritisation many communities are left waiting for their bridge to be returned to capacity, with consequent economic and environmental costs. Again, BA 75/98 identifies some of the considerations that will need to be made for prioritisation; however, each local authority has wisely been left to identify its own process of prioritisation.

The measures identified in the Advice Note are

- the relative residual risk to structures where interim measures have been applied, taking account of the effectiveness of the interim measures, with reserves of strength, etc.
- traffic delay costs which are caused by the implementation of interim measures and will be eliminated when strengthening is complete

- other social, environmental and economic consequences caused by interim measures to businesses and communities, in addition to traffic delay costs
- the adequacy of alternative routes (including winter conditions) and the environmental consequences of their use
- the cost effectiveness of strengthening, taking into account the ratio of costs and benefits
- other benefits which result from strengthening.

The above is a broad shopping list of issues which will distil to an assessment of the structural considerations for the bridge, and the strategic importance of the route. The strategic importance should attempt to establish the economic consequences of delayed reconstruction or strengthening.

A particular difficulty for local authorities is that in many cases many of the parameters cannot be established by a desk top study, but will in fact flow from the implementation of interim measures, such as weight restrictions. Hence the importance of consultation and feedback, and the acceptance that prioritisation has to be a dynamic process which can respond to new factors.

Managing records, public utilities, abnormal loads, retaining walls and cellars

Other papers in this book deal comprehensively with detailed issues of bridge management and inspection. Nevertheless, the importance of records cannot be over emphasised. Any bridge management system relies upon quality records which are well maintained and it is essential that local authorities recognise the importance of this area of work as it sustains the glossier side of our work. Records of monitoring and the evaluation of changes are essential parts of the BA 75/98 guidance. Monitoring must have real value if it is actually to provide security against structural failure.

The increased pressures of managing the expanding workload of assessment and strengthening has, in some cases, overshadowed the central function of controlling public utilities and managing heavy load movements. We must not lose sight of the importance of these activities and there is a need to continue our efforts to improve our procedures both locally and nationally.

Most local authorities have already identified many of their retaining walls and it must be recognised that retaining walls fall within the remit of TPP submissions, and indeed can present some of the most high risk locations. Cellars and vaults under the highway will need to form a second tranche of assessment and evaluation, beyond 1 January 1999 and that must be recognised. The cost of assessment and dealing with retaining walls and cellars to support the 40 tonne lorry have yet to be identified.

Managing maintenance — cyclic, remedial and preventative

Other papers also deal in detail with the inspection and repair strategy necessary to achieve the reliability criterion. They seek optimisation of inspection and repair programmes and are to be commended. The well-used descriptions of cyclic, preventative and remedial maintenance still provide a sound foundation for managing maintenance. Local communities also rightly expect maintenance to take account of aesthetics, i.e. appearance. Bridges are often seen by the community as being a focal or reference point and indicative of that community and, again, adequate funding is essential to ensure cost-effective bridge management.

Conclusions

One principal conclusion is that each bridge is unique in its own environment and contributes uniquely to the infrastructure around it. Therefore, bridges on all categories of road have to be assessed, maintained, strengthened or reconstructed in accordance with the needs of the locality.

Summary

Four principal areas of bridge management have been identified as being of particular importance and include the following key points

1. The assessment programme needs continued funding into 1999–2000 to deal with residual assessment and refined analyses facilitated by BA 75/98.
2. Funding for strengthening has been curtailed in the last two years and weight restrictions are unavoidable as a consequence.
3. While local authorities can prioritise spending and focus available funding to the highest priority locations, many communities, industries, businesses and agriculture will be affected for an unacceptable period of time and therefore greater funds are required to deal with the backlog.
4. Greater effort needs to be applied to managing public utility works and heavy load movements.
5. The assessment of retaining walls and cellars will require funding beyond 1 January 1999.
6. The funding of maintenance should not be neglected.

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Management of the national bridge stock in Italy: toll motorway bridge management for structural safety and customer needs

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Introduction

The management of motorway bridges is a question of ever increasing importance in civil engineering. However, the resources earmarked for it have a relatively insignificant incidence on the balance sheets of the entity designed to manage them.

With infrastructural works such as bridges, two functions must now be performed: first, checks and inspections to prevent the occurrence of deterioration that reduces to the ultimate safety of a structure, and compromises the state of its general conditions overall, and, second, guaranteeing the duration of bridges over time and client safety.

The key aspect of the correct management of such structures consists in their being monitored so as to assess their real conditions and their durability. By so doing it will, consequently, be possible to identify in terms of time and technical action, the priorities and the most immediate repair measures to be taken.

The experience obtained from the monitoring of road structures, furthermore, enriches our store of knowledge and enables us to optimise our conceptions of new motorway engineering works.

Background

The Autostrade Company manages about 3000 km of motorway and approximately 3000 bridges and viaducts of over 10 m in length and utilises a series of control and maintenance programmes co-ordinated within the framework of the partially automated SAMOA programme (surveillance, auscultation and maintenance of structures). By virtue of this programme which entails the classification and recording of motorway structures by their different structural components, we can obtain an accurate survey of the defects present, chosen either by ease of identification or by their importance in assessing the state of health of the structure. To overcome the problem of subjective interpretation by different inspectors, data forms are compiled on the basis of standardised methods set out in special operator manuals, and then inputted into a series of compatible computers in the territories covered by the Autostrade network.

Automated monitoring programme

The high average age of the structures and the growth in their numbers have reached such a point that surveillance based solely on the visual inspection of the state of the structures and a diagnosis based on individual inspector's experience is no longer reliable. It thus became necessary to devise specific criteria to ensure a more uniform and sound assessment of the state of health of certain structures, and also to predict the probable evolution of this state over time.

Starting from this global assessment or diagnosis of the entire population of structures under surveillance, it is possible to formulate three distinct sub-groups of structures (or structural components)

- definitely reliable
- definitely requiring maintenance
- not perfectly defined in terms of reliability.

The two sub-groups referring to precarious or suspect conditions, presumably comprising a very limited number of items with respect to the total population, are then subjected to more detailed inspection (and thence diagnosis) using more sophisticated instruments and tests with respect to simple visual inspection.

The specific aims of the SAMOA programme are the

- creation of a database and related software for the management of the morphological data on the structures and the maintenance interventions performed on them
- research and development of rapid, nondestructive control systems for the automatic acquisition of such data
- development of structural verification programmes to assess the level of safety and need for intervention.

The first part of the management activity consisted in recording the registration data on all the structures of the network. Drawing on design and cost data from existing files, combined with information obtained on possible site visits, the next step was to compile special 'morphological records' for each of the structural elements comprising the structure. Various type groupings were defined such as, for example, piers, foundations and all the other elements making up the structures, as well as indicating the dimensions and compositions occurring in the individual structure in question. The data subsequently noted on the computer constitutes the historical database of the structures.

As far as inspections are concerned, surveillance is essentially performed by means of close visual examination of the individual structural parts. Any defects which may be present are identified and noted, with particular care being taken to observe the developments in those found previously. Visual checks, despite their limitations, remain the fundamental method of surveillance as they are, at least, able to provide general indications of the overall state of conservation of the structures.

In order to guarantee the greatest possible uniformity in classifying the different types of defects and the most important parameters for their description and control, 'defect charts' are used on which such characteristics are defined in unequivocal fashion. In fact, inspections are conducted in accordance with a special manual on damage survey methods. After the forms have been compiled in the field, the data is then entered into a computer, where it is processed using special algorithms.

The processing criteria yield a global assessment of the state of the structure or of its single component parts. These, in turn, serve as the basis for establishing criteria for prioritising interventions. Thus global assessment being very quick to use and also reasonably accurate, serves to distinguish definitely reliable structures from structures which require more detailed examination. The latter require special attention (measurements, determination of the restoration measures to be taken), and hence are subject to more frequent surveillance or, if necessary, constant monitoring. Reliable structures, however, are only subjected to ordinary routine surveillance save for the repetition every two years of the reliability inspection forming part of the global assessment.

Contrary to visual inspection procedures, the specialised control techniques are employed on an *ad hoc* basis, i.e. the need arises to have more detailed information on a single structure or on those located on a particular section.

Proper integration of the various methods used is fundamental for correct interpretation of the data measurements. Indeed only in this manner is it possible to effect a type of iteration which, taking into account the various approximation factors proper to each of the systems, makes it possible to realistically quantify the state of conservation.

Data processing method

The processing software contains defect assessments of varying degrees of seriousness in relation to the type of structure involved, the component materials, as well as the extent and location of the deterioration. The procedure adopted will depend on the objective required. The choice between the different analytic viewpoints is a matter of selecting between distinct aspects: structural safety, state of conservation, aesthetics, joints, waterproofing, etc. It is necessary to select the viewpoint at the very beginning, so that the processing operation can be carried out automatically without interruption.

Given the fact that the assessment is conducted separately for each component part of the structure, we shall concentrate our attention on a specific application — in this case a deck. The deck in question comprises simply supported beams and the final partial output (one of the types of data processing results yielded by the program) consists of the aggregation of all the spans of a structure.

The list of 'suspicious' bridges is aggregated by spans and degree of deterioration.

In particular, the groups associated with each span are organised according to two different output possibilities

- aggregation of spans by single structure, with related indication of the type of group to which they belong
- aggregation of spans by group, with related indication of the structure to which they belong.

The final output is then represented by an overall assessment of the conditions of all the decks of a structure and the ranking of the latter in a classification of overall condition based upon the ratings of the condition of the various structures of a selected population.

The bridge management system is continually improved over time by constantly up-dating its database and thus enhancing the management of the traditional tasks by human operators. Thus, we have surveillance, auscultation and intervention as integrated moments of a single process with the substantial but friendly assistance of the computer.

Interventions on structures

Starting with the list of priorities provided automatically by the computer program and following whatever specialised testing may be required, we proceed to the design of the intervention in accordance with defined reference criteria. The general criteria for interventions to be carried out on civil engineering works are as follows

1. *Ordinary and extraordinary maintenance work.* A series of operations necessary to guarantee the efficiency of a structure on the basis of its original characteristics through work upon those structural elements exhibiting deterioration but which do not affect the static load of the structure when the work in question is carried out.
2. *Static rehabilitation.* A series of measures designed to restore the original load-bearing capacity of a bridge that has partly lost its original static properties.

3. *Upgrading or retrofitting*. A series of measures designed to enable a structure to handle greater or different stresses, which substantially respect the initial geometry and original static scheme.
4. *Restructuring*. A set of measures aimed at rehabilitating or increasing the load-bearing capacities of the original structure which require modification to the geometrical characteristics or static scheme of the original work.

Categories (2), (3) and (4) refer to the load-bearing structures of civil works (beams, transverse beams, slabs, foundations and anti-seismic supports) while category (1) also refers to accessory components of the work (paving, waterproofing, joints, supports, drainage, crash-barriers).

The problems faced by the designer planning maintenance interventions on existing structures are considerably more complex than those found in the design of new structures. Some of the additional constraints encountered include the need to operate in the presence of traffic; the need to select materials which yield reliable results within a very short period of time and in the presence of traffic-induced vibrations; the very strict limits on intervention time; the need to assess material strengths in situations where deterioration is present and the need to achieve higher performance standards than those adopted in the original design of the structure.

As regards existing structures, all work falling into the categories of upgrading and restructuring as well major work in the category of static restoration requires the full specification of the design approaches — in terms of the techniques and standards adopted — in relation to the reference standards for the load-bearing capacity of the structures in question. However, there are no requirements on the activities coming under the heading of ordinary and extraordinary maintenance — which accounts for the greater part of the interventions on our motorway network.

In this context these types of application do not call for the drawing up of specific technical reports or calculations, except for particular measures such as the lifting of the deck, the installation of rigid barriers or the replacement of supports and joints. In all such cases it is held that the resistance of the structure should be ascertained so as not to produce an increase in stress incompatible with the effective state of the structure in question.

With regard to such work, when there are no cogent prescriptions in the present regulations, and in agreement with the engineering company involved, reference is made to the standards in force at the time the structure was designed unless we, as Autostrade, request that reference be made to more prudent criteria in order to increase the durability of the structure in question.

For work falling under the heading of static restoration, reference is made to the standards in force when the structure was built, unless it is decided to apply more rigorous standards in relation to the real damage sustained by the structures, and, again, in concert with the engineering company.

For work classified in the categories upgrading or retrofitting and restructuring reference is made to the regulations in force, unless it is decided to adopt more recent and more prudent methodologies and criteria. In terms of seismic precautions various options are open to us, regardless of the category of work involved, which enable us to obtain different reductions in seismic risk for the work in question.

New structures: integrated design and maintenance

Thanks to 30 years' experience of surveillance of motorway engineering works a specific store of knowledge has been built up which has already been put to use in the more recent infrastructural designs.

Containing running costs and the extension of the working life of a bridge are possible if the design does not just limit itself to the ascertainment of structural safety but also satisfies a series of requirements, among which is the best way to maintain the structure by devising apparently marginal improvements and taking care of some particular construction details. However, this requirement, although one of the principal ones is not the sole requirement.

On the basis of the experience acquired in the surveillance on structures, it has been possible to draw up a checklist of good design pointers which meet the following five objectives

- increasing the durability of the structure
- reducing the probability and size of accidental damage
- making certain parts of the bridge accessible for surveillance
- facilitating maintenance work
- guaranteeing the safety of users and personnel as well as the safeguarding of adjacent environments.

If we consider the statistics of the interventions taken, a first balance can be made, for example, of structures with freely supported beams. Although exhibiting notable advantages in terms of construction, the weakness of this type of structure has been shown to be its joints (Fig. 1).

The use of continuous structures with few and high quality joints overcomes this problem brilliantly, making radical reductions in the frequency of maintenance and to the benefit of the user by improving the driving comfort and eliminating, at the same time, a source of deterioration and noise.

The design of accessory parts and the constructive details was more often than not overlooked either because it was left to the contractor or

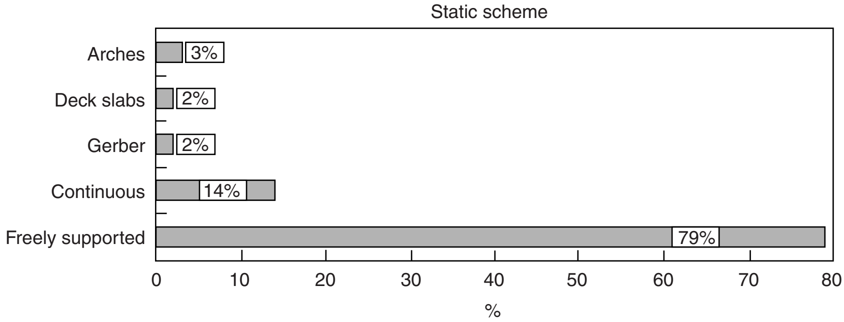

Fig. 1. Static scheme

because it was regarded as a negligible matter. But also here the examination of the deterioration observed on existing structures indicates that the designer of a new structure must dedicate attention to the design of such parts as they have direct repercussions on the overall state of conservation of the bridge.

Conclusions

In connection with the complete privatisation of Autostrade Company, a new agreement has been made with the Government whereby toll tariffs have been correlated to 'Quality' (Q), i.e. if quality improves, so may the tariffs, but vice versa if quality deteriorates the tariffs must also diminish. This provision was introduced to prevent the new proprietors of the company from reducing expenditure on maintenance in order to increase profits. The measurement of Q is based upon a series of indicators of performance which are measured by parameters classified into the following levels — very good, good, average, sufficient, insufficient. Successively, these levels undergo a percentage breakdown into classes for purposes of correlating the indicators with the levels. The sum of the product of the indicators of performance and the weightings for the single characteristics of quality as measured by the relative indicator gives the overall quality.

$$Q = \sum_i I_i p_i \tag{1}$$

where I_i is the indicator of performance, variable between 0 and 100, and P_i is the weighting attached to the single characteristics of quality as measured by the relative indicator of performance.

At present, 14 indicators of performance have been identified which refer to three types

- ☐ Pavements: adherence, regularity and load-bearing
- ☐ Structures:

(*a*) bridges, viaducts, tunnels
(*b*) geotechnical stability
(*c*) hydraulics
(*d*) road signs
(*e*) safety barriers

☐ Safety and Services:
(*a*) accidents:
(i) On flatland
(ii) In the mountains
(*b*) availability of services
(i) information on traffic (services along the road)
(ii) time to remove vehicles involved in accidents
(iii) time during which roadway is occupied for roadwork/number of works)
(iv) toll payment time.

In the specific case of structures, the indicator of performance is represented as shown in Table 1.

At the moment the indicator of the structures has not been accepted as it was held to be too subjective. Those accepted for the road paving are adherence and regularity and as regards *Safety* accidents on the plain and in the mountains. However, as regards maintenance we use the indicators of

Table 1. Simplified table of marks

	Mark	Level	Ref. classes
0	No fault		
1	The deficiency will not produce other deficiencies		
2	The deficiency may cause other deficiencies but these do not call for maintenance work	Very good/ Good	
3	The deficiency may cause other deficiencies which will call for maintenance work		
4	The deficiency requires for an intervention in the long-term	Sufficient	
5	The deficiency requires for an intervention in the medium-term	Insufficient	C%
6	The deficiency affects the statics but does not significantly reduce safety coefficients	Bad	D%
7	The deficiency determines a reduction in safety coefficients		
>7	Non-functioning structures	Very bad (to be excluded)	E%

structure until they are definitively revised. As performance indicators for structures we apply the following algorithm

$$I_{\mathrm{p}} = 100 - (12\,\mathrm{E}\,\% + 8\,\mathrm{D}\,\% + 4\,\mathrm{C}\,\%). \qquad (2)$$

The approach to improvement in the sense of making judgements less subjective is to update the projection process of the various forms of deterioration encountered by addressing the deterioration process of the single parts. In this sense, the damage can be made visible by the Markovian process which calculates the probability with which each condition will be followed by any other. The validity of this procedure has been verified by the behaviour found in other similar work, in similar conditions.

As regards deterioration forecasts use is made of deterioration trends obtained from the SAMOA databank. Furthermore, the conditions of the deterioration are related to structural functioning and as a result it is possible to discriminate between structures assigned the same number of marks or points. This in its turn permits the identification of the more dangerous parts as regards both the technical reliability of the structure and the perceptions of the user.

In addition, further improvement has been obtained with the adoption of dynamic programming. This is a convenient description for a motorway system because it has shown itself to be a useful tool for purposes of optimising the performance of a set, represented as a network of critical nodes and connecting branches with differential probabilities of failure.

An optimum solution has been found in the use of a backward inductive algorithm. Consequently, it is first necessary to determine the optimal decision for the final stage. As regards the preceding stages, an optimum decision will be one which minimises the probability of failure of the network with a reduced level of resources with respect to the quota already assigned to the subsequent stages. The procedure is completed when the examination of the first stage is finished.

The use of the backward inductive algorithm and the whole procedure is recommended when planning has to be made in the presence of economic constraints, for instance when the available resources are not sufficient to repair all the critical elements of a network or vice versa when the entire amount of available resources is not yet known.

In order to assist dynamic programming it is necessary to identify additional criteria of choice with respect to those of generalised deterioration. In conclusion it is necessary to supply operative instruments with reference to the design phase and the measures to be carried out.

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Part 2. Bridge management methodology

Development of a comprehensive structures management methodology for the Highways Agency

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Introduction

The Highways Agency, which is an executive agency of the Department of Environment, Transport and the Regions (DETR), is responsible for the maintenance of the trunk road network in England, including its structures. From the early part of this century, UK government departments responsible for transport have been directly involved in the development of national procedures for the assessment of road bridges in respect of their load carrying capacity.[1] After the Second World War, road traffic in general and the numbers of heavy goods vehicles grew rapidly and the safe load carrying capacity of the structures, particularly that of the older bridges, became increasingly doubtful. Thus, by the mid-1960s bridge assessment and management became important issues and the Department of Transport, in liaison with other UK bridge authorities, initiated the first national bridge rehabilitation programmes, the *Bridgeguard Operation*. This was followed by the current 15 year programme which was started in 1988.

In parallel with the rehabilitation programmes, the Department of Transport, together with the Scottish Office, The Welsh Office and the Northern Ireland DOE, have implemented a number of national standards and advice notes for bridge inspection, numbering and other management activities. The current versions of these Departmental Standards and Advice Notes are contained in the *Design manual for roads and bridges*.[2] The Highways Agency's specific requirements and procedures for maintenance are given in the *Trunk road maintenance manual*.[3] Individual component activities relating to structures, such as data management, inspection and assessment, called up by the manual, are carried out in compliance with the above-mentioned standards and advice notes.

Many of these procedures for structures maintenance were set up some years ago; for instance the current *Bridge inspection guide* was published in 1983. Most are in need of revision in the light of experience gained hitherto from their use, as well as to take on board new developments, both procedural and technical. These changes have necessitated a thorough review of the needs and the procedures for structures management.

Recent developments

One significant new development in the area of highway maintenance in England has been the new maintenance agency arrangements being adopted by the Agency. These are in effect ending the long standing arrangements with local authority highways departments for maintaining trunk road structures and introducing fixed term agency commissions. This could, in practice, mean periodic changes of maintenance personnel. In future therefore all requirements and procedures will need to be much more precise and detailed than before.

Another development is that the current 15 year bridge rehabilitation programme, which was primarily intended for dealing with a backlog of essential maintenance work, is approaching its end. Hitherto, all non-routine maintenance activities for the Agency's structures were carried out under this programme. New procedures will be required soon to cover parts of this programme which will necessarily continue into the future.

The present stock of structures was built largely between the early 60s and the late 80s, with a peak of construction in the 70s. Many of these structures are beginning to suffer from age and their maintenance needs will become increasingly important, both in volume and complexity. Procedures will need to be developed, firstly to forecast these future needs accurately, and secondly to carry out the necessary work effectively.

International co-operation

For the reasons described above, the Highways Agency for some time has been reviewing the whole spectrum of its engineering procedures for structures management and their objectives. These reviews started in the early 90s with a comprehensive examination of the state-of-the-art in respect of bridge management systems [BMSs] carried out by consultants High-Point Rendel.[4] The examination covered many overseas developed systems, notably Pontis and Bridget from the USA and the BMSs developed by the central road authorities in Denmark and Finland. This was followed by a Highways Agency in-house review of the overall structures management methodologies including inspection, assessment and other maintenance procedures. This internal review was founded on the extensive R&D work involving reliability-based techniques already under development through various Highways Agency projects.

During the reviews carried out by High-Point Rendel and by the Agency itself detailed liaison was necessary with overseas government officials, system developers and academics, who have all been extremely helpful throughout. It can be said that the proposed developments which are described in the rest of this paper are largely an outcome of international co-operation.

The purpose of this paper is to describe these developments, but first it discusses the principles which form their basis.

Objectives of structures management

Risks to highway structures

Throughout their intended functional life, structures are required to be safe for the users and the general public in terms of loss of life or injury (life safety), as well as to remain available for full functional use. Risks to structures can therefore be primarily of two types, risk in respect of life safety and risk in respect of loss of use.

In the case of a bridge, total system failure (collapse) clearly has life safety implications, and the probability of this occurring must be kept at an extremely low level. Fortunately, loss of life from bridge failure is extremely rare, and whenever this has occurred, in most cases, it has occurred as a result of unforeseeable combinations of events. Individual element failures resulting from exceeding their structural capacity (the ultimate limit state), however, do occur at a relatively frequent rate. From a recent survey, Menzies[5] found that on average two member failures took place per year in the UK out of a total of some 100 000 bridges. This represents a lifetime (120 years) failure rate of $2{\cdot}4\times10^{-3}$. The real failure rate for element failure is probably greater since many failures are dealt with within normal maintenance and are unlikely to be reported or recorded.

The ultimate limit state failure of a bridge member may not have any immediate or significant safety implication, but can nevertheless be very important to the owner and the road user. For example, if a permanent crack appears in the deck, or a noticeable sagging or rotation takes place, the authority has to undertake remedial measures although the overall bridge may be in no immediate danger of collapsing. The remedial measures will most probably involve unscheduled disruption to traffic. Such disruptions on a heavily trafficked road can result in considerable road user delay costs. Considering that the probability of failure of the first element may be much higher than that of total collapse, the risk of disruption to functional use is very real.[6]

To illustrate the point, let us assume that purely on financial terms, taking into account road user delay costs, the cost of the total collapse of a bridge is £2 million and that resulting from the failure of a major element is only £150 000. Let us also assume that the probability of a total collapse is 10^{-8} whereas the probability of the element failure is 10^{-3}. If risk is defined as the consequence times the probability of occurrence, the risk from collapse, 0·01, is smaller than the risk from element failure which is 150.

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.

Risk factors

During their functional life, particular groups of structures, or their elements, may be considered to be at risk of failure or collapse unless some remedial actions are taken quickly. The main causes of concern are usually the following

1. material deterioration and development of structural faults
2. inadequate original specification of materials and methods, e.g. the use of ASR (alkali-silica reaction) prone aggregate in concrete or colliery shale as structural backfill
3. increased traffic loading since original design
4. inadequate original design requirements.

Some of the structures deemed to be at risk from causes (2), (3) and (4) do not necessarily develop any significant or noticeable signs of distress at the time of consideration. This is because the extreme load conditions and the worst circumstances may not yet have occurred for these bridges.

For this reason, *it is not sufficient for the bridge authorities to repair or strengthen only those structures which have shown deterioration. Other structures which are at risk due to other factors also require attention.* Indeed the current Highways Agency bridge rehabilitation programme covers, in addition to steady state maintenance which deals with general deterioration, bridges and other structures deemed to be at risk for a variety of reasons.

Reliability-based management

Similar structures designed and constructed to the same requirements, for various reasons, end up with different safety levels, even when they are just constructed. Fig. 1 shows the whole life safety (load carrying capacity) profiles for 15 concrete bridges on UK trunk roads which were constructed 25–30 years ago. The profiles have been calculated by Prof. P. Thoft-Christensen of Aalborg University, Denmark, and Cambridge University, in the course of a current Highways Agency project. The figure shows that, even as constructed, the bridges had widely different load capacities, although they were designed to the same design requirements (45 units of HB, for all but one). Some of the bridges shown have such margins of safety in respect of the assessment requirements that even after extensive deterioration they may remain safe, whereas some of the others may

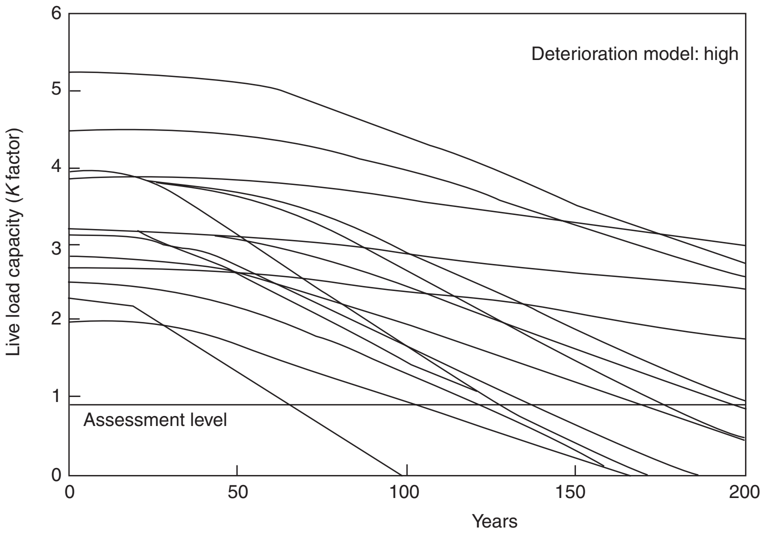

Fig. 1. Whole life load capacity profiles of 15 concrete bridges

become unsafe with only minor amounts of deterioration. The implication is that *structures management needs to be based on structural adequacy (reliability) rather than on the extent of deterioration. This is also important because a minor fault in an important part of the structure may signify a greater risk than a more extended fault in a less critical area.*

Need for preventative maintenance

Reliability, in the form of p_f, the probability of failure, or β, the reliability index, is an accepted means for describing the overall risk to the safety of a bridge. Let us assume, for the purpose of a schematic representation, that the reliability index β for the whole population of the bridge stock is individually calculated and the number of bridges for each value of β is expressed in a distribution graph as shown in Fig. 2. The newer bridges are likely to be on the right of this distribution and some at risk bridges such as those with severe deterioration are likely to be located towards the left. Let us assume that the bridges to the left of β_{cr} are those that have been assessed to be sub-standard, i.e. have been calculated to have a critically low factor of safety.

It is reasonable to expect that, without any management action, the overall reliability distribution of the bridge stock will tend to move leftwards, i.e. many bridges will become progressively less safe with time. Some bridges will of course deteriorate much more slowly than others. If the bridges with β less than β_{cr} are the only ones repaired or replaced at year 0, after a period of time, say at year x, the number of bridges to be repaired

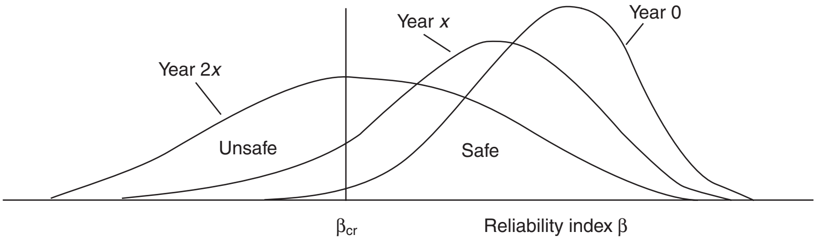

Fig. 2. Bridge safety deterioration

will be much greater, as shown in Fig. 2. After a number of similar periods the numbers of bridges to be strengthened could reach unmanageable proportions. The overall purpose of bridge management is to keep maintenance at a steady level as far as possible and prevent unmanageable backlogs of work from developing. For this reason, *it is not sufficient to strengthen or replace only those structures which are found to be inadequate at any point of time, but others may also require preventative maintenance to avoid future logistical and funding problems resulting from backlogs*.

In order to carry out such preventative maintenance, as large numbers of structures may be involved, groups of these have to be selected carefully for such action. Whole life performance assessment and costing, incorporating potential future reduction of structural capacity, will be required to identify where any preventative action will be cost effective.

Whole life costing

The cost of any maintenance activity consists not only of the immediate costs but also of future costs arising from the chosen strategy. For instance, if only a minor repair is carried out instead of a required full rehabilitation, the effect will be to necessitate further work at a later stage. In order to obtain best value for money from maintenance work it is essential to compare options on the basis of whole life costs. However, the use of whole life costing of structures maintenance options has two associated problems.

First, all costs have to be discounted to their present value (PV) and therefore future costs become progressively insignificant compared to the immediate costs. Hence, the options that postpone major work till later almost always win in terms of whole life costs. However, the postponed works accumulate as the years go by and eventually become a mountain of backlog work requiring major funding and logistical effort to clear. *It is therefore essential to have a long term strategy for maintenance work in parallel with the assessment of annual bids on the basis of whole life cost.* This is also important because the use of whole life costing may give the best strategies for individual structures, nevertheless these in total may not represent the best strategy for the network.

The second problem with the use of whole life costing applied to highway structures is that the road user delay costs tends to overshadow all others. However, the delay costs are not entirely real costs and hence such applications introduce an additional element of uncertainty to the decision process.

Flexibility for continuous development

The existing structures' stocks contain a large variety of construction types. In general, they all have exhibited, or are expected to do so in future, different problems which will require different management approaches for inspection, investigation, assessment and repair. In essence, structures management has been, and can be expected to remain in future, a constantly developing scene. No fixed procedure can be expected to be useful for any long period. Hence, *the procedures need to be flexible, i.e. able to incorporate changes in the component activities without unduly affecting the rest*. This means that the procedures, particularly the databases, need to be in a linked but modular form.

Proposed developments

Management of highway structures involves a large number of activities, which can be broadly grouped as

- ☐ Structures inventory details
- ☐ inspection
- ☐ assessment
- ☐ maintenance bids, prioritisation and allocation
- ☐ works data and outturn
- ☐ network structures condition monitoring
- ☐ planning and forecasting
- ☐ database.

All these activities depend on each other for input and output of information, and have to take place in sequence, except that the overall information system, in the form of a database, has to be involved at each stage. It was clear from the current review that the procedures for most of these activities needed to be upgraded, and some new ones added, if the objectives were to be met satisfactorily. The proposed developments therefore cover all these areas.

The following are brief descriptions of the more significant aspects of the developments, more details of which are given in some of the other papers presented at this symposium.

Structures data

Each structure will be sub-divided into elements, and elements into segments, to make the details more directly applicable in inspections and assessments, and secondly, to allow the inclusion of photographic, video and virtual reality (in special cases) records which the revised database SMIS (structures management information system) will be able to store.

Inspection

There have been two main criticisms of the current inspection procedures: firstly, the standard of inspections carried out on similar structures by different people do not appear to be consistent, and secondly, the results are not sufficient or suited for subsequent use, for example in assessments or condition monitoring. Based on a study carried out by consultants, it is proposed that the somewhat expensive and generalised Principle Inspections, at present carried out at approximately six year intervals, will be replaced by item specific Particular Inspections at different intervals depending on the importance of the item.

The current *Bridge inspection guide* will be replaced by five inspection manuals.

Assessment

One major limitation, as far as structures management is concerned, is that the present form of structures assessment only determines whether the structure is safe at the time of assessment; it does not provide information on how the structure will perform in the future, or what the consequences of adopting alternative management strategies would be.

In order to overcome this limitation, whole life performance-based assessment procedures are currently being developed by the Agency which will enable the assessing engineer to estimate future safety levels of a structure and how these levels will be influenced by alternative management strategies. The new procedures will also enable the assessing engineer to estimate works costs, traffic management costs and traffic delay costs, for any future period, corresponding to different management activities. The intention is that these costs will be input into the proposed bid assessment and allocation program, which is referred to later as BAPS.

Whole life assessments will require cost data as well as structure specific traffic management and traffic delay data which will be available from SMIS. Similarly, all reports and forms containing results of assessments will need to be formatted in a way that SMIS will be able to input. The results will be focused on critical aspects so that future regular reassessments can be carried out conveniently.

It is also proposed that assessments in future will imply not only formal calculation-based assessment of structural capacity, but also observational or engineering judgement-based assessment of components, etc.

The following new standards which will be essential for bidding for maintenance work are in various stages of development

- whole life performance based assessment of structures
- whole life costing of design and maintenance options for structures
- the management of sub-standard highway structures.

Bid assessment and prioritisation

At present bids for structures maintenance are input by Maintenance Agents into the structures database NATS, at appropriate points in the bid cycle. The information provided is very limited and certainly not adequate to fully assess the risks and other implications of not carrying out the work. Only one bid is provided for a scheme, with no option.

The Agency is currently developing a computerised bid assessment and program system, which will enable the MAs to input a number of management strategies for each structure covering a period of years. The associated costs including traffic delay costs will also be input. The Agency will consider the options and risks for the proposed list of work along with other strategic or central requirements and produce a network bid through BAPS. The system will also determine the implications of not meeting the bid fully.

Network structures condition monitoring

In order to assess whether maintenance has been effective, two indicators are being developed. The Condition Index (CI) will indicate the physical condition of an individual structure, the whole structures stock, or a part of it. The Safety Index (SI) will similarly indicate the calculated safety levels of the structures. The Safety Index will more directly indicate the required maintenance work level at any time. It is intended that these indices will be updated following each inspection and assessment.

Planning and forecasting

The bid assessment and allocation process deals with the funding requirements for one year at a time related to individual structures and schemes.

There will still be the need for overall strategic planning similar to the current 15 year programme. This will pick up special issues such as post tensioned bridges, fatigue of steel components, etc. and produce an overall programme of maintenance. It will then be possible to check that the annual bids and outturns satisfy the overall objectives. For instance, if it was assumed that £80 million would be spent on rehabilitation in one year, and only £40 million is proposed, it will necessitate an examination as to whether there will be a resulting backlog in the future.

Structures database

The Agency at present uses the structures database NATS to record all structures inventory data, inspection and assessment information and bids, allocations and outturns. It is a single entity and is not user-friendly by today's standards.

The database has to serve a number of management activities. It is very likely that the procedures for most of these will need to be modified from time to time, at least in parts, as new issues appear and new developments take place. In order to make the central database capable of dealing with these changes without undergoing major amendments, it is proposed that a new database entitled *Structures management information system* (SMIS) is developed in a modular form, with pre- and post-processors linking the modules. Data from NATS can be used as a component of SMIS, and BAPS can be the first new module to be added. Apart from improving all the facilities that NATS contains, the proposed system will have two new features, the management plan database MPLAN and the condition database COND. MPLAN will contain all significant programming information including the dates at which inspections, assessments and other programmed activities are due for individual structures. COND will store the condition and safety indices.

Significant items of output from SMIS will be displayable through the structures part of a proposed network display system *State of the network* (SON).

Programme timetable

The main outputs, with respective target dates, are as follows

- new cost effective inspection procedures (five manuals) — June 1999
- three new standards to assist with whole life performance-based bidding — March 1999
- annual strategic maintenance programme review — April each year
- new bid assessment and prioritisation system and probability-based allocation procedure — October 1999
- annual reporting of condition and safety indices through state of the network report — April each year
- structures management information system — October 2000

□ new quality assurance, audit procedures and performance indicators for structures management activities — October 2000.

A number of these items are already in progress through various projects. A long term programme of maintenance expenditure covering 40 years has been carried out and software projects for developing SMIS are in hand.

Conclusions

The overall purpose of proposed procedures is to enable the structures management process to fulfil the following objectives

(*a*) to provide relevant information for answering the following questions
 (i) Why are the maintenance activities necessary?
 (ii) What are the likely consequences if the works are not carried out?
 (iii) Are the funds being used effectively?
(*b*) To obtain better value for money through whole life costing of work options
(*c*) To make safety the primary objective while minimising disruption to functional use
(*d*) To take into account long term sustainability in respect of future logistical and funding requirements based on strategic forecasts of maintenance needs.

The proposed developments are tailored to the Highways Agency's specific needs, although the principles behind them are clearly relevant to other bridge management systems. While greatly benefiting from the developments that have been carried out by others in a number of countries in recent years, the proposed methodologies incorporate some important new concepts.

Acknowledgements

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The Danish bridge management system DANBRO

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Philosophy

Bridge management involves a number of activities, such as the collection of inventory data, inspections, assessment of damaged structures, management of special transports, allocation of funds for repair and maintenance, etc., all with the purpose of ensuring traffic safety and maintaining the bridge stock in the desired condition at the lowest possible cost. The purpose of a bridge management system is to assist in the management of these activities.

When managing bridges, decisions have to be made all the time. As a help in making the right decisions, guidelines should be set up and followed, but normally it is not possible to set up objective rules for these decisions. Therefore, bridge management cannot be performed by computer systems. The human mind is involved all the way. And consequently, a complete bridge management system is not just a computer program. It is

- ☐ a set of interrelated activities for handling bridges
- ☐ a set of codes and guidelines for the activities
- ☐ a database holding data resulting from the activities
- ☐ a set of computer tools for processing the data in the database.

In the development of the DANBRO database system, it has been kept in mind that a comprehensive computer system is not the goal. The goal is to assist the decision makers and the administration in doing their job. DANBRO comprises guidelines and management tools to be used at all levels within the field of bridge operation and maintenance

- ☐ the executive level
- ☐ the planning level
- ☐ the administrative level
- ☐ the maintenance level.

A golden rule in deciding the amount of data to be stored has been that the data necessary for the administration are collected and kept up-to-date. And no more than that.

In fact, the development of DANBRO was decided when it was realised that the existing bridge databases contained a lot of data which nobody used and which were very expensive to keep up-to-date.

The structure of DANBRO

As a consequence of the realisation that different bridge owners have different needs, the structure of DANBRO is modular. Of course, there are inter-relations between the individual modules, but in general each bridge administration can choose which modules to implement. And within the administration, each individual user may be given access to selected modules, in order to distribute the responsibility for data. Each module of the complete DANBRO system is described below.

Inventory

All documentation regarding the design and construction of the bridges is stored on paper and microfilm in the archives. In order to facilitate the daily management, selected data for which easy access is required are registered in the DANBRO inventory module.

The inventory contains

- *administrative data*: Road register, bridge identifications, etc.
- *technical data*: Bridge types, dimensions, materials, etc.
- *passage data*: Data on roads, waterways, etc. including clearances and load carrying capacity classes for the bridges
- *archive references*: Information on the contents of the archives
- *chronological overview*: A list of important events in the 'life' of each bridge, such as construction, inspections, rehabilitation works, including the most important data from each event.

The inventory module contains report programs to print out selected data on a single bridge or on a selection of bridges, and to provide various statistics on the bridge stock. The facilities of the inventory module are also used for the administration of special transports (excessively high or heavy vehicles).

The database may be connected to an electronic bridge map which enables the user to find data for a specific bridge by pointing it out on a map shown on the screen, or to show on the map all bridges with specified properties, e.g. weak or narrow bridges.

Principal inspection

The Principal Inspection is the key activity in the monitoring of the condition of the bridges and is a visual inspection of all visible parts of the bridge. The purpose is to maintain an overview of the general condition of the whole stock of bridges, and to reveal significant damage in due time, so that rehabilitation works can be carried out in the optimum way and at the optimum time, taking safety and economic aspects into consideration.

For the inspection, the bridge is divided into 15 standard components, one of which is 'the bridge in general'. For each standard component, the following is registered

- a condition rating, ranging from 0: 'No damage' to 5: 'Ultimate damage/complete failure of the component'
- a short description of significant damage
- need for routine maintenance/cleaning (yes/no)
- need for Special Inspection (technical and economic analysis or only economic)
- need for repair works (type of work, extent, cost estimate, execution year).

Damage which does not require remedial action is not described, and in any case the damage description is brief. A Special Inspection including a detailed damage registration will always be carried out before major repair works, so there is no need to do this at the Principal Inspection.

As a help in making the cost estimate for repair works, a catalogue of unit prices of common works is provided (see section on the price catalogue)

An essential part of the Principal Inspection is to determine the year of the next inspection for the individual bridge. If a bridge is in bad and still deteriorating condition, the interval may be as short as one year. If the bridge is new and in good condition, the interval may be up to six years. This is part of a general policy of concentrating the effort on the areas that really need attention, thus getting the most out of the limited funds for bridge maintenance. Each year a list is printed, specifying the bridges to be inspected in that year.

The main output from the Principal Inspection module is

- statistics on the general condition of bridges
- cost estimates for all rehabilitation works five years ahead
- list of bridges to be inspected in a specified year
- list of bridges that require Special Inspection
- statistics on the performance of routine maintenance.

Lately, the Principal Inspection module has been supplemented with the possibility of including digital photos. They may be taken from video tapes, scanned from paper copies, or taken by digital cameras, and they can be shown on the computer monitor and printed with the inspection reports.

For very large bridges, tunnels, ferry berths and other special structures for which the fixed division into standard components is not applicable or sufficient, a special module has been added. In this module, components can be specified in a four-level hierarchical structure. For each component and component level, condition ratings and cost estimates for routine works and rehabilitation works can be registered and summarised.

Routine inspections and routine maintenance

As the Principal Inspections are carried out at intervals as long as six years, they are far from frequent enough to monitor the safety and the day-to-day serviceability of the bridges, or for planning the routine maintenance. Therefore, frequent routine inspections have to be carried out as well.

The routine maintenance is organised by the resident bridge engineer. As a tool for planning and monitoring the work, the Routine Maintenance module was developed. The use of the module ensures that the maintenance works are carried out systematically, in accordance with the needs, and using the proper materials.

The module comprises a list of standard bridge components (a more detailed list than that used for Principal Inspections). For each component, standard works are linked with corresponding unit prices and — when relevant — specifications for the materials to be used. For each bridge, the principal dimension (length or area) of each standard component is registered. From the module, a list of possible maintenance works for each bridge is printed. At the routine inspection once a year, the need for each work is registered.

Based on these registrations, work orders for each individual bridge are automatically generated and printed, stating the type of work, the extent and the materials. Having completed the work, the maintenance crew foreman signs the work order, including the extent of the work executed and the date of execution. The signed work orders are collected by the resident engineer as a means of monitoring the progress of work.

The module comprises facilities for tendering out the routine maintenance works. Based on the needs for routine maintenance registered at the extended routine inspections, the program prints out bills of quantities for the tendering. Later on, when a contractor has been selected, work orders for each individual bridge are printed.

Special inspections

Normally, Special Inspections are initiated at the Principal Inspection. When the inspector is not certain about the cause of damage, the extent of damage, or the proper rehabilitation method, a Special Inspection is recommended. Such inspections are carried out by engineers with experience in deterioration mechanisms, inspection methods and rehabilitation design.

The Special Inspection comprises destructive and nondestructive tests carried out in situ, as well as collecting samples for laboratory tests. Based on the results of these tests, the state of damage is assessed as well as its probable future development, and various different rehabilitation strategies are evaluated.

An economic evaluation is carried out, comparing the relevant strategies and assessing — for each strategy — the consequences of postponing the strategy. (These consequences are to be used for the priority ranking, see below).

The ‘net present value method’ is used for the economic evaluation, giving the net present value of each strategy and of each postponed strategy, including direct and indirect costs to society within the next 25 years. The strategy with the lowest net present value is the economic optimum for the bridge, and will normally be the one proposed as the conclusion of the inspection report.

Optimisation of rehabilitation works

In Denmark — as in most countries throughout the world — the funds allocated for bridge maintenance are not sufficient for carrying out all the proposed works. Therefore, some sort of priority ranking must be made.

As one of the main purposes of a bridge management system is to ensure the safety and serviceability of the stock of bridges in the most economic way — on the long view — the economic analyses from the Special Inspections are taken as the basis for the optimisation. This means that it is not necessarily the bridges that are most severely damaged that are given high priority. The bridges selected for repair are those for which the economic consequences to society of postponing the works are the worst. These consequences — the extra cost of not allocating sufficient funds — may be additional direct costs caused by a larger amount of repair works, or they may be road user costs, e.g. in case the bridge must be temporarily closed or a weight limit applied.

Input to the Optimisation module are the budgets for the coming five years, the discount rate and the data from the economic evaluations of the Special Inspections. Through an iterative process the program finds the set of strategies — one for each bridge — for which the two criteria are met

- ☐ The total cost estimate lies within the budget each of the first five years.
- ☐ The economic consequences (the extra costs) are the lowest possible.

After the automatic optimisation procedure, the result is studied carefully, and the choice of strategies may be altered, taking into account factors that are not included in the automatic process, because they cannot be expressed in terms of money, e.g. aesthetics or environmental aspects, or co-ordination with other works on the same road.

Budget and cost control

DANBRO comprises a module for managing the flow of money used for bridge maintenance. It controls the development of budgets for the individual rehabilitation works as well as the total for all works, thus keeping track of the budgets throughout the fiscal year.

Long term budgeting

In addition to the five-year budgets created by the Principal Inspection module, there is a need for long-term estimates for bridge rehabilitation. For this purpose, the Long Term Budgeting module has been developed.

The general idea is that average repair intervals and corresponding average repair costs, as well as average service lives and replacement costs are registered for the standard components of the bridges. For each bridge is registered the year of construction and the type and dimension of all standard components. Based on these data, the program calculates total future budgets. Of course, the uncertainty is relatively large, but the purpose of the module is only to indicate the trend of the costs, and for that, the accuracy is considered sufficient.

Price catalogue

For several tasks in bridge management, cost estimates for repair works are needed. As a tool for collecting and updating unit prices of common rehabilitation works in a systematic way, the DANBRO Price Catalogue was developed.

The basis is a standard Bill of Quantities which must be used for all bridge repair works. The standard Bill of Quantities is composed of commonly used items. A description of each item ensures that each standard item comprises the same operations in all repair projects, making it possible to compare unit prices from different projects.

The Price Catalogue module is used for preparation of the Bill of Quantities for a specific project, and when the quantity of each item has been inserted, the program prints the Bill of Quantities for the tender and calculates the client’s estimate. After the tendering, the unit prices from the two lowest bids are entered in the database.

The standard repairs used in the Principal Inspection module are composed solely of sub-items from the standard Bill of Quantities. In this way, automatically updated cost estimates are always available for the standard repairs, and thus for the five-year budgets derived from the Principal Inspections.

Administration of heavy transports

An important part of any bridge management system is the administration of heavy transports. In Denmark, the administration is based on assigning classes to the bridges and to the vehicles. The normal class of a bridge is related to the heaviest of a series of standard vehicles — each assigned a vehicle class — that can pass the bridge with the prescribed safety.

In addition to the normal class, the bridges are assigned special classes based on the vehicle being alone on the bridge, travelling on a specified lane on the bridge, and with reduced speed. The class of a specific vehicle is determined by comparing the effect of the vehicle on several standard bridge spans with the effects of the series of standard vehicles.

As a general rule, a vehicle may pass the bridge if the class of the vehicle is not greater than the class of the bridge. DANBRO will check this criterion for a specific vehicle on a specified route.

Experience of use in Denmark and abroad

Denmark

Since the implementation of the first modules in 1988, DANBRO has been used by the Road Directorate and the counties of Denmark. In addition, a substantial number of municipalities are using a simplified version of DANBRO, comprising the basic modules only.

The general impression is that DANBRO fulfils its main purpose of supplying the people responsible for bridge administration and maintenance — at all levels — with the information required to perform their tasks, thus making it possible to ensure traffic safety and the optimum maintenance of the bridge stock.

The development of DANBRO is led by a task group, composed of active users from all levels of the administration, and the programmers. Revisions of existing modules as well as the development of new ones are initiated and followed by the group.

It is considered very important that the development is governed by the users, i.e. the personnel responsible for the bridge operation and maintenance at all levels — and not by software suppliers. This is probably the main reason for the position DANBRO has obtained as the dominant bridge management system for the administration of road and highway bridges in Denmark.

Through the DANBRO task group and through meetings to which all users are invited, many suggestions for minor adjustments as well as major innovations are received. This is taken as a proof that DANBRO is really used intensively as a tool in the daily management of bridges.

DANBRO in other countries

Bridge Management Systems based on DANBRO have been implemented for the national highway administrations in Saudi Arabia, Mexico, Colombia, Honduras, Croatia and Malaysia.

When implementing a bridge management system — or any management/administrative system — it should be borne in mind that the computer programs form an integral part of the bridge administration and must reflect the organisation and the rules of the society in which it is to operate. It is not considered realistic, or advisable, to change the organisation and traditions to suit a foreign computer program, and therefore it is necessary to make the programs suit the administration.

In all cases, the implementation must start with a thorough analysis of local needs. This phase should end up with the specifications for the local bridge management system (BMS). The programs are changed accordingly, and a training programme is set up for the coming users at all administrative levels, so that they will be fully capable of using the system in all aspects of their work.

The amendments to programs and manuals, training and initial use of the whole system must be carried out in very close collaboration with the users. Therefore, the implementation of a DANBRO-based bridge management system is normally carried out in a project with a duration of one or more years. The implementation is not considered complete until the BMS has proven its capabilities through routine use.

As was the case in the development of DANBRO in Denmark, it is normally advantageous to start the implementation of a bridge management system with the basic modules, Inventory and Principal Inspection. When they are in use and work as desired, the addition of supplementary modules can be considered. In that way, the local users are better able to define the exact requirements for the coming modules, and the probability of success is greater.

Current development

Demands, wishes, and technical possibilities are constantly changing, and consequently, development is still on-going. The following major amendments are being carried out presently or are planned to be carried out within the coming year.

Conversion to the Windows environment

Most of the DANBRO modules have been converted to a Windows 95/Windows NT version, and the remaining modules are under way. The missing modules are mainly those used by the Road Directorate only, so for the average user, the Windows version is complete.

As a part of the conversion to Windows, a comprehensive help facility is introduced, giving assistance in the proper use of the computer programs as well as access to an online version of the technical section of the manual, explaining the significance of each data field and giving the rules and guidelines applying to the data and functions in the actual DANBRO screen from which the help system was activated.

Contract administration

The Contract Administration module is intended for use in the administration of rehabilitation and maintenance contracts. For rehabilitation contracts, it will be used for administering payments on account and the final bill, based on keyed-in quantities for each sub-item in the bill of quantities. For maintenance contracts, bills for payment on account will be generated, based on quantities for each bridge, registered by the contractor. Facilities for the administration of securities and guarantees for the contracts will be included as well.

Sharing data

For the administration of special transports (high and heavy), it is necessary to register vertical clearance and carrying capacity classes for all road and railway stretches passing the bridge. Therefore, all passages on and under the bridge are registered with route identification and kilometre.

In the previous versions of DANBRO, the identification of a bridge was composed of the identification of the primary (the most important for the actual administrator) passage and a serial number. In the Windows version, the identification of a bridge is an independent registration number which is used by the programs as the key to the bridge data. For the user, the bridge identification is the same as before, but the new internal key makes it possible to easily redefine which passage is the primary one on which the bridge is numbered.

Consequently, different bridge administrations with interests in the same bridge can share the same set of data on the bridge, each seeing the data from its own point of view. For example, a railway passes under a bridge carrying a highway. For the railway administrator, the bridge is number 35 on the railway, and the bridge is called an underpass under the highway. For the highway administrator, the bridge is number 17 on the highway, and is called an overpass over the railway.

Facilities are planned to automatically redefine all bridge identifications depending on who is logging on to the system, enabling different administrations to use exactly the same database and programs.

Conclusions

The success of DANBRO — in Denmark and abroad — is mainly due to the fact that it is a system developed by bridge engineers to be used by bridge engineers, and that it has been developed to fulfil precisely defined purposes. Much emphasis has been put on making the computer programs an integral part of the bridge administration.

This is probably the main reason why most users find that DANBRO is a help in their job. For the same reason, you are not likely to succeed if you buy a management system 'from the shelf' at a software supplier. It may be developed for a different kind of administration or different kinds of structures. However, the basic principles are so widely applicable that DANBRO can be adjusted to suit the exact needs of any bridge administration.

The Finnish practice and experience regarding bridge inspection and management

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Introduction

A computer-based bridge management system (BMS) was designed in Finland by the Finnish National Road Administration (Finnra) to assist in high-level bridge policy, long term planning and programming of investments on the network level and to help bridge engineers when preparing annual work programmes to reach the accepted condition level on short term.

The system applies probabilistic deterioration models to find a condition distribution of the bridges that minimises maintenance and rehabilitation costs for the existing bridge stock and deterministic repair and reconstruction indexes to organise individual bridges for annual programming.

Finnra started the bridge management system development for its 10 596 bridges and 2757 culverts (span length $\geq$ 2·00 m) with the total length of 308 km and the deck area of 3·1 million m^2 in 1986. The network level system has run as a prototype since 1996,[1] while the project level system was taken into use in the nine road districts by the end of April 1998.

The goal of this management system is to provide a reliable support tool in decisions related to fund allocation for maintenance, rehabilitation and replacement (MR&R) of existing bridges. The system will minimise MR&R costs while keeping the bridges safe and on a required level of service.

The aim is to find the economically optimal long-term condition distribution of the bridge stock within the safety and minimum service levels. The long term optimum solution is a combination of the optimal condition distribution and the optimal repair measure distribution.

The system will be employed by the central administration of Finnra and its nine districts to assist in high-level bridge policy, long term planning and programming of MR&R investments, and short-term evaluation of bridge repair needs and their costs. The work schedules and recommended bridge repair programmes are prepared in the districts.

The elements of bridge management system

Bridge database

The whole bridge management system is based on a thorough bridge inspection and condition evaluation. The damages and deterioration detected during the inspections, their exact location and extent are recorded. Also, information on the effect of the damages on bridge bearing capacity, on repair urgency class and the inspector's proposals for repair measure and their costs will be described and recorded.

All this information is stored in the bridge database together with bridge structural, administrative and traffic data. Also historical data and information on previous repairs and their realised costs are gathered for further research and bridge age behaviour modelling.

Thus the database is completely reorganised and improved for the bridge management use. The first new version of the bridge register was implemented as a PC-version in every road district in 1990. A second, improved multi-user client-server version was in the districts' use in 1994. Today, a third generation database with new demands of the computer environment and bridge management side is under development and will be in use in 1999.

The elements of the bridge management system are described in Fig. 1.

The network level bridge management system

The network level bridge management system consists of two parts: the long term module to find the ideal optimal condition distribution for the bridge stock and the short term module to find out how to get the bridge stock from the present condition distribution to the optimal distribution.

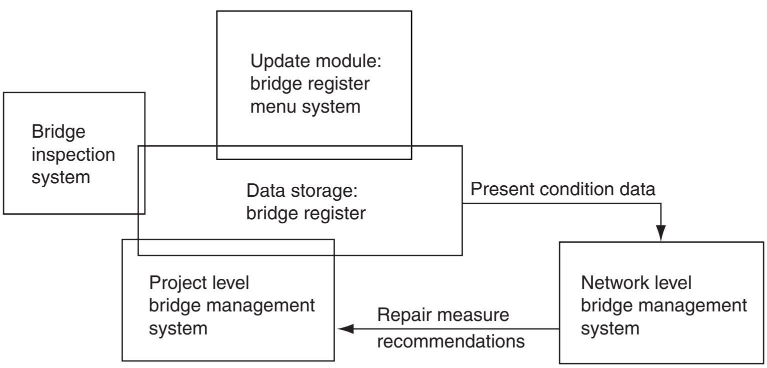

Fig. 1. Connections between elements of bridge maintenance system

The long term analysis is based on the general idea that the bridge stock has an optimal condition distribution. This optimum is intermediate in the following sense: keeping all bridges in an excellent state at all times would be excessively expensive and, conversely, allowing a severe deterioration of the bridge stock would cause expensive major repairs. Somewhere in between there is an optimum where the bridge stock can be kept on the same condition level year by year with the smallest possible amount of funding, yet adhering to the safety level and service requirements.

The optimal condition distribution corresponds to a certain optimal set of repair measures. These repair measures would, in the ideal case, be applied to the same extent year by year, although naturally to different individual bridges. The set of optimal repair measures and the extent of each, i.e. the optimal repair measure distribution, will ensure the permanency of the optimal condition distribution of the bridge stock.

The short term analysis provides an economically optimal way to reach the long term optimum condition distribution during the next few years. There are separate short term solutions for each coming year. Each short term solution represents a step closer to the long term optimum.

In reality the long term optimum will change somewhat year by year due to changes in the variables that influence it. Changes can be expected in repair method costs because of new repair methods and materials. The road policy could change and the level of service standards with it. New improved deterioration modelling could affect the optimum, etc. The network level system offers the possibility for 'what-if' experiments with respect to the safety and minimum service level policy, repair measure costs, budget limits and other variables. The system will also provide detailed information for future bridge designers on the deterioration mechanisms of bridge elements and on the life-span cost of different bridge types.

The project level bridge management system

The project level system, which deals primarily with individual bridges, uses the results from the network level system to decide on the repair measures in individual repair projects. This project level system is the key tool for everyday bridge repair planning in the road districts. It is an interactive computer program that helps the bridge engineer to plan and schedule the repair projects for individual bridges based on the recommendations from the short term model and the damage data in the database.

The project level system also includes a life-cycle-cost-analysis. This analysis compares the repair measure combinations, recommended by the network level, with each other instead of using the traditional calculation method with its repair measure combination given beforehand. Thus the most advantageous repair measures for an individual bridge are found out.

This gives the bridge engineer the flexibility needed and a possibility to opt for repair measures which result in the minimum optimal total cost during a bridge's life span.

Mathematical approach on network level

The purpose of the network level system is to minimise the total of the yearly bridge repair costs under given restrictions by carrying out the right repairs at the right moment in the life-span of bridge components.

The mathematical solution uses a set of probabilistic Markov chain models to predict deterioration of the various bridge structural members in the bridge stock. Together with data on possible repair measures for damage of every description, the respective cost of these measures, minimum bridge condition requirements and budget limits, linear programming (LP) models can be formulated and solved by computer to yield a recommended long-term optimal solution for the condition state distribution of the bridge stock (Fig. 2). The LP models also give the distribution of the repair measures required each year to maintain the optimal cost-effective state year by year.[2]

The short term model, Fig. 3, uses a modified method to recommend repair measures for several consecutive years starting with the present, to move the current distribution of condition states in the bridge stock towards the long term ideal situation, minimising cost along the way.

The optimising long and short term models will be used to study different repair strategies and to support budget allocation decisions on both national and district level. In addition, the network level results are a key input to the project level system.

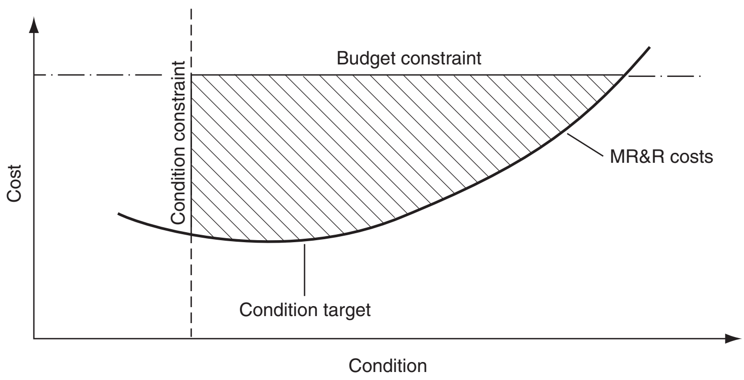

Fig. 2. Calculation of long-term target condition

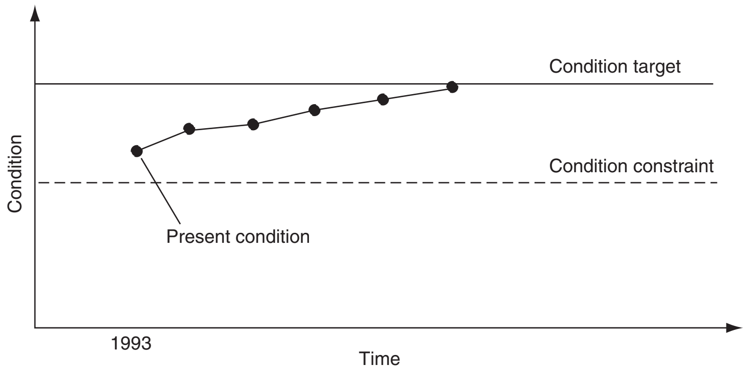

Fig. 3. Calculation of recommended annual repair measures

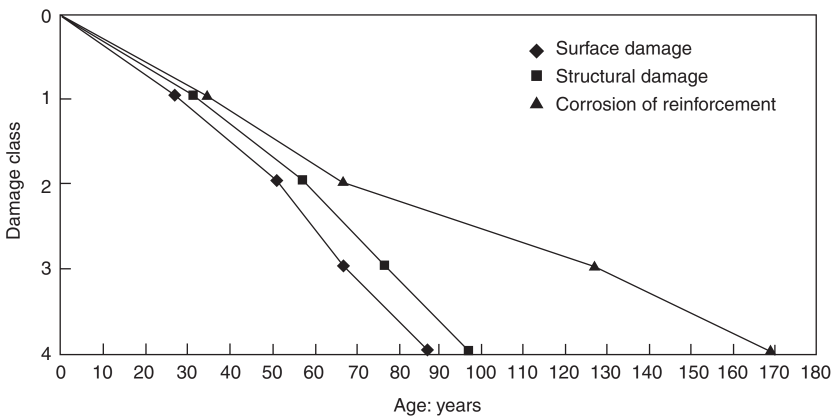

Fig. 4. Deterioration of precast prestressed concrete superstructures on salted main roads (Delphi study)

Bridge deterioration and age behaviour modelling

Deterministic deterioration approach

The modelling of bridge deterioration acceleration and age behaviour is based on the information of damages gathered during the inspections. Because there is still not enough information of this kind, opinion surveys (Delphi studies) or expert evaluations are also carried out in order to setting up the first age behaviour curves and models. One result from these expert evaluations is given as an example in Fig. 4.

The present condition of the bridge stock is calculated from stored data of the actual observed damages on the bridge structures. Each damage is related to a specific structural part, and each part can have several types of damage. For each damage the bridge register contains severity, extent and location of the damage in addition to several other data. The severity is classified on a scale from 0 (no damage) to 4 (serious damage), according to rules given in the Finnra *Bridge inspection manual*.[3] An example is given in Table 1.

Probabilistic deterioration approach

The network level optimisation algorithm works with populations of bridge structural parts that are similar in their construction and use. Superstructures, substructures, surfacing structures and bridge furnishings are treated separately. A breakdown of building material, bridge type and construction technique is also made. Altogether 25 categories of bridge items are treated in the system.[4–6] There are two environmental categories: salted main roads and other roads, resulting in a total of 50 categories.

The condition of bridge structural parts is evaluated with respect to one, two or three damage groups. Most structural parts were evaluated with respect to 'surface damage' and 'structural damage'. All deck structures were evaluated with respect to 'water leakage'. The damage groups may depend on each other, i.e. the process of a particular damage group may be influenced by the other damage groups occurring at the same time. The Markov models account for the shortening of service life due to damage group interdependency.

The deterioration models, repair measure models, repair measure costs as well as budget and condition constraints may be formulated as a linear programming model. Each category of bridge items corresponds to one LP model. The objective is to minimise repair costs due to given condition constraints or to minimise social cost, i.e. the sum of repair and bridge user costs, for those LP models where user cost can be modelled. If an overall budget constraint is given, it can be necessary to relax some condition constraints. This is done by giving each LP model a weighted proportional budget constraint which will govern the relaxation of condition constraints in models for which user cost modelling is not used.

Repair models

The *Bridge Inspection Manual* recommends repair measures for each damage class and each type of structure (Table 1). These repair measures or repair measure levels (A, B, C and D) are combined with the above mentioned damage groups in the network level Markov repair models.

On the project level the inspector gives his or her judgement to a repair measure recommendation regarding the observed damage on an individual bridge. Every repair measure recommendation is saved in the database together with its expected cost. This information is further used to convert the network level repair measure recommendations to match individual bridges.

Table 1. Classification of concrete surface deterioration and recommended repair procedures

Damage class	Damage	Type of structure					
		Normal reinforcement		Prestressed reinforcement		Special stress	
		Bending	Other	Bending	Other	Edge beam	Water table range
1	The surface of the concrete shows map cracking. The surface cement mortar has come loose, but no coarse aggregate is visible.	A	A	A	A	–	–
2	The depth of deterioration or wear is 0 to 10 mm. The coarse aggregate is visible.	A, B	A	B, C	B	–	–
3	The depth of deterioration or wear is 0 to 20 mm. Cement mortar from around the aggregate has come off.	C	B	D	C	B	C
4	The depth of deterioration or wear exceeds 20 mm. The coarse aggregate has come loose and the reinforcement may be visible.	C, D	C	D	C, D	C, D	C, D

Key
A Surface treatment may be considered. A special inspection shall be undertaken, in order to determine the degree of reinforcement corrosion as well as the chloride concentration and depth of carbonation. A specification shall be drawn up.
B Local damage is repaired according to the guidelines concerning patching of concrete, generally by using patching mortar or by ejection. Larger areas are treated as set out in point C
C. The damaged concrete is removed by chiselling or with a jack hammer and a new concrete cover, better suited for the conditions, is made, generally by applying gunite or by casting. A specification shall be drawn up.
D All damaged concrete is removed by chiselling or using a jack hammer and repaired using casting or applying gunite. The reinforcement is repaired to a necessary extent. Calculations are used to determine the need for additional strengthening of structures and possible service limitations. A special inspection is carried out and a repair plan is drawn up. In the case of prestressed structures, the effect of the damage on tendons and cables must be determined.

Bridge condition description on project level

To arrange the bridges in an urgency order in the work programme respecting the repair recommendations given as the accepted condition target, an index describing the repair needs of individual bridges was used on the project level. This repair index is a function of the bridge structural part's estimated condition (*EC*), damage class (*DCL*) and the repair urgency class (*UCL*). All these measures are given by the bridge inspector and saved into the database.

The score is calculated for every registered damage. The most serious damage with maximum score will be taken as such, the scores of other damages are multiplied by a reduction factor. The final number of points, the repair index (*RI*), is the sum of scores received as described above. An importance factor could be used when taking into account the seriousness of the damage compared with the other damages of the bridge.

$$RI = \mathrm{Max}_i(EC_i \times DCL_i \times UCL_i) + \gamma\left[\sum_{j,j\neq jmax}(EC_j \times DCL_j \times UCL)_j\right] \qquad (1)$$

The repair index distribution of the bridge stock in Finland at the end of the year 1997 is described in Fig. 5.

The given condition target from the network level long term analysis refers to a repair index level, for example $RI \geq 140$, where the bridge structural part's estimated condition equals 1 (quite good), damage class 4 (serious damage) and the repair urgency class 1 (must be repaired during the next year). Thus the total bridge deck area and the individual bridges, which need repair, will be known.

The reconstruction index is used to find bridges which have functional deficiencies like narrowness or inadequate load carrying capacity or the

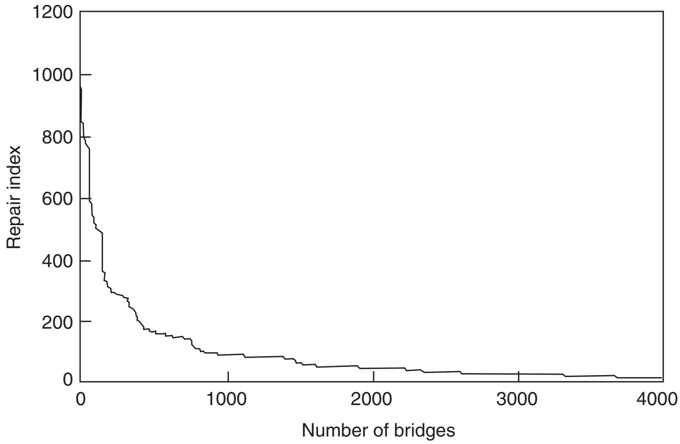

Fig. 5. Repair index distribution of bridge stock

bridge has reached the end of its economical and functional life. Several factors and bridge properties result in the sum of points, which describe the necessity of replacing the bridge with a new one.

The life cycle analysis allows the bridge engineer the possibility of comparing the effects of different management strategies on the remaining economical and functional life of the bridge under a given budget frame. In this analysis also deterioration on the project level will be used.

A stock of reference bridges

A set of about 120 bridges has been selected for regular, special observations to improve knowledge of bridge age behaviour and durability. This reference group consists of bridges of different bridge material and type, age and condition, geographically situated throughout the country.

The research programme consists of studies of bridge materials, repair materials and repair methods. Concrete chloride content and carbonation are of special interest. Samples are analysed at the Technical Research Centre of Finland. The information gained is used to improve age behaviour modelling in the management system.

The reference bridges are also used to compare bridge maintenance costs and life span costs for different bridge types. The economical and structural suitability of different bridge types and materials for various purposes are analysed to improve future bridge design.

Further research

The bridge management system, i.e. the bridge register, the network level bridge management system and the project level bridge management system together make up a comprehensive tool for Finnra. To develop it further means that still more effort must be put in the age behaviour research of bridges. The quality of the models also needs to be improved.[7] Also, to really optimise the MR&R costs more experience in the road user costs is needed: the reality and the formulas describing user costs on the network level must be better fitted together.

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A risk-based maintenance strategy for the Midland Links Motorway viaducts

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Introduction

The paper includes a brief history of the viaducts and the effect of acute chloride induced corrosion on the reinforced concrete crossbeams. Reference is made to the ten year *whole life cost repair and maintenance strategy*. This strategy was developed in the late 1980s to identify a cost effective repair and maintenance programme. The duration of the repair programme was to be ten years and repair works commenced in 1990. The strategy was designed to be flexible and required an annual review of the assumptions and data to identify future programmes of work. After five years it became evident that the repair programme could not be achieved within the ten years without a considerable increase of resources.

There was a need, therefore, for,

- a rational justification for increasing the duration of the repair programme
- a means of identifying priorities for repair.

To address these issues a new strategy has been developed which identified relative intervention dates for the repair of the viaduct crossbeams. The calculations were based both on published codes of practice for design and assessment and the use of reliability analysis to define the uncertainties in the assessment process.

History and form of the Midland Links

The Midland Links Motorways form a vital part of the national road network, connecting the M5 and M6 motorways through the heart of the West Midlands conurbation. To avoid extensive demolition in the heavily built up area, much of the construction involved the use of elevated sections of motorway carried on viaduct. In all, 21 km of the 37 km of the Midland Links Motorways are carried on viaducts. Fully opened to traffic in 1972 the Links carry traffic flows in the range 120 000 to 160 000 vehicles per day of which approximately 30% are heavy goods vehicles.

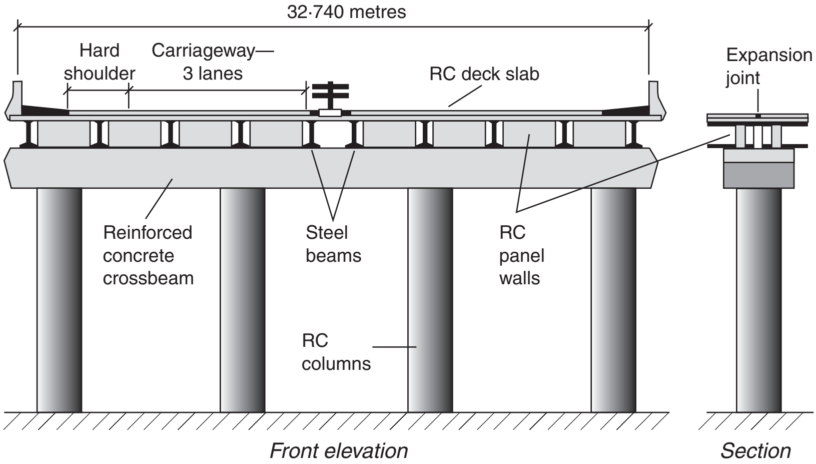

Fig. 1. Typical viaduct crossbeam arrangement

The form of the viaducts was partly dictated by the terrain and obstructions over which they pass including numerous roads, railways, canals and rivers as well as industrial premises and open land. The structural elements were standardised as far as possible to give economies in both cost and construction time. The longitudinal spans vary from 15 m to 46 m with over 90% of the spans being less than 21 m. The standard viaduct construction comprises 'simply supported' composite decks consisting of reinforced concrete slabs and steel beams. There is an expansion joint between each deck span, and the deck beams are supported on a reinforced concrete substructure by 'free sliding' steel bearings. A typical substructure is shown in Fig. 1. Within the Midland Links viaducts there are some 1200 crossbeams.

The problems

Extensive failure of the buried expansion joints occurred about four years after opening. The buried joints originally specified, and in common use at the time, could not withstand the arduous conditions of the viaducts and it is the legacy from the leaking deck expansion joints that has, and will for several years to come, present an engineering and economic challenge. Detailed investigations, put in hand in the early 1980s, identified high levels of chloride contamination in the reinforced concrete substructures arising from road salts used at times of de-icing. Both general and pitting corrosion was widespread and on some viaducts delamination of the cover concrete was identified. Carbonation of the concrete was found not to be significant.

In summary, the initial investigations identified that the principal mechanism of deterioration was chloride induced corrosion of the reinforcement resulting in loss of reinforcement section together with cracking and spalling of the cover concrete.

Initial strategy — whole life cost repair and maintenance strategy

The scale of the deterioration on the Links and the number of structural elements affected was such that it was imperative that a process of rational selection of the most appropriate repair and maintenance options was established. A strategy was therefore developed to enable a cost effective repair and maintenance programme to be identified which also maintained the integrity of the structures.[1] This was based on a system of crossbeam classification, an assumed global deterioration rate and a number of repair and corresponding maintenance options. These factors, together with assumptions on the suitability, durability and effectiveness of particular repair options to certain crossbeam classifications were used to determine the priority for repair within the ten year timescale. Beyond the initial ten years it was assumed that the crossbeams would require only minor maintenance for the following 30 years. It was thus possible to produce a 'whole life cost' repair and maintenance strategy for an overall timespan of 40 years.

A review of the assumptions was made annually together with an update of the cost model to bring it into line with current contract prices.

The need for a strategy review

Year 1 of the ten year repair programme was 1990. By the end of 1995 over half of the crossbeams had been repaired. However, it was also evident that with the more difficult and non-standard crossbeams to repair and competing demands on financial resources, the programme could not be achieved within the ten years identified. The annual update in 1996 noted that unless additional resources could be made available the programme would be prolonged.

The predicted condition state of the crossbeams depended on its current condition and the assumed rate of deterioration. By their nature these predictions were uncertain and the degree of uncertainty increases with time. Extending the repair programme beyond the initial ten year period introduced additional unknown levels of risk. Therefore, it was imperative that an indication of the effect of a prolonged repair programme was identified to ensure the longer term integrity of the viaducts.

Assessment

Structural assessments had been undertaken as part of the initial strategy to ensure the structures had sufficient reserves of strength to give confidence that the ten year repair programme involved a low risk of failure through on-going deterioration. These assessments have been complemented by the need to assess all of the Midland Links viaducts as part of the Highways Agency's Stage II–15 year bridge rehabilitation programme.[2]

While only a few structures were shown to be inadequate by the Stage II assessment process, the exercise demonstrated the vulnerability of the structures to deterioration and exposure to risk by the relatively subjective choice of condition factors.

As a large number of structural elements were exposed to such risks it was essential that a more detailed assessment was undertaken which took account wherever possible of bridge specific parameters. In addition, it was necessary to determine how the strength characteristics were reduced through the process of deterioration with time.

Previously published papers had identified five levels of assessment[3]

- Level 1: assessment using simple analysis and requirements and methods
- Level 2: assessment using more refined analysis
- Level 3: assessment using better estimates (bridge specific design values of load and resistance, using probabilistic estimates where possible)
- Level 4: assessment using bridge specific target reliability
- Level 5: assessment utilising full scale reliability analysis.

Risk-based maintenance strategy

The development of the new strategy began in the summer of 1996, with the aim of having a workable repair programme for the 1998–99 financial year. The main stages of the development process are illustrated in Fig. 2.

Qualitative assessment

An initial screening process defined which structures were to be considered in more detail. The hazards to the various structural elements were then identified by considering the ways in which the elements could fail. All structures were considered in the initial screening exercise, categorised as

- satisfactory strength, not deteriorating
- deficient strength, not deteriorating
- deficient strength, deteriorating
- satisfactory strength, deteriorating.

Structures which had adequate strength but were not deteriorating were deemed to require only routine maintenance and so were excluded from the strategy. The criteria for determining the adequacy of the strength was taken as the results of the Stage II assessments. Structures which failed the

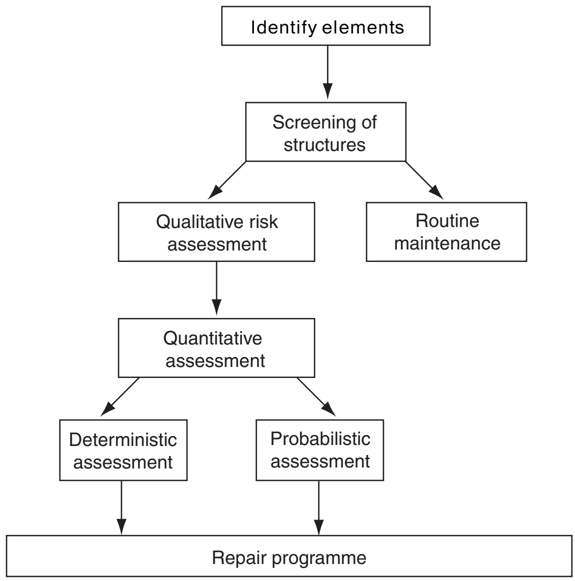

Fig. 2. Flow chart of activities

assessments were deemed to require strengthening before December 1998 and were therefore excluded from the reliability maintenance strategy. Structures which had been shown by the assessments to have adequate strength, but deteriorating, were considered in more detail in the qualitative and quantitative stages of the strategy. There were approximately 450 such structures in this category.

To rank the structures in order of priority the generally accepted definition was used

$$risk = probability\ of\ failure \times consequence$$

The probability of failure was assessed by considering the capacity ratios for the various failure modes, the vulnerability to failure due to particular construction details and the condition of the structure. The consequences of failure were assessed by considering the importance of the particular section of motorway carried by the structure and the significance of any hazard crossed.

The qualitative risk assessments identified the large number of deteriorating reinforced concrete crossbeams as the primary structural elements which, unless suitably addressed, would pose a significant risk to the structural integrity of the viaducts. To ensure an acceptable level of reliability was maintained for the viaducts, the latest time at which remedial measures should be undertaken for each crossbeam needed to be determined.

Quantitative assessment

The aim of the quantitative assessment was to define intervention dates for the repair of the crossbeams, incorporating the concept of time related deterioration. Two approaches were used to provide a greater degree of confidence in the results

- A deterministic approach (Level 3) using, wherever possible, existing codes and standards (code implicit target reliability) with an intervention date defined as the date at which the capacity ratio reduces to unity.
- A probabilistic approach (Level 5) to take account of the uncertainties inherent in the assessment process to determine the probability of failure of the structure at the Level 3 intervention date.

Deterministic calculations were carried out on a selection of crossbeams, covering all ranges of qualitative priority ranking. The crossbeams were analysed to identify the sections which became critical as they deteriorated. Reliability analysis was then carried out on the critical sections identified from the deterministic analysis.

The various analytical models which are required to calculate the strength of a deteriorating structural component with time are illustrated in Fig. 3.

Load model The Midland Links carries a high level of traffic flow with a large proportion of heavy goods vehicles (HGVs). Therefore it was necessary to develop a bridge specific load model for the reliability analysis and which would be beneficial for the deterministic analysis. Data on traffic using the motorways were gathered including

- data from a weigh-in-motion (WIM) site in the West Midlands for 1996
- traffic flow rate and percentage HGVs from the Midland Links for 1996
- abnormal vehicle data.

A Monte-Carlo simulation was used to determine a probabilistic distribution of the maximum traffic load effect for use in the reliability analysis. Characteristic values of loading were developed for use in the deterministic analysis.

Structural model The determination of bending and shear forces applied to the structure are calculated using well established structural modelling techniques such as grillage analysis or finite element analysis.

Deterioration model The main deterioration process on the Midland Links structures was known to be chloride induced reinforcement corrosion,

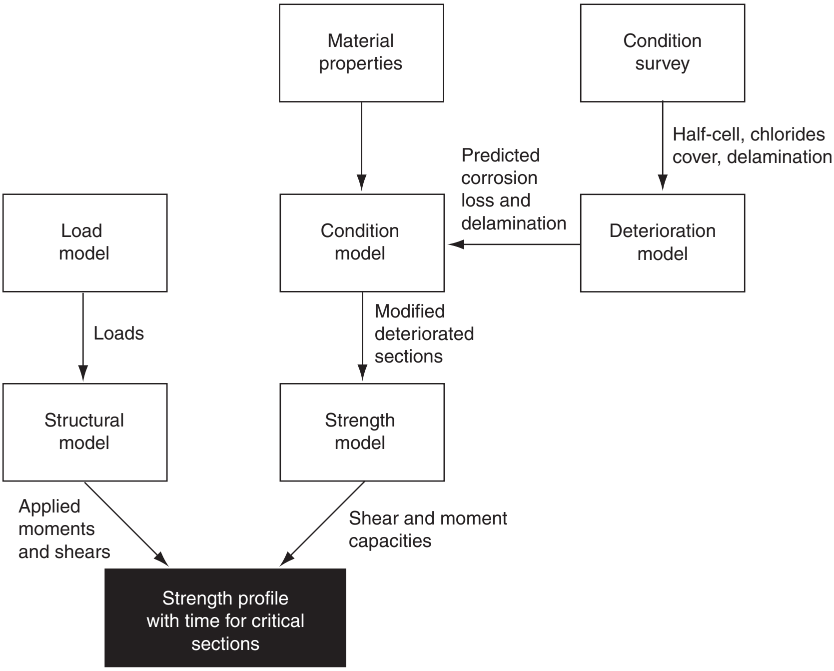

Fig. 3. Analytical models for the quantitative analysis

resulting in reinforcement section loss, delamination and spalling of the concrete cover. To calculate the capacity of a crossbeam which is deteriorating with time, a deterioration model was required which could predict the loss of reinforcement and the subsequent extent of delamination of the cover concrete with time.

Detailed data of the loss of section arising from chloride induced corrosion was available from previous repair trials carried out in 1987, together with nondestructive half-cell mapping, cover surveys and chloride contamination data. These data were used to develop a Midland Links specific deterioration model. By using statistical manipulation of the data, characteristic values were obtained for use in the deterministic analysis and probability distributions for use in the reliability analysis.

The derived deterioration model was validated with data from subsequent repair contracts. While a reasonable fit with the available data was found, it is acknowledged that the model is built on limited data and will require further development to provide statistically sound predictions.

Condition model The condition model established the rules for the application of the deterioration model so that an equivalent net cross-section suitable for analysis could be determined.

Strength model The strength model identified the possible modes of failure under consideration and established the rules for analysis. From previous knowledge of the Midland Links structures it had been identified that the cantilevers of certain crossbeams were particularly vulnerable to delamination of the concrete cover where some of the reinforcement was curtailed. The vulnerability of the cantilevers had been borne out also by the results of the qualitative assessment. This effect was quantified by testing scale model beams with simulated delamination to the steel. The results were used in the quantitative assessments by defining a bond factor applied to curtailed bars where delamination is predicted.

Implementation of methodology A range of crossbeams with various priority rankings from the qualitative assessment was selected for analysis and the necessary site condition survey data gathered for the implementation of the deterioration model. This was used to predict reinforcement section loss and delamination throughout the crossbeam at various future dates. The crossbeams were analysed at closely spaced sections along their length in their deteriorated states for the various timespans until the capacity ratio of the most critical section reduced to 1·0 for each limit state. A typical plot of capacity ratio versus time for shear, bending and bond limit states is shown in Fig. 4.

Having determined the critical sections and modes of failure for each of the crossbeams from the Level 3 analysis, the probability of failure at these sections was calculated using reliability analysis. This was used to provide complementary information on the reliability of the crossbeams at the Level 3 intervention date.

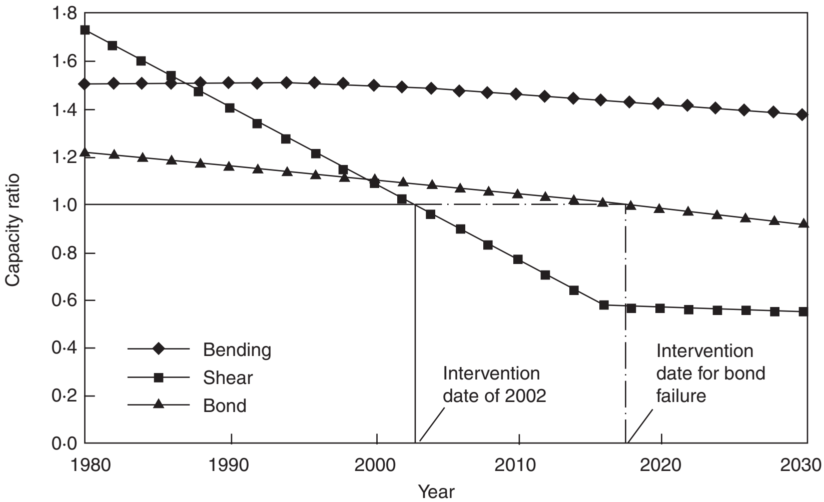

Fig. 4. Typical plot of capacity ratios against time for Level 3 analysis

Development of repair programme

The quantitative analysis was carried out on only a selection of crossbeams as it would have been impracticable to undertake detailed analysis on all 450 or so beams. A method was therefore developed to enable the intervention dates for other crossbeams to be estimated, by taking into account their condition at critical sections.

These intervention dates were used as the basis for identifying the repair programme, particularly for the immediate future.

Future developments

The reliability based strategy has demonstrated that it is possible to determine intervention dates for the Midland Links crossbeams. However, further development in certain key areas is required to increase the level of confidence in the results and to develop a strategy which includes whole life costing of the repairs. Data are being gathered from repair contracts currently underway, and this will be used to refine the deterioration model.

If more specific weigh-in-motion data could be gathered it would give greater confidence in the bridge specific load model already developed, and allow for simpler updating of the load model to take account of changes in traffic load patterns and usage.

The previous repair strategy had identified the most cost effective repair method for each crossbeam. This information can be incorporated in the methodology which is under development to determine more accurately the whole life costing of repairs.

Conclusions

The aim of the risk based strategy was to provide rational justification for extending the Midland Links repair programme.

Using a combination of code based and reliability analyses it has been possible to define priorities for repair with greater confidence than previously, and to demonstrate that an increase in the length of the repair programme is acceptable.

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Issues of practical concern

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Introduction

As mentioned in the Preface, most of the papers included in this book were presented at the conference *Management of highway structures*, held in London in June 1998. In the discussions that followed the individual presentations, a number of issues were raised concerning the practical implications of the new developments. The theory behind the procedures and the software systems are at the cutting edge of technology, yet practising engineers are familiar only with the current methods. There seems to be a gap developing between current practice and the forthcoming procedures which will need to be bridged rapidly if serious disruption in bridge maintenance activities is to be avoided.

The problem experienced both in the UK and most other countries is clearly that of large bridge stocks which are aging under the influence of ever increasing traffic volume and weight and are subject to material deterioration caused by factors such as the extensive use of de-icing salts in the past. This increases the pressure to maintain the existing stock with the given funds in the most possible efficient manner. The objective of the Highways Agency has been identified as the preservation of past investment at optimum whole life cost together with the need to plan ahead and identify funding and the logistical consequences of funding shortages.

The new developments described in this book include new structures inspection, management and maintenance strategies as well as the new structures management information system (SMIS) which is currently under development and has already been finished in parts. Next, approaches to bridge management abroad are also presented together with current research on inspection planning and whole life performance based assessments.

The discussions after each session took subjects raised in the presentations further and touched on new areas raising a multitude of issues which attracted wide interest. This paper records a few of the issues that came through as being of greatest concern to practising engineers regarding the new developments described elsewhere in the book.

Risk-based bridge assessment and management

Terms such as 'risk', 'reliability' and 'risk assessment' are nowadays used frequently but most practising engineers are not familiar with them. Reliability analysis and risk assessment have for a relatively long time been used in other sectors such as nuclear safety and in the offshore industry where they have proved extremely useful. In the bridge assessment and management area they represent new and innovative techniques which are becoming increasingly popular. With regard to bridge management, reliability-based techniques lend themselves to a multitude of tasks such as optimum inspection and repair planning, structural integrity assessment and whole life performance based assessment.

In the UK, it has often been felt since the 15 year bridge rehabilitation programme has started, that assessments undertaken in accordance with current codes can be unduly conservative, as a result of which too many bridges fail assessment. It is here where reliability analysis has been found to be a particularly useful tool. One of the problems with reliability based assessments is to decide what the target reliability should be.

Target reliability

While there is a general agreement that safety should be the most important criterion in any maintenance or repair decision it gets somewhat more difficult when one tries to establish suitable target levels of safety. The key question which keeps coming up in any discussion involving risk assessment is 'how safe is safe enough?' There is no easy answer to this question and different parties such as the bridge owner, the taxpayer or the road user might have different ideas on what represents a suitable safety level. Approaches taken include the comparison of the risk involved in everyday activities to the risk of injury or death as a consequence of bridge failure in order to establish socially acceptable minimum safety levels. More commonly, code implicit safety levels have been established and knowing that they have proved satisfactory in the past new codes have been calibrated against those.

The main problem is that it is impossible to judge whether a given safety level is unduly safe. It is only when failure has occurred that it becomes obvious that the safety level provided was too low, given the failure was not the consequence of some unpredictable freak event. Thus the only possible way would seem to be to gradually lower safety margins while bridges where this is applied would have to be very carefully monitored for signs of distress. However, this would not be wise since increasingly large numbers of bridges may start showing signs of distress and it will be beyond the current logistics to deal with them all at once.

The issue of target reliability has been previously dealt with in greater detail by Das.[1,2]

Results of reliability analysis

There is considerable unease in the engineering community concerning the interpretation of the results one typically gets from a reliability analysis or a risk assessment. This is due to the difficulty in translating typical figures such as a probability of failure of 10^{-5} yielded by a reliability analysis into more understandable measures. It has been emphasised on various occasions that the computed reliability is relatively meaningless as an absolute value and one must not misconceive a figure of, say, 10^{-5} in the sense that one bridge in every 100 000 is going to fail every year. All that a probability of failure of 10^{-5} is telling us is that when compared to another bridge with a probability of failure of 10^{-6} which was calculated using the same variable distributions and analysis methods the first one is less safe.

Reliability indices and probabilities of failure are very sensitive to the underlying basic variable distributions used and in order to be able to compare a number of bridges it has to be ensured that a consistent approach is taken. Work is currently underway to standardise reliability analysis procedures, so called Level 5 assessments, which it is hoped will enable maintenance engineers to account for bridge specific safety characteristics and assess the actual safety of bridges which fail deterministic (Levels 1–3) assessments and thereby avoid unnecessary strengthening. These guidelines will allow reliability analysis to become more widespread and will, hopefully, subsequently increase the level of confidence in it.

Although a low reliability index and therefore a high probability of failure do not necessarily indicate an imminent failure, one must realise that if no maintenance or repair was undertaken over some time then within an entire bridge stock one or two failures might eventually occur. The need is thus obvious to relate the condition of an element or the bridge to the safety and reliability of the structure and allocate funds in a way that ensures an adequate level of safety at the lowest possible life cycle cost.

Acceptable risk

As a possible solution to the question ‘when is risk acceptable?’ one possible way is to measure risk in monetary terms and to set acceptable targets in terms of that. Risk can be of two different types, namely risk to life safety and risk of loss of use. Following the definition of risk as the probability of failure times the consequences, the risk of element failure works out as 150 ($10^{-3} \times$ £150 000) compared to 0·2 for total collapse ($10^{-8} \times$ £2 million) which shows that although element failure does not usually have immediate life safety implications, it is of key importance from a financial point of view to avoid any element from failing.

System analysis

Another issue which is of concern is the difference between element and system failure. Capacity is always checked on a component level and the reserve strength of the structure is not taken into account. Depending on the form of construction this reserve strength however can be significant. For a simply supported RC-slab bridge for example, local failure of the slab can directly lead to total collapse whereas failure of one girder in a multi-girder bridge will not cause total failure of the bridge; a number of further elements would have to fail to cause total collapse. System reliability techniques offer a possibility to account for reserve strength; however, to date, they are not widely used and this area has been identified as one of the key areas for future research. The degree of redundancy for different structure and construction types needs to be established and methodologies need to be developed which allow system effects to be accounted for in a simplified manner.

New Highways Agency procedures

The Highways Agency have over the last few years thoroughly reviewed all the activities involved in the structures management process and have developed a set of new structures management arrangements and databases involving innovative safety based techniques with the aim to ensure maximum effectiveness and value for money. Safety will be the primary objective while at the same time efforts are being made to minimise traffic disruption. A number of papers in the book describe parts of the new structures management information system (SMIS) in more detail and the discussions reflect the strong interest of all participants in this system.

Inspection and inspectors

A particularly positive consensus seems to exist on the issue of the new inspection manuals which are currently being developed. A wide agreement between delegates from the UK and abroad could be seen on the problem of bridge inspector training and the lack of consistency in the collected data. The effectiveness of any inspection procedure is under close review and the different approaches taken by the various countries highlight the scope for change and improvement. The Danish road administration for instance, only employs qualified engineers as inspectors and it is the inspectors' role in the field to evaluate the condition of the bridge and forecast the future condition over the next five years. Finland also has a strict control of the inspection procedures and quality. For predicting future deterioration, however, they use a Markov chain process in their bridge management system. The Markovian model has been derived by expert elicitation.

Nevertheless, the predominant problem faced by all countries is to obtain reliable and consistent condition data from inspections. It has been suggested that probabilistic approaches could be used to deal with the inherent uncertainty in any statement and thereby to update the computed reliability of the structure. One way of improving consistency will be the photographic guidance given in the new inspection manuals on different levels of defects. Photos for both severity and extent of every defect will be included.

Another area of importance raised was how to relate inspection results to material properties and load-carrying capacity. The possibility of developing probabilistic models for this task was identified alongside the need to quantify the detectability of defects when using different inspection methods. From experience collected in the United States, the expense of using more sophisticated nondestructive evaluation methods seems well justified because these methods usually have a higher probability of detection. Therefore, money is saved in the long term due to earlier detection of defects which allows more efficient maintenance and repair planning.

Uncertainty of future funding levels

There are some concerns regarding the difficulty associated with estimating future funding levels correctly following the introduction of the new planled approach. This approach is designed to derive the best maintenance strategy and to avoid future backlogs by planning ahead and assessing future maintenance and funding requirements. Actions based on this ‘forward look’ are, however, not easily sustainable if funding does not match the anticipated level. One important task of the management process is therefore to demonstrate the logistical and financial consequences of under funding in the medium and long term and to get warning in good time so that an effective maintenance strategy including preventative maintenance can be planned within the budgetary constraints.

Bridge management systems

This book contains a wide overview about the implementation and use of bridge management systems (BMSs) in the UK and abroad. One key point that has become obvious is that BMSs are not just computer programs but a set of diverse activities carried out by a number of different parties such as maintaining agents, funding bodies, engineers and network administrators.

While systems in Finland and Denmark which have been operational for a while take a condition-based approach to the planning of maintenance and repair, a move towards a safety and reliability-based system has taken place in the UK. Denmark has so far only used reliability analysis for a small number of long span bridges and the administration of exceptionally heavy vehicles crossing these bridges, but it is hoped to implement reliability-based techniques for the management of the entire stock in the near future. The level of implementation of Pontis, the main system in the United States, is still relatively low, few authorities use the system's full capacity and repair work is still prioritised mainly on engineering judgement.

Prioritisation of maintenance and repair work

The question of how to prioritise repair work most effectively is a major issue. It has became obvious that rather than a purely financial or purely safety led approach, a somewhat more holistic approach might have to be taken in order to account for the prolific number of factors which influence maintenance and repair decisions.

As far as the optimisation process is concerned it is widely agreed that traffic delay costs which incur either due to weight restrictions and functional deficiencies or as a result of on-going repair works have to be included in the economic appraisal of any project. A BMS will have to give in the future the ammunition necessary to seek funds that are needed in order to minimise these traffic delay costs in a rational way.

A problem, which is often encountered with any sort of optimisation, is that the benefits of the planned action are both complex and multiple and not all of them are obvious straight away. In the case of impending collapse it is essential to undertake work immediately and the benefit is quite clearly the reduction in risk. However, it is not always that straightforward: for instance, for preventative work which is carried out in order to reduce future costs, benefits can only be expected in the long term.

Whole life costing

As a tool for comparing different maintenance options, the need for whole life costing is generally acknowledged. For value for money, one must look beyond the immediate or short term returns and the whole life performance of the structure in terms of future maintenance costs must be taken into account. A common problem experienced in relation with whole life costing is the value of the discount rate, currently 6% for investment in transport related projects in the public sector in the UK. Such rates are set by the Treasury to reflect public preference for enjoying benefits earlier and incurring costs later. However there are significant difficulties in applying whole life costing to bridges and these are discussed elsewhere in this book.

In the UK an innovative application of whole life assessment procedures is already taking place on a number of large scale rehabilitation projects and it was reported that a NCHRP project is under way in the United States which aims to develop a standardised methodology for life cycle costing analysis of highway bridges; so far no standardised and generally accepted methodology has been in place.

Conclusions

This paper highlights a number of key areas involving bridge management and safety which concern practising engineers who have to apply the results of the state-of-the-art developments taking place in many countries at present. The way ahead seems to be the trend towards safety and reliability-based bridge management techniques accompanied by further research in fields like system reliability, deterioration modelling and prediction, the relation between condition and reliability of a structure and the modelling of uncertainties. The usefulness of more sophisticated nondestructive evaluation techniques and the effectiveness of preventative maintenance are further areas where closer investigation is desirable.

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Part 3. Procedural standards

Advice Note on the management of sub-standard highway structures

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Introduction

Requirements for the assessment and strengthening of trunk road bridges in the UK are contained in the Departmental Standard BD 21/97[1] and its predecessors. The Standard provides criteria and procedures for assessing the adequacy of highway bridges to carry current and future heavy vehicle types, in particular the EU 40 tonne vehicles that will be permitted from 1 January 1999. If assessment shows a bridge to be inadequate, then action (termed *Formal Interim Measures* in the Advice Note) is required as follows

- vehicle weight and/or lane restrictions or propping of the bridge
- monitoring of the condition of the bridge if further structural deterioration is considered likely to occur despite vehicle weight and/or lane restrictions
- closure of the bridge if it is assessed to be incapable of carrying the lowest traffic load
- replacement or strengthening of the bridge.

In addition, the Standard requires urgent action to be taken if, in the course of an assessment, a bridge is found to be so inadequate that there is a potential risk to public safety, i.e. it is an immediate-risk bridge.

Many authorities have found it difficult to comply fully with the required actions. The main reason for this has been the excessive traffic disruption and other costs any BD 21 recommended measure would cause for certain bridges. Since many of these bridges seemed to show no apparent signs of distress, these measures were considered disproportionate to the perceived risks. Various risk assessment methods have been devised to justify such departures from the Standard, either formal or otherwise.

The Highways Agency, who were consulted regarding the appropriate use of risk assessment methods, became concerned in late 1996 that, unless a coherent national policy was developed regarding the appropriate interim measures for sub-standard highway structures, the difficulties faced by authorities in this area might lead to public safety being compromised. A National Working Group, comprising all the major highway authorities in the United Kingdom, was therefore established to prepare advice on the management of these situations. A list of the individual representatives of the group, which was chaired by the Highways Agency, is given in Appendix 1.

The Group worked through 1997 to draft the new Advice Note BA 79/98 *The management of sub-standard highway structures*,[2] which has now been published. This paper describes the principal features of the Advice Note, in particular the assessment and monitoring processes, which are dealt with in appendices in the Advice Note and which are not attached here due to their size.

Process of assessment

The Advice Note explains the need for structural assessment to be made using criteria and methods which yield an appropriate balance between safety and economy. Unduly conservative assessments can lead to unnecessary strengthening while the use of lax rules or inconsistent application can result in bridges which have an unacceptable risk of failure remaining in service without appropriate actions being taken.

The principles and levels of calculation-based assessment are described in an appendix to the Advice Note. The essential feature of such assessment of an existing bridge is that it proceeds in stages. The objective is to determine structural adequacy with minimum effort. Initially an assessment is made using assumptions known to be conservative (e.g. simple analysis and full partial factors). If the requirements of the assessment are satisfied then no further action is needed. However, if the bridge does not meet the requirement, it must then be considered to be provisionally sub-standard. Refinement of the assumptions is the next step to enable a further assessment which may demonstrate structural adequacy, e.g. more refined analysis and better structural idealisation. Where this stage is unsuccessful, further stages of increasing refinement (e.g. use of bridge specific assessment live loading) and complexity may follow until either adequacy is demonstrated or it must be concluded that the bridge is sub-standard.

As an aid to structuring this approach the Advice Note describes five levels of assessment of increasing sophistication, Level 1 being the simplest and Level 5 the most sophisticated. Conservatism in the assessment is gradually reduced as assessment stages proceed from Level 1 towards Level 5.

Some new terminology is introduced to facilitate the description of the management of the assessment process. A sub-standard bridge is one which is found to be inadequate in relation to the requirements of BD 21/97. Where a bridge is assessed to be inadequate during assessment and before the process is completed, it is termed a provisionally sub-standard bridge. *Formal Interim Measures* is the term used to describe the actions required by BD 21/97 for an inadequate bridge. These actions are distinguished from *Other Interim Measures* which are defined as measures short of or different from the Formal Measures. They may include monitoring alone or monitoring with other measures. Monitoring-appropriate bridges are those which are considered appropriate for monitoring as Other Interim Measures. In this context monitoring, which may range from visual inspection to continuous measurement using instrumentation, is used to detect deterioration of structural behaviour or condition and thereby give warning that action may be needed to ensure safety of the bridge.

The management of the assessment process and related measures is illustrated by the flow chart given in Fig. 1 which is included in BA 79/98.[2] It shows the sequential and cyclical nature of the assessment process. It also shows the actions which need to be considered at each stage of the process. Crucial points in the process occur after a bridge is determined as provisionally sub-standard and sub-standard. An assessment of safety is then needed by consideration of the assessment findings to date, the local circumstances of the bridge and the traffic carried. For a provisionally sub-standard bridge, decision is required on the next assessment stage and also on whether it is an immediate or a low risk bridge. A low risk bridge is one which is judged to have substantial capacity even though the Level 1 assessment indicates it may be sub-standard. For a sub-standard bridge, the measures required to increase safety and the priority for strengthening action have to be determined. A review of the measures is required every two years if the bridge has not been strengthened in the meantime.

Monitoring-appropriate bridges

A key feature of the Advice Note is that it recognises the contribution which monitoring can make to the assurance of the safety of a bridge. For this purpose the essential requirement is that the monitoring process detects deterioration of bridge condition or performance in sufficient time for safeguarding action to be taken before significant loss of safety occurs. This requirement can be met depending upon the type of bridge and the performance of the monitoring system.

Only some types of sub-standard bridge are monitoring-appropriate. Essentially, these are bridges that will give a warning of failure which can be detected by monitoring before a critical situation develops. The main criteria (1) and (2) given for determining that a bridge is monitoring-appropriate must both be met. Briefly they are

1. bridges with no sign of significant distress, where no hidden distress or weakness is likely to be present or, where observed, distress is not detrimental to safety
2. bridges where failure is likely to be gradual over time progressing from local signs of distress.

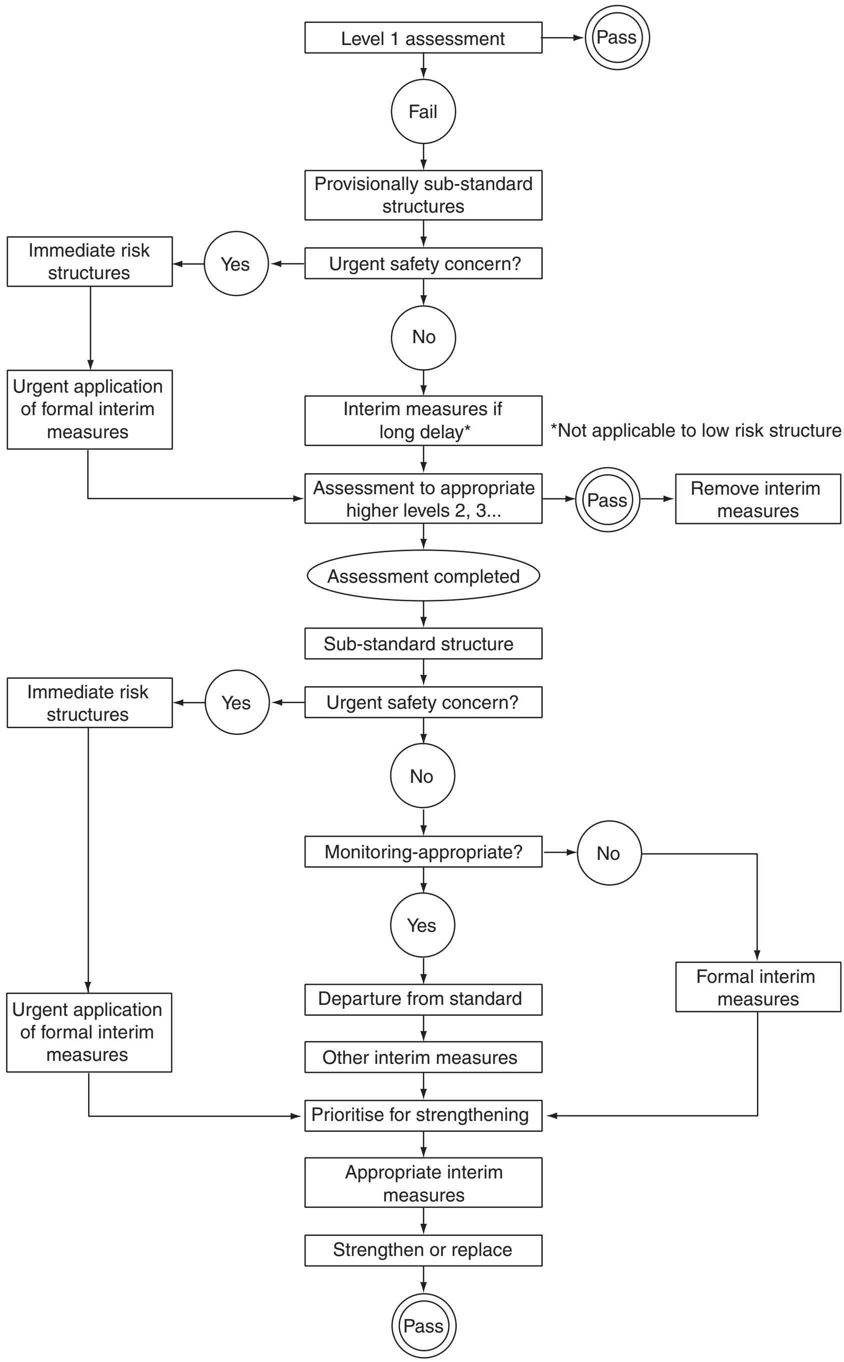

Fig. 1. Management action flow chart

The Advice Note suggests types that are likely to be monitoring-appropriate, including

- reinforced concrete slab bridges or composite steel and concrete slab bridges with flexural inadequacies
- bridges with an inadequate element or connection whose failure would not precipitate sudden collapse and whose failure can be observed.
- bridges in a sound condition with span less than five metres where the consequence of failure is low.

Types of bridge which are not normally monitoring-appropriate include those which are sub-standard through tendon, shear or anchorage inadequacy where failure would precipitate bridge collapse.

Monitoring

Monitoring is defined as periodic or continuous observation and recording of information pertaining to structural behaviour, the primary purpose being to detect deterioration should it occur. It should be noted that this definition includes visual inspection and recording information by hand.

The performance required of the monitoring system is defined in terms of five essential requirements. Their principal features are

- Monitored parameters must be directly related to the predicted modes of failure and type of inadequacy of the bridge.
- Alarm levels for the monitored parameters must be set.
- The monitoring system must be able to detect warning of distress of critical elements associated with the predicted mode(s) of failure. It must also be able to respond sufficiently quickly to enable safeguarding action to be taken before catastrophic collapse occurs.

Monitoring of a provisionally sub-standard or a sub-standard bridge may be used alone or with Other Measures. The overall purpose is to contribute to the assurance of the safety of the bridge enabling it to remain in service. For this purpose the extent (frequency and intensity) of monitoring should generally be greater than that involved in General and Principal Inspections.

The extent of monitoring will depend upon the type and condition of the bridge, its structural inadequacies and current circumstances, and the Other Interim Measures proposed. It should be continued until the bridge has been strengthened or replaced, or Formal Interim Measures have been implemented.

The choice of monitoring regime will be influenced by the nature of the structural inadequacies (more than one type of inadequacy may be present)

- Assessment calculations will give the basis for identifying critical areas for monitoring where they indicate the original design loading was lower than now required or the original design used less onerous principles and criteria than now adopted in assessment.

- Assessment calculations provide the basis for monitoring the deficient part(s) of a bridge or the development of deterioration where an error in design or construction causing weakness has been found, or where deterioration or damage has reduced capacity which otherwise would be adequate.

In planning a monitoring regime it is essential to recognise that any of these inadequacies may be present without the bridge showing any visible signs of structural distress, e.g. cracking due to reinforcement corrosion may be present where it is hidden from visual inspection.

The Advice Note identifies the key issues to be considered in deciding to implement a monitoring regime. They are the requirement that the bridge meets the criteria for being monitoring-appropriate; the specific purpose of the monitoring and what events, distress or deterioration may possibly occur; the ability to detect them sufficiently early, and the consequences should they not be detected.

Distress of a minor nature may not invalidate the use of monitoring. However, it is pointed out that monitoring in service may not be appropriate where distress is recent, significant or has resulted from live load effects. Clearly, careful consideration of the presence of structural distress is required based on substantial engineering experience.

To assist in the selection of an appropriate monitoring regime, three classes of monitoring are described

- basic monitoring — visual observation and recording
- detailed monitoring — visual observation and recording supplemented appropriately by quantitative measurement (e.g. extent of deterioration, level survey, nondestructive testing), displacement or strain measurement at typical or critical positions of defects/damage, traffic loading survey
- extensive monitoring — more extensive and frequent or continuous monitoring at typical or critical locations where change is predicted to progress to bridge collapse in a short time. This class will often require use of data loggers, remote techniques and automatic alarm systems.

Monitoring specification

The Advice Note recommends the use of a specification for monitoring. It should be a clear, unambiguous procedure document prepared following a special inspection of the bridge. It should normally include descriptions of

- the structural inadequacies
- the reasons for the observed satisfactory performance in service

- ☐ the anticipated mode(s) of failure, likelihood and consequences
- ☐ the parameters to be monitored
- ☐ acceptable ranges of observations and alarm/warning levels
- ☐ procedures to be followed if alarm/warning levels are reached
- ☐ recording and reporting requirements
- ☐ requirements for review of the monitoring regime.

Conclusions

The Advice Note provides guidance on the management of provisionally sub-standard and sub-standard highway bridges. A key feature is that the contribution which monitoring can make to the assurance of the safety of a monitoring-appropriate bridge is recognised. The guidance includes description of the process of assessment, and of requirements for a bridge to be designated monitoring-appropriate. It also provides descriptions of classes of monitoring and the factors to be considered in choosing an appropriate monitoring regime. Overall the Advice Note should assist in the complex task of maintaining the safety of sub-standard bridges during assessment and the subsequent period before replacement or strengthening.

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Appendix 1. Membership of the National Working Group

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Whole life perfomance-based assessment of highway structures

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Introduction

The UK has a long history of involvement in the development of tools and procedures for the assessment of the load carrying capacity of structures. Technical Note BE 4[1], published in 1964, was used for an assessment programme known as *Operation Bridgeguard*. The assessment criteria used were less severe than the then current design loading and in addition a 25% overstress on metal bridges was permitted. Structures that failed the assessment were weight restricted and earmarked for strengthening; those which passed were, however, to some extent still sub-standard bridges. In the early 1980s it was recognised that a new Assessment Code was necessary incorporating an assessment loading which adequately dealt with the effects of present day traffic, in particular the very large growth in heavy goods vehicles. The Assessment Code BD 21, introducing the first limit state approach to the assessment of structures, was published in 1984 and has been widely used both in the UK and abroad. When the current programme of bridge assessment and strengthening is complete some 50 000 structures in Great Britain will have been assessed using the code.

The limitation of the assessment approach adopted to date is that only the performance at the time of assessment is considered. The Highways Agency is developing tools and techniques to address this limitation and enable whole life assessments to be undertaken.

This paper indicates the role whole life assessment will play in the Highways Agency's future steady state assessment programme, highlights the significant new concepts used and considers the whole life assessment of a concrete deck by way of illustration. A new Highways Agency standard, is in the course of preparation.[2]

Background

Once the UK's current assessment and strengthening programme is complete there will remain an on-going need to carry out assessments of the performance of structures. Whether this is based on calculations, as in the case of a bridge deck, or on engineering judgement, for components such as bearings and expansion joints, it will be an essential part of the steady state maintenance of the structures stock. Most of the existing trunk road structures were built in the period between the mid-50s and the late 80s. As they are beginning to age, need for major rehabilitation, at least on the earlier structures, is already becoming apparent. It is expected that this need will progressively rise from now on.

The condition of the structures stock will be an indicator for selection of structures in the steady state assessment phase which will follow completion of the Highways Agency's current assessment and strengthening programme. Work is planned to develop a safety index which will enable vulnerable structures to be identified and prioritised for assessment. It is intended that a rolling programme of assessments, consistent with maintaining an acceptable safety level for the stock as a whole, will be undertaken each year. Some assessments will arise as a direct result of deterioration identified through inspection.

Methods are required not only to assess the adequacy of a structural element or component at the time of assessment, but also to predict future performance levels under different maintenance strategies. On-going research is beginning to provide deterioration models which can be used for this purpose.[3,4] Using these and whole life costing principles in the Highways Agency's new bid assessment and prioritisation system (BAPS) for allocating structures maintenance funds, will enable appropriate maintenance treatments offering value for money to be selected.

A new standard is being drafted which will provide a framework for the whole life performance assessment of highway structures. While the current BD 21[5] and associated assessment standards and advice notes will continue to provide the analytical basis for the assessment of bridges and structures, new implementation documentation, similar to BD 34[6] which introduced the requirements for Stage 1 of the current assessment programme, will also be required.

New concepts

The standard introduces the following classifications of assessment

1. *General Assessment (GA).* General Assessments are full assessments of structural adequacy at the time of assessment as required by BD 21. General Assessment of structural components such as bearings are mainly observation based assessment of their adequacy. Methods for this are given in the standard and will use guidance given in new bridge inspection manuals, which are currently under development.
2. *Particular Assessment (PA).* Particular Assessments are periodic re-assessments of specific critical structural elements or their sections.
3. *Whole Life Assessment (WLA).* Whole Life Assessments are Particular

Assessments where current performance and prediction of future performance is carried out.

General principles

Structural components and elements start their life at or above the design requirement level. Over time the performance level reduces, until at the end of its functional life the level has reached a minimum acceptable level.

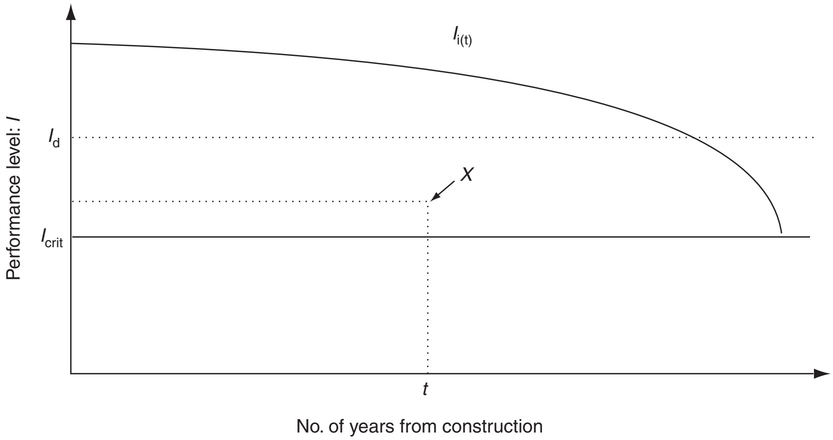
If an element or component in an existing structure is found to have a performance level below the minimum acceptable level, then it must be replaced or strengthened as soon as possible. Such maintenance work is to be considered as *Essential* and is safety related. If, however, its performance level is assessed to be at or above the minimum level, then it may still be advisable to carry out maintenance work on it, but such work is to be considered as *Preventative*. Preventative work has to be justified on economic grounds alone.

Performance indicators

The purpose of assessment is to determine performance levels and compare these with target levels, thus enabling the assessor to decide if any maintenance action is necessary at any point of time. It is therefore necessary to define appropriate performance indicators for different elements and components. The standard considers these as follows

- Critical performance level indicator (I_{crit}). In the case of bridge deck elements, for instance, this will be the assessment load level indicated by the appropriate load capacity factor C. It will be individually defined for components such as bearings and expansion joints.
- Design performance level indicator (I_d). This will be the indicator corresponding to the exact design requirement for the item. In some situations the design level will be the critical level.
- Ideal performance level indicator (I_i). Structures designed to a particular requirement in reality at the 'as built' state have considerable reserves of strength in excess of the design requirement I_d. This reserve of strength enables a structure to remain serviceable and safe for a long length of time without requiring major maintenance. The ideal performance level indicator will be the indicator corresponding to the performance of the best examples of the type of item at any point in time through its expected functional life. The ideal level will be age related.

Figure 1 shows the relationship between the three performance indicators.



I Performance level
t No. of years from construction
$I_{i(t)}$ Ideal performance level at time t
I_d Design performance level
I_{crit} Critical performance level
X Performance level I of the structure being assessed, at time t

Fig. 1. Performance level indicators

Whole life assessment

The purpose of Whole Life Assessment (WLA) is to determine options for maintenance strategies and to provide inputs for maintenance works bids to the new Highways Agency bid assessment and prioritisation system.

WLA is used to determine whether any maintenance action is required to any structural element or component in the foreseeable future. This is done by predicting the future performance of the element or component and considering the effects on performance by intervention with maintenance actions. WLA provides the means for determining and comparing any number of maintenance options, taking account of future performance levels.

As maintenance needs are related to individual elements or components, WLA is also element or component specific. For structures, WLA therefore is based on the results of the Particular Assessment (PA). When dealing with performance in future years, an element of approximation is inevitable; hence it is not necessary to achieve the same degree of accuracy in the forecasts as required for PAs carried out for the present state.

Example: Whole life assessment of a concrete bridge deck

WLA for a bridge deck element considers how corrosion of the reinforcement caused by chloride ingress may affect load carrying capacity over time. Two sets of whole life performance targets — I_{crit} and I_I are

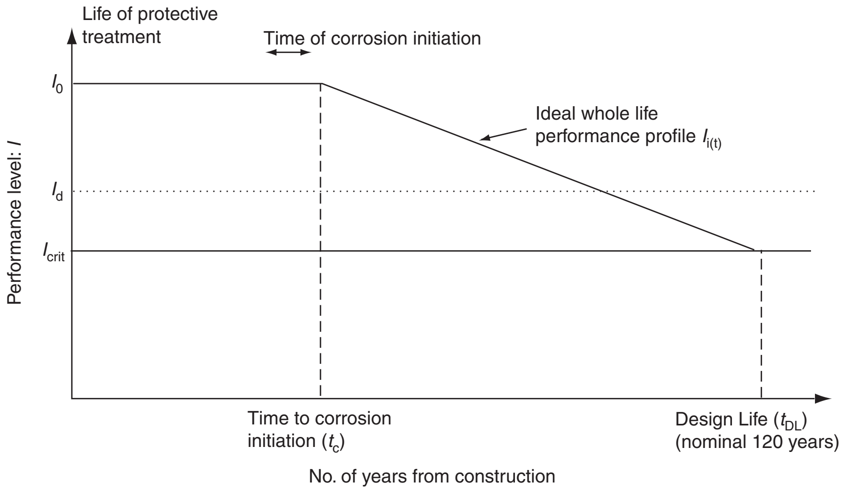


Fig. 2. Ideal performance profile for a bridge deck

required. The method of constructing a performance profile $I_{i(t)}$ for the element is shown in Fig. 2. The structure is assessed from the original drawings and the performance level at the time of construction (I_0) is calculated. From this level, the performance profile to the initiation of corrosion (i.e. the life of the protective treatment + the time to corrosion initiation) is assumed horizontal after which it is assumed to fall-off linearly.

When the design life of the element has been reached the performance level is considered to have reached the critical performance level. I_{crit} level shall be the appropriate live load capacity factor C as given in BD 21.

The time to first exposure to chlorides will depend on the life of any protective treatment, such as a waterproofing membrane, that has been applied to the deck. The time taken to initiate corrosion in the reinforcement of the deck, from first exposure, is a function of the cover depth and the extent of corrosion is a function of the time the reinforcement has been exposed to chlorides and the chloride concentration. Corrosion loss of the reinforcement can be predicted and the reduced steel area used to calculate a revised section capacity at any point in time.

Conclusions

The standard is a developing document. Further work is required to quantify the effects on whole life performance of intervention through the application of maintenance options. The service life of elements and components is the subject of on-going research by the Highways Agency.

Further research is assessing the cost effectiveness of maintenance treatments.

The Highways Agency sees whole life performance based assessment as an important tool in the evaluation of maintenance needs and the selection of appropriate cost effective maintenance solutions. Much work still remains to be done in this area but the standard provides a framework and represents an important first step.

Acknowledgements

This paper is presented with the kind permission of Mr Lawrie Haynes, Chief Executive of the Highways Agency.

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Whole life considerations for bridges and other highway structures

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Introduction

The purpose of whole life costing (WLC), also known as life cycle costing (LCC), is to compare the merits of alternative options for projects in a rational manner by taking account of all future costs in addition to the initial (building) costs. Such comparisons are made while considering new projects, as well as during the life of a project when choosing management strategies.

The future costs considered in whole life costing generally include all maintenance, operating, decommissioning and replacement costs. Although the process is referred to as costing, all future benefits and incomes also need to be taken into account. For assessing public sector investments, future costs are converted into their present value (PV) at a given base year, using a discount rate, known as the test discount rate (TDR) (currently 6% for transport related projects), which is stipulated by the Treasury. There are various justifications for the TDR, discussions on which is beyond the scope of this paper. However, one obvious reason for its use is that, without using discounting, one could justify unrealistically large initial capital investment for long life projects which may be expected to produce benefits *ad infinitum* into the future. Discounting of future costs and benefits to present value, using a high discount rate (e.g. 6%) has the effect that all calculations, beyond about 30 years or so, become insignificant. As such, WLC is particularly appropriate for short life projects. Long life projects, such as highway structures, pose a number of difficulties regarding WLC.

The Department of Transport Standard BD 36/92[1] and Advice Note BA 28/92,[2] *Evaluation of maintenance costs in comparing alternative designs for highway structures*, embody the principle of whole life costing for highway structures. These documents require that, when considering alternative design options, or options for alterations to existing structures, in addition to the initial costs, future maintenance requirements are also to be taken into account.

Although the BD and the BA are concerned with the evaluation of design options for highway structures, the same principles of whole life (or more precisely, remaining life) costing are applicable when considering different management options for a structure at any given point of time. Indeed, this aspect is at present of primary concern to the Highways Agency as the maintenance of the existing road network has now assumed greater importance. The paper therefore also covers the question of comparing different maintenance options.

Recent developments regarding the procedures used for procuring and managing roads and bridges, such as the increasing use of DBFO contracts and the new maintenance agency arrangements, mean that there is now a need to review the wider issues of whole life consideration for highway structures. The purpose of this paper is to highlight these issues and explore a possible way forward. The following discussions are essentially confined to bridges, although the principles referred to are equally applicable to other highway structures.

Current requirements

BD 36/92[1] and BA 28/92[2] require that discounted future maintenance costs for a proposed structure, or a proposed structural alteration, are to be added to the initial building costs in considering options. For this purpose, the standard gives recommended discounted values of maintenance costs for steel and concrete bridges and components. In addition, it is required that as part of the maintenance costs, traffic delay costs are also to be taken into account, for which methods are provided.

Limitations

The principle of whole life costing is intended to encourage the use of more durable low maintenance structures. The adoption of the principle is therefore a significant advance on the earlier approach of considering the initial (building) costs only for new designs. However, a number of difficulties have been encountered in applying whole life costing methods to bridges, which have cast doubt regarding the validity of its use in the case of such structures. These difficulties are discussed in the following paragraphs.

Uncertainty of maintenance needs

Bridges are traditionally long-life structures, their engineering life being generally in excess of 100 years. Trying to determine the maintenance needs of bridges being built today for the whole of their life must therefore be considered as a doubtful exercise. Furthermore, a large proportion of the bridges become functionally redundant long before the end of their engineering life. The exercise of whole life costing may turn out to be somewhat futile for such bridges. The use of a high TDR has a mitigating effect in this regard by reducing the effective time horizon to 40 years or less. However, as we will see later, this also causes some problems.

Historically, in earlier times, the building rate of bridges was sufficiently slow to allow for well proven methods to be used. However, more recently, the expansion of road building has necessitated a large number of bridges to be built quickly with new materials and methods, with little or no history of use behind them. This has resulted in many untried features and techniques, many of them potentially with problems which are yet to be discovered, being introduced into the stock, which makes any estimation of life expectancy and maintenance needs, somewhat unreliable.

Long life structures

Bridges are required in large numbers for a road network and it is physically impossible to replace more than a small proportion of the stock each year. In the UK, the current expected engineering life of 120 years for bridges means that no more than about 1% of the bridges can reasonably be expected to be renewed or rehabilitated each year. Any reduction of the engineering life for new bridges will require a corresponding increase of replacement rate from a future date. However, using whole life costing with cost discounting makes it impossible to take account of such future increases of overall needs of the bridge stock.

The use of cost discounting means that all long term future costs (beyond about 30–40 years when using the present recommended rate of 6%) become negligible. This gives an in-built advantage to short life or low initial maintenance options. Conversely, it also makes it difficult to justify even small increases in the initial costs which may result in substantial long term benefits. If such procedures are rigorously applied over a period of time, the current bridge replacement rate (of, say, 1% of the stock per year) could in time rise considerably (to, say, 3% per year), reflecting a general lowering of bridge stock life.

Need to consider options

As a number of new (and some not so new, such as masonry) materials are now being put forward for bridges and bridge components, it is becoming increasingly necessary to allow freedom of choice provided it can be justified through rational comparison. Similarly it may also be necessary to consider radical management options for an existing bridge, such as the do-minimum option. The present procedures, for which the expected life of long life structures is not relevant, are not particularly useful for comparing short term options on a rational basis.

Traffic disruption cost

In calculating maintenance costs, the costs to the road user in terms of traffic disruption (known as user delay costs) need to be considered. User delay costs are calculated using the DETR computer program QUADRO. Certain implications arise from the use of such costs in whole life costing. These are

- The user delay costs tend to be so high that all other costs become unimportant by comparison. This perhaps is not illogical since the purpose of the network is essentially functional.
- The user delay cost is basically a 'notional' rather than a 'real' cost, although it is partly based on estimates of real cost of delays to commercial operators. It is questionable whether such notional costs should be included in the same equation with real costs such as capital costs.
- When taking a decision to choose an option with a lower user delay cost, but with a higher initial capital cost, the authority incurs a net increase of its expenditure for the project, which eats into its available funds. However, the beneficiary of the decision will be the road user. User delay cost being a notional quantity it is not possible to reimburse the spending agency directly for taking decisions, benefits of which are enjoyed more widely.

Bridge elements and components

Although bridges are referred to as single entities, they are composed of individual elements such as decks, substructures and foundations, components such as expansion joints, bearings and parapets, and protective coatings such as waterproofing, painting and galvanising, which have different expected service lifes, and may be replaced or repaired with varying degrees of ease. An effective whole life costing method should be able to take account of these different features.

Design and build and DBFO contracts

The government is committed to the increasing use of the 'Design and Build' and 'Design, Build, Finance and Operate (DBFO)' type of contracts for new road works as well as for the maintenance of existing roads. In these forms of contract, the client has no direct control over the choice of structure type in terms of long term durability. In DBFO contracts such interests are to some extent safeguarded by the operators' own need to maintain the bridges in a cost-effective manner for the duration of the concession period, and the subsequent handover obligations.

The situation is not so clear-cut with the Design and Build contracts. The use of BA 28 is usually included as client requirement; however, it is difficult to ensure that it is being applied fairly in practice. Furthermore, to force a contractor to choose an option which is more costly to him in real terms, while delivering 'notional' benefits to others, goes against the principle of minimum interference from the client which is the basis of such contracts.

Proposals

Despite all the difficulties encountered in using whole life costing for bridges, the principle of taking into account future maintenance costs cannot be questioned. Central government and other public authorities are committed to it. Furthermore, in the case of projects involving the road network, the need to adequately take into account the road user delay costs arising from any traffic disruption, is also undeniable.

At the same time, it is also to be remembered that the public today enjoys the benefits of a road network largely established by their ancestors through many centuries which already had in it a large number of very durable bridges and other structures. This is why the rate of replacement or rehabilitation is still manageably low. It is therefore an obligation on the part of today's decision makers to maintain the overall long term durability of the stock so that such benefits can be equally enjoyed by future generations. For any whole life costing to be justifiable and credible, ways must be found so that they do not contradict this historic obligation.

With the above issues in mind, the following proposals are being put forward as a means for rationalising the use of whole life costing for bridges.

Test discount rate

As discussed earlier, the current TDR of 6% shortens the time horizon in the calculations to 30 years or so. Societal expectations and the economic and technological developments in the transport field being very unpredictable in the longer term, this rate seems to be reasonable.

Long life elements

Clearly, the use of a high TDR will have the effect of progressively lowering the serviceable life of certain bridge elements which have traditionally been designed for very long life. In order to prevent this from happening, those elements that cannot be replaced easily, say within 30 years or so, such as foundations, substructures and main deck sections, should continue to be designed for the current target design life of 120 years.

Replaceable/renewable elements

The replaceable or renewable elements of the bridge, such as bearings, expansion joints, waterproofing, paint systems, etc. should be chosen on the basis of whole life costing and should not be subject to design life requirements. This will ensure that their initial costs and short term maintenance costs will be low, and in the longer term, such elements will be renewed in any case. This may also have the effect of encouraging the use of more replaceable parts in the main bridge elements such as the decks, if this will result in lower initial costs.

Maintenance cost data

A major problem with whole life costing for bridges is the lack of maintenance cost data, and a determined effort should be made to collect data on the maintenance needs and service life of different bridge types, elements and components.

There is also a need to improve the way maintenance cost data are used in the current method. For instance, the concrete deck rehabilitation cost is derived on the basis that there is a 2% probability of a major rehabilitation in every 20 years. Thus the estimated cost is multiplied by 2% for the year 20. The next future costs from year 40 onwards become negligible in any case. In reality, however, the probability of requiring rehabilitation could be 2% at year 20, the cumulative probability increasing progressively to a peak (unity) at the maximum expected life of the type of bridge.

Data for maintenance needs and costs should be related to the problems encountered, rather than the elements involved. For instance, steel elements, with regular repainting, can be expected to remain sound for the foreseeable future. Yet, a number of major bridges have suffered from significant fatigue cracks within 15 years of their construction.

D&B and DBFO

Design and Build and DBFO type of contracts have brought into focus a major limitation of the more conventional 'conforming design' contracts where the Consulting Engineer prepares the full designs that go into the tender documents. The latter has often inhibited the use of alternative designs by tenderers because of the delays and complexities involved in obtaining technical approval during the tendering process.

There is thus a need for a fundamental review of the standards and specifications used in the contract documents, so that the necessary degree of flexibility can be introduced in all forms of procurement while safeguarding the high quality implicit in the current requirements. It is proposed that each of these documents, i.e. standards, advice notes and specification clauses, are examined to identify the core client requirements which need be included in tender documents. The rest of the current requirements essentially form compliance standards, which could be maintained and enhanced, as necessary, by industry and the professional bodies.

Conclusions

The paper discusses the issues relating to the whole life costing of highway structures brought to the fore by recent developments, as well as those encountered in applying the current standards. It is proposed that a thorough review of the procedures and the principles behind them should be carried out in order to find rational solutions.

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Part 4. Inspection and assessment

Inspection manual for bridges and associated structures

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Introduction

The Highways Agency is responsible for the maintenance of some 16 000 structures, including 10 000 bridges and large culverts, on the trunk road network in England. To date, these structures have been inspected at regular intervals, using a regime based on General Inspections every two years and Principal Inspections every six years. These are essentially visual inspections, although limited testing of concrete has been included in recent years.

As part of the development of a new structure management methodology, the Highways Agency's inspection procedures have been reviewed.[1] The aim has been to concentrate effort where it is needed most and improve the quality and consistency of inspection and reporting. New procedures are proposed, the main change being the replacement of Principal Inspections with Benchmark Inspections at less frequent intervals and with Particular Inspections, which will target areas of suspected deterioration and include a range of tests.

A new inspection manual is being prepared covering the various types of bridges, retaining walls and miscellaneous structures. The manual will provide comprehensive advice on the new procedures and on defects, methods of testing and reporting. This paper outlines the new inspection procedures and describes the progress being made on the manual.

Field trials will be arranged within the next few months, inviting a number of Maintaining Agents to test both the new procedures and the manual. The Highways Agency then intends to introduce the manual, along with other bridge management improvements, in 2000.

Current practice

The current Highways Agency requirements for inspection are set out in Vol. 3 of the *Design manual for roads and bridges*.[2] The four main categories of inspection are

- ☐ *Superficial Inspection*: a cursory check for obvious deficiencies which might lead to accidents or high maintenance costs. Superficial

Inspections usually entail maintenance staff being vigilant and reporting anything needing urgent attention. They are also undertaken when a problem has been observed and reported by other staff, the police or the public. There are no set programmes or reporting requirements.

- *General Inspection*: a visual examination of representative parts of the bridge to ascertain condition and note items requiring attention. General Inspections are normally undertaken every two years. Inspection is from ground or deck level, using binoculars if appropriate. The results are reported on a Form BE11[3] which allows only one entry for each type of structural element regardless of the number present or the range of defects.
- *Principal Inspection*: a close examination of all parts of the bridge, generally at intervals of six years. Access equipment and traffic management are usually needed to enable all parts of the structure to be inspected. Originally Principal Inspections were confined to visual examination, but limited testing (half-cell potential, chloride concentration, cover and carbonation) has been included in recent years for concrete bridges.[4] Full reports form part of these inspections.
- *Special Inspection*: a close inspection or testing of a particular area or defect which is causing concern. Special Inspections are undertaken for a wide variety of reasons, including following up a defect identified in an earlier inspection, investigating a specific problem discovered on similar structures, checking for scour after flooding, or monitoring at regular intervals.

Other inspections include Underwater Inspections, Paint Surveys prior to repainting steelwork, and Joint Inspections and initial Principal Inspections prior to taking over responsibility for a structure.

The need for change

A review of the current procedures found that the existing system of inspections provides a sound basis for management of structures, comparing favourably with those used in several other countries, but that there is scope for improvement. The following shortcomings were noted

- Principal Inspections require the whole of a structure to be inspected closely, irrespective of the importance of each component or its likely deterioration
- more use could be made of the growing range of non-destructive testing techniques
- the limited testing of concrete structures is not always as effective as it might be

- ☐ standards of inspection and reporting vary greatly between Maintaining Agents
- ☐ reporting requirements for General Inspections do not provide adequate scope for reporting a variety of defects or for dealing with multi-span bridges.

Procedures also need to take account of the recent developments whereby inspections will be undertaken by Maintaining Agents which have won competitive tenders for the work. The contracts are normally for three to five years, so successive inspections could often be carried out by different firms. In addition, the Highways Agency is developing a computerised bridge management system, the success of which will depend to a large degree on the accuracy and reliability of data derived from inspections.[5]

Proposed procedures

Cost-effective management of the maintenance of a structure relies on detailed, up-to-date information about the current condition and rate of deterioration. This objective can best be achieved through an inspection programme tailored to meet the specific requirements of each structure. Therefore, the new inspection procedures are based on a combination of regular visual inspections of the whole structure and a programme of more detailed investigations concentrating on known or suspected areas of deterioration. The inspection schedule for each structure may be unique to that structure but will be designed to provide the appropriate level of information.

For all structures, two basic types of regular inspection will be carried out

- ☐ A *General Inspection* at two yearly intervals will record the visual condition of each part of a structure which is visible from ground level, bridge deck level or other vantage point. Those parts which are not visible from the ground will be inspected at intervals not exceeding six years.
- ☐ A *Benchmark Inspection* will be carried out on first taking over responsibility for a structure and at intervals of between 6 and 24 years thereafter. This will record each and every defect and blemish on that structure in the form of scale drawings. Photographs and text will also be used where appropriate. The inspection will establish precisely the condition of the structure at that time such that the report can be used as a reference until the next Benchmark Inspection.

For the majority of structures, other inspections will be carried out when required

- *Superficial Inspections* will be carried out whenever personnel visit a structure for any form of cursory check, inspection or maintenance (e.g. following impact damage). Many such visits will therefore be unprogrammed.
- *Particular Inspections* will be detailed inspections at programmed intervals concentrating on the condition of particular parts of the structure where deterioration is known or suspected. They may include detailed testing and monitoring as appropriate and will be carried out at intervals ranging from about six months to 12 years. A schedule of Particular Inspections will be drawn up for each bridge or family of bridges depending on its form of construction, condition, materials, location and maintenance history.
- *Special Inspections* will be similar in scope to Particular Inspections but undertaken for a specific reason rather than at programmed intervals.

The various types of specialist inspection, such as chloride investigations, paint surveys or underwater inspections, will be forms of Particular or Special Inspections. Unlike the current system, the programme of inspections will not be the same for all bridges. Greater care will be needed, therefore, to keep track of the inspections required each year. However, with the use of computer databases, this should not pose problems. Table 1 summarises the current and proposed inspection regimes, showing how they compare.

Segmental inspection

Inspection procedures in the United States of America frequently require each element of a bridge (such as a girder, column, capping beam, etc.) to be inspected and given a condition rating. The condition ratings are then used in the structural assessment of each element. Frangopol and Hearn[6] propose that elements be subdivided into 'segments', which they define as a part of the element bounded by physical landmarks on the bridge or a single physical unit of the element. The segments are represented diagrammatically so that, during an inspection, the condition rating of each segment (based on visual inspection) can be marked on the diagram. If structural assessments are carried out to determine the ratio of applied load to capacity of each segment, the condition ratings from inspections can be used to ascertain the residual strength of each segment and, hence, of the whole bridge.

Used in this way, segmental inspection can be an efficient method of updating structural assessments. Its accuracy clearly depends on the choice of segment, and it relies on the condition rating being determined by visual inspection. However, the method of subdivision also provides an effective

Table 1. Comparison of current and proposed inspection procedures

Current procedures			Proposed procedures		
Inspection type	Interval	Remarks	Inspection type	Interval	Remarks
Superficial	When needed	Cursory inspection. No standard report	Superficial	When needed	Cursory inspection. Simple report format
General	2 years	Visual inspection from ground	General	2 years	Visual inspection from ground. Improved report
				6 years	Visual inspection of other areas
Principal	6 years	Close visual inspection. All defects recorded	Benchmark	From 6 to 24 years depending on condition	Close visual inspection. All defects recorded
	6 years	Limited testing of specified areas	Particular	From 6 months to 12 years depending on condition	Detailed testing of particular areas
Special	When needed	Detailed testing of particular areas to suit structure	Special	When needed	Detailed testing of particular areas to suit structure
Joint	At construction completion	New structures	Special	At construction completion	No change
Initial Principal	At end of maintenance period	New structures	Benchmark	At end of maintenance period	Close visual inspection
Underwater	6 years	Part of Principal Inspection	Particular	6 years	No change
Scour	When needed	Special Inspection	Special	When needed	Detailed advice to be issued
Paint survey	When needed		Particular or Special	When needed	No change

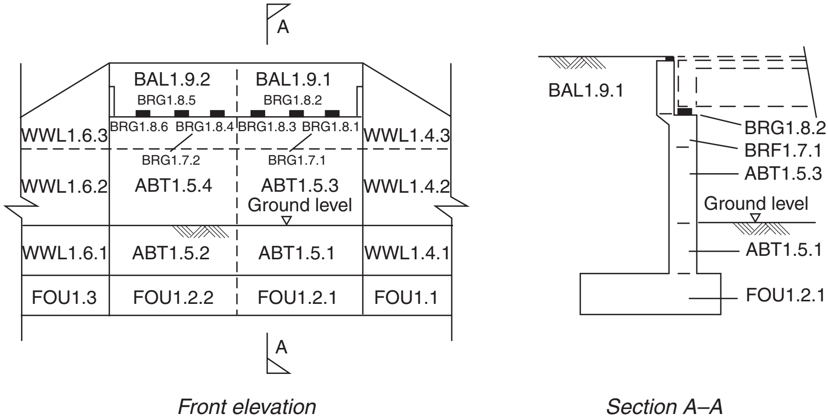

Fig. 1. Typical segments for a bridge abutment

means of defining the areas of the bridge which may be subject to different exposure conditions or different defects. Subdivision is therefore useful even if not used in conjunction with structural assessment.

The Highways Agency proposes to adopt a form of segmental inspection as part of the new procedures. Each bridge will need to be subdivided into segments, using guidelines in the new inspection manual, and diagrams prepared defining segment boundaries and numbering. The list of segments will form an integral part of the bridge records, with information relating to the bridge inventory, condition, inspection and maintenance records and planned future maintenance being recorded against each segment.

An example of subdivision of a bridge abutment into segments is shown in Fig. 1.

Use of information technologies

Information technologies are rapidly changing the ways in which bridge inspection and reporting are carried out. It is expected that data loggers, notebook computers and digital cameras will steadily replace traditional clipboard-and-paper methods of recording information during inspections. Similarly, computer databases are being used increasingly for storing the results. Used correctly, the newer methods can greatly improve efficiency and can provide ready access to data for retrieval, comparison and use in other applications such as the preparation of bids.[7]

Despite these innovations, it is expected that inspection results will still need to be reported using a combination of text, forms, photographs and drawings. Whether a form be a screen on a notebook computer or a pre-printed paper form on a clipboard is not necessarily important. Of prime importance are the content and clarity. With this in mind, the report formats for the new inspection procedures have been designed to be straightforward. They may be used in either paper or computer versions, but it is expected that, initially, paper forms will normally be filled in on site for later transcription to the computer database.

The new inspection procedures are intended to be used in conjunction with a computerised bridge database for the bridge inventory, inspection records and inspection programme. However, they have been designed so that they can also operate as a purely paper system. This is important, as it will enable the procedures to be implemented even by Maintaining Agents who do not have such a database.

Inspection manual

The proposed *Bridge inspection manual* will be divided into six volumes covering

1. inspection procedures
2. concrete bridges
3. steel and steel/concrete composite bridges
4. masonry arch bridges
5. retaining walls
6. other highway structures.

Volume 1 will set out the inspection procedures to be adopted on all highway structures. It will have chapters on

- *Planning*: desk study, planning and programming the work, equipment, training of inspectors and environmental considerations
- *Access*: advice on the various means of obtaining access and on the particular problems of working in areas such as on carriageways and in confined spaces
- *Health and safety*: general aspects of health and safety and risk assessment
- *Inspection procedures*: details of the procedures for the various categories of inspection, including reporting requirements
- *Inspection guidelines*: guidance on inspecting the various parts of bridges and culverts.

Volumes 2, 3 and 4 will provide guidance on the defects which are encountered on highway structures built of concrete, steel and masonry respectively. They will also contain advice on the test methods which are available for diagnosing defects. Comprehensive sets of photographs illustrating typical defects of varying severities will be included.

The final two volumes will contain advice on the inspection of other highway structures: retaining walls, high masts and sign gantries. Defects particular to these structures will be described and illustrated, and advice will also be given on defects in other structural materials, such as cast and wrought iron, timber and advanced composites.

The intention is that the manual will be self-contained, incorporating all the information needed to inspect bridges and other highway structures. At present this information is spread across several documents such as Departmental Standards and Advice Notes as well as the old *Bridge inspection guide*.[8] The manual will consolidate guidance into one authoritative publication.

New methods of testing bridges and diagnosing defects are being developed rapidly. The manual will not, therefore, attempt to provide definitive information on tests to be used. Instead it will outline current test methods, both commonly used and specialised, providing general advice, noting possible shortcomings and referring to more detailed publications. Inspection and reporting procedures will be such that the basic principles can be applied to any form of testing.

Since one of the aims of the manual is to improve the quality and consistency of reporting, examples of completed inspection reports will be provided as appendices. The new inspection procedures will be intended for use by Maintaining Agents on the Highways Agency's structures in England. However, the manual will also contain details of the existing procedures, which will continue to be used for trunk road structures in Scotland, Wales and Northern Ireland.

Drafts of the first two volumes — inspection procedures and concrete bridges — are nearing completion and will soon be issued to a few Maintaining Agents for field trials. Meanwhile, a start has been made on preparation of the remaining volumes. It is hoped that the full set will be available for general use by the spring of 2000.

Conclusions

The existing procedures for the inspection of bridges contain a number of shortcomings. Principal Inspections require the whole of a structure to be inspected closely irrespective of the likelihood of deterioration of any component. Also little use is made of nondestructive testing.

Following a comprehensive review, revised inspection procedures are proposed. Principal Inspections will be replaced by Particular Inspections, which will target areas of suspected deterioration and will include a range of tests. Reporting, especially of General Inspections, will be improved. It is also proposed that bridges be subdivided into segments for inspection purposes. The segments will also be used for recording bridge data in the bridge management system which is being developed in parallel.

A *Bridge inspection manual* is being prepared, setting out the new procedures and providing comprehensive guidance on many aspects of inspection. The manual will replace the *Bridge inspection guide*. Draughting of the first two volumes, covering inspection procedures and concrete bridges, is almost complete. A number of Maintaining Agents will soon be invited to undertake field trials to test both the procedures and the manuals before their adoption by the Highways Agency.

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Bridge condition index

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Introduction

As one of a suite of tools to assist in the management of their stock of highway structures the Highways Agency is developing a bridge condition index. The development of the bridge condition index has been entrusted to High-Point Rendel and Taywood Engineering (the Consultants). The authors form the main part of the Consultants' team working on the development of the bridge condition index; the Highways Agency's contract officers are Gerry Hayter and Paresh Tailor.

Objectives

The bridge condition index (BCI) can be expected to serve a number of purposes

- At its simplest level it provides an indication of the change in condition state of an individual bridge (or an element of a bridge) over a period of time. More importantly it can provide an indication of the change in condition state of the entire bridge stock or part of that stock.
- By considering the entire stock the BCI can, over a long period of time, provide an indication as to whether the levels of funding being provided nationally are adequate to maintain the stock in a steady state of repair.
- By considering different types of construction, the funding applied and the BCI, it might also be possible at some time in the future to provide an indication as to which types and materials of construction are truly the cheapest in the long run, a subject upon which many bridge engineers have argued and upon which there are many entrenched views.
- By considering the funding levels provided and the BCI, it might be possible to achieve an indication of the performance of an agent authority when compared against another, or against the national average. This might prove useful in comparing agents when the agency arrangements change with time. However, it is recognised that when

comparing agents in different geographic areas, differences in bridge type, material of construction and age distribution will mean that this comparison can only be made with caution.

The following constraints on the applicability of a BCI are recognised

- It is *not* intended that the bridge condition index will provide information on the issue of functionality of a bridge from a strategic or traffic point of view (unlike some of the existing indices developed overseas).
- It is *not* intended that the bridge condition index will address the issue of prioritisation of funding for individual bridges, other management tools are being developed to serve that purpose.
- The bridge condition index might be used to address the issue of prioritisation of funding between agents but only in the sense that an agent may be seen to have a stock with an above average, or a below average, condition index requiring an adjustment to the base level of funding.
- Safety is not a primary concern in developing and using the bridge condition index, but safety is likely to be a factor in considering the consequences of the failure of a component or element.

In the long term the data gathered as part of the inspection process to 'feed' the new decision methods which will form part of the structures management information system (SMIS) are likely to provide the basis for the development of a bridge condition index with a truly statistical base. However, SMIS is still in the early stages of development and it is likely to be some years before the data gathered would be sufficiently mature to provide the basis for a truly statistical, element by element, condition index. In the meantime, and probably for many years to come, a condition index of a more traditional kind is seen as a very useful and important management tool. Inevitably, if a statistically based index is ultimately developed as an offshoot from SMIS, there is likely to be a period where both indices would operate in parallel.

From the start of the development of the index it was recognised that the various management tools under development and being discussed at this conference interact. In particular it was recognised that the BCI affects and is affected by the updating of the inspection manuals and will have an influence on future inspector training, as the index is reliant on the quality of codified inspection results.

Basic principles

For some years a national road maintenance condition index has been in use by the Agency and the bridge condition index is intended to provide a similar function, for bridges and other highway structures. The national road maintenance condition index is determined using inspection data on a rolling sample of the trunk road and motorway network, part of the sample changing each year. A study, carried out several years ago, to define the scope of the development of the BCI had recommended a similar approach, with special inspections carried out on a selected and rolling sample of bridges forming the database for the calculation of the BCI.

Very early in the development work by the Consultants team this approach was seen to have a major shortcoming, given the variability of structural form and material of highway structures the sample used would have to be considerably larger, as a proportion of the whole stock, than that used for the national road maintenance condition index, and the cost of special inspections for such a large sample was seen as prohibitive. Instead, after discussions with the Agency's officers, the decision was taken to develop an index which used data gathered during routine inspections, and stored in the national structures database (NATS). However, it was recognised that there might be a need for a briefing to inspectors to ensure that data are gathered in a sufficiently consistent manner to be of use in calculating the index.

Initial development of the condition index

Study of existing methods

The study commenced with a review of existing methods of describing bridge condition used by authorities in the UK and overseas.

The methods studied were

- the methods used in the bridge management systems of two English counties, Denmark, Holland and the State of Pennsylvania
- the US national coding guide for bridges
- the Swedish Department of Transport's method.

The Swedish method provided some very useful ideas which helped in the study of possible alternative methods, even though their data formats were a little different. The US method also provided useful information and ideas, but was somewhat limited by the fact that the US method incorporates many other factors, such as traffic capacity, into an overall index.

Without exception the methods used in the various bridge management computer systems were all devised with the basic aim of providing a simple measure of priority for maintenance expenditure. Although of general interest their aims were sufficiently different to those of the proposed condition index for them to be of little relevance.

Study of existing NATS data, and recommendations for future data formats

Data were downloaded from the NATS INGRES database and transferred to a database established specially for the project. In total, inventory (BE 13) and inspection (BE 11) data were provided covering 5248 structures. The inspection records went back to the start of NATS in the late 1970s. The total number of inspection reports provided was 22 112, made up of 60% General, 38% Principal and 2% Special. On the 22 112 inspection reports there were 188 100 lines of element by element report data. Clearly, this volume of data was a large enough sample (over 25% of all Highways Agency structures were covered) to be considered statistically significant.

Apart from those data fields which identify the structure and define the inspection (date, type, by, etc.) the data recorded on the BE 11 forms is

- *element type*
- *estimated cost*
- *extent* (A to D)
- *severity* (1 to 4)
- *work* (recommended)
- *priority* (for work)
- *PD* (indicating premature deterioration)
- *comment* field.

These are repeated for each element on the bridge. In total there are 33 element types currently recognised.

Estimated cost and *comments* whilst important, fields were not considered as usable as input to the BCI. *PD* (premature deterioration) is a relatively new field. As might reasonably be expected of a field which is intended to flag up special circumstances, even over the short period that it has been in use, there are very few entries in the NATS data in this field. This field also was abandoned as input to the BCI. Efforts focused on deriving a BCI using the fields: *element* type; *extent* of defect; *severity* of defect; *work* recommended; *priority* for work recommended, with the fallback that if these fields were insufficient for the calculation of an effective condition index additional data fields would be recommended.

The current and historical usage of the *extent*, *severity*, *work* and *priority* fields were studied. Prior to mid 1984 the method of describing defect condition was a G(ood) F(air) or P(oor) entry. These entries were stored in what is now the *extent* field. After mid 1984 the additional (*severity*) field was created and the current system adopted, although there have been changes over the years in the definition of what are acceptable data combinations in the *extent* and *severity* fields. Because the G.F.P. definition of element condition could not reasonably be used in a compatible way with the current extent and severity system, the data using the old system were abandoned and efforts were concentrated on the inspections carried out using the current system. These numbered 17 347 inspections with 147 195 lines of element data, so there was still a large enough database to provide sound information.

The historical usage of the *work* and *priority* fields was reviewed. It was seen that there is some confusion over their use and recommendations were made for future guidelines to be incorporated in the new inspection manuals. However, the fields are quite properly left blank for a very large proportion of the entries and because of this they were seen as not useful fields for input to the BCI.

By that stage in the study it was clear from data reviews and other studies that a workable index could be created using only *extent, severity* and *element* type. The proposed index was to be numerical and normalised to indicate that 100% is 'as new'. An important principle in establishing the BCI was seen to be that the numeric index derived for a particular bridge should accord with the interpretation of condition that a reasonably experienced engineer would have in inspecting the bridge.

The current acceptable usage of extent and severity is defined by the matrix

A1	–	–	–
–	B2	B3	B4
–	C2	C3	C4
–	D2	D3	D4

In the current system *severity* 4 is defined as 'severe defects where urgent action is needed'. There is no provision for an element to have failed completely.

This might be logical if the major elements of a bridge are considered, such as the beams or the foundations. However, that is not logical as a general principle, as there can be many cases where an element may fail completely but the bridge can remain in service. Perhaps the best examples are expansion joints and some types of bearings such as elastomeric strips. In an ideal world we would replace such damaged elements as a matter of urgency as their continued use will, in time, damage the bridge. However, strategic and funding restrictions often mean that such repairs are delayed and the team have seen hundreds of instances of bearings and expansion joints continuing in service for years after their failure.

It was recommended that the reporting and data recording systems be altered to accept an *E5* extent severity condition. This will allow engineers to report nonfunctional elements without the need for alarm where this is appropriate.

The current definitions of *severity* mix the *severity* of a defect, that is its potential to damage the bridge, with the *priority* for repair, creating duplication with the *priority* field and some confusion. The following definitions for *severity* were proposed

1. No significant defect.
2. Minor defects which currently do not appear to be causing damage to the structure, for which no action is currently needed but which should be observed in future inspections.
3. Moderate defects which appear to be causing damage to the structure, and which should be considered for repair if funds permit.
4. Severe defects which are clearly causing damage to the structure and which must be repaired.
5. Element nonfunctional.

Development, field trials and adjustment of the condition index algorithm

A BCI was proposed based on two major factors: the extent/severity factor S_f and an element factor E_f. Both were given a value between 1 and 10.

Initial trials were carried out on a series of bridges on the A3 and the A1, referring to previous inspections as recorded in NATS. The bridges were inspected by the Consultants' team and the old inspection data compared with current observed condition. The BCI was calculated using both the old inspection data and compared with the team's 'view' of the condition of the bridge on a 1 to 100 scale. The first round of trials with the initial proposal for the BCI algorithm showed too small a spread of index for the range of condition actually observed.

To weight the index, the concept of primary and secondary elements was introduced. Primary elements being those (such as foundations, main beams, etc.) the failure of which would either bring about the collapse of the structure or at least render it unusable.

After a series of studies into the sensitivity of the algorithm to different factors and after further field trials the formula below was proposed for the bridge condition index

$$\mathrm{BCI} = 100 - F_1 \times [F_2 \times (E_{\mathrm{fp}} \times S_{\mathrm{f}})/N_{\mathrm{p}} + F_3 \times (E_{\mathrm{fs}} \times S_{\mathrm{f}})/N_{\mathrm{s}}] \qquad (1)$$

where *BCI* is the condition index, E_{fp} is the element factor (primary elements), E_{fs} is the element factor (secondary elements), S_f is the extent/severity factor, N_p is the number of primary elements on bridge, and N_s is the number of secondary elements on bridge. F_1, F_2 and F_3 are a series of factors.

With the proviso that if any primary element has an *E5* extent/severity factor the condition index for that bridge should be set to 0 (zero), that is the bridge should be considered as non-functional.

The age factor

The BCI formula described above will produce a value of 100 for a bridge if all elements are reported on and if all elements have an extent and severity of A1. However, if the BCI is to be used as a measure of the effectiveness of maintenance in any way, such as considering funding levels for the whole stock or part of it, it is important that perfection, the 'as new' condition, is not a target. It must be recognised that as a structure ages some deterioration is both inevitable and acceptable and therefore the target for ideal maintenance is to maintain the structure at a level which will optimise the structure's working life against expenditure.

The concept of an age factor has been introduced to allow for this. If the acceptable condition of a bridge at age 50 years is a BCI of (say) 70 then it is wasting money to try and repair it to achieve a BCI of 80. The target for ideal maintenance for any bridge (or summed for a group of bridges) then becomes

$$\text{current BCI} \times \text{age factor}/100$$

The pilot trial

During the first phase of the study, efforts were concentrated on bridges. A pilot trial is now in progress during which the ideas developed for bridges in the first phase (and trialled on relatively small numbers of bridges) will be trialled on a large number of structures and the BCI adjusted if necessary. The condition index is also being extended to cover other types of highways structure; gantries, culverts and retaining walls.

Data handling methods have been extended to allow rapid re-processing of large volumes of data if changes in the algorithm need to be made. The data available in the study database covers 3486 bridges, 659 culverts, 416 gantries and 425 retaining walls. It is possible that additional data may need to be added later to properly cover dry stone retaining walls but other types of structures, style of construction and material are reasonably well represented in the data currently in use.

Figure 1 shows the average condition index plotted against time for all bridges in the sample. Although the change from a 'good, fair, poor' method of condition reporting started in 1984 there is a significant amount of data going back to 1979 reported in the current extent/severity format. It is assumed that some agents re-set their data when the 'new' system was introduced in 1984. The data for the 1979 to 1984 period is, however, less per year than for later years. The tail off in the data plotted in Fig. 1 after 1995 is assumed to represent that fact that the data was taken from NATS early in 1996 and the probability that many 1995 and 1996 inspections simply had not been entered into the database at that time.

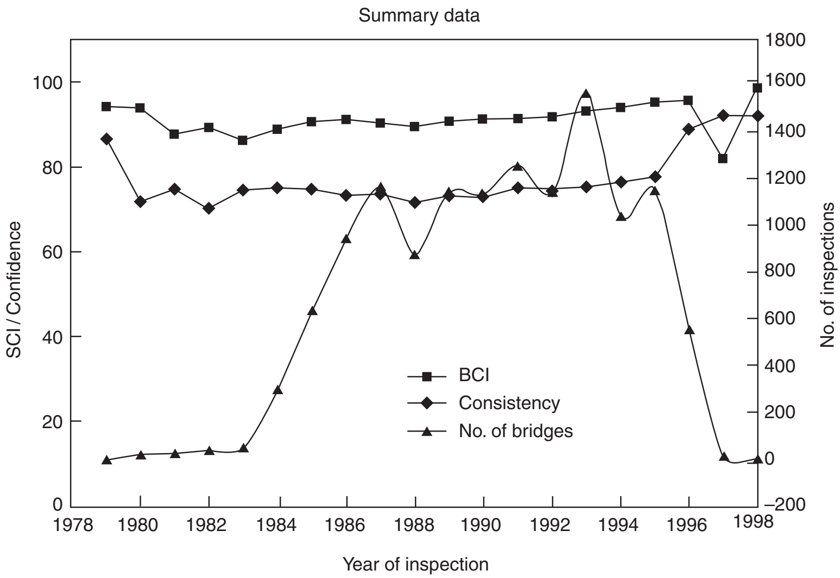

Fig. 1. BCI versus time

A sensitivity study is currently underway and has shown a rather alarming incidence of nonreporting on elements. This does not, as might be expected, simply apply to General Inspections. To review this nonreporting a 'confidence factor' has been introduced. The confidence factor is defined as

Number of elements reported on/number of elements which exist in the bridge $\times$ 100

Figure 2 shows BCI and confidence factor against time for Principal Inspections on in situ reinforced concrete bridges.

A small number of the inspections with very low reporting (confidence factors <40%) might be Special Inspections of parts of the bridge incorrectly reported and this is being studied, but such a large proportion of principal bridge inspections with only part of the structure reported on is, to say the least, a matter of concern.

In the long term, when dealing with inspections carried out to the new methods being developed, the confidence factor will become an irrelevance, or at best a means of checking that inspections are being fully reported on. In the short term it allows efforts to focus on areas of the existing data in which the data are reliable.

The new inspection manuals currently being developed are expected to recommend a segmental approach and that inspectors report on a 'per defect' basis rather than a 'per element' basis. That is, there will in many cases be several entries in the inspection report per element and a greater

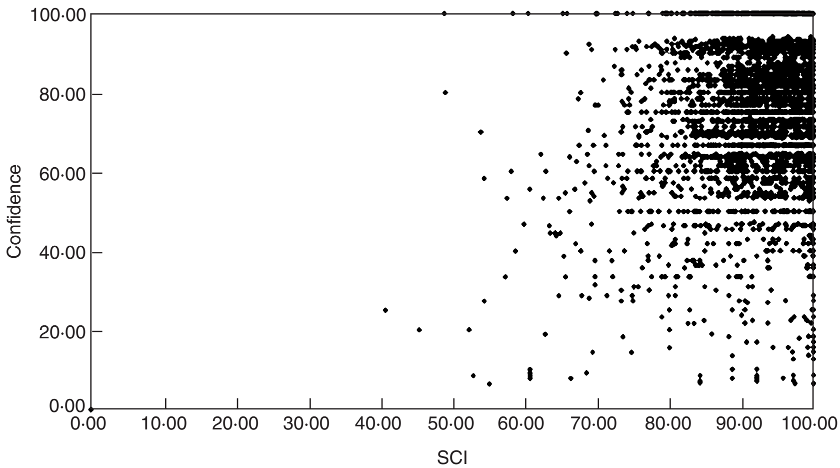

Fig. 2. BCI for concrete bridges

number of elements. The algorithms described above are being studied to allow for the change in data format, but early indications are that this will not represent a serious impediment to the generation of the BCI.

Implications for the future

A number of recommendations relating to the data that is stored in NATS (or SMIS) and to the way that inspections are reported have already been made as a result of the studies leading to the development of a bridge condition index. It is likely that as the pilot trials continue further recommendations will be made.

One thing is, however, very clear; if a condition index of any sort is to be used as a management tool for highways structures, inspectors *must* report on all elements of a bridge at each inspection, whether the element is visible or not. Current practice is to say the least, very variable.

Advanced methods of assessment for bridges

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Introduction

The process of assessment is of crucial importance for maintaining highway bridges in a safe and serviceable condition. The objective of assessment is to evaluate the safety of an existing bridge quickly and with a minimum of effort. However, the assessment rules and criteria need to be established rigorously and judiciously. If assessments are unduly conservative, structures will be unnecessarily strengthened, or needless load restrictions will be imposed. Conversely, if the rules are too lax some bridges could actually fail during service.

Codes and standards for design and assessment employ partial safety factors to ensure an appropriate level of safety for bridges. These factors guard against extreme variations in design parameters (e.g. material properties, applied loads, etc.) which could occur during service. In order to ensure that the design rules are simple for routine use the values of the partial factors are chosen such that they cater for a wide range of structure/component types and failure modes. Of necessity, therefore, the rules tend to be conservative for the majority of bridges and the level of conservatism varies considerably from structure to structure.

The partial factors used in BS 5400: Part 3[1] for steel bridges have been derived from extensive reliability-based calibration. This process ensures a more uniform level of safety (or, more precisely, 'reliability') across the most commonly used bridge and component types. However, the criteria could still become over conservative for a particular structure which may be significantly different from the norm. The partial factors for reinforced and prestressed concrete bridges have not similarly been derived using extensive reliability-based calibration and as a result greater variation in their safety levels could be expected.[2,3] Furthermore, the safety is checked at a component level using in most cases elastic methods of analysis and, therefore, do not take account of the reserve strength of the whole bridge system. The assessment criteria do not take into account consequences of failure, for example a bridge on a motorway and a small culvert on a local road are assessed to the same criteria.

There have been no reported cases of highway bridge failures as a result of traffic overloading. The design rules are therefore considered to provide an adequate level of safety. However, this does not necessarily mean that the safety level is optimal, it may well be conservative and the extent of conservatism will vary for every structure. In order to ensure that the assessment rules provide a more consistent level of safety across the bridge stock, and to avoid unnecessary strengthening costs due to over-conservative assessments, it is necessary to develop criteria which can be tailored to each bridge by taking into account its particular safety characteristics.

Levels of assessment

Assessment of an existing structure may be carried out in stages of increasing complexity. Initial stages may involve simpler but conservative means of assessing the load carrying capacity of a structure. If the structure is shown to be adequate using these simpler methods, then no further analysis would be required. However, if the structure is found to be inadequate, advanced methods could be used to remove any conservatism in the calculations in order to avoid unnecessary strengthening of the structure.

Based on the methods and tools available at present, Das[4] identified five levels of assessment for highway bridges, with Level 1 being the simplest and Level 5 the most sophisticated. These levels have now been formally recognised in the draft BD 75,[5] and are summarised below.

Level 1. (Simple) This level uses simple analysis methods and code specified material properties combined with full values of partial factors given in the standards.
Level 2. (Refined) This level involves refined analysis and better structural idealisation and may involve grillage analysis, finite element analysis and yield line analysis where considered beneficial.
Level 3. (Bridge specific) This level uses structure specific material properties and loading. Characteristic values for material properties may be established from records. Alternatively, worst credible values for material properties may be established by taking core samples from the structure. The derivation of bridge specific assessment live load (BSALL) may be beneficial for a long span bridge located on a lightly trafficked road. For short span bridges, the recent BD 21/97[6] already provides different levels of live load based on traffic density and pavement condition, and it is generally not considered cost effective to derive BSALL. Consideration may also be given to load testing in accordance with BA 54.[7]
Level 4. (Modified partial factors) Levels 1 to 3 assessments are based on the full values of the partial factors given in the current standards. The Level 4 method goes further to amend the assessment criteria, taking into account any additional safety characteristic to the structure being assessed. This is discussed in detail later in this paper.

Level 5 (Reliability analysis) This level involves the direct application of structural reliability analysis techniques. This method does not use the partial factors; instead, it characterises the uncertainties in the basic design parameters, using probability (or statistical) distributions, and evaluates the probability of failure for the structure. This is discussed in detail later in this paper.

As can be seen from the above, each higher level of assessment uses one or more refinements over the lower levels of assessment. It is not necessary that all the refinements mentioned above are implemented in every case. The most appropriate method(s) should be chosen for the structure concerned. There is no requirement in every case to continue the analysis up to Level 5 nor is it necessary to apply the above levels sequentially. However, it is likely that assessment to Levels 1, 2 or 3 would be carried out prior to the Level 4 or 5 assessment.

At present, guidance for carrying out assessments at Levels 1, 2 and 3 are contained within current DMRB (*Design manual for roads and bridges*) Standards and Advice Notes, whereas such guidance is not available for Level 4 and 5 assessments. WS Atkins Consultants have recently been commissioned by the Highways Agency to develop procedures for Level 4 and 5 assessments.

The Level 4 method will be calibrated using reliability analysis techniques, hence the Level 5 method needs to be developed first. In the following, the key issues involved in the development of these advanced methods are discussed along with proposed approaches for their development. This is followed by an example illustrating the application of these methods and the benefits that can be expected.

Level 5 method

The Level 5 assessment involves the direct application of structural reliability methods. The probability of failure of a structure depends on the uncertainties in load and resistance parameters, as well as other factors such as gross error, freak events, poor workmanship, etc. Structural reliability methods are now widely considered to be a rational means for treating the uncertainties in design parameters. In this approach, the uncertainty in each design parameter is modelled using an appropriate probability distribution function and the probability of failure, or equivalently the reliability index, for the component or the overall structure is calculated. The techniques for determining the probability of failure are now available and can be used relatively easily to complete an analysis in modest time frames.

The probability of failure computed from a reliability analysis should be treated as a 'notional' value to be used in a relative sense to compare reliabilities of different structures. It should not be interpreted as a measure of the frequency of failure which could be expected in service, i.e. in the sense that one out of so many bridges would fail. This is because the probability distributions used in a reliability analysis are intended to represent inherent variability in design parameters which could be expected for structures which are designed, constructed, operated and maintained to good engineering practice. They do not account for gross errors in design (e.g. incorrect calculation of loads or capacities), construction (e.g. the use of wrong steel reinforcement, missing bars, poor quality of construction) or in operation (e.g. gross overloading of vehicles) which are seen as the cause for most failures observed in practice.

In addition to the probability of failure, a reliability analysis gives sensitivity factors for the uncertain variables which give a measure of the relative importance of each variable (on a scale of 0 to ±1) to the probability of failure of the component. These factors will help in identifying important parameters for which further data should be collected in order to reduce the uncertainty and hence to increase the computed reliability of the component.[3]

The Level 5 method provides a greater flexibility for incorporating service data (e.g. load testing, material testing, measurement of dimensions, deterioration, etc.) and any additional safety characteristics (e.g. warning of failure, consequences of failure) for the specific structure and thus provides a more rational assessment of its safety. However, extensive statistical data for all the variables are not readily available, and the results are seen to be sensitive to the statistical distributions and the methods of structural analysis used.

Guidelines for using the Level 5 method are being developed for the following aspects of reliability assessment

- overview of the procedure
- limit states
- modelling of component capacity
- modelling of system capacity (or reserve strength)
- probabilistic modelling of basic variables
- computational methods for the calculation of probability of failure
- acceptance criteria (or target reliability)
- interpretation and assessment of results.

The guidelines will be presented in the form of a BA document. In order to ensure that assessments carried out by different engineers are consistent there is a need to standardise the reliability analysis procedures for bridges. The input probability distributions and the target reliability levels also need to be standardised so that assessment engineers are not faced with this daunting task each time a reliability assessment of a bridge needs to be performed.

The experience from other industries suggests that, in order for the guidelines to be widely accepted and correctly used, a careful balance needs to be established between the flexibility allowed for the user on the one hand and the need to ensure consistency on the other. The guidelines should be easy to use, unambiguous, and should be structured so that practising bridge engineers can perform these assessments with appropriate help from reliability specialists. The Advice Note will include clearly illustrated and fully worked out examples for a number of bridge types.

Target reliability

In order to assess a bridge using reliability analysis it is necessary to specify a target reliability value above which a bridge may be considered to be acceptable. At present target values have not been specified in the UK bridge codes, although some of the North American codes specify these values for bridge assessment.

In choosing a value for target reliability, one is essentially faced with the question of *How safe is safe enough*? It involves complex technical, social and economic issues and requires value judgements. In principle, three approaches can be recognised for establishing target reliability levels[8]

- socially acceptable risk levels derived from historical data
- calibration to existing codes and standards
- economic optimisation.

The first approach, adopted for example by Menzies,[9] is limited in that it cannot easily be related to the 'notional reliabilities' computed from a reliability analysis. Furthermore, as bridge failures due to overloading have not occurred in the UK, direct calibration against bridge failure statistics is not possible. The difficulty with the economic optimisation approach is in the accurate evaluation of all direct and indirect consequences of a bridge failure. The second approach, although not free from difficulties, seems to be the only practicable way forward at the present time. This, in effect, provides a means for taking into account the conservatism in current standards for particular structures.

Careful consideration will need to be made in deciding whether separate target values should be used for each structure type (for example, RC slab bridge, RC girder-slab bridge, etc.) or a single target to be used for all bridge types. It is now commonly accepted that the target reliability levels should take into account the warning of failure and consequences of failure of a structure. Thus bridges with low consequences, for example those carrying a minor road over a small span, could have a lower target reliability than a bridge carrying a motorway. This, in effect, adjusts the code levels of safety for higher and lower than normal risks.

Clearly, the target reliability levels proposed should be consistent with the inputs and methods used for reliability analysis.

Level 4 method

As discussed previously, the partial factors in present codes have been chosen to cover a wide range of structure types, component types and failure modes. As a result they may be over-conservative for a particular structure. The Level 4 assessment method aims to account for any additional safety characteristic to that structure and amends the assessment criteria accordingly. The Level 4 method will be incorporated into the standards as a BD document.

The criteria for Level 4 assessments would be developed based on detailed reliability analyses of a number of bridge and component types. The precise format of the method needs to be chosen carefully. It is tentatively proposed that the current safety checking format and the values of partial factors used in Level 3 assessment are retained for general use, and any amendments to the criteria are derived in the form of modifications to these partial factors for the specific bridge being assessed.

As part of the Level 4 methodology, modifications to partial factors would be derived to take account of bridge specific characteristics such as

- measurements of actual dimensions and thickness of surfacing
- live load to dead load ratio or live load factor (LLF)
- age of the structure and its expected remaining service life
- reserve strength and redundancy
- inspection and monitoring regime linked to warning of failure
- consequences of failure/removal of the bridge from service.

It is not intended at this stage to perform an extensive reliability-based code calibration covering all bridge and component types. To start with, those cases that are most likely to benefit from a Level 4 assessment will be identified and the calibration exercise limited to these cases. The calibration could later be extended to cover other bridge types.

Example

The following example is based on work carried out on the Midland Links viaducts which presented particularly difficult problems with regard to the assessment of the deteriorated condition of the structures which are suffering from chloride induced corrosion.

The scope of the structural assessments was to determine the 'latest intervention date' at which a crossbeam becomes 'critical' from a safety point of view. These results were then used to develop a maintenance/repair programme for the deteriorating crossbeams by taking into account whole life costs, practical constraints and the technical feasibility of available repair methods. The work was carried out jointly by WS Atkins Consultants Ltd and Maunsell Ltd for the Highways Agency.[10]

While this example, which includes the time dependency of deterioration, is more complex than would normally be the case for Level 5 assessments, it illustrates the method and the benefits which may be realised. Further examples can be seen in Shetty *et al.* (1997).[3]

Level 2 assessment

A Level 2 assessment of the crossbeams was initially carried out as a part of Stage II of the 15 year rehabilitation programme. The load effects were generally determined using a grillage analysis of the decks and a plane frame analysis of the crossbeam supported on RC columns. These assessments were focused on evaluating the as-built capacity of the crossbeams. Where crossbeams had shown signs of severe deterioration, a Condition Factor was applied to the calculated as-built capacity. The factor was evaluated subjectively based on a Condition Rating determined from Principal Inspections.

Level 3 assessment

The assessment method used in Level 2 was significantly refined in order to evaluate accurately the capacity of deteriorating crossbeams. The main areas of refinement included the derivation of a Midland Links specific assessment live load model, development of a deterioration model and the establishment of the bond strength of delaminated sections by testing.[10] Typical results from the Level 3 assessment showing the variation in Capacity Ratio with time can be seen in Cropper *et al.* (1998).

Level 5 assessment

The partial safety factors given in the current codes do not take into account the additional uncertainties involved in the rate of deterioration and the strength of deteriorated components. For this reason, a Level 5 assessment was carried out to provide assurance that the Level 3 method results in adequate levels of safety.

The probability distributions for material parameters, geometric quantities and model uncertainty variables were established based on the information published in literature combined with judgement and experience. The distribution for traffic loading was derived by simulating traffic on the Midland Links viaducts as discussed previously. The distributions for deterioration parameters such as rate of corrosion, delamination length, etc. were derived by statistical analysis of corrosion

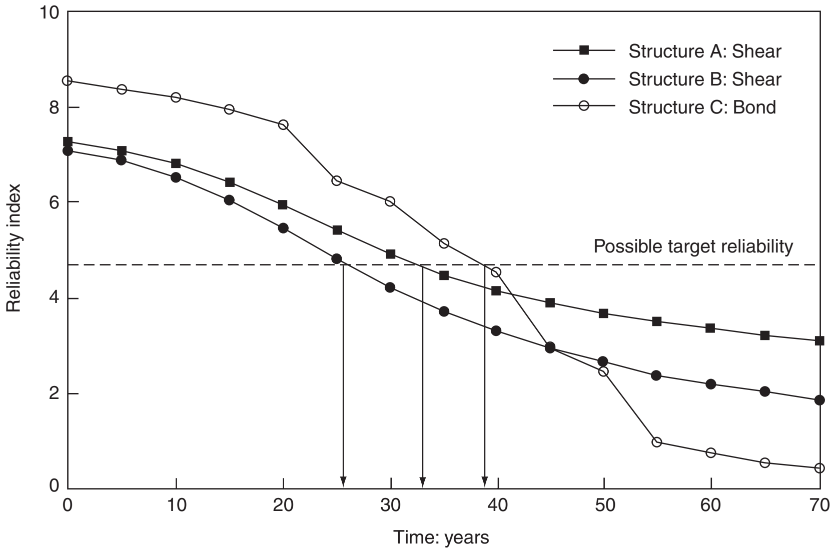

Fig. 1. Examples of reliability profiles

loss measurements on crossbeams, and for the bond strength of delaminated bars from experimental data on scale models of crossbeams.

The probability of failure was calculated using proprietary software, which utilised generally accepted methods of reliability analysis. Both first order and second order reliability methods (FORM and SORM) were used.[11] Due to the combined effects of corrosion of longitudinal and shear steel, and the delamination of concrete cover over the reinforcement, the capacity of a crossbeam, and hence its reliability decreases with time. The variation in reliability can be established by repeating the reliability analysis for various time intervals.

Typical reliability profiles for bond and shear failure modes are shown in Fig. 1. As discussed previously, a target reliability value for Level 5 assessment needs to be established by a careful calibration to current standards. Based on limited studies, a provisional target reliability value of 4·7 (equivalent to a notional annual probability of failure $= 1 \times 10^{-6}$) was chosen. Based on this, the time at which a crossbeam becomes critical could be determined as shown in the figure. Typical plots of sensitivity factors for shear failure are shown in Fig. 2, and those for bond in Fig. 3.

Typical plots of sensitivity factors for shear failure mode are shown in Fig. 2, and those for bond in Fig. 3.

The sensitivity factors for the shear failure mode show a similar trend for the different crossbeams. The reliability of the crossbeam in earlier years is seen to be dominated by the uncertainty in shear strength model, load effect analysis and the live load. As the corrosion begins to take hold of the crossbeam, the sensitivity factor for the corrosion rate of shear steel

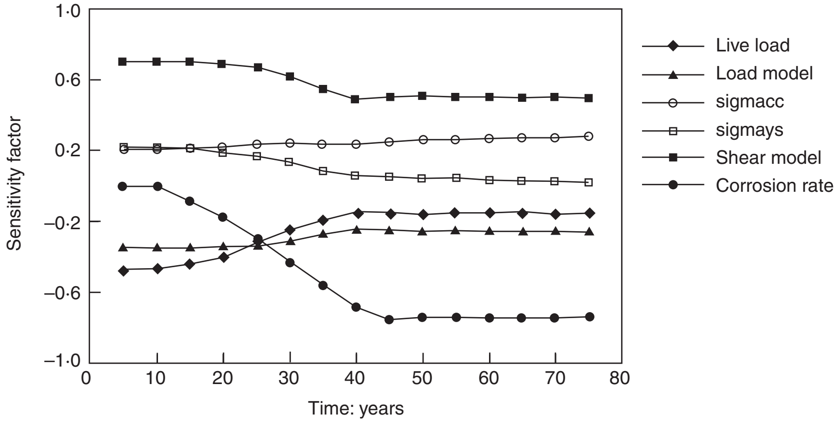

Fig. 2. Sensitivity factors for shear failure

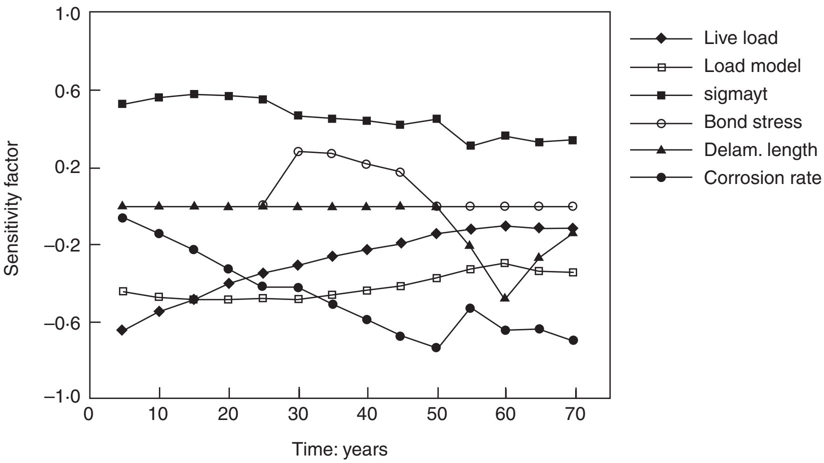

Fig. 3. Sensitivity factors for bond failure

increases rapidly (as the reliability decreases) while the sensitivity factors for other variables decrease. After a certain time the sensitivity factors stabilise and in this regime the reliability is predominantly influenced by the uncertainty in corrosion rate.

On the other hand, for bond failure the sensitivity factors vary considerably between different crossbeams and at different points in time. The sensitivities depend on the anchorage length of bars, number of layers of reinforcement, initial length of delamination as a proportion of the anchorage length, etc. For crossbeams with multiple layers of reinforcement or for sections with lapped bars, the probability of bond failure is seen to be very low, as is to be expected. While for crossbeams which have a single layer of main reinforcement and short anchorage length, the reliability is seen to be very sensitive to the variability in delamination length, bond strength and corrosion of main and shear steel.

Level 4 assessment

Within the Midland Links project, a full Level 4 method was not developed since the Level 5 method was directly applied to a number of crossbeams. From the extensive comparisons made between the Level 3 and Level 5 results, the following approach is suggested for developing the Level 4 method for deteriorating RC elements.

In addition to the partial factors given in current codes for loading and resistance parameters, new partial factors would need to be established for the bond stress of delaminated bars, corrosion rate, delamination length and an overall resistance partial factor to account for the considerable uncertainties involved in the estimation of deterioration and the capacity of deteriorated RC sections. The current values of partial factors for loading and material variables may need to be reduced as the sensitivity of reliability with respect to these variables reduces as the element deteriorates with time. The values of partial factors can be derived using a reliability-based calibration method, see Thoft-Christensen and Baker (1982).[12]

Summary and conclusions

In order to avoid unnecessary strengthening of bridges while maintaining the safety of road users, the assessment rules and criteria need to be established judiciously. Based on the methods available at present five levels of assessment have been recognised which enable the assessment of an existing bridge to be carried out in stages of increasing complexity.

Guidance for carrying out assessments at Levels 1, 2 and 3 are contained within current standards while procedures for Level 4 and 5 assessments are under development. The paper discusses the key issues involved in the development of these advanced methods.

The Level 5 assessment involves the direct application of structural reliability methods. Although these techniques are now well developed, there is a need to standardise the procedures for application to bridges. The Advice Note under preparation for Level 5 assessment aims to standardise the basic variables and their probability distributions, incorporation of service data, methods of reliability calculations and interpretation of results. Recommendations will also be included on target reliability levels.

The Level 4 method allows the partial safety factors to be modified taking account of the safety characteristics specific to each bridge. Modifications would be provided for factors such as measurement of actual dimensions, live load to dead load ratio, reserve strength and redundancy, consequences of failure, etc. The Level 4 method will be calibrated using the Level 5 method and will be presented in a Departmental Standard.

The application of various levels of assessment has been illustrated for RC crossbeams of the Midland Links viaducts which have undergone chloride induced corrosion. Starting from the conventional Level 2 method, the assessment process was refined in stages to obtain a more accurate evaluation of deterioration and load carrying capacity. The Level 5 method was employed to take into account the considerable uncertainties involved in estimating the rate of deterioration and the capacity of deteriorated RC sections, and to provide an assurance that the lower levels result in a safe assessment of the crossbeam.

The example has demonstrated that Level 5 reliability analysis can be effectively carried out on complex structural assessment problems and can be a viable tool for bridge assessments.

Acknowledgements

The work described in this paper was carried out under two separate projects for the Highways Agency in London and Birmingham. We would like to thank Dr Parag Das, Mr David Cropper and Mr Lionel Wellappili from the Highways Agency, and a number of colleagues from WS Atkins and Maunsell Group for their contribution. Thanks are also due to the Highways Agency for its permission to publish these results.

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Part 5. Software systems development

Structures management information system (SMIS)

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Background

The Highways Agency is responsible for a stock of some 15 000 trunk road and motorway structures in England including around 10 000 bridges. Maintenance of the stock is undertaken by maintaining agents (MAs) who until recently were local authorities operating within the authority's boundary under long standing agency agreements. These arrangements are being replaced by 24 larger maintenance areas which are tendered competitively. Within the Highways Agency each area will be managed by an area manager and particular routes managed by route managers.

In 1996 the National Audit Office (NAO) examined and reported progress on a programme,[1] which was commenced by the Department of Transport in 1987, to upgrade and strengthen motorway and trunk road bridges and other structures to current standards. It also examined the Agency's management of structures maintenance resources and considered the scope for improving systems to enhance future performance. With regard to management information systems, weaknesses were identified in the way that the Agency's existing national structures database (NATS) operates and deficiencies in the range and timeliness of the information collected. The need for the Agency to have greater assurance centrally about the funding of, and work done by, maintenance agents was also highlighted.

The Highways Agency recently undertook a review of the whole spectrum of its engineering procedures and requirements for structures management.[2] This paper describes proposals for the development of a new structures management information system (SMIS) which will bring together information from a number of management tools to provide rational and consistent methods for the prioritisation of works and the determination of future maintenance needs. The following are considered

- The existing national structures database (NATS)
- Current bridge management systems
- Overview of SMIS
- SMIS architecture
- SMIS development programme.

The existing national structures database (NATS)

The Agency's existing national structures database (NATS) is an INGRESS-based system developed in the early 1980s to aid management and detailed record keeping of the stock. It was later expanded to include financial data to assist in the allocation and management of the annual maintenance funds. The main elements are:

- inventory
- inspection records
- assessment and strengthening
- bid, allocation and outturn information on maintenance funding.

A number of problems have been identified with the system

- too unwieldy, difficult to access or analyse data. There are some 320 pieces of information/structure accessed through 57 screens
- insufficient data validation
- slow data entry
- little on screen help and supporting manuals are cumbersome.
- some information and reports users require are not readily available and may need computer specialist help to access

Two contracts have recently been completed which have addressed the key issues. The lack of data validation has been a major factor in the quality of information which is held. One contract has sifted through the NATS data and identified default and incorrect data, and lists have been sent to maintaining agents for completion. Recommendations on data validation will be taken forward from this contract and applied in the SMIS development. The second contract looked at the NATS user interface and the scope for introducing a Windows based front end involving a small workshop of users in a review of the proposals. The findings from this work will be an important input to the future development of SMIS. Experience with NATS has shown that the system for data entry and procedures to ensure accurate and timely input are key to ensuring the system is well populated with data and widely used.

Current bridge management systems

A number of computer based bridge management systems (BMSs) are being developed and tried at present, of which Pontis[3] seems to be at the forefront. Pontis has been developed on behalf of the US Federal Highway Administration, where as a similar system Bridgit[4] had been developed through the AASHTO (American Association of State Highway and

Transportation Officials) sponsored National Cooperative Highway Research Program. Other government sponsored BMSs are the Finnish 'SIHA'[5] and Danish BMS 'DANBRO'[6] In addition there are a number of BMSs which have been developed and marketed commercially.

These systems process inventory, inspection and condition details and use deterioration models to predict future deterioration and produce optimised maintenance strategies at network level. The Highways Agency's system will not combine information in a fully automated process. The modular nature of SMIS will provide the necessary flexibility to accommodate network management issues which often arise and intervene in the overall maintenance strategy.

A major difference from other BMSs is that the identification of maintenance needs used by the Highways Agency will be based on structural adequacy or safety rather than on the condition state of the structures. Similar structures designed and constructed to the same standards will for various reasons have different inherent safety levels even just following construction. This means that in use some bridges will have large margins of safety even with extensive deterioration, whereas others may become unsafe with only minor deterioration. Structures management decisions need to consider safety requirements rather than just observed or measured deterioration. The Highways Agency is developing a methodology for a Safety Index for bridges.

Overview of SMIS

SMIS will hold data and provide software tools for all the structures management procedures. Development will be on a modular basis so that as new maintenance procedures are finalised they can be added. Using modules will also allow new processes and procedures which may arise in the future to be readily integrated in the system.

Figure 1 shows how SMIS will interact with the structures management operational procedures within the Highways Agency. An annual strategic programme review[7] is carried out to set the priorities for the bidding/allocation of maintenance funds for a particular year and advice issued to Maintaining Agents in the *Trunk road maintenance manual*[8] prior to preparation of bids. The Highways Agency is developing and piloting a route strategy approach to work on its network and in due course this will influence structures maintenance work allocations and programmes. SMIS will also interact with other Highways Agency initiatives. The state of the network project is being designed around a map based computer system to present key data on the state of the trunk road and motorway network to area and route managers. SMIS will provide the structures information.

The seven modules which make up SMIS are listed below.

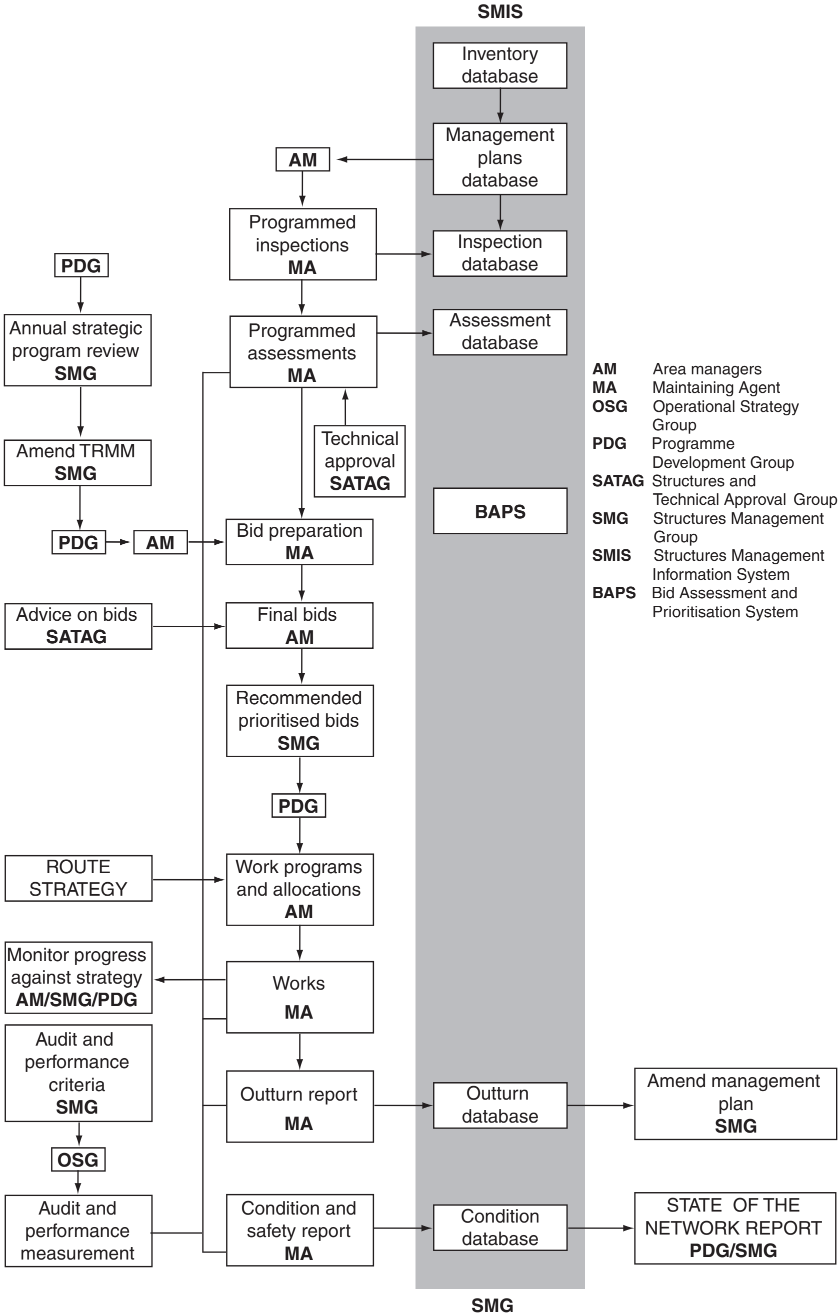

Fig. 1. Interaction between SMIS and structures management operational procedures within Highways Agency

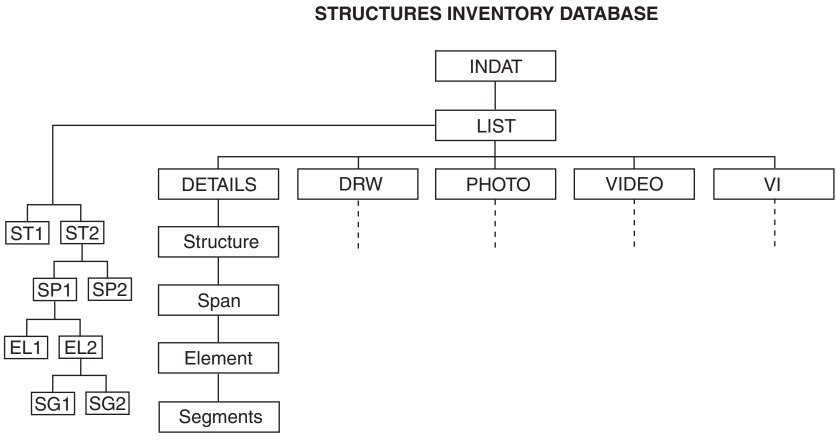

Fig. 2. Proposed arrangement of INDAT

Structures inventory database (INDAT)

The structures inventory database will be the core of the system with the existing NATS data being transferred to the new database. It is envisaged that the inventory will over time hold new information on structures in the form of drawings, photographs, video and virtual images.

The proposed arrangement of INDAT is shown in Fig. 2. The LIST feature will be the identifier that links the components of the database. The LIST, along with the other components, will contain information at different levels working down from the Structure (ST), to structure span (SP), to structural element (EI), to segment (SG), as appropriate.

Management plans database (MPLAN)

Management plans[9] will be the focus for a plan led approach to structures maintenance. The plans will identify all the work that needs to be undertaken on a structure and provide a longer term view of maintenance needs. Timing of past and future actions will be recorded in diary format. The proposed arrangement of the database, MPLAN, which will hold this information is shown in Fig. 3.

Inspection database (INSP)

Data from the revised inspection procedures which the Highways Agency are currently developing[10] will be held in the INSP database. This database will be directly linked to MPLAN.

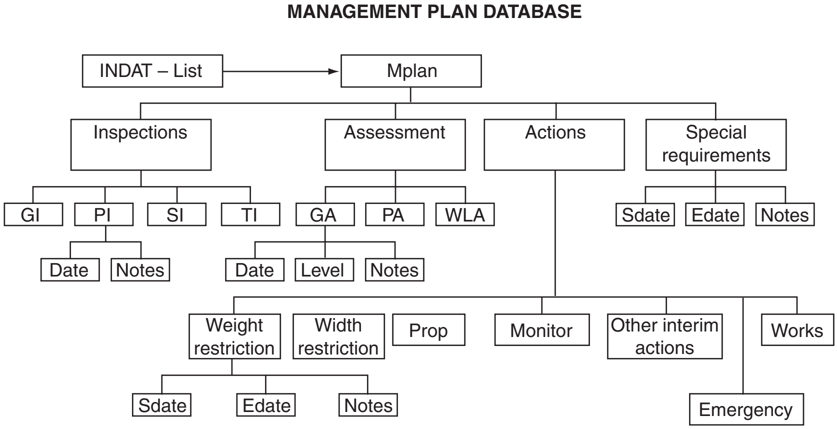

Fig. 3. Proposed arrangement of MPLAN

Assessment database (ASSESS)

There will be a continuing need to undertake assessments as part of the on-going management of the structures stock. Comprehensive details of the results of all assessments including those for abnormal loads will be held in ASSESS.

Bid assessment and prioritisation system (BAPS)

The new Highways Agency's bid assessment and prioritisation system (BAPS)[11] which it is intended will replace the current bidding system using BE14 screens in NATS is undergoing trials and is likely to be the first module of SMIS to be completed. Fig. 4 shows the interaction with whole life assessment input prepared by MAs.

Outturn database (OUTTURN)

The Outturn Database will hold a comprehensive record of the outturn costs of structures maintenance work.

Condition database (COND)

The Condition Database will hold information on the Condition Index (CI)[12] and Safety Index of the structures stock. The proposed arrangement of the database which will hold this information is shown in Fig. 5.

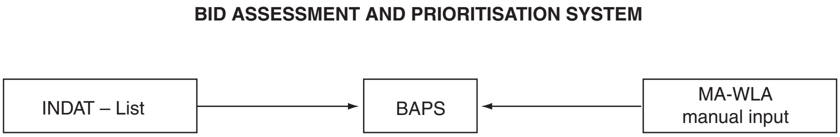

Fig. 4. Interaction between BAPS and whole life assessment

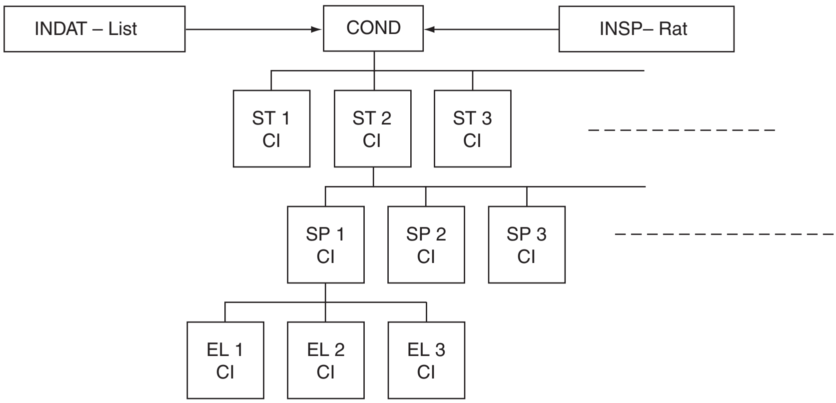

Fig. 5. Proposed arrangement of COND

SMIS architecture

The primary architecture of the SMIS database has been established and draft specifications for the technical requirements of the user interfaces for LIST, DETAILS (the construction details held within INDAT), BAPS and MPLAN are being prepared.

LIST is a front end application, which will define each structure on the road network in terms of structure, span, elements and segments. The need for segmentalisation of a structure is driven by the proposals for segmental inspections and assessments whereby information relating to a specific element or segment may need to be entered onto the database.

It is envisaged that for each structure, LIST will hold schematic diagrams at each level of subdivision illustrating the various components of span, element and segments. The diagrams are intended to provide guidance to the user in selecting the correct component reference number for accessing the relevant database records held against that component.

The ‘Structure’ Diagram will be a basic plan layout of the structure and will contain sufficient details of the location to verify that the correct structure has been selected (Fig. 6).

The ‘Span’ Diagram will illustrate how a particular span is subdivided into the component elements. The elements will be defined and numbered following the procedures set out within the proposed new *Bridge inspection manual* currently under development. There could be up to 20 elements

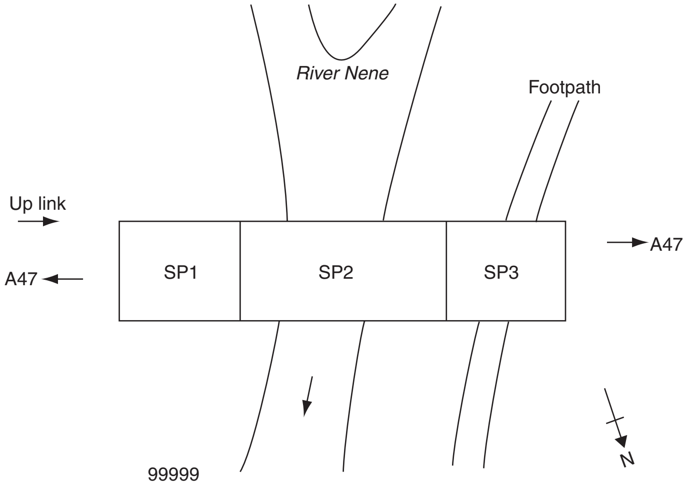

Fig. 6. Structure diagram

within one span, and the illustration shown is a very simplistic representation (Fig. 7).

The Element Diagrams will fulfil a similar role in defining the segments of which it is comprised. The need for Segment Diagrams will depend on the level of detail recorded against that segment within the database.

Once the structural subdivisions have been defined, all data held in SMIS within each module (e.g. DETAILS, MPLAN, BAPS, etc.) will be attributed to either the structure generally, or to the relevant span, element or segment component. The data will be accessed by means of a suitable

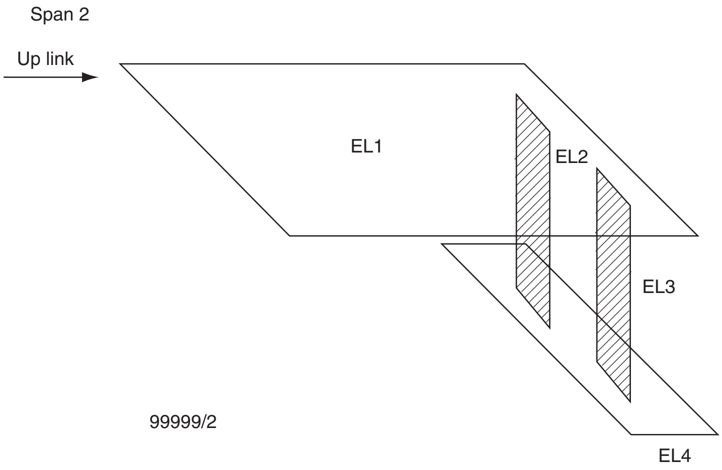

Fig. 7. Span diagram

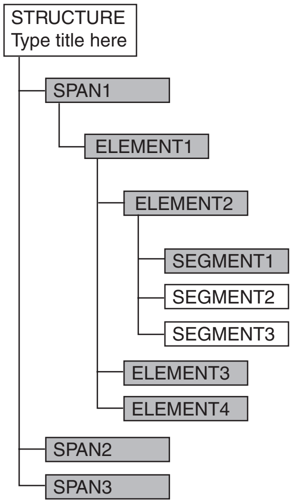

Fig. 8. Drill-down mechanism showing means of data access

drill-down mechanism, such as an expanding tree type explorer (Fig. 8). Not all components of every structure will necessarily hold data, and consideration will be given to giving each tree node a marker indicating whether data is held against that component in each module.

As-constructed information

This is currently entered onto NATS using the BE13 form, and supplemented by hard copy ROADS 277 forms. Within SMIS, this information will be either cleaned and transferred from NATS, or re-entered into DETAILS using new SMIS data sheets. These options are currently under consideration.

In general, the header and location file fields on the BE13 form will be allocated to a LIST header screen, and most of the fields in the bridge, small culvert, retaining wall files, etc. allocated at structure level within DETAILS. Most of the fields within the bridge type span file will be allocated to span level within DETAILS. The SMIS elemental subdivisions will permit greater detail to be allocated to all types of element than is currently possible utilising the component (parapets, joints, etc.) and element files within NATS.

Querying and reporting within the SMIS database

It is believed that although the current NATS general report writer facility is a versatile tool, it is neither universally accessible to all users, nor fully appreciated in terms of its capability. It is an important objective of SMIS to provide a user friendly and versatile querying and reporting facility for each of the SMIS modules, to the benefit of all end users.

SMIS development programme

The target date for completion of SMIS is the end of the year 2000. BAPS is expected to be the first module to be completed followed by INDAT and MPLAN. Software development for other modules will follow as the remaining maintenance procedures are finalised.

Acknowledgements

This paper is presented with the kind permission of Mr Lawrie Haynes, Chief Executive of the Highways Agency.

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Strategic planning of future structures' maintenance needs

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Introduction

The Highways Agency's duties include ensuring that trunk road and motorway structures are maintained adequately. This involves both long and short term responsibilities: not only must bridges be maintained safely this year and next, but also the general level of maintenance must be such as to avoid a general deterioration which would lead to future funding crises. Currently the Agency spends approximately £170 million per year on bridge maintenance.

The 15 year bridge rehabilitation programme was launched in November 1987 in order to carry out a planned rehabilitation of trunk road structures and, in particular, to ensure that bridges were adequate for the proposed 40 tonne vehicles. Although the programme is nearing its end, there will always be a need for rehabilitation as bridges continue to deteriorate.

This paper describes a review which is being undertaken of predicted maintenance costs. The results are being used to prepare a rolling programme rather than a fixed period programme. Various maintenance strategies have been considered and the work is to be developed to form the basis for strategic planning of maintenance of the Agency's bridges. The review has been based on a methodology developed by the Highways Agency,[1] together with Maunsell, which supplied the cost information, and the Transport Research Laboratory, which carried out the necessary computation.

Structure maintenance strategy

The prime objective of bridge management is to maintain the structures in a safe condition. The Highways Agency has to secure sufficient funds to enable it to discharge this responsibility. In order to justify these funds, the Agency needs to have a strategy for maintenance and needs to be able to answer

- why is the maintenance necessary?
- are the funds being used effectively?
- what will happen if the maintenance is not carried out?

The current study is attempting to supply the answers.

The previous 15 year rehabilitation programme was described in terms of steady-state maintenance, assessment and strengthening, upgrading and routine maintenance.[2] As part of the revised strategy the work has now been divided into routine, preventative and essential maintenance. These are defined as

- *routine* Minor work carried out on a regular basis, such as clearing of drains and bearing shelves, etc.
- *preventative* Maintenance work which repairs defects, replaces components or otherwise slows the rate of deterioration, and may enhance the strength of the structure to some extent. Examples are steelwork repainting, expansion joint replacement, silane impregnation and cathodic protection.
- *essential* Rehabilitation work undertaken when a structure is considered to be (or about to become) structurally inadequate. The work will strengthen the structure. Examples are major concrete repairs, replacement of structural elements and strengthening arising from the bridge assessment programme.

A feature of essential maintenance is that, if the work were to be delayed, it would be necessary to implement some interim measure to ensure the safety of road users. Interim measures could include weight restrictions, temporary propping or even complete closure and would normally involve both maintenance costs (capital and traffic management) and road user delay costs.[3,4]

In an ideal situation, with a uniform mix of bridge types and ages, and with adequate funds, the expenditure would be as shown in Fig. 1. With this regime, sufficient routine and preventative maintenance are carried out each year for the essential rehabilitation work to be kept at a steady level. If, however, insufficient funding were provided each year, the amount of essential work required for structures to remain in service would start to increase. After a while these rehabilitation needs would take up most of the available funds so little, if any, preventative work would be carried out (Fig. 2). Unless significant extra funds were provided, large scale weight restrictions or road closures would eventually be required.

Clearly, the situation depicted in Fig. 2 must be avoided, but the ideal of Fig. 1 is unlikely to be achievable in reality. There will always be special programmes arising from time to time and the actual mix of bridge types and ages will lead to peaks and troughs in the need for essential maintenance. It is the purpose of the strategic long term plan to identify the optimum expenditure profile due to these variations and to estimate the consequences of other courses of action.

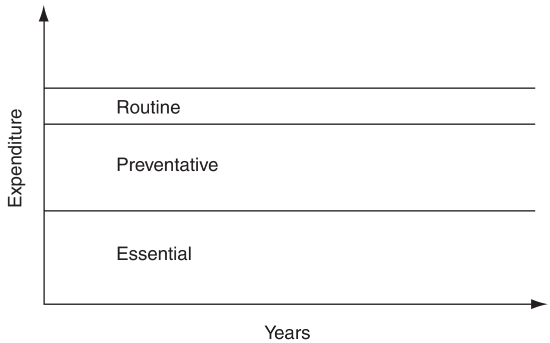

Fig. 1. Ideal bridge maintenance programme

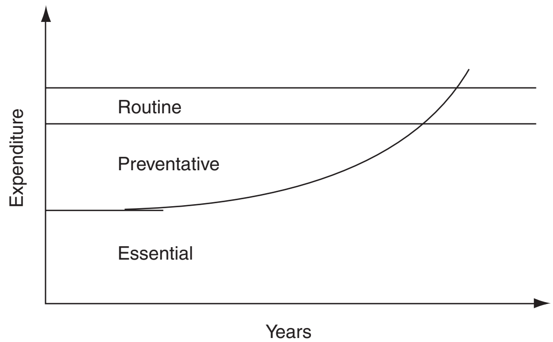

Fig. 2. Effect of long-term underfunding

Typical maintenance costs

The first task of the review was to estimate typical maintenance costs which could be applied to the whole stock of Highways Agency structures. Using the Agency's national structures (NATS) database the bridge stock was sub-divided into four broad types — steel/concrete composite, in situ reinforced concrete, pretensioned concrete and post-tensioned concrete. These categories cover 85% of the bridge stock; the remaining 15% of bridges were distributed among these four categories to ensure the whole stock was considered. For each of the four materials, two typical bridges were selected, an overbridge and an underbridge, to represent the average size of bridge for that type. Thus, for example, the typical reinforced concrete overbridge, determined from the information on the NATS database, is 56·8 m long by 11·5 m wide with 3·3 spans.

Four Maintaining Agents were consulted to obtain costs for a range of maintenance operations and the associated traffic management. These costs were then pooled to derive an average cost for each activity. The Agents also provided information on the extent of work normally required for each item.

For each bridge, two programmes of maintenance were devised and the costs estimated: one assuming regular preventative maintenance and the other assuming no maintenance until essential rehabilitation work is required. Preventative maintenance was assumed to include

- repainting structural steelwork
- replacing deck joints
- rewaterproofing the deck
- minor concrete repairs
- re-application of silane impregnation to exposed concrete
- regrouting post-tensioned tendons.

Cathodic protection was also considered as an option for the reinforced concrete bridges but, in the event, these costs were not used in the review.

Essential maintenance which would eventually be required after a regime of preventative maintenance was assumed to include

- major concrete repairs to abutments, piers and deck ends
- replacement of bearings
- replacement of steel or aluminium parapets
- tendon replacement or enhancement on post-tensioned bridges
- normal preventative maintenance due at the time major works are undertaken.

For the regime with no maintenance until essential work was required, it was assumed that the following would eventually be needed

- complete replacement of the deck
- replacement of piers above foundation level
- major concrete repairs to the abutments.

All the costs calculated were divided by the plan deck area of each bridge, since it was considered that a rate per unit area would give a better representation of overall costs than a cost per bridge. The costs calculated for the reinforced concrete overbridge are shown in Table 1.

Predicting the numbers of bridges needing rehabilitation

The age of the bridge when essential maintenance is required depends on its rate of deterioration and the effectiveness of any preventative maintenance it may have received. Therefore, with information about the number of bridges built each year, previous maintenance work and the rate of deterioration, it is possible to estimate the number of bridges needing rehabilitation in each future year.

However, while the numbers of bridges of each type built in each year can be obtained from the Highways Agency's records (NATS), adequate

Table 1. Estimated maintenance costs for a reinforced concrete overbridge

	Preferred regime: Preventative maintenance (cost/cycle)	Preferred regime: Essential maintenance Sub-structure	Preferred regime: Essential maintenance Super-structure	Alternative regime: Essential maintenance after no maintenance Sub-structure	Alternative regime: Essential maintenance after no maintenance Super-structure
Cycle time (mode) Years	20	80	90	35	45
	£/m^2	£/m^2	£/m^2	£/m^2	£/m^2
Preventative maintenance					
Expansion joint replacement	15				
Waterproofing decks	16				
Silane impregnation	6				
Concrete repairs	10				
Essential maintenance					
Concrete repairs		37	10	33	–
Deck/pier replacement		50	–	46	539
Bearing replacement		–	40	–	20
Parapet replacement		–	94	–	20
Preventative works		6	30	–	–
Total works	47	93	174	79	579
Design and supervision	6	11	21	11	66
Traffic management	16	21	38	26	76
	£69/m^2	£125/m^2	£233/m^2	£116/m^2	£721/m^2
Road user delay costs	£157/m^2	£398/m^2	£343/m^2	£1,420/m^2	£4,988/m^2

details of preventative and essential maintenance are not readily available. Prediction models for rates of deterioration were therefore derived from expert opinion. A number of Maintaining Agents were consulted for their opinions, from records and engineering judgement, of the likely times to essential maintenance for the types of bridge considered. Using the information obtained, distribution curves were fitted to produce a set of distribution functions for each bridge type. Those for reinforced concrete are illustrated in Fig. 3.

By combining the distributions with the age profile data, the number or area of bridges of each type requiring essential maintenance in any year could be obtained. However, in order to do this it was necessary to estimate the beneficial effects of preventative maintenance in the past. Data were not readily available, so some simple assumptions were made. In essence, it was assumed that bridges built since 1984 will have received preventative maintenance but that the older bridges will have had no preventative work until their first essential maintenance. From these deliberations the

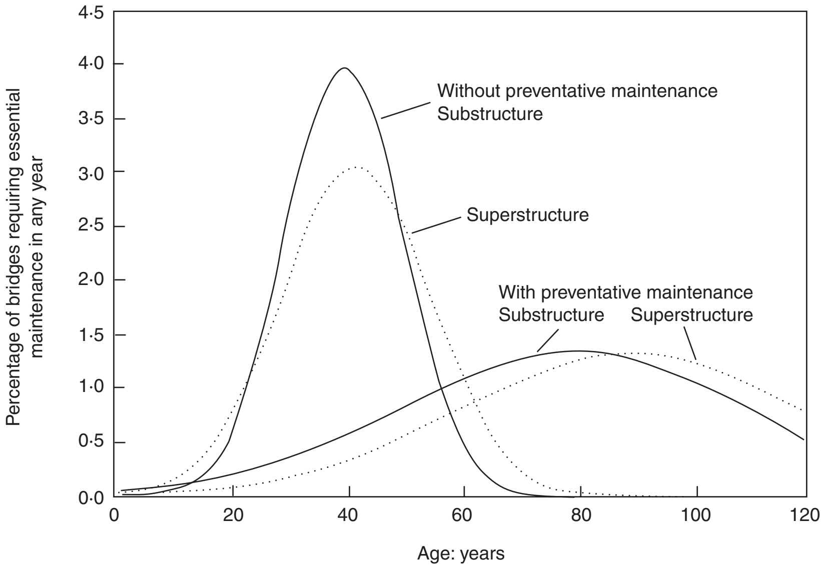

Fig. 3. Essential maintenance cycle times for reinforced concrete bridges

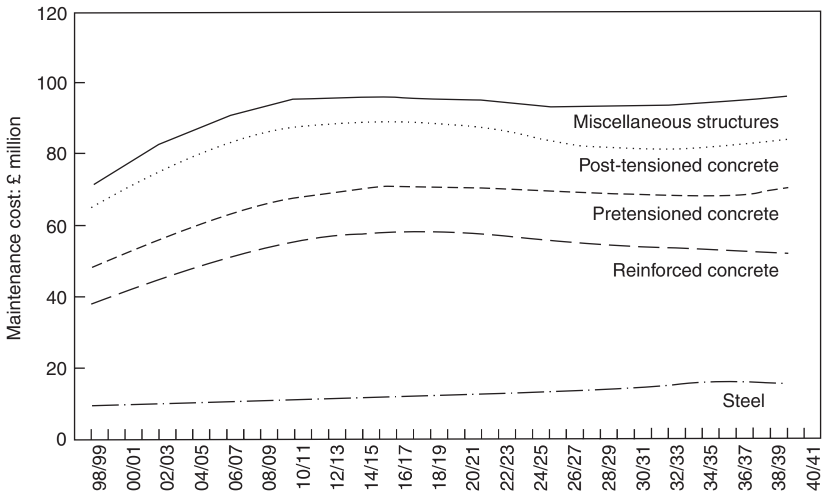

Fig. 4. Essential maintenance costs

essential maintenance costs for the whole bridge stock in each year could be derived (Fig. 4).

A similar procedure was adopted to predict future preventative maintenance costs as a constant annual amount. A simple triangular distribution was used, assuming that all bridges required preventative maintenance at cycle times of between 10 and 25 years, with a mode of 20

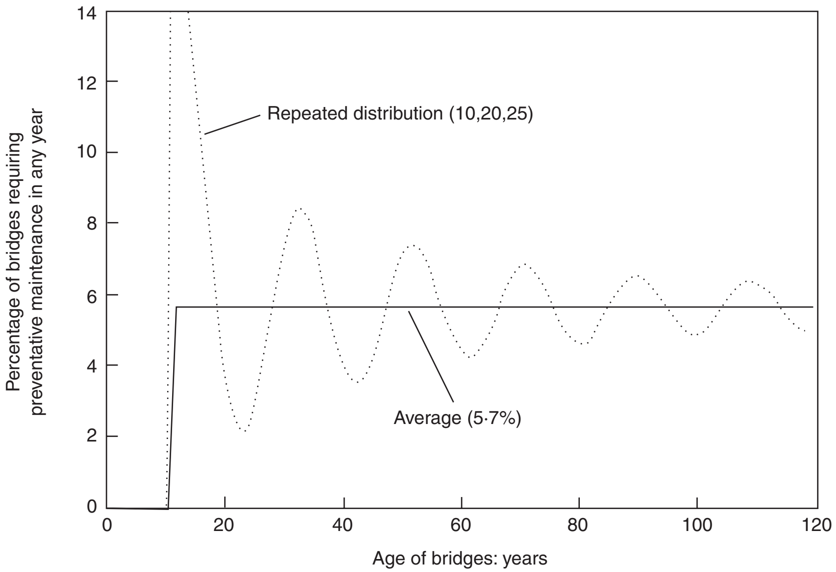

Fig. 5. Preventative maintenance cycles

years. Repeated application of this distribution showed that the probability that preventative maintenance will be required approaches about 0·057 in the long term (see Fig. 5). Thus where preventative maintenance is appropriate it has been assumed that a uniform 5·7% of the stock will be treated each year.

Similar calculations were also undertaken to estimate preventative and essential maintenance costs for other types of highway structure: tunnels, culverts, retaining walls and gantries.

Routine maintenance and inspections have cost about £20 million annually for the past few years. It was assumed that similar funding would be required in the future.

Backlog of work

In addition to the maintenance discussed above, there is a backlog of other work arising from

- ☐ strengthening as a result of assessments for 40 tonne vehicles
- ☐ replacement of sub-standard parapets
- ☐ strengthening of piers vulnerable to impact.

Estimates of the cost of the remaining strengthening for 40 tonne vehicles were derived from figures supplied by Maintaining Agents. Preliminary estimates were also derived for the costs of pier and parapet strengthening, pending the results of a risk-based assessment which is currently underway. These costs were added to the global estimates assuming that strengthening for 40 tonne vehicles will be completed by 2001 and the pier and parapet work by 2006.

Road user delay costs

The costs of delays to road users due to maintenance works can be substantial, often exceeding the direct costs. The review therefore estimated the delay costs using QUADRO.[5] There are so many variables that the average delay costs calculated for a typical bridge can only give a broad indication of the order of magnitude. Nevertheless, the estimates showed that delay costs would, on average, be around twice the direct costs for preventative and essential maintenance, but considerably higher when major reconstruction is undertaken (Table 1).

If essential maintenance were not carried out, a bridge would deteriorate to an unacceptable level, such that emergency interim measures would be required to avoid overloading. Load or weight restrictions may be needed or the bridge may even have to be closed.[3,4] Most of the interim measures cause delay to the traffic over or under the bridge.

Assumptions were made concerning the frequency with which each possible option would be adopted as an interim measure on various types of bridge. From these, it was possible to derive the average road user delay costs for a typical bridge awaiting strengthening: it was about £3·65 million per year (£10 000 per day). Other calculations had already shown that the average essential maintenance cost of a typical bridge, following a regime of preventative maintenance, would be about £640 000. Thus, for each £1 million of essential maintenance not carried out in a year, road user delay costs of about £5·7 million per year would be incurred. These costs would continue each year until the essential maintenance was carried out.

While these calculations have been based on a series of very broad simplifications and are sensitive to the assumptions, it is considered that they give a reasonable indication of the order of road user delay costs when applied to a large bridge stock.

Conclusions

The overall estimated costs of maintenance for the Agency's structures are shown on Fig. 6. This shows that, once the backlog maintenance is out of the way, an annual spend of about £145 million at present day prices will be required to maintain the structures. The increasing annual cost over the next ten years, caused by the ageing bridge stock, will largely be masked by the backlog.

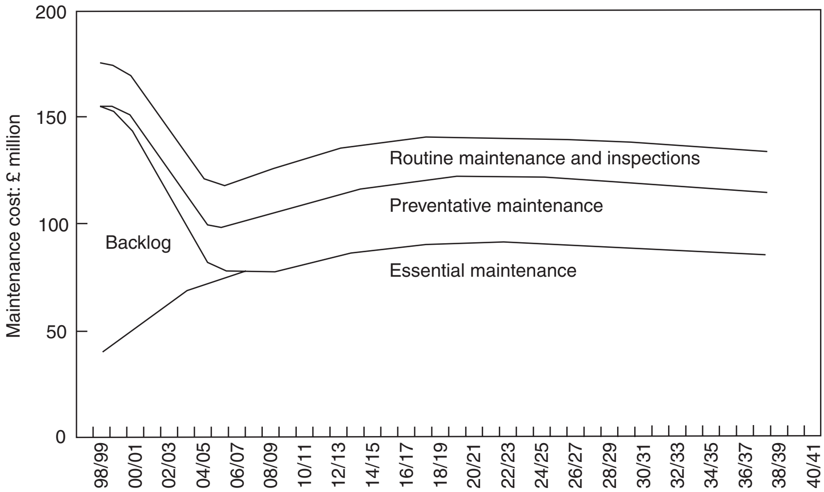

Fig. 6. Total maintenance costs

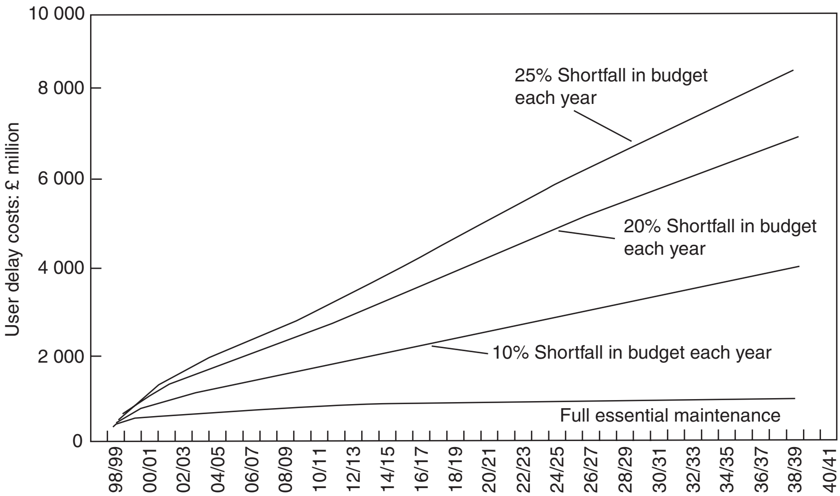

Fig. 7. Traffic delay costs due to underfunding

Underfunding would, in time, lead to bridges being closed or restricted while awaiting repair. The main effect would be road user delay costs of about £5·7 million a year for each £1 million of essential maintenance not undertaken. Fig. 7 shows the cumulative effects of underfunding: such a scenario would soon become unacceptable due to the disruption caused.

The strategic review has adopted a broad brush approach, making fairly coarse assumptions. There are many areas in which it could be refined. Examples include

- ☐ improved assumptions concerning the current condition of bridges
- ☐ collection of more data on rates of deterioration
- ☐ calculation of whole life costs and benefits of various preventative maintenance operations
- ☐ sub-division into more structure types
- ☐ allowing for future changes to the bridge stock.

The review is still in progress at the time of writing and final figures will not be available for several weeks. The details given in this paper are based on preliminary results and are therefore liable to change. Nevertheless, the review is already providing a sound basis for planning future expenditure. It has also shown that huge traffic delay costs would be incurred if essential maintenance were to be underfunded.

Acknowledgements

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Bid assessment and prioritisation system (BAPS)

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Introduction

The Highways Agency currently hold its structures information on its national structures database known as NATS.[1] Maintaining Agents use the financial input data forms BE14 and BE15 of NATS to submit bids for schemes for the maintenance of its structures on motorways and all purpose trunk roads. Bids for rolling programmes relating to major schemes and for the assessment and strengthening programme are similarly submitted. NATS is also used by Maintaining Agents to return the outturn costs. The existing system, however, does not look beyond the bid year in terms of future work profiles nor does it provide information on the consequences of not funding any part of the bid. Further, only one maintenance proposal is put forward for each scheme.

Current bridge management philosophy has moved towards structures maintenance based on longer term network strategy-based systems. There is a need to forecast maintenance needs better, to protect the Agency's structures stock asset and to make optimal use of limited resources. It is therefore necessary to develop rational systems which integrate the maintenance works with network structures maintenance strategies. The existing system, while meeting the needs of the past, is no longer suitable to confront the funding challenges of today.

The Highways Agency's proposed new computer-based structures bid assessment and prioritisation system known as BAPS stems from the Highways Agency's overall review of the management of the trunks roads which is detailed elsewhere.[2] BAPS will replace BE14 and BE15 in the future. The system will combine the strategic needs of the network with the maintenance needs of individual structures, taking into account alternative maintenance strategies, the application of whole life costing principles and the assessment of risk in terms of road user delays for not carrying out the work.

Underlying principles of BAPS

Structures are considered in terms of their constituent elements, each of which requires a particular maintenance regime. Bids are submitted for any element which requires work to be undertaken in the next bid year. These bids are based on formal structural assessments or on assessments based on engineering judgement of previous inspections. From these assessments the current performance level is determined which is used to determine the type of maintenance work required, i.e. whether the work is essential or preventative.

The type of maintenance work is determined by comparing the current performance level of an element with the level at which the performance of that element becomes unsafe. Maintenance work on elements below the critical level are categorised as *essential*. It follows that without maintenance the element would become unsafe and measures such as weight limits would have to be put in place. Such measures would lead to traffic disruption which can be estimated in terms of user delay costs.

If the performance level is above the critical level then the work is regarded as *preventative*. The argument for doing preventative work is that it will cost more later if the work is not done now. However, justification based on future cost increases alone is not a very strong one for securing funds for bridge maintenance. Traffic disruptions resulting from future large-scale weight restrictions imposed on unsafe bridges, however, is likely to be a more compelling reason for allocating funds.

By ensuring that appropriate maintenance work is undertaken at the correct time, a build-up of a backlog of work in future years would be averted.

Maintenance options

The Maintaining Agent will be required to consider and submit a number of bid options to undertake the maintenance work. Each option will cover a period of 30 years. Generally, three or four options would be considered by the Maintaining Agent but in practice the scope to consider more than one option may be limited due to the type and location of the element. The reason for requiring more than one option is that the best option for a particular scheme may not be the best option when strategic considerations of the whole bridge stock are taken into account. Where the maintenance work is a continuation of an existing contract then the work is regarded as *committed* and only one option is necessary.

In addition to the works itself, consideration would be given to programming aspects of the work i.e. when the maintenance work affects part of a series of related structures forms part of a hybrid scheme linked to highway works. Other factors to consider would be the deferring of work to future years, access restrictions (for example due to rail track possessions) and carrying out only minimal work in the bid year sufficient to maintain the element in a safe condition.

In proposing maintenance options the Maintaining Agent will also look at the implications of not doing the work. In such cases traffic disruption would need to be assessed and an estimate of the costs to road users evaluated. It should be noted that all decisions regarding the engineering and programming aspects of the maintenance work are dealt with outside BAPS and undertaken by the Maintaining Agent and the network manager who are best placed to make such decisions.

For each element, the bid comprises the incurred cost, and the traffic delay cost, for each year in which maintenance work is proposed. Each project/scheme may comprise work on a number of elements together. The cost benefit of combining work should be reflected in the bids through linking. The bids will include the following items

- year of occurrence
- works cost
- preparation cost
- supervision cost
- traffic management cost
- traffic delay cost
- work description
- work type (committed, essential, etc.)
- maintenance action type (concrete repairs, etc)
- link with other work including the resulting cost benefit.

Figure 1 shows the schematic representation of a 30 year bid for a number of elements.

The Maintaining Agents and regional offices will undertake the first sift of maintenance options to ensure that programme and other constraints which cannot be fully modelled by BAPS are taken into account in selecting whole life costed maintenance options to be offered to headquarters in the bidding cycle. A full sift of the maintenance options submitted by all the Maintaining Agents will be carried out at network level by the Highways Agency.

When both the network managers and the Maintaining Agents have prepared their bids and are satisfied that the works proposed can be carried out in their works programme, the structures maintenance bid options are submitted to the Highways Agency for assessment.

Options entered by maintenance agent

Element or structure	Maint. option	Y1	Y2	Y3	Y4	Y5	etc.
1	1		●	●		●	
	2	●	●	●	●	●	
	3					●	
	4			●			●
2	1	●				●	
	2			●			
	3			●		●	●
	4	●		●		●	
3	1	●	●	●	●	●	●
	2			●			
	3					●	
	4	●	●				●
etc.	etc.						

Fig. 1. Schematic representation of a 30 year bid profile

Prioritisation process

The first stage of the prioritisation process is by the work type, the order of importance being committed, essential and preventative. Other types of work may be added in the future as needs arise. Prioritisation by work categories forms the next stage. Work categories are analogous but not necessarily identical to the 24 work categories listed in Annex 2.8.1 of the *Trunk road maintenance manual*.[3] The order of priority given to these categories is determined by the Highways Agency and would reflect the Agency's maintenance strategy for the particular bid year. Different items may assume different levels of priority in subsequent years.

Once this ordered list is obtained the lowest 30 year discounted cost option is selected from the maintenance proposals for each element or structure and all the selected bid items totalled up and checked against the strategic plan costs. Where a 'preferred option' has been flagged up by the MA, this option is selected in preference to the lowest 30 year discounted cost option. The preferred option facility provides the MA with the flexibility to programme work more effectively.

For preventative work it has often been found that the 'do minimum' option becomes the cheapest maintenance option in present value terms. If this happens the implications of such bidding in terms of backlogs arising at a future date would be examined. If, on examination, the potential for backlogs looks likely, the bid would be amended to bring forward work in this category. Similarly, if the bids for essential works for a particular bridge type do not add up to the corresponding strategic plan item, it could

Work type	Work category	Structure/ element	Bid Year Cost (BYC)	Cumul. BYC	Traffic Delays (TD)	Cumul. TD
Committed	C1	–				
	C2	–				
	C6	–				
Current (inspections etc)	R1	–				
	R6	–				
	R8	–				
Essential	E3	–				
	E12	–				
	E17	–				
Preventative	P23	–				
	P24	–				

Fig. 2. Prioritised list of bids

mean that, in some cases, partial strengthening has been the cheapest present value option. Where this may result in premature rehabilitation, the bids would be revised to allow for full strengthening.

The prioritised list will be of the form shown in Fig. 2. It will show, for instance, from the bottom of the list, decreasing funding level and increasing traffic delay cost for not providing the full funding. Those involved with considering the options for different funding levels will be able to draw a line at the proposed level and get an indication of the cost of traffic disruption that is likely to take place from weight restrictions, etc. which would be necessary for maintaining safety if the full bid is not funded.

Once a set of proposals has been selected and the funding agreed, an allocation of funds is made available to each Maintaining Agent. The Maintaining Agent would then organise and carry out the works in that bid year. Following completion of the work, the Maintaining Agents would advise the Highways Agency of the outturns costs.

Software development and supporting documentation

Maunsell Ltd were commissioned by the Highways Agency to carry out a seven month project to develop a pilot computer-based management system for prioritising maintenance bids for the Agency's structure stock.

The overall objectives of the project were twofold

- ☐ to develop a methodology for assessing and prioritising the maintenance bids
- ☐ to develop a pilot computer system and associated documentation to demonstrate the viability of the proposed methodology.

Before the methodology could be developed, there were a number of issues which had to be considered

- □ what the Highways Agency wanted to consider in determining priority
- □ what information Maintaining Agents could provide
- □ how to compare the priorities for different types of elements
- □ how to minimise the amount of work required in creating the bid data.

After consideration of a wide range of possibilities the methodology outlined above was agreed with the Highways Agency. At this stage, it was obvious that the proposed bidding system would require somewhat more work to be carried out by the Maintaining Agents than under the current system, and a conscious decision was made to limit the changes as far as possible by utilising existing NATS data. For example, element types from Table II in BD 62;[3] work categories based on *Trunk road maintenance manual* — Annex 2.8.1.[4]

While the methodology was being established, a number of prototype systems were developed, so that the Highways Agency could see the types of screen displays which BAPS might use. The software was to run on a stand-alone 486DX PC running Microsoft Windows. There was to be no interface between the pilot software and the existing NATS database, and no direct connection between the Maintaining Agents and the Highways Agency. It was therefore necessary to devise a system which met all these constraints but which could be implemented on a full-scale network system with minimal modifications.

Consideration of all these constraints led to the adoption of a Visual Basic user interface with Microsoft Access used to provide data storage. Bid submission and notification of allocations and outturns were to be transferred by diskette, but were also readily adaptable to online network.

The system divided naturally into two sections; one for the Maintaining Agents to enter their bid and outturn data and one for the Highways Agency to carry out their assessments and prioritisation.

To simplify data entry and to minimise development time, drop-down lists and look-up tables were used wherever possible. This would also restrict Agents to selecting data of pre-determined types, enabling subsequent analysis of the data by the Highways Agency.

Pilot software and field trial

A field trial of BAPS followed on from the development work to demonstrate the technical viability of the proposed methodology and to assess the amount of work involved for the Maintaining Agents. The opportunity was also available to gain feedback into what additional features might be usefully incorporated.

One Maintaining Agent from each of four Highways Agency regions were invited to participate in the trial, together with representatives from the regional offices. The trial commenced with all participants attending a seminar. This explained the engineering principles, use of the software, data transfer procedures and reporting requirements of the trial. The appropriate sections of the software were then installed on a computer at each of the Maintaining Agents' sites and at the Highways Agency, accompanied by a hands-on training session in the use of the software.

In addition to the software, the participants received copies of the user manual, the Highways Agency's draft Departmental Standard *Whole life performance based assessment of highway structures and structural components*[5] and guidance on traffic delay costs. Additional help during the field trial was available via a telephone and e-mail hotline. All requests to the hotline were logged, with a record of the actions taken.

The field trial was to simulate two complete bid cycles. The Maintaining Agents were asked to submit bids for elements on ten structures, six in the first bid year and the remaining four in the second. The structures were to be selected from a list of twelve structure types, to encourage the participants to consider a wide range of structures.

The participants' reactions to the field trial varied widely. It was found that those Agents which used staff for the trial who had not attended either the seminar or the initial training session experienced considerably more difficulty than those whose staff had received training.

Particular difficulties were encountered in arriving at suitable element definitions and in deriving the traffic delay costs for not doing the work; all the participants requested further guidance in both these areas. That significant difficulties had been encountered with element definition was particularly disappointing because it had been hoped that existing Maintaining Agents with substantial experience of their structures would be able to do this relatively easily. Agents with less experience of their structure stock would find considerable difficulty without much more extensive guidance. Such guidance will, however, be provided in the revised *Bridge inspection manual*,[6] which is currently in preparation, but was not available for the field trial.

The Maintaining Agents envisaged using BAPS for strategic planning of their maintenance work as was also foreseen by the development team. However it is intended that strategic planning will be carried out in an entirely separate SMIS module which is still at a very early stage of consideration. It is important to remember that the purpose of BAPS is to assess and prioritise the maintenance bids, although in the longer term it may extract data from and provide data to other databases.

The overall feedback gained from the trial is that the general principles are good, particularly the ability to assess, in financial terms, the consequences of not funding maintenance; however the work involved in applying these principles is too complex. This is, at least in part, due to the lack of documentation explaining how to apply the engineering principles to the software. It is intended that this will be addressed in guidance documents which are currently being prepared.

Future developments

Much has been learned from the BAPS development work and the associated field trial. The Highways Agency are currently working on proposals to take forward BAPS to a fully networked system for use by the Agency and its Maintaining Agents. The network system will take on board necessary changes/refinements identified from or since the field trial.

Running in parallel with this development will be the publication of the *Whole life performance assessment of highways structures and structural elements* Design Standard which will enable the bid engineers to prepare their bids, guidance notes on the preparation of the bid data required for input into BAPS and the updating of the procedures in the Department's *Trunk road maintenance manual*.[2]

Summary

In summary, the Highways Agency's bid assessment and prioritisation system when implemented will combine the strategic needs of the network with the maintenance needs of individual structures, taking into account alternative maintenance strategies, whole life costing principles and assessment of risk in terms of road user delays. Where funding is limited, the implications on traffic disruption caused if the work is not undertaken will also be evaluated. While providing an objective means of assessing structures maintenance bids, BAPS will require that all engineering decisions remain with the Engineer and the Network Manager who are best placed to take these decisions.

Acknowledgments

This paper is being published with the kind permission of Mr Lawrie Haynes, the Chief Executive of the Highways Agency.

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Structure management plans

Neil Loudon and Andrew Wingrove, *Highways Agency, UK*

Introduction

Other papers at this symposium (published in this volume, see pp. 49 and 153) described the development by the Highways Agency of new structures management arrangements and databases. This paper focuses on two important aspects of these developments, the management plans database MPLAN and the whole life costing of maintenance options. The two items are broadly related in that once the best or most optimum whole life maintenance strategy is chosen for a particular structure, the key activities and their dates are to be recorded in the management plan.

Management plans

Management plans are intended to provide a snapshot of a particular structure at any point in time and a ready reference to crucial information for bridge managers. They will provide a historical record of significant past maintenance activities, decisions and incidents, and also allow a forward look for future works that are identified through inspections, testing and assessments as well as the programmed dates of routine activities such as inspections and assessments.

Need for management plans

The overall background for developing the Highways Agency structures management information system (SMIS) including the associated management plan database MPLAN, is the changing Highways Agency focus, with emphasis on the operation of the highway network and maintenance rather than new construction, and directed at the needs of the customer. There has also been a perceived need to develop a forward rolling plan-led programme of work. Previously there had been a tendency to look forward only to the next year's work, without co-ordinating and planning all the work necessary on a structure or group of structures on a routine basis.

The Highways Agency appoints a large number of Maintaining Agents to carry out all the maintenance work needed on its structures stock. Until now there have been 92 of these agents, now being reduced to 24 and they will cover larger individual areas. Experience has shown in recent years that collecting information from Maintaining Agents can be a very difficult and long drawn out process. Simple information such as how many bridges are assessed to be sub-standard or weight-restricted at any point in time takes many months to gather. It is hoped that the management plan database MPLAN will allow immediate access to such network wide information.

One aspect of the current changes to the Maintaining Agency arrangements is that until now most of the Agents were local authorities, who were highway authorities in their own right. They had more or less permanent tenure of the agencies. However, the new agents will be appointed for three to five years at a time, and it is possible that some agencies will change hands from time to time. It will therefore be more important from now on to have a permanent record of the past activities carried out on each structure and the corresponding future plans. There is therefore an urgent need for the database.

Overview of MPLAN

Use of management plans is not intended to replace existing documentation and records, but to pull together essential information in an easily assimilated format. The management plan database MPLAN will be an integral part of the Highways Agency's structures management information system, for structures on the trunk road and motorway network. It can be thought of in terms of an index to a book, where reference is made to chapters (or files) for greater details. This other information may be elsewhere in SMIS or in hard copy form in archives. So far as possible, data entry will be kept to a minimum, and will be downloaded from the information system and structures inventory. However like all systems, it will be only as good as the data entered, and will require to be regularly reviewed to keep it up to date.

The principal aim of adopting management plans is to provide a snapshot picture of each structure, and quick access to crucial information, and to provide an 'alert' for the bridge or network manager. Each plan is intended to contain, in principle, the following items

(*a*) Past activities
- (i) maintenance works
- (ii) assessments
- (iii) inspections/testing
- (iv) departures from standards
- (v) events and incidents
- (vi) management decisions

(*b*) Forward look
- (i) planned maintenance
- (ii) planned inspections
- (iii) planned assessments

The overall structure of the information to be held in MPLAN is shown in Fig. 1.

The database MPLAN is intended to operate in conjunction with the LIST component of the structures inventory database. LIST identifies all the structures of the network, their spans, elements and segments. For each structure, span, element and segment, MPLAN will contain various types of information, mainly dates. As shown in Fig. 1, such information could be dates of different types of inspections, dates and levels of assessments, dates and form of interim actions such as weight restrictions and monitoring, etc. MPLAN will also contain useful specific information for the network manager, for instance when there are specific conditions (such as maintaining a smooth road surface) attached to an assessment pass.

Potential benefits

It is hoped that MPLAN will enable the network managers to take a longer term view and allow the development of a more coherent forward programme, which will in turn feed into the Highways Agencies proposals for the adoption of route based strategies. It will also hopefully allow the co-ordination of all the structures work required (assessment, testing, inspection, maintenance, strengthening), together with non-structural works taking account of other operational issues. Overall, it should lead to better structures knowledge for management purposes and, in particular, to easier access to some of the historical information, which may not have been properly recorded in the past. It will provide benefits to the Maintaining Agents, in that there should be better planning of works on their part.

Records management will be significantly improved, and the process of changing Maintaining Agents in the future will be assisted.

Programme

The current state of the Highways Agency development is that a consultant commission has been let to produce a simple paper based system, together with the associated documentation. This will be rapidly converted to a computerised MPLAN module of the SMIS system, for implementation in 1999.

Management plans will apply to all new and in-service structures. Planned bridges will require an outline plan to be set up in association with the technical approval procedures, and as such will be linked to the developing ideas of whole life costing and management strategies.

It is intended that once the database is available, a concerted programme will be undertaken to populate it as a matter of urgency. For this purpose, Maintaining Agents will be required to input information for each structure in the outline forms (yet to be finalised) as shown below.

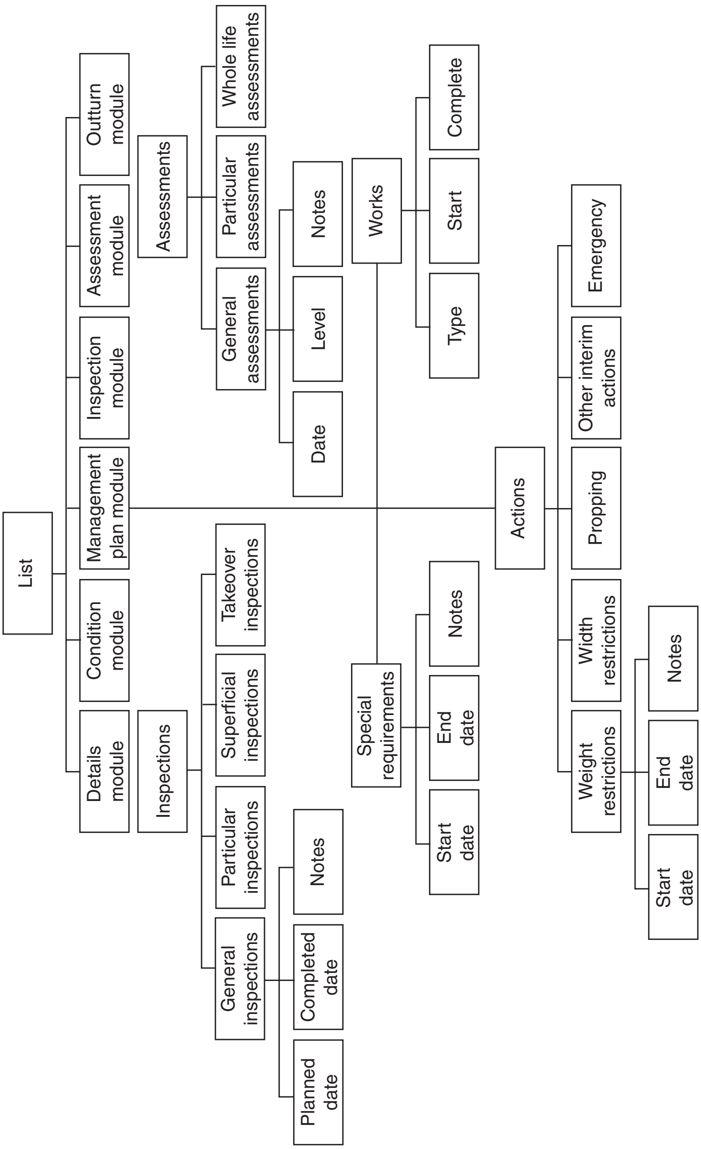

Fig. 1. Schematic view of management plan database MPLAN

Typical details to be shown in a management plan
NAME OF STRUCTURE
STRUCTURE NUMBER
STKEY (Computer reference)
TRUNK ROAD/MOTORWAY
TENS ROUTE
GRID REFERENCE
OWNER
AGENT/DBFO CONTRACTOR
COUNTY
REGION/AGENCY AREA
OTHER TRANSPORT UNDERTAKINGS

BRIDGE TYPE
YEAR OF CONSTRUCTION
ANCIENT MONUMENT/SCHEDULED/CONSERVATION AREA
ROUTE RATING
BRIDGE CONDITION INDEX
SAFETY INDEX
STATUTORY UNDERTAKERS APPARATUS
NRSWA DESIGNATION
HEALTH AND SAFETY SPECIAL RISKS
SECURITY
SPECIAL STRUCTURE
SCOUR RISK

HEAVY OR HIGH LOAD ROUTE
ABNORMAL LOAD PROVISION ON M/W OR T/R
DESIGN LOAD RATING
MINIMUM HEADROOM ON M/W OR T/R

MINIMUM HEADROOM ON ROAD CROSSED
RESTRICTIONS (width/height/weight)
MONITORING
OTHER APPARATUS (e.g.Trafficmaster)
OTHER INFORMATION

PAST AND PROGRAMMED WORK

In the above list, under types of work and actions it is proposed that all inspections, testing, assessments, maintenance, strengthening, structural modifications, and other activities having an influence over structural integrity and performance, e.g. significant accidents, will be recorded.

The final system must be practical and user-friendly, useful for Agents and the Highways Agency, provide easy access to important management information and an at a glance view of bridge condition and alerts to potential safety issues.

Management plans database

Purpose

The management plan database will be an integral part of the structures management information system (SMIS) with management information held at structure, span, element and segment level.

Its purpose is as a planning and monitoring tool for maintaining agents, Highways Agency area/route managers and central policy makers. It will hold all the relevant information from previous management decisions/ actions and record planned future maintenance actions.

Benefits

The perceived benefits are

- Agents responsible for maintaining the structures will input directly actions, decisions and their dates into the database
- it will give a clearer picture of the future workload and current state of the bridgestock at a glance
- recording management decisions and planning future actions in this manner will ease the transition between outgoing and incoming agent
- it will assist in the longer term planning on route strategies
- the responsibility for keeping it up to date and accurate will be the Maintaining Agents, this will provide motivation to make best use of the system and its data.

It will not supplant engineering thinking nor will it replace existing documentation and records.

Levels and types of information

Managers will only need to record information on screens down to the level of structure relevant for planning purposes, i.e. where a General Inspection is planned for the whole structure the date need only be held at structure level. In contrast, a Particular Inspection of a part of the structure, say a bearing, would be recorded at structure and down to segment level.

Main screen headings include

- inspections
- special requirements
- actions

- ☐ assessments
- ☐ works.

Sub-screen headings divide each main heading into actions relating either to mandatory requirements, management decisions or special requirements, some of which are outside the control of the Agency.

The types of information held will include all the past, current and future actions on the structure ranging from inspections to emergency works.

Future maintenance actions will be either

1. a mandatory requirement of a standard
2. part of the structures maintenance strategy dictated by assessments inspections and service life prediction, or
3. a result of the structures physical shape and size and location.

(1.) Where actions derive from standards, as in General Inspections and general assessments, dates will be automatically generated indicating when they are to be completed or if they are outstanding. Transferring these and other planned actions to completed actions will follow a process of validation by the client body, only when the actions are satisfactorily completed.

(2.) Information on the structures maintenance strategy will have essential and preventative actions with intervention periods derived from whole life assessments, inspections and the serviceable life of structural components. This would need reviewing every year prior to submission of the strategy for funding. Examples of these actions would be concrete repair, painting of steelwork, waterproofing and silane impregnation.

(3.) The structures location, geometry and history will have produced actions particular to the structure of a non-structural nature such as: land access arrangements, emergency services requirements, traffic management arrangements, diversion routes or traffic signalling. These may be related directly to the assessment of the structure or be a general route requirement.

Where planned maintenance is deferred through lack of funding or the requirements of the route as a whole, the item itself will remain on the database and as with inspections will be indicated as outstanding. The Agent may, as a result, have to revise the maintenance strategy of the structure; however it is not anticipated that a single year's funding will affect this, as future funding will be based on longer term maintenance strategies.

Interface with SMIS

A set of basic information necessary to access the data for each structure will be resident on all MPLAN screens. These are

- ☐ Structure number
- ☐ Structure key
- ☐ Structure name

- Roads over/under
- Agent
- Region
- Area.

It will also be possible to view the diagrammatic representation of the structure to pinpoint the particular part of the structure relating to the data entry.

To ensure managers can obtain the necessary information at a glance without viewing numerous screens, MPLAN draws its information from other modules in SMIS. Each of these modules can be viewed and interrogated individually and users can view corresponding data at the required level of structure in any module of the database. MPLAN shares data with the majority of modules within SMIS. The sources of common data will be held at a single position in the database, and will be capable of amendment at any location. The completed amendment will require validation by the Highways Agency.

Data will be shared across (but without limitation) the following modules

- Assessment
- Inspections
- Bids assessment and prioritisation system
- Outturns
- Inventory database.

Reporting

Users will be able to produce reports from individual screens, modules or throughout the whole database and then print or download these to compatible formats for short term manipulation.

Research has indicated that the method and type of reporting needs to be as flexible as possible to meet the needs of the Highways Agency and its Maintaining Agents. It is therefore envisaged that apart from generation of reports from screens on view, the user will be able to produce reports using proper English, key words or codes throughout the database.

Part 6. Research

Whole life performance profiles for highway structures

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Introduction

Previous research[1] has developed a methodology for obtaining whole life profiles of structural performance for short-span concrete slab and beam-and-slab highway bridges. This methodology is currently being verified and the scope of its application broadened in order to obtain a simplified format for the assessment of current bridge condition and also the prediction of future performance taking into account deterioration of structural components.

The information provided by these whole life profiles may then be used by bridge managers to predict the stages at which the structural performance drops to critical levels. As a result, maintenance and repairs or even strengthening or replacement may be programmed to optimise the allocation of resources. As such, the work forms part of a bridge management methodology formulated by the Highways Agency.[2] The bridge management system uses bridge specific assessment loading and whole life performance related targets in a risk-based approach which is intended to be both rational and flexible and which will represent a major improvement on the current methods of assessment.

The current project is aimed solely at reinforced concrete highway structures, however, similar research is currently underway for steel and composite highway structures, as well as associated structures such as retaining walls.

Methodology

Following this methodology, bridges are assessed to existing standards using elastic or plastic analysis methods. A live load capacity rating is obtained for the bridge assuming it to be in its original undeteriorated condition (K_0 factor). The live load capacity rating at the present time can then be calculated using deteriorated properties of the bridge. These deteriorated properties can be obtained using either the results of testing or from theory. A theoretical deterioration model has been proposed[1] which assumes corrosion of the concrete reinforcement results solely from chloride ingress due primarily to the application of de-icing salts to the bridge deck. Chlorides attack the reinforcement and the resulting corrosion brings about a reduction in the area of steel. As a result of this the capacity of the structure is reduced. Using this theoretical model for deterioration of the bridge, whole life profile curves can then be formulated. This information regarding the likely future rate of deterioration of the structure can be used to compare one bridge with another in order to prioritise maintenance work and estimate the time at which the risk of failure becomes unacceptably high.

Deterioration model

The deterioration model used in the formulation of the whole life profiles is based on Fick's law of diffusion and assumes that all the deterioration within the bridge deck occurs as a result of chloride penetration. The effects of carbonation, capillary action and cracking are not considered.

In alkaline environments, the reinforcement steel is protected by a layer of gamma ferric oxide which prevents further oxidation and therefore corrosion. When the chloride ion concentration in the concrete exceeds a certain threshold level, this protective layer is destroyed and a galvanic corrosion cell develops. A certain minimum concentration of chloride ions in water and oxygen must be present for the corrosion process to proceed. Water and oxygen are present in concrete and so the chloride ion concentration is the only variable.[3]

Diffusion through a porous medium follows Fick's Law

$$\frac{\delta c(\mathrm{x},\ t)}{\delta t} = D_C \frac{\delta^2 c(\mathrm{x},\ t)}{\delta \mathrm{x}^2} \tag{1}$$

where c is the chloride concentration; x is the distance from the surface; t is time and D_C is the chloride diffusion coefficient.

The standard solution to this equation for an isotropic semi-infinite medium was presented by Crank[4] as

$$C(\mathrm{x},\ t) = \mathrm{C}_0\left\{1 - \mathrm{erf}\left(\frac{\mathrm{x}}{2\sqrt{D_\mathrm{C}t}}\right)\right\} \tag{2}$$

where $C(\mathrm{x}, t)$ is the chloride concentration at depth x after time t for equilibrium concentration C_0 at the surface.

The corrosion initiation period for corrosion of reinforcement can then be described by the equation[1]

$$T_1 = \frac{(\mathrm{d}_1 - D_1/2)^2}{4D_\mathrm{C}}\left(\mathrm{erf}^{-1}\left(\frac{\mathrm{C_{cr}} - \mathrm{C}_0}{\mathrm{C_i} - \mathrm{C}_0}\right)\right)^{-2} \tag{3}$$

where T_I is the corrosion initiation period (years); $(d_I - D_I/2)$ is the concrete cover (mm); d_I is the initial distance from adjacent concrete surface under consideration to the centroid of reinforcing bar (mm); D_I is the initial diameter of (uncorroded) reinforcement bar (mm); D_C is the chloride diffusion coefficient (mm^2/year); C_{cr} is the critical chloride concentration (%); C_0 is the chloride content on the surface (%); and C_i is the initial chloride concentration (%).

The area of reinforcement at time t is then described by the equation[1]

$$nD_i^2\frac{\pi}{4} \text{ for } t \leq T_I \tag{4}$$

$$nD_i^2\frac{\pi}{4} \text{ for } T_I \leq t \leq T_I + D_I/0\cdot 023i_{corr}$$

$$n(D(t))^2\frac{\pi}{4}\text{for } t > T_I + D_I/0\cdot 023i_{corr}$$

where

$$D(t) = D_I - 0{\cdot}023i_{corr}(t - T_I) \tag{5}$$

and n is the number of reinforcing bars; T_I is the time to initiation of corrosion (years); D_I is the initial diameter of reinforcement bars (mm); $0{\cdot}023i_{corr}$ is the rate of corrosion (mm/yr); i_{corr} is the corrosion current density (μA/cm^2), and t is the time (years).

The coefficient value of 0·023 which partly defines the rate of corrosion, is related to the chemical composition of the reinforcement steel and the observed mode of corrosion. Further details of this parameter are given in our interim report on whole life assessment for concrete bridges.[5]

Three corrosive environments have been proposed, these being defined as low, medium and high. The values used for these three environments are given in Table 1.[1] The proposed corrosion model was originally identified for use in the probabilistic modelling of bridge performance and so the parameter values are each defined in terms of a mean, standard deviation and statistical distribution type (i.e. N for normal or U for Uniform). In the current research, mean values have been assumed.

In each of the environment models, the critical chloride concentration (C_{cr}) is assumed to be 0·3%[1] and the initial chloride concentration C_i in the concrete is assumed to be zero. All of the above values relating to chloride concentration are given in terms of total chloride concentrations by % weight of concrete.

Guidelines similar to those given in the current codes of practice for design[6] could be provided in order that an Engineer could identify the appropriate corrosion environment for his/her bridge.

The proposed model values are based on a combination of theoretical and laboratory test results.[1] A representative range of actual measured values for the corrosion parameters on bridges in the UK is given below.[5]

Table 1. Parameters defining proposed corrosive environments

Corrosion Environment	Corrosion parameters	Parameter values (mean, standard deviation)
low	Diffusion coefficient, D_C	N (25·0, 5·0) (mm^2/year)
	Surface chloride concentration, C_0	N (0·575, 0·038) (%)
	Corrosion current density, i_{corr}	U (2·0, 3·0) ($\mu A/cm^2$)
medium	Diffusion coefficient, D_C	N (30·0, 5·0) (mm^2/year)
	Surface chloride concentration, C_0	N (0·650, 0·038) (%)
	Corrosion current density, i_{corr}	U (3·0, 4·0) ($\mu A/cm^2$)
high	Diffusion coefficient, D_C	N (35·0, 5·0) (mm^2/year)
	Surface chloride concentration, C_0	N (0·725, 0·038) (%)
	Corrosion current density, i_{corr}	U (4·0, 5·0) ($\mu A/cm^2$)

- ☐ Diffusion coefficient, D_C: 3·0–300·0 (mm^2/yr)
- ☐ Surface chloride concentration, C_0: 0·0–0·15 (%)
- ☐ Corrosion current density, i_{corr}: 0·1–1·0 ($\mu A/cm^2$)
- ☐ Critical chloride concentration, C_{cr}: 0·1 (%)

Values of the corrosion parameters are affected by factors such as the C_3A content of the cement, the water/cement ratio, the method of exposure to chlorides, the method of curing used during construction and also temperature. A number of the corrosion model parameters, for example the diffusion coefficient and the surface chloride concentration, are also time dependent. The parameter values chosen for the deterioration model are intended to take account of this time dependency and so are effectively average values for the life of the structure, rather than realistic actual values at any one point in time. It is noted that no validation of the model has been attempted against data obtained from real structures. However, one of the main reasons for this is that there is little data from in situ measurements on concrete bridge decks.

It is recognised that the deterioration model adopted is very limited in its ability to represent the actual processes of corrosion in a concrete bridge deck. For example, uniform corrosion over the entire bridge deck is assumed whereas there will almost certainly be some spatial variation in the location and degree of corrosion over the area of a bridge deck. Similarly, once corrosion is initiated it is assumed to be active around the full bar perimeter whereas in practice loss of section often progresses from one side, typically that closest to the surface. Pitting corrosion is also often observed in bridge decks and this can result in a far more severe loss of section than would be predicted by the uniform corrosion model assumed here. However this severity is mitigated by the fact that it is unlikely to affect a large number of bars in the same location. Products of corrosion, which build up around a reinforcement bar, occupy more volume than reactants and so produce stresses which result in cracking and spalling of the concrete. Bond strength is also affected by the build up of products of corrosion on the surface of the reinforcement bar. It is possible that the effects on the section capacity of cracking, spalling and loss of bond may be more onerous than the effects of a loss of reinforcement area.[5]

The complex nature of corrosion may require a purely empirical model in order that the behaviour of real structures can be accurately predicted. However, it is likely that all corrosion will be bridge specific and so any model will be an approximation to what is actually occurring. Until further research becomes available in this area, it is intended that the above model shall be used in the formulation of whole life profiles for highway structures.

Whole life profiles

Previously, a sample set of good bridges had been identified and whole life profiles were formulated using plastic methods of analysis.[1] This sample set has now been extended to include a number of poor bridges and these have been analysed using both plastic and elastic analysis methods. The sample set of good bridges have also been analysed using elastic analysis and a comparison of the two methods has been made.

The good bridges had all passed the current 40 tonne assessment criteria using elastic analysis methods and were judged to be in good condition for bridges of their particular age and type.[7] The poor bridges had all failed the current 40 tonne assessment criteria when analysed using elastic analysis methods but were found to pass when re-assessed using plastic analysis. The plastic analysis was undertaken using a yield-line program (COBRAS) developed previously at Cambridge University.[8] The elastic analysis was carried out using a torsionless grillage analysis. Shear capacity was evaluated using only elastic analysis.

Example geometric details and material properties for two bridges (one good and one poor) are given in Table 2. Both bridges had one layer of bottom longitudinal reinforcement and one layer of bottom transverse reinforcement. No top steel was taken into account in the analysis.

Figures 1(a) and 1(b) show sample whole life profiles for the flexural capacity of these two bridges, obtained using plastic analysis. A medium corrosion environment was assumed and mean values for the corrosion parameters were used in the calculation of the deteriorated area of steel. The whole life profile shown in Fig. 1(a) is plotted in terms of the K factor against time, where the K factor is defined as in BD 21/97 Clause 5.28[9] by Equation 6.

Table 2. Details for two sample bridges

Property	Good bridge	Poor bridge
Span	9·75 m	3·294 m
Width	20·2 m	8·15 m
Skew	0 degrees	16 degrees
Slab thickness	550 mm	254 mm
Longitudinal reinforcement	R40 – 125	R25 – 140
Transverse reinforcement	R12 – 200	R16 – 114
Cover to longitudinal reinforcement	40 mm	52 mm
f_{cu}	30 N/mm^2 (nominal)	30·7 N/mm^2 (w.c.s.)
f_y	250 N/mm^2	250 N/mm^2

$$K \text{ factor} = \frac{\text{Available live load capacity}}{\text{Live load capacity required for HA loading}} \tag{6}$$

The whole life profile in Fig. 1(b) is plotted in terms of the normalised K factor (K/K_0) where K_0 is equal to the original undeteriorated K factor for the bridge. This normalised parameter is used to allow comparison between different structures.

Figures 2(a) and 2(b) show the sample whole life profiles for the same two bridges, also for flexure but obtained using elastic analysis. The deterministic K_0 factor obtained using elastic analysis was 80% of that obtained using plastic analysis for the good bridge and 50% of that obtained using plastic analysis for the poor bridge.

Comparing the normalised K factor profiles (Figs 1(b) and 2(b)) it can be seen that the good bridge deteriorated more slowly when analysed using elastic methods as opposed to plastic. However, the poor bridge deteriorated more rapidly when analysed using elastic methods as opposed to plastic. These differences are a function of the different failure modes exhibited by the two bridges and also the varying dead-to-live load ratios resulting from the different methods of analysis. Despite these differences, the plastic and elastic profiles of the normalised K factor are relatively similar.

Figures 3(a) and 3(b) show the sample whole life profiles for shear, obtained using elastic analysis. Comparing the whole life profiles for shear with those given previously for flexure, it can be seen that both bridges were limited by their flexural strength. It can also be seen that, as expected, the effects of loss of area of steel are much more severe in terms of the flexural strength of the bridge than of the shear strength, which is heavily dependent upon the concrete strength.

The profiles shown in Figs 1–3 are intended to act as examples of the effects of deterioration on the K factor values for two particular bridges. In order that this method can be applied to any bridge without the need to

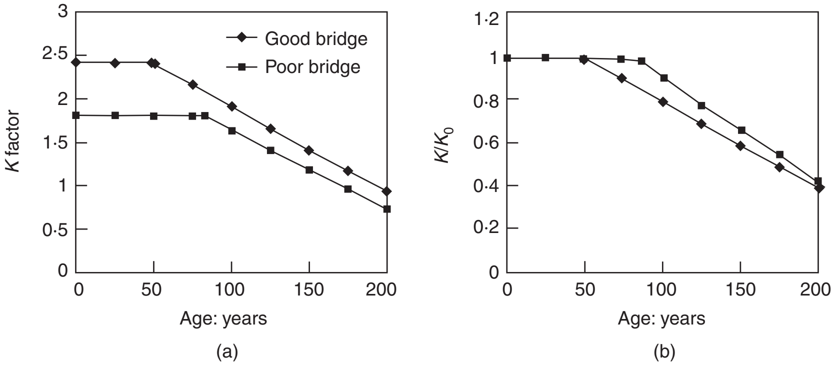

Fig. 1. Whole life profiles for flexure obtained using plastic analysis: (a) whole life profile for K factor; (b) whole life profile for K/K_0

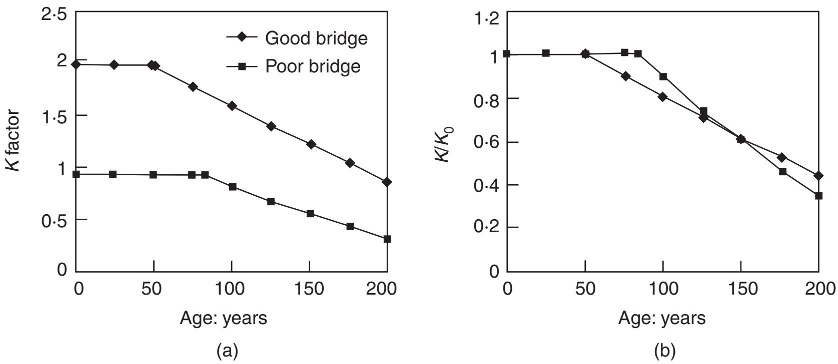

Fig. 2. Whole life profiles for flexure obtained using elastic analysis: (a) whole life profile for K factor; (b) whole life profile for K/K_0

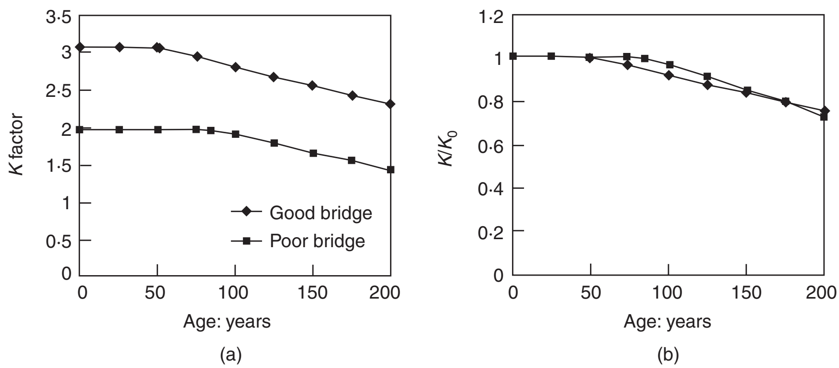

Fig. 3. Whole life profiles for shear obtained using elastic analysis: (a) whole life profile for K factor; (b) whole life profile for K/K_0

derive the individual whole life profiles for that bridge, some generalised form of the whole life profile is required.

Generalised presentation of whole life profiles

There are two distinct phases of the deterioration model. The first phase models the time prior to the initiation of corrosion, during which the chloride concentration levels are below the threshold level (Equation 3). The second phase is that which models the period of active corrosion (Equations 4 and 5).

For a given corrosion environment, the time to initiation is governed solely by the cover to the reinforcement. Fig. 4 shows the variation of initiation time with cover for a medium corrosion environment.

The rate of loss of area of steel is dependent not only upon the specific corrosion environment, but also upon the size of bar. From Fig. 5 it can be seen that the effect of a specific rate of corrosion on a small bar is much more significant than the same depth of corrosion on a much larger bar. Fig. 5 is plotted assuming a medium corrosion environment. It is apparent therefore that the cover depth and the size of the bar are two important parameters in modelling the deterioration of a reinforced concrete structure.

Having determined the rate of loss of area of steel for a structure, the next step is to relate this to the K factor. Using the simplified equation from BD 44/95 Clause 5.3.2.3[10] the relationship between moment capacity of a reinforced concrete section and area of steel is given by Equation 7.

$$M_{\mathrm{u}} = A_{\mathrm{s}} f_{\mathrm{y}} d - \frac{k_2 A_{\mathrm{s}}^2 f_{\mathrm{y}}^2}{k_1 f_{\mathrm{cu}} b} \tag{7}$$

where M_{u} is the ultimate moment capacity of section; A_{s} is the area of steel; d is the effective depth to reinforcement; k_2 is the coefficient describing the ratio of the depth of the centroid of the assumed stress block to the neutral

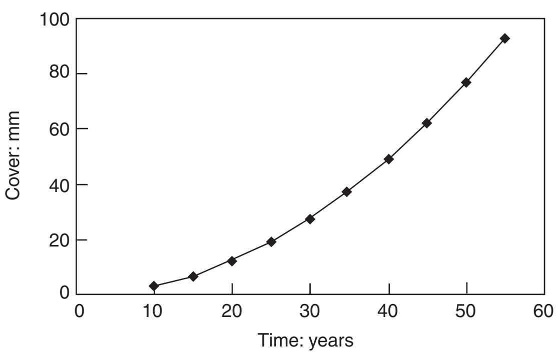

Fig. 4. Variation of initiation time with cover for a medium corrosion environment

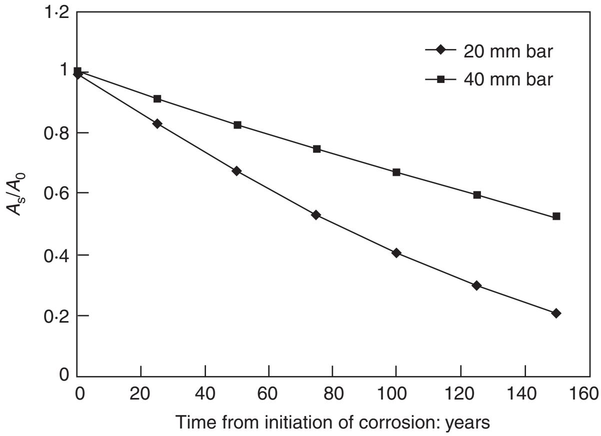

Fig. 5. Effect of active corrosion on different bar sizes

axis depth; k_1 is the coefficient describing the ratio of the average compressive stress to the characteristic concrete strength for the assumed stress block; f_y is the yield strength of reinforcement; f_{cu} is the characteristic concrete strength, and b is the width of section.

In the analysis, a value of 0·5 was taken for k_2 and a value of 0·67 was taken for k_1.

Figure 6 shows the effect on flexural capacity of varying the percentage area of steel from the minimum required by BS 5400[6] for the design of concrete bridges, to the percentage required for a balanced section. Fig. 6 was obtained assuming a value of f_y of 250 N/mm^2. The appropriate minimum area of steel was therefore 0·25%. The analysis assumed a concrete strength of $f_{cu} = 30$ N/mm^2. Partial safety factors on characteristic material strengths were used in accordance with BD 44/95.[10]

In Fig. 6: M_0 is the initial undeteriorated ultimate moment capacity of section and A_0 is the initial undeteriorated area of steel in section. The maximum difference between the two curves is approximately 10% of the M_u/M_0 value. A sensitivity analysis showed that the most important parameter governing the variation of M_u/M_0 with A_s/A_0 was the percentage area of steel in the original section. The yield strength of the reinforcement also had a noticeable effect on the results, as did the partial safety factors. The effects of varying k_1 and k_2 were not considered.

The shear capacity of a section is calculated using Equation 8 which is taken from BD 44/95 Clause 5.3.3.1.[10]

$$V_u = \left(\frac{550}{d}\right)^{\frac{1}{4}} \frac{0 \cdot 24}{\gamma_{mv}} \left(\frac{100\,A_s}{bd}\right)^{\frac{1}{3}} (f_{cu})^{\frac{1}{3}} bd \tag{8}$$

where V_u is the ultimate shear capacity and γ_{mv} is the partial safety factor.

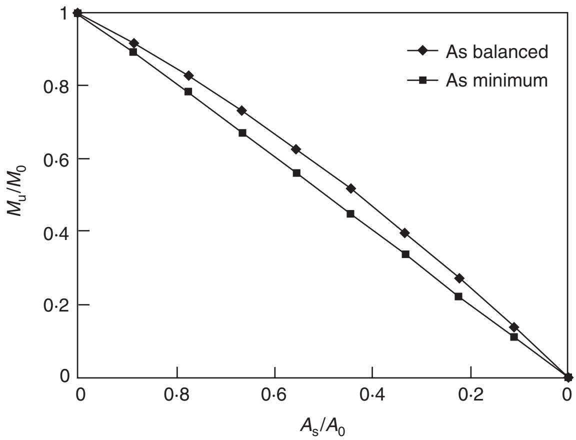

Fig. 6. Variation of moment capacity with area of steel

Figure 7 shows the relationship between the area of steel and the ultimate shear capacity of a section. This relationship was unaffected by the initial percentage area of steel, concrete strength or chosen value of γ_{mv}. In Fig. 7: V_0 is the initial undeteriorated shear capacity of section and A_0 is the initial undeteriorated area of steel in section.

Finally, the relationship between the section capacity and K factor needs to be defined. For both flexure and shear this relationship is defined by the dead-to-live load ratio of the structure or section. Fig. 8 show this effect for various dead-to-live load ratios.

It can be seen that the effect of the dead-to-live load ratio is non-linear between different values of dead-to-live load ratio. It can also be seen that

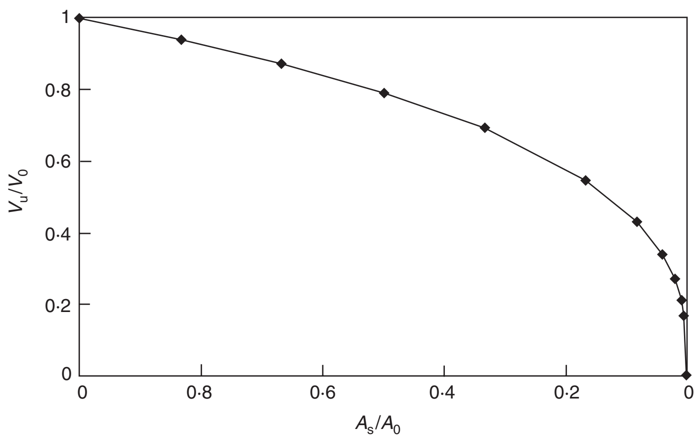

Fig. 7. Variation of shear capacity with area of steel

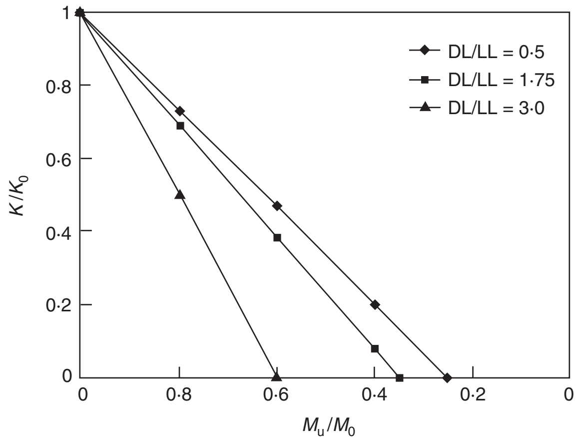

Fig. 8. Effect of dead-to-live load ratio on relationship between section capacity and K factor

the effect of the dead-to-live load ratio is a significant variable when considering the relationship between the section capacity and the K factor.

The important variables defining the whole life profile, i.e. the relationship between time and the ratio K/K_0, have been shown to be the cover, the size of the reinforcement bar, the yield strength of reinforcement bar, the percentage area of steel and the dead-to-live load ratio. The whole life profile is therefore likely to be bridge specific and any generalised method of presenting the whole life profile will need to take each of these variables into account.

Conclusions

A deterioration model has been defined which models the loss of area of reinforcement steel as a result of chloride ingress to the bridge deck. The model is simplistic when compared with the complex and variable nature of the deterioration processes acting upon a real bridge deck. However, it provides a starting point from which predictions of the whole life performance of concrete bridges may be made. These may subsequently be modified on the basis of any data from in situ measurements.

Whole life profiles have been formulated using both plastic and elastic analysis methods. The results showed large variations in K factor values depending on the method of analysis; however the normalised K/K_0 values were relatively similar.

The results of the elastic and plastic analysis showed that for both of the sample concrete slab bridges, flexure was the critical mode of failure.

It was shown that the important parameters for modelling the whole life profile for a general bridge were cover, size of reinforcement bars, yield strength of reinforcement bars, percentage area of steel and dead-to-live load ratio. It is concluded therefore, that the whole life profile is likely to be bridge specific.

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Optimum design of bridge inspection/repair programmes based on lifetime reliability and life-cycle cost

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Introduction

Although current bridge inspection/repair practice recognises the importance of minimising the expected life-cycle cost while maintaining an acceptable level of structural reliability, very few studies were provided to optimise the inspection/repair programmes for new and/or existing structures.[1–6] While various methods for optimisation of bridge management decisions are under development in several countries, current bridge inspection/repair programmes are based on oversimplified assumptions without integrating and balancing properly the life-cycle cost and lifetime reliability. This study, which borrows from previous work of the authors[7,8] proposes a more realistic and comprehensive approach for optimising the design of bridge inspection/repair programmes based on lifetime reliability and life-cycle cost. This approach is consistent with the new whole life performance-based bridge assessment rules which are being currently developed by the Highways Agency.[9] As indicated by Das,[9] these rules

> *will enable assessing engineers to determine both safety-related as well as economically justified preventative maintenance needs.*[9]

Methodology

The general methodology for optimising the lifetime inspection/repair strategy for a deteriorating structure consists of the following ten steps[7,8]

1. define the structure and the criterion which constitutes its failure
2. specify how the structure deteriorates over time. Develop a deterioration model
3. specify the inspection methods available to detect the deterioration. Quantify the detection capability and cost of these methods

4. define the available repair options and compute their costs
5. quantify the probability of making a repair if a defect is detected
6. formulate the optimisation problem based on the optimisation criterion, failure constraints, expected service-life of the structure, and any other constraints that should be imposed such as minimum and/or maximum time intervals between inspections
7. use an event tree to account for all of the repair/no repair decision possibilities that must be made after every inspection
8. for a discrete number of lifetime inspections, optimise the timing of these inspections for a specific inspection technique
9. repeat the problem for other numbers of lifetime inspections and inspection techniques to find the optimum strategy
10. update the optimum strategy after every inspection using the new information provided from the inspection results.

The general methodology described above is first applied to the lifetime inspection/repair optimisation of a deteriorating member. Then, the approach is extended to simple systems. Finally, it is used for new and existing bridges in a whole life perspective.

Single member

As a first example a single deteriorating member is considered. Its resistance R and load P are considered time-independent random variables, while its deterministic cross-sectional area A is assumed time-variant. The criterion which constitutes failure of the member is $R \leq P/A(t)$, where $A(t)$ is the time-variant cross-sectional area. The deterioration of the cross-sectional area over time, t, assuming that no repairs are made, is

$$A(t) = A_{\text{initial}} - 2{\cdot}0(0{\cdot}051\, t^{0{\cdot}57}) \tag{1}$$

where A_{initial} is the cross-sectional area. For the case $\mathrm{A}_{\text{initial}} = 1{\cdot}0$, only half of the initial cross-sectional area remains after about 17 years.

Given the deteriorating member defined by the main descriptors of resistance, $\overline{R} = 14{\cdot}0$ and $\sigma(R) = 1{\cdot}4$, and load, $\overline{\mathrm{P}} = 8{\cdot}0$ and $\sigma(P) = 0{\cdot}8$, where $\overline{\mathrm{X}}$ and $\sigma(\mathrm{X})$ are the mean value and standard deviation of X, respectively, the goal is to develop a strategy that will minimise the expected total cost $E(C_{\text{tot}})$ of the lifetime inspection/repair programme and prevent the member from deteriorating to an unacceptable level of reliability at any point during its service-life. The assumed service-life of this member is ten years and the expected value of the reliability index $E(\beta)$ will not be permitted to fall below $\beta_{\min} = 2{\cdot}0$. There will be two, three, or four inspections conducted over the service-life of the member. The design variables are the inspection technique and the inspection times.

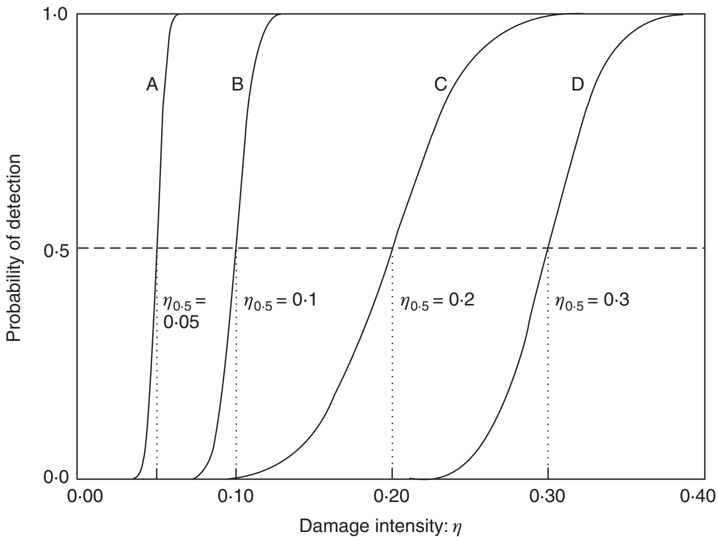

Fig. 1. Probability of detection for four inspection techniques

Table 1. Main descriptors of four inspection techniques

Inspection technique	$\eta_{0\cdot5}$	σ_{insp}	$\eta_{\min}$	$\eta_{\max}$	Inspection cost
A	0·05	0·005	0·035	0·065	1·50
B	0·10	0·010	0·070	0·130	1·00
C	0·20	0·040	0·080	0·320	0·75
D	0·30	0·030	0·210	0·390	0·50

Four inspection techniques A, B, C and D with normally distributed damage detection capabilities are considered (Fig. 1 and Table 1). The ability of these methods to detect deterioration is based on the intensity of the structural damage which relates to percent section loss, as follows

$$\eta_{\text{str}}(t) = [A_{\text{initial}} - A(t)]/A_{\text{initial}} \tag{2}$$

The main descriptors including costs associated with the four inspection techniques are shown in Table 1, where $\eta_{0\cdot5}$ is the damage intensity at which there is a 50–50 chance of detection; σ_{insp} is the standard deviation of the detection ability of the inspection; $\eta_{\min}$ is the damage intensity below which detection is impossible; and $\eta_{\max}$ is the damage intensity above which detection is absolutely certain. The values for $\eta_{\min}$ and $\eta_{\max}$ are associated with three standard deviations below and above $\eta_{0\cdot5}$, respectively.[10, 11]

The probability of a defect being detected P_{det} at any time t is dependent on the damage intensity of the member η at time t, $\eta(t)$, and the inspection technique being used[11]

$$P_{\text{det}} = \phi\left(\frac{\eta(t) - \eta_{0\cdot 5}}{\sigma_{\text{insp}}}\right) \tag{3}$$

where ϕ is the distribution function of the standard normal variate.

If a defect has been detected, the probability of making the repair P_{rep} is calculated as indicated in Estes.[10] Assume that a repair will return the structure to its initial strength level. This assumption could be easily modified to return a structure to some specified percentage of its initial strength level after a repair. That specified percentage could even decrease over time indicating the increasing difficulty of returning an aging structure to its initial strength level.[11]

After an inspection, a decision regarding whether or not to repair the structure based on the degree of damage that was detected in the inspection must be made. The repair decision made after the first inspection affects the later decisions. As the number of inspections n increases, the number of decision paths, also called decision branches, increases by 2^n. Using an event tree, Figs 2 and 3 illustrate these paths for two and four inspections during ten years of service-life. In these figures, the timelines indicating when the inspections will be conducted (i.e. t_1 and t_2 in Fig. 2, and t_1 to t_4 in Fig. 3) are also shown.

The probability of taking any path or branch i, P_{b_i}, is equal to

$$P_{\text{b}_\text{i}} = \prod_{\text{j}=1}^{\text{m}} P_{\text{sub}_\text{j}} \tag{4}$$

where P_{sub_j} is the probability of taking any sub-branch j, along the path i, and m is the number of sub-branches. The probability of taking a sub-branch which involves making a repair, denoted by R^+ in Figs 2 and 3, is equal to the probability of detection multiplied by the probability of making a repair if damage has been detected, P_{rep}, as follows

$$P_{\text{sub}_{\text{R}+}} = P_{\text{det}} P_{\text{rep}} \tag{5}$$

This probability accounts for both the damage intensity and the ability of the chosen inspection technique to detect the damage. Similarly, the probability of taking any sub-branch where a repair is not made, denoted by R^- in Figs 2 and 3, is equal to

$$P_{\text{sub}_{\text{R}-}} = 1 - P_{\text{sub}_{\text{R}+}} \tag{6}$$

For each $Branch_\text{i}$ on the event tree, the probability of failure of the single member structure given that $Branch_\text{i}$ was taken, P_f $(Structure|Branch_\text{i})$, is multiplied by the probability of that branch being taken P_{b_i}. The probability

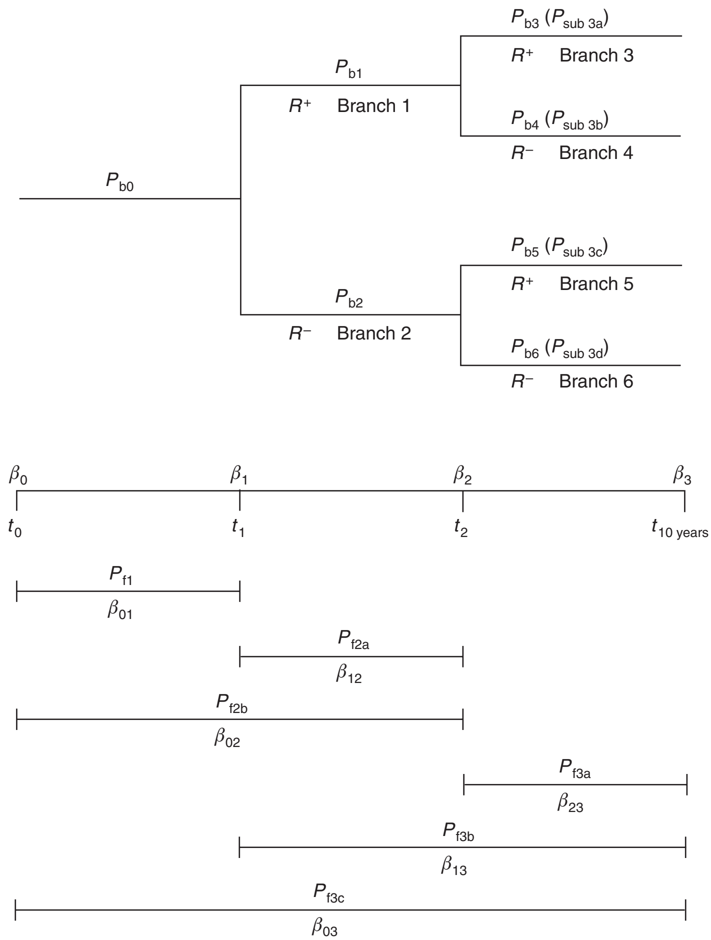

Fig. 2. Decision paths and timeline for two lifetime inspections

of failure is equal to the sum of the product P_f $(Structure|Branch_i)$ P_{b_i} over all branches. Consequently, the expected reliability index, $E(\beta)$, can be expressed as

$$E(\beta) = -\phi^{-1}[\sum_{i=1}^{2^n} P_f(Structure|Branch_i)P_{b_i} \qquad (7)$$

where n is the number of lifetime inspections.

The cost of repair C_{rep} is the sum of a fixed cost C_{fix} which occurs every time a repair is made (i.e. planning, getting to the site, exposing the element) and a variable cost C_{var} which depends on the degree of damage (i.e. the percentage of material that needs to be replaced η_{rep}). Therefore, C_{rep} is expressed as

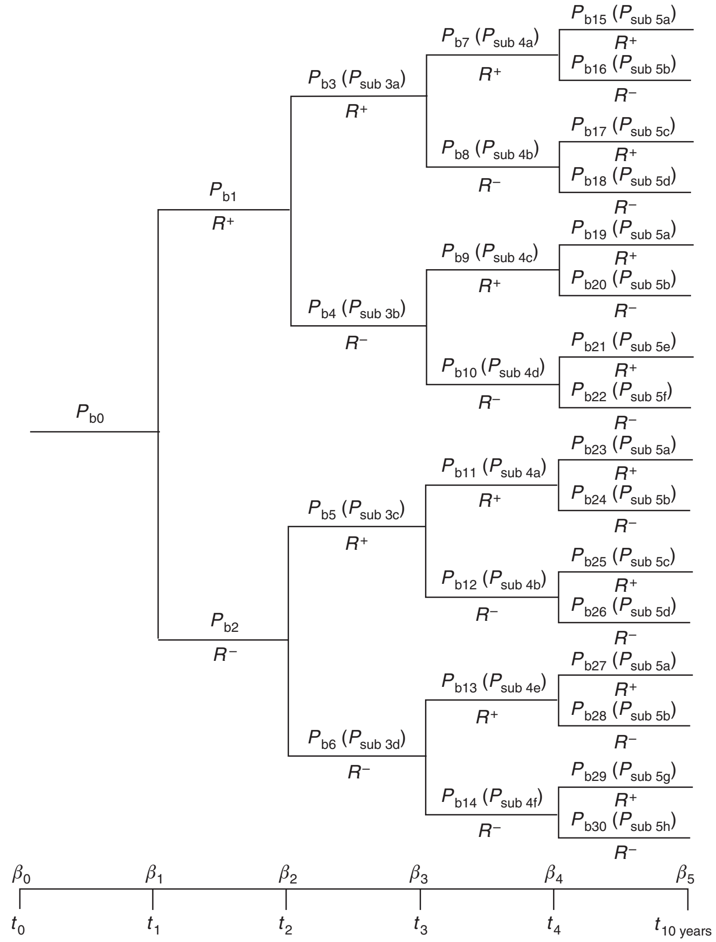

Fig. 3. Decision paths and timeline for four lifetime inspections

$$C_{\text{rep}} = C_{\text{fix}} + C_{\text{var}} \tag{8}$$

Assume $C_{\text{fix}} = 5{\cdot}0$ and $C_{\text{var}} = 5{\cdot}0\,\eta_{\text{rep}}$. The expected total cost $E(C_{\text{tot}})$ is the sum of the lifetime inspection costs $C_{\text{insp}_{\text{life}}}$ and the expected cost of lifetime repair $E(C_{\text{rep}})$. That is

$$E(C_{\text{tot}}) = C_{\text{insp}_{\text{life}}} + E(C_{\text{rep}}) = nC_{\text{insp}} + \sum_{\text{i}=1}^{2^n} P_{\text{b}_\text{i}} \sum_{\text{j}=1}^{m} C_{\text{rep}} < \psi_{\text{j}} > \tag{9}$$

where n is the number of lifetime inspections, m is the number of lifetime repairs ($m \leq n$), P_{b_i} is the probability of taking branch i on the event tree, and ψ_{j} is a bracket operator which is equal to 1 when the sub-branch on the event tree under consideration involves a repair and 0 if the sub-branch

Table 2. Formulation of the optimisation problem

Number of lifetime inspections		
Two	Three	Four
Minimise: $E(C_{tot})$ Such that: $\beta_{t1} \geq 2{\cdot}0$ $\beta_{t2} \geq 2{\cdot}0$ $\beta_{10\,years} \geq 2{\cdot}0$ $0{\cdot}5 \leq t_1 \leq 7{\cdot}0$ $0{\cdot}5 \leq t_2 - t_1 \leq 7{\cdot}0$ $t_2 \leq 10{\cdot}0$	Minimise: $E(C_{tot})$ Such that: $\beta_{t1} \geq 2{\cdot}0$ $\beta_{t2} \geq 2{\cdot}0$ $\beta_{t3} \geq 2{\cdot}0$ $\beta_{10\ years} \geq 2{\cdot}0$ $0{\cdot}5 \leq t_1 \leq 7{\cdot}0$ $0{\cdot}5 \leq t_2 - t_1 \leq 7{\cdot}0$ $0{\cdot}5 \leq t_3 - t_2 \leq 7{\cdot}0$ $t_3 \leq 10{\cdot}0$	Minimise: $E(C_{tot})$ Such that: $\beta_{t1} \geq 2{\cdot}0$ $\beta_{t2} \geq 2{\cdot}0$ $\beta_{t3} \geq 2{\cdot}0$ $\beta_{t4} \geq 2{\cdot}0$ $\beta_{10\ years} \geq 2{\cdot}0$ $0{\cdot}5 \leq t_1 \leq 7{\cdot}0$ $0{\cdot}5 \leq t_2 - t_1 \leq 7{\cdot}0$ $0{\cdot}5 \leq t_3 - t_2 \leq 7{\cdot}0$ $0{\cdot}5 \leq t_4 - t_3 \leq 7{\cdot}0$ $t_4 \leq 10{\cdot}0$

does not involve a repair. It is the expected total cost $E(C_{tot})$ that will be minimised to find the optimal method of inspection and the optimal inspection times. In this example, the time value of money is not considered.

The formulation of the optimisation problem for two, three and four lifetime inspections is presented in Table 2. In this Table, t_1 through to t_4 are the times (in years) when the four inspections will be conducted. The time constraints ensure the inspections are at least six months apart but not more than seven years apart, and that the service-life is not above ten years.

The results of the optimisation process are shown in Tables 3, 4 and 5 for two, three and four lifetime inspections, respectively, and for different values of the mean resistance $\overline{R}$. Note that the coefficient of variation of resistance is the same, $V(R) = \sigma(R)/\overline{R} = 0{\cdot}10$, for all cases. Fig. 4 compares the associated inspection and repair costs for each case when only two inspections are allowed over the service-life. Figs 5 and 6 list the same for the cases of three and four lifetime inspections, respectively.

It is interesting to note that the less expensive techniques will not work for members with smaller mean resistances where early detection of the damage is essential to maintain the member above minimum reliability index. For example, when $\overline{R} = 13$, inspection technique D is unsatisfactory for two, three, or four lifetime inspections. Inspection technique C will work for $\overline{R} = 13$ but only if four inspections are allowed. However, inspection techniques A and B will work for all cases when $\overline{R} = 13$.

At one extreme where $\overline{R} = 12{\cdot}18$, only inspection technique A with four lifetime inspections will solve the problem without violating any constraints. At the other extreme where $\overline{R} = 17$, any technique will work

Table 3. Optimum inspection strategy and associated expected total cost for two lifetime inspections

Inspection technique	Mean resistance	Optimum inspection times (years) t_1	t_2	Expected total cost
A	13·0	3·42	6·76	14·6
A	14·0	4·40	4·90	10·1
A	15·0	3·33	3·83	8·6
A	16·0	0·51	1·21	5·1
A	17·0	0·51	1·01	4·7
B	13·0	3·42	6·76	13·8
B	14·0	4·45	4·96	8·1
B	15·0	3·36	3·88	7·1
B	16·0	0·50	1·00	3·5
B	17·0	0·50	1·01	2·6
C	14·0	4·67	5·17	7·6
C	15·0	4·05	4·66	6·4
C	16·0	2·12	2·90	2·7
C	17·0	0·50	1·00	1·5
D	15·0	4·20	7·55	6·1
D	16·0	4·00	5·92	2·2
D	17·0	2·91	3·50	1·0

Table 4. Optimum inspection strategy and associated expected total cost for three lifetime inspections

Inspection technique	Mean resistance	Optimum inspection times (years) t_1	t_2	t_3	Expected total cost
A	12·5	2·56	5·08	7·56	21·7
A	13·0	3·18	6·23	6·74	17·5
A	14·0	3·95	4·47	4·97	13·1
A	15·0	2·43	2·94	3·45	11·1
A	16·0	0·50	1·01	1·52	7·9
A	17·0	0·51	1·02	1·53	7·5
B	13·0	3·24	6·35	6·85	14·6
B	14·0	4·11	4·62	5·14	10·1
B	15·0	2·53	3·02	3·52	8·7
B	16·0	0·50	1·00	1·50	5·1
B	17·0	0·50	1·00	1·50	4·9
C	14·0	4·25	4·77	5·27	8·4
C	15·0	3·17	3·92	4·43	7·3
C	16·0	1·41	2·11	2·88	3·5
C	17·0	0·50	1·01	1·53	2·3
D	15·0	4·01	6·03	7·32	6·5
D	16·0	3·98	5·11	5·70	2·7
D	17·0	1·67	2·44	3·17	1·50

Table 5. Optimum inspection strategy and associated expected total cost for four lifetime inspections

Inspection technique	Mean resistance	Optimum inspection times (years) t_1	t_2	t_3	t_4	Expected total cost
A	12·18	2·00	4·00	6·00	8·00	29·0
A	12·2	2·00	4·00	6·00	8·00	28·7
A	12·5	1·85	3·91	5·70	7·8	25·7
A	13·0	0·51	3·25	6·25	6·75	20·4
A	14·0	0·51	3·97	4·48	4·99	15·7
A	15·0	0·50	2·45	2·95	3·45	13·4
A	16.0	0·51	1·02	1·52	2·03	10·9
A	17·0	0·51	1·01	1·53	2·03	10·4
B	12·2	2·00	4·00	6·00	8·00	26.7
B	12·5	1·85	3·91	5·70	7·80	23·7
B	13·0	2·80	3·32	6·39	6·90	15·9
B	14·0	0·50	4·12	4·62	5·13	11·0
B	15·0	0·50	2·50	3·01	3·52	9·7
B	16·0	0·51	1·03	1·53	2·03	7·5
B	17·0	0·51	1·02	1·52	2·05	7·2
C	13·0	3·60	4·84	7·13	8·12	13·8
C	14·0	0·54	4·26	4·75	5·28	9·2
C	15·0	0·58	3·34	3·87	4·37	8·0
C	16·0	0·67	1·18	1·71	3·04	4·2
C	17·0	0·51	1·02	1·53	2·03	3·3
D	14·0	2·00	5·48	5·98	7·99	8·0
D	15·0	2·00	4·01	6·03	7·32	7·0
D	16·0	2·00	3·97	5·10	5·70	3·2
D	17·0	2·00	2·51	3·13	3·63	2·0

but inspection technique D with two lifetime inspections provides the lowest expected cost solution. This is due to the fact that in this case the probability of repair is so small that the cost of inspection is the only cost that counts.

The selection of the proper inspection technique for the problem at hand is important. Looking, for example, at the four lifetime inspection case (Fig. 6), if one chose inspection technique A for the deteriorating member having the mean resistance $\overline{R} = 17$, one would be paying over five times (i.e. 10·4/2 = 5·2, see Table 5) as much as if inspection technique D was chosen. However, if $\overline{R} = 13$, using inspection technique A would cost about 50% (i.e. 20·4/13·8 = 1·48, see Table 5) more than the optimum (i.e. inspection technique C). In this case, inspection technique D lacks the detection ability to keep the member from falling below the minimum reliability index. The consequences of a poor selection of an inspection technique are either additional cost or potential violation of the imposed reliability constraints.

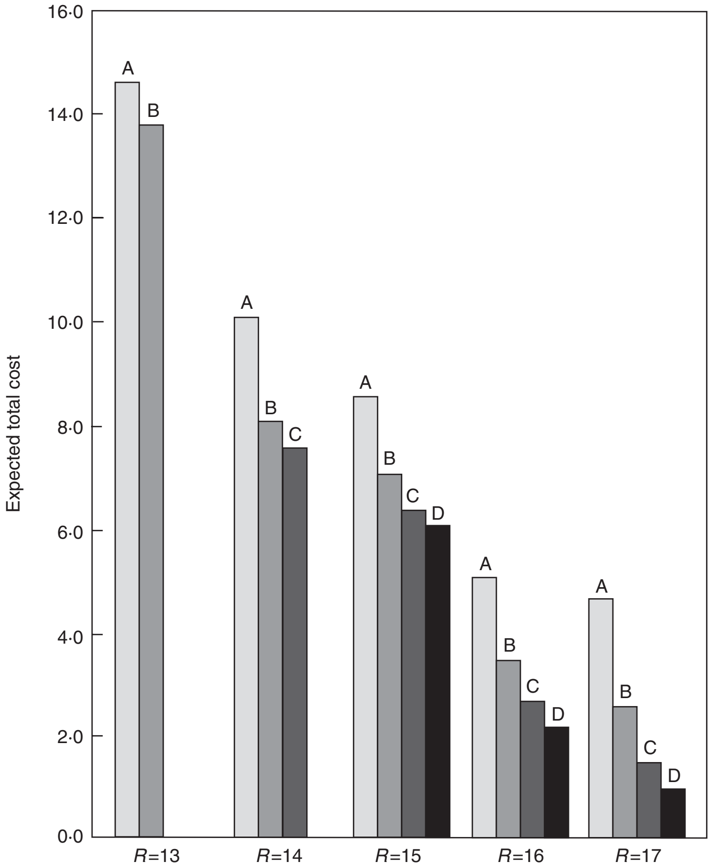

Fig. 4. Two lifetime inspections: expected total costs associated with inspection techniques A, B, C and D for different mean resistances of a single member structure

The cost effect of the choice of inspection technique is greatest when the expected number of repairs is small. In the cases of two, three or four lifetime inspections, the percent difference in costs for the different techniques is greatest when $\overline{R} = 17$ — the number of expected repairs is small and the total cost is dominated by the inspection cost. As the expected number of repairs increases such as for $\overline{R} = 13$, the cost of repair dominates the expected total cost and the percent cost differential between the inspection techniques drops.

For these particular examples, the general conclusion is that if the number of repairs is expected to be small, the cheapest inspection technique that will do the job should be chosen. The cost savings will be significant and there is little chance of violating the constraints. However, if the number of repairs is expected to be large, the best inspection technique

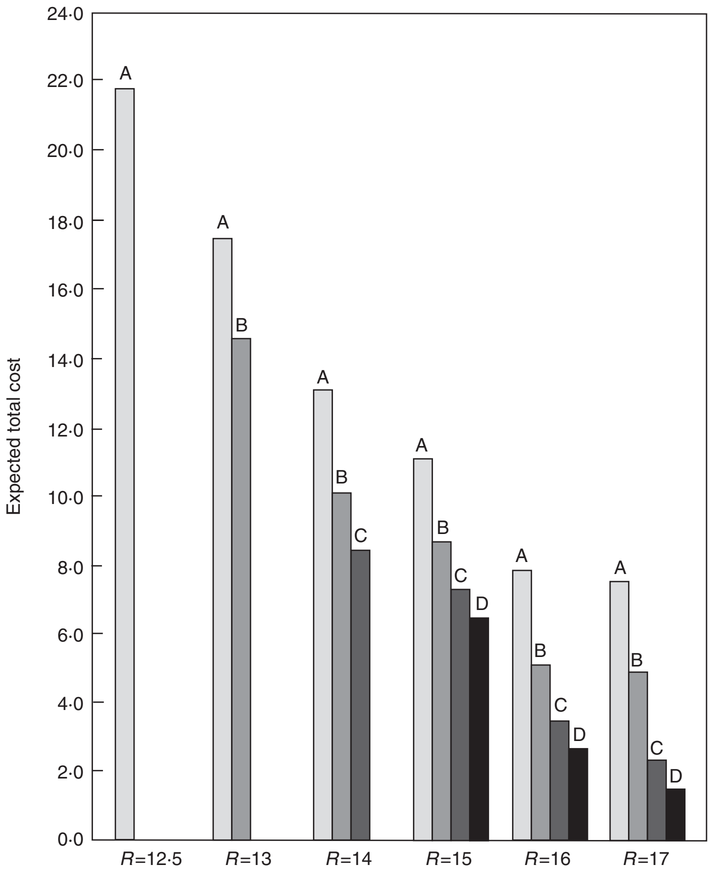

Fig. 5. Three lifetime inspections: expected total costs associated with inspection techniques A, B, C and D for different mean resistances of a single member structure

available should be selected because the extra cost is small relative to the expected total cost of inspection/repair and the penalty of a bad choice is a high likelihood of violated reliability constraints. In these examples, the cost of inspection is relatively close to the cost of repair. For cases where the cost of repair is much larger than the cost of inspection, that conclusion is probably invalid.

The timing of inspection and likelihood of repair for different single member structures are investigated next. It is assumed that the number of inspections and the inspection technique are fixed. Fig. 7 shows the case of two lifetime inspections using inspection technique A. Figs 8 and 9 show the cases of three lifetime inspections using inspection technique B and four lifetime inspections using technique C, respectively. The results are similar in all three figures. At the higher mean resistances, the probability of repair is low and the effect of the repair barely appears on the graphs. As the mean

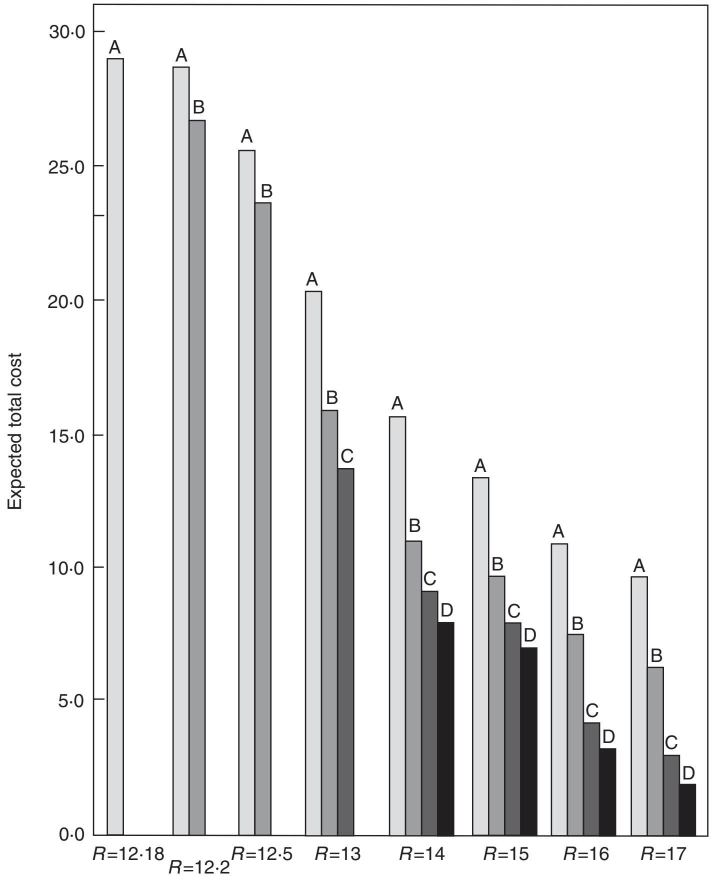

Fig. 6. Four lifetime inspections: expected total costs associated with inspection techniques A, B, C and D for different mean resistances of a single member structure

resistances become lower, the probability of repair becomes greater and the effect of the repair is much more visible.

For high mean resistances, the inspections tend to be as early in the life of the structure as the constraints allow. Fig. 7 shows this for the structures where $\overline{R} = 16$ and $\overline{R} = 17$. As the mean resistance decreases, the optimum timing of the inspections comes later in the life of the structure and the interval between inspections becomes larger. Figs 8 and 9 demonstrate as well that for high mean resistances the inspections are scheduled as early in the life of the structure as possible. As the mean resistance decreases, the effects of the third and fourth inspection are more pronounced. The most extreme case is shown in Fig. 10 for the four lifetime inspection case with inspection technique A where $\overline{R} = 12{\cdot}18$. The timing of the inspections is stretched out to 2, 4, 6 and 8 years with maximum probability of repair at

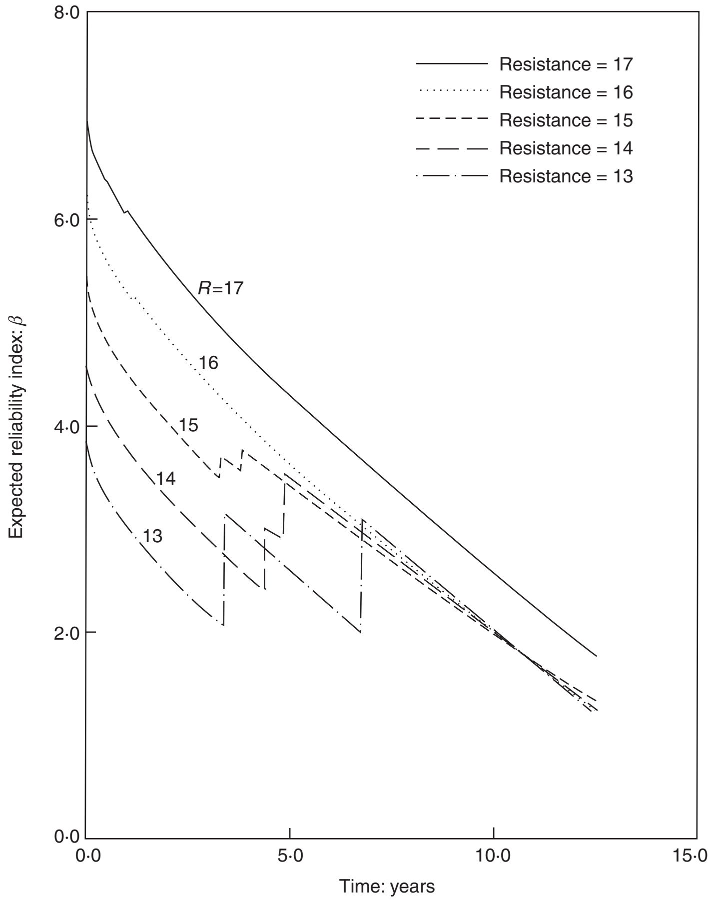

Fig. 7. Two lifetime inspections using technique A: optimum timing of inspections of a single member structure with different mean resistances

each inspection. If the mean resistance was reduced even slightly, there would be violated constraints.

The effect of inspection techniques on optimum inspection times is shown in Figs 11 and 12. Fig. 11 shows the case of two lifetime inspections when the mean resistance is $\overline{R} = 14$ and Fig. 12 shows the four lifetime inspection case when the mean resistance is $\overline{R} = 15$. Note that the coefficient of variation of the resistance is again the same for all cases, $V(R) = 0{\cdot}10$, and that $\eta_{0.5}$ and σ_{insp} in Table 1 are denoted as μ and σ in Figs 11 and 12. The optimum timing of the inspections for the higher quality techniques occurs earliest in the life of the structure and grows progressively later as the quality of the inspection technique drops. The probability of making the repairs increases with the quality of inspection techniques because the probability of detection is larger. In Fig. 12, the

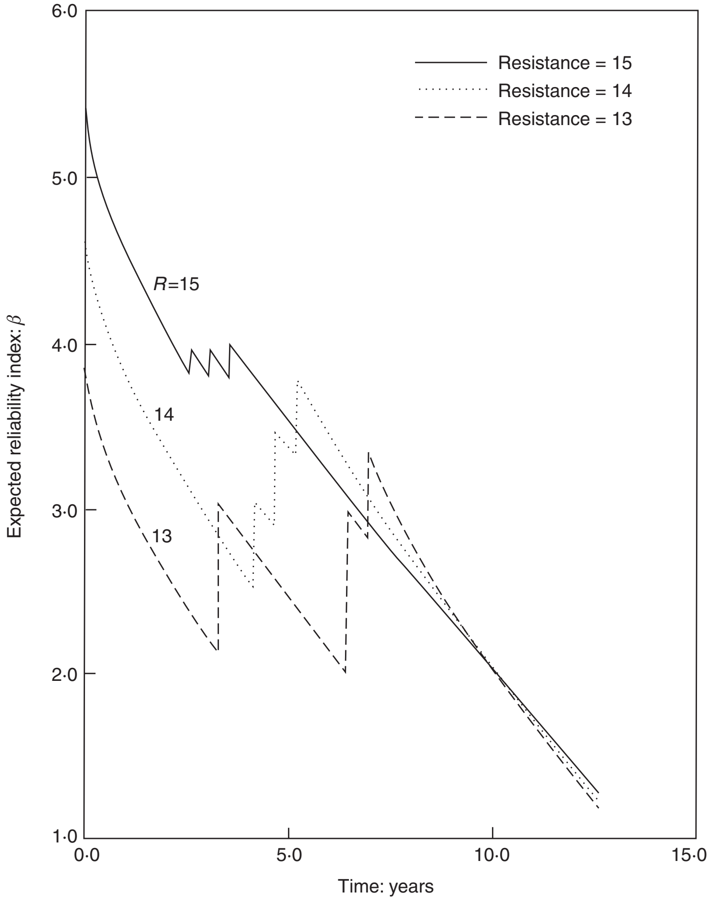

Fig. 8. Three lifetime inspections using technique B: optimum timing of inspections of a single member structure with different mean resistances

expected probability of making the fourth repair is so small that the effect of this repair is not visible on the graph. This indicates that the fourth inspection was not needed in this case. In fact, the graph for three lifetime inspections is almost identical to the four lifetime inspections graph in Fig. 12.

Finally, the global optimum lifetime inspection strategy for a given structure can be selected. Fig. 13 shows the expected total costs for all feasible combinations of inspection techniques (i.e. A, B, C and D) and number of lifetime inspections (i.e. two, three and four) for the single member structure when $\overline{R} = 14$ and $V(R) = 0{\cdot}10$. The best strategy for this structure is to conduct two inspections during the service-life using inspection technique C. The optimum inspection times are 4·67 years and 5·17 years, and the optimum expected cost is 7·6. Other acceptable solutions

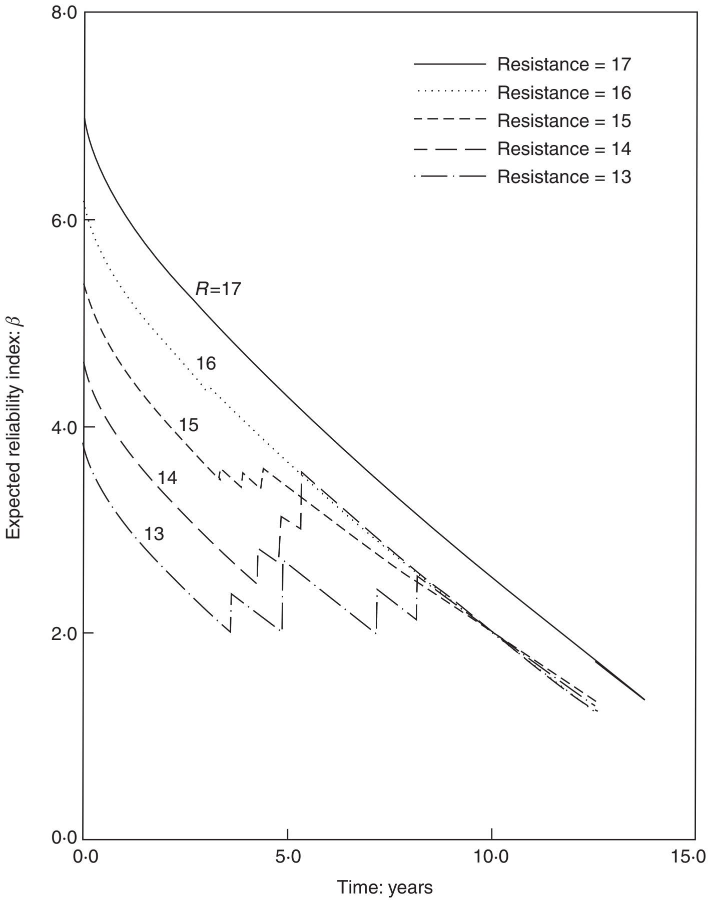

Fig. 9. Four lifetime inspections using technique C: optimum timing of inspections of a single member structure with different mean resistances

where the expected cost is within 10% of the optimum would be four lifetime inspections with technique D, three lifetime inspections with technique C, or two lifetime inspections with technique B. Use of inspection technique A would be a waste of money in this case.

The global optimum lifetime inspection strategy for different values of the mean resistances $\overline{R}$ based on minimum expected total cost is shown in Table 6. Note that $V(R) = 0{\cdot}10$ in all cases shown in Table 6.

Systems

A similar optimisation is now performed on a series (i.e. weakest link) and parallel (i.e. fail safe) systems. Considering two lifetime inspections, Tables

Table 6. Optimum inspection strategy for different mean resistances

Mean resistance	Optimum inspection technique	Optimum number of lifetime inspections
12·18	A	4
12·50	A	3
13	B	2
14	C	2
15	C	2
16	D	2
17	D	2

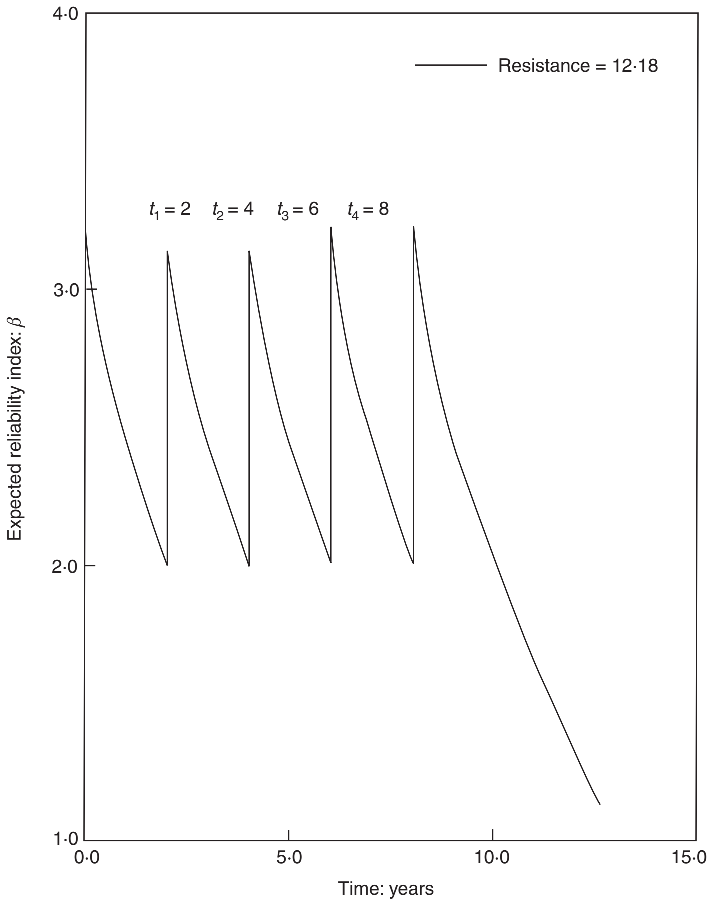

Fig. 10. Four lifetime inspections using technique A: optimum timing of inspections of a single member structure with mean resistance equal to 12·18

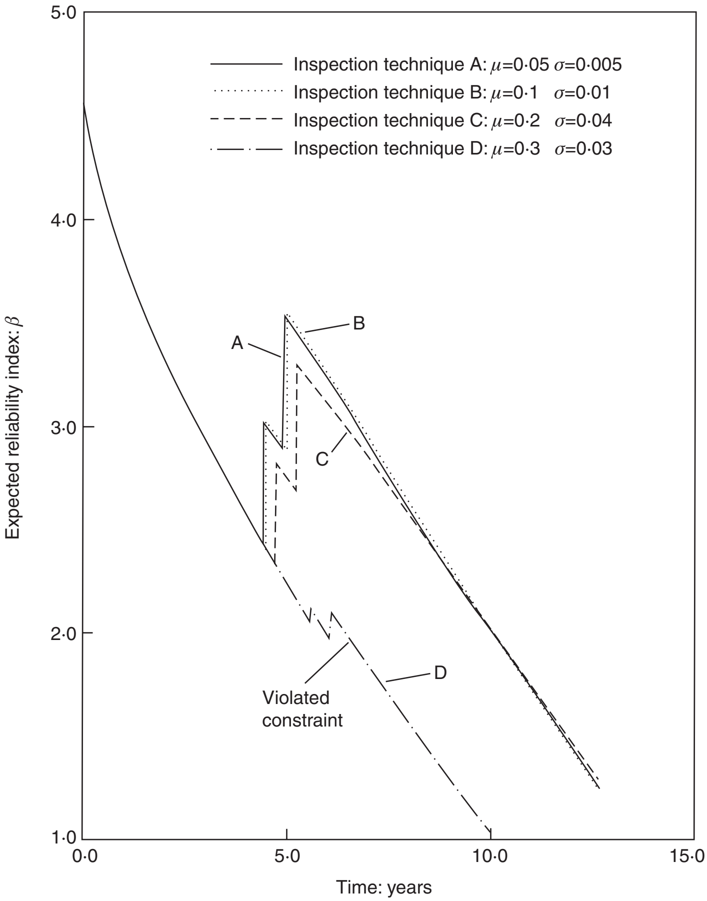

Fig. 11. Two lifetime inspections: effect of various inspection techniques on the reliability of a single member structure with mean resistance equal to 14

7 (see also Fig. 14) and 8 (see also Fig. 15) show the optimum inspection strategies for the cases of a two-member system with equal independent resistances, $R_1 = R_2$, in series and parallel, respectively. In the case of the parallel system, the mean value of the single member resistance (i.e. $\overline{R} = 14$) was halved (i.e. $\overline{R}_1 = \overline{R}_2 = 7$) in order to make a fair comparison with the series and single member structure considered earlier. The coefficient of variation was unchanged (i.e. $V(R_1) = V(R_2) = 0{\cdot}10$). Even though the mean resistances of the members in the parallel system were halved relative to the series and single member structure, the parallel system was still more reliable. Inspection techniques A and B provided answers with no violated constraints for mean resistances as low as 6·0. Neither the single member nor series system came close.

Table 7. Optimum inspection strategy and associated expected total cost for two-member series system with two lifetime inspections

Mean resistance $\overline{R}_1 = \overline{R}_2$	Inspection technique	Optimum inspection times (years) t_1	t_2	Expected total cost
13·5	A	3·83	6·18	15·0
14·0	A	4·70	5·19	11·8
15·0	A	3·90	4·39	10·3
16·0	A	2·18	2·69	7·7
17·0	A	0·50	1·01	4·9
14·0	B	4·59	5·17	9·4
15·0	B	3·85	4·43	8·6
16·0	B	2·09	2·72	6·3
17·0	B	0·51	1·01	3·0
14·0	C	4·83	5·42	9·9
15·0	C	4·55	5·05	9·1
16·0	C	3·55	4·05	7·0
17·0	C	0·50	1·00	1·5
15·0	D	6·86	7·40	10·4
16·0	D	3·93	7·22	6·7
17·0	D	2·77	3·38	1·0

Table 8. Optimum inspection strategy and associated expected total cost for two-member parallel system with two lifetime inspections

Mean resistance $\overline{R}_1 = \overline{R}_2$	Inspection technique	Optimum inspection times (years) t_1	t_2	Expected total cost
6·0	A	3·28	6·73	17·07
6·25	A	4·0	6·0	14·43
6·5	A	4·52	5·03	11·29
6·75	A	4·04	4·55	9·67
7·0	A	3·61	4·13	8·49
7·5	A	1·95	2·45	5·96
6·0	B	1·93	2·45	4·63
6·25	B	4·0	6·0	13·44
6·5	B	4·38	5·02	8·89
6·75	B	3·94	4·57	7·69
7·0	B	3·57	4·18	6·79
7·5	B	1·93	2·45	4·63
6·5	C	4·27	5·63	6·62
6·75	C	4·23	5·09	6·00
7·0	C	4·17	4·67	5·39
7·5	C	3·12	3·63	4·06
7·0	D	6·22	6·73	4·97
7·5	D	5·62	6·13	3·50

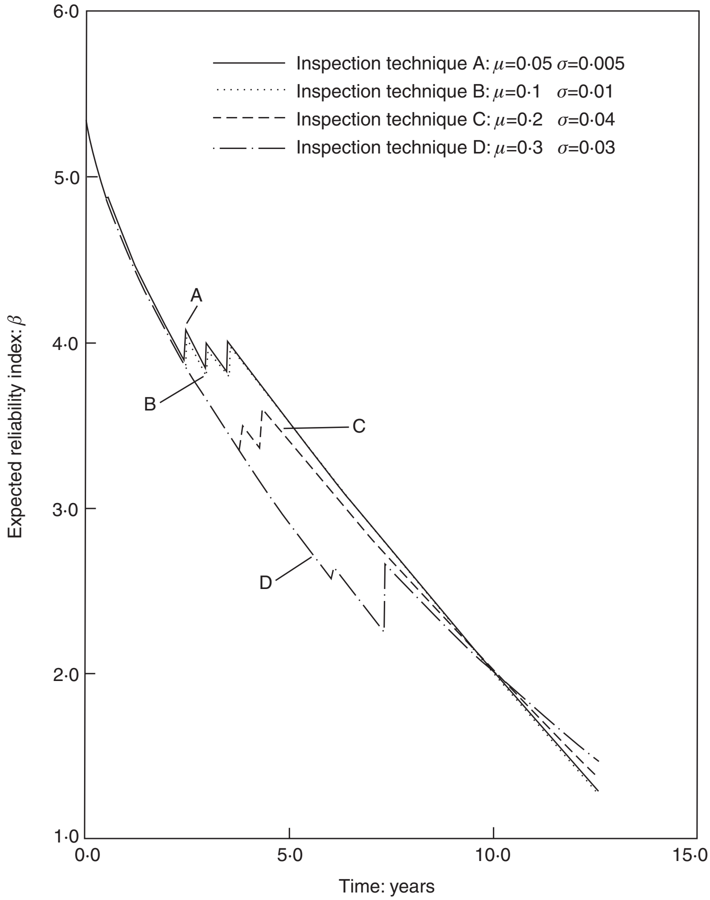

Fig. 12. Four lifetime inspections: effect of various inspection techniques on the reliability of a single member structure with mean resistance equal to 15

New bridges

The proposed optimum lifetime inspection/repair methodology was applied to the design of bridge inspection/repair programmes for new reinforced concrete T-girder bridges. The total life-cycle cost to be minimised consisted of a routine preventative maintenance cost to be incurred every two years, a cost of inspection, and a cost of repair, to accompany the initial construction cost and cost of failure. The time value of money is also taken into account. As an example, Fig. 16 shows the optimum inspection/repair programme over 75 years for a reinforced concrete T-girder bridge under corrosion attack with a corrosion rate $\nu = 0{\cdot}0114$ cm/year.[12] For details on the corrosion process and on the optimisation methodology, the reader is referred to Lin,[12] Thoft-Christensen *et al.*,[13] Frangopol *et al.*, [11,14] and Frangopol.[15,16]

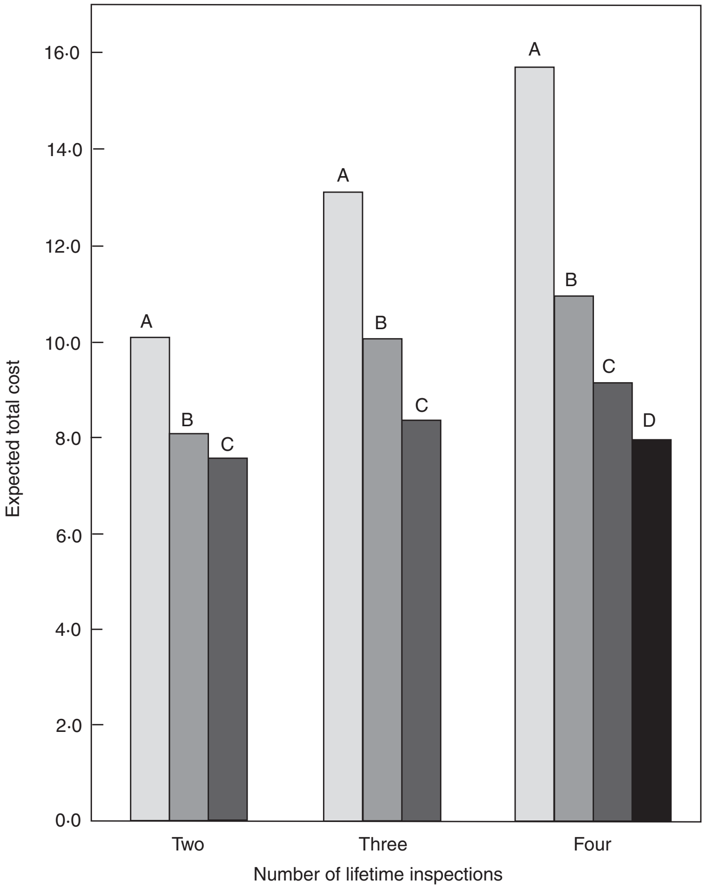

Fig. 13. Expected total costs for a single member structure with mean resistance equal to 14 for two, three and four lifetime inspections using techniques A, B, C and D

Existing bridges[7, 8]

The proposed methodology was next applied to an existing concrete bridge deck using the half-cell potential inspection method and realistic cost and inspection/repair data. The structure is a 42·1 m by 12·2 m concrete bridge deck. As road salts are applied to the deck, the chlorides penetrate the concrete. When the chloride concentration reaches a critical threshold concentration at the reinforcing steel, corrosion begins. This eventually causes spalls and delaminations in the concrete. The deck will be replaced when the active corrosion is underway in at least 50% of the deck, which is consistent with Colorado Department of Transportation policy. The mean chloride initiation time for the concrete deck was computed as 19·60 years and the standard deviation as 7·51 years.

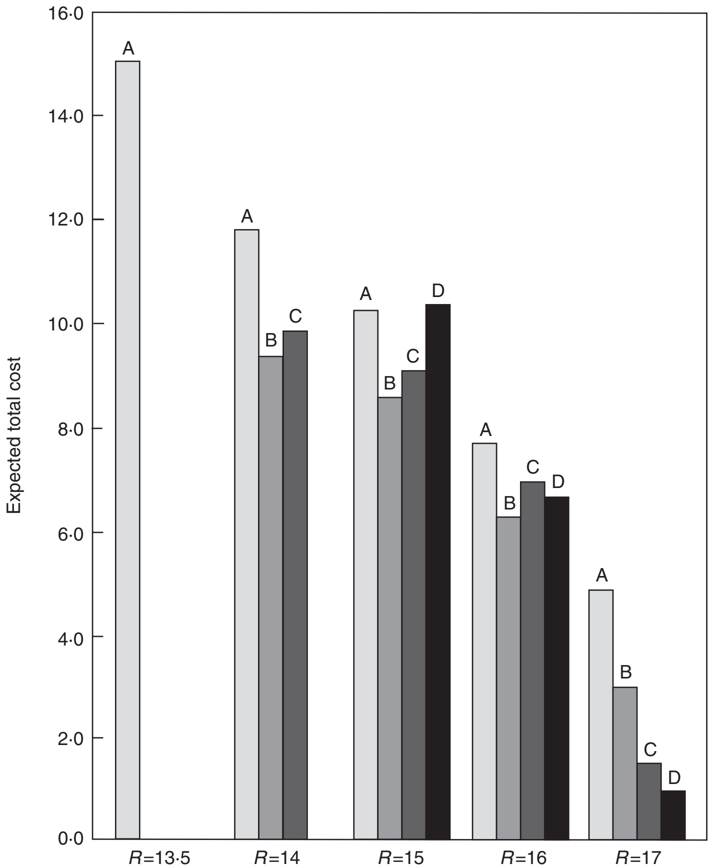

Fig. 14. Expected total cost for a two-member series system with equal mean member resistances, $\overline{R}_1 = \overline{R}_2$, for two lifetime inspections using techniques A, B, C and D: effect of mean member resistance

The uncertainty of assessing the condition of the entire deck from a finite number of half-cell readings is considered using three different inspection techniques I_1, I_2 and I_3, where the differences are the spacing between half-cell readings (i.e. 1·52 m, 3·05 m, and 6·10 m, for methods I_1, I_2, and I_3, respectively) and inspection cost (i.e. \$1027, \$604 and \$408, for methods I_1, I_2, and I_3, respectively). The inspection cost (in 1996 US\$) was estimated in consultation with specialists from the Colorado Department of Transportation. These costs include travel time to the site, traffic control, test set-up, recording readings, and preparing a final report.

The only repair option considered is replacement of the deck at a repair cost (in 1996 US\$) of $C_{\text{rep}} = \$225\,600$.[5] The probability of making a repair is a function of the number of half-cell readings, the interpreted results of the inspection and the bridge manager's approach to repair. Four repair

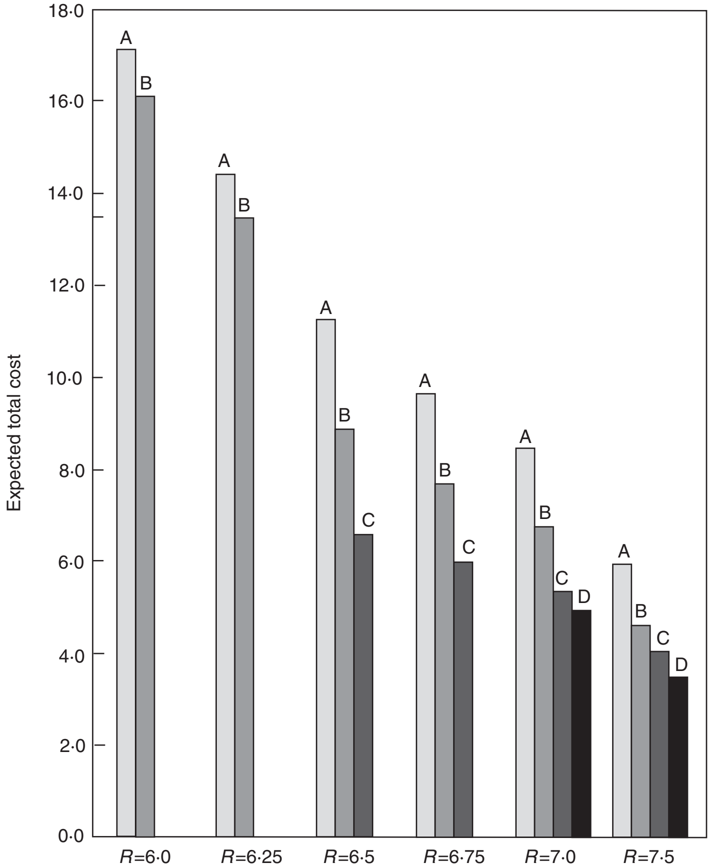

Fig. 15. Expected total cost for a two-member parallel system with equal mean member resistances, $\overline{R}_1 = \overline{R}_2$, for two lifetime inspections using techniques A, B, C and D: effect of mean member resistance

approaches (delayed, linear, proactive and idealised) are used.[7] The repair approach relates the interpreted damage for the deck to the bridge manager's willingness to make the repair based on past performance.

A discrete optimisation of the bridge deck was conducted for one, two, three and four lifetime inspections. For the case of four lifetime inspections, the optimisation problem which minimises the expected value of the total cost $E(C)_{\text{tot}}$ is formulated as

Minimise: $E(C_{\text{tot}})$
Such that:
$E\,(Damage)_{\text{t1}} \leq 0{\cdot}50;\ E\,(Damage)_{\text{t2}} \leq 0{\cdot}50;\ E\,(Damage)_{\text{t3}} \leq 0{\cdot}50;$
$E\,(Damage)_{\text{t4}} \leq 0{\cdot}50$
$E\,(Damage)_{\text{Life}_{\text{service}}} \leq 0{\cdot}50$ (10)

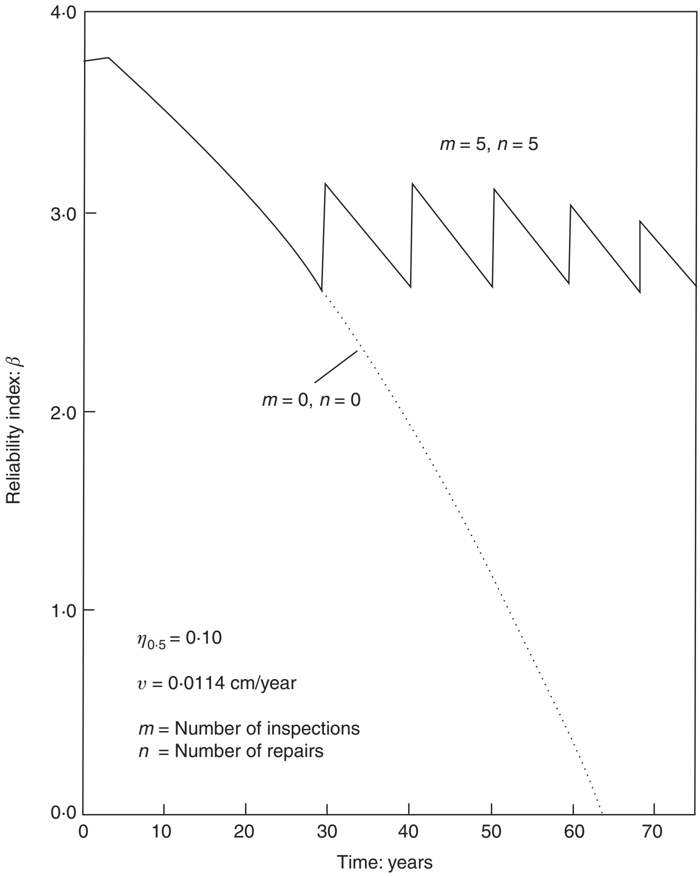

Fig. 16. Optimum inspection/repair programme for a new reinforced concrete T-girder bridge

$$2{\cdot}0 \le t_1 \le 20{\cdot}0;\ 2{\cdot}0 \le t_2 - t_1 \le 20{\cdot}0;\ 2{\cdot}0 \le t_3 - t_2 \le 20{\cdot}0;$$
$$2{\cdot}0 \le t_4 - t_3 \le 20{\cdot}0;\ t_4 \le Life_{service} \tag{11}$$

where t_1, t_2, t_3, and t_4 are the times when the four inspections will be conducted. Equation 10 ensures that the expected damage of the deck *E(Damage)* at any point in time t_i never exceeds the 50% damage limit established by the replacement policy, and Equation 11 ensures that the inspections are at least two years apart but not more than 20 years apart.

After an inspection, a decision regarding whether or not to repair the structure based on the degree of damage that was detected in the inspection is again made using an event tree. The probability of taking any branch or sub-branch on the event tree is computed using Equation 4, and the probability of making a repair is given by Equation 5.

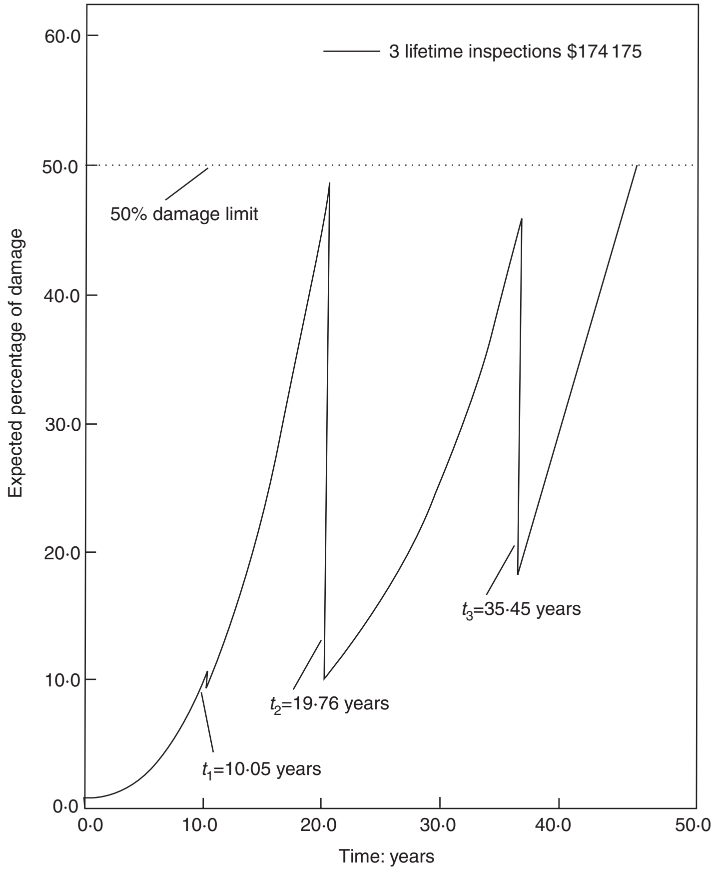

Fig. 17. Optimum inspection/repair programme for an existing reinforced concrete bridge deck

The repair criterion in this example is based on expected value of damage rather than on an expected reliability index. The expected damage computation is similar to that shown for the expected reliability index computation in Equation 7. For each $Branch_i$ on the event tree, the expected damage to the structure given that $Branch_i$ was taken, $E(Damage|Branch_i)$, is multiplied by the probability of that branch being taken P_{b_i}. The total expected damage to the structure *E(Damage)* is equal to the sum over all branches

$$E(Damage) = \sum_{i=1}^{2^n} E(Damage|Branch_i)P_{b_i} \quad (12)$$

where n is the number of lifetime inspections.

Finally, the expected value of the total cost $E\,(C_{\mathrm{tot}})$ to be minimised is equal to

$$E(C_{\mathrm{tot}})=C_{\mathrm{insp_{life}}}+E(C_{\mathrm{rep}})=C_{\mathrm{insp}}\sum_{i=1}^{n}\frac{1}{(1+r)^{t_i}}+\sum_{i=1}^{2^n}P_{b_i}\sum_{j=1}^{m}C_{\mathrm{rep}}\frac{<\psi_j>}{(1+r)^{t_j}} \quad (13)$$

As indicated, the equations for computing the lifetime cost of inspection $C_{\mathrm{insp_{life}}}$ and expected cost of repair $E(C_{\mathrm{rep}})$ have changed from those presented in Equation 9 to accommodate a discount rate, r, which accounts for the time value of money. Assuming a discount rate $r = 2\%$, an expected service life of the bridge of 45 years, a proactive approach to repair, and inspection technique I_1 (i.e. $C_{\mathrm{insp}} = \$1027$), the optimum inspection strategy requires three lifetime inspections at 10·05 years, 19·76 years and 35·45 years. The expected optimal total cost is $E\,(C_{\mathrm{tot}}) = \$174\,175$. Fig. 17 shows the expected value of damage at each inspection and the expected effect of the deck replacement. There appears to be little probability of replacing the deck after the first inspection, but a fairly high likelihood of replacement after the second and/or third inspection.

While the initial optimum strategy is for three lifetime inspections at 10·05, 19·76, and 35·45 years, the inspection/repair programme will be updated after each inspection to account for the new information that the inspection provides. After the first inspection, the replacement decision will be made and half of the eight paths can be eliminated. With that additional information, an updated optimum inspection plan was proposed.

Conclusions

The objective of this study was to develop a methodology for optimum design of bridge inspection/repair programmes based on lifetime reliability and life-cycle cost. The ten steps of the proposed methodology which minimises the expected lifetime inspection/repair cost and maintains an allowable level of reliability for a deteriorating bridge were stated. The methodology was illustrated using both hypothetical and real structures. Once the specific nondestructive evaluation inspection techniques are identified, the methodology proposed can be used to optimise the number and timing of these inspections. The result identifies the optimum inspection technique, number of lifetime inspections/repairs, the timing of these inspection/repairs, the reliability level of bridge, and the expected minimum lifetime inspection/repair cost during the service-life of a deteriorating bridge. This optimum maintenance strategy must be updated after every inspection as more information becomes available. Procedures for updating the time-variant system reliability of deteriorating concrete bridges[17,18] are currently under study at the University of Colorado.

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Fatigue assessment of steel and concrete bridges

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Introduction

Consideration of fatigue is important in bridge management. A fatigue performance of structure depends on strength of materials and load spectra. The most important load parameters are amplitude and frequency of loading. To investigate fatigue of bridges loaded with heavy trucks, it is convenient to use the load model based on weigh-in-motion (WIM) measurements. WIM can be used to calculate statistical stress parameters for girders. The results indicate that magnitude and frequency of truck loading are strongly site-specific.

In their service life, bridges are exposed to traffic loads, sometimes very heavy, especially on the high volume roads.[1] Multiple application of dynamic load may lead to fatigue-specific changes in the structural materials. Fatigue damage affects especially short span bridges where dead load is relatively low, and therefore, the live load stress ranges are higher than in the case of long span bridges. The available data, used to develop the fatigue load models show that the number of trucks on the slow lane of highways can be very high, in some cases 5000 per day were observed. It gives over 180 million vehicles during a lifetime of 100 years. This number of trucks corresponds to many more cycles in structural elements because each axle may generate a separate load cycle. Usually, a real number of load cycles during the service life of the bridge is greater than is assumed in design codes, for example Eurocode specifies 100 million cycles.

Material response has been studied by many researchers. For steel girders, so called S–N curves were developed for various categories of details in steel structures. The distribution of the number of cycles to failure can be approximated as normal, with the coefficient of variation decreasing for decreasing stress levels. For reinforced concrete components, the fatigue-caused reduction of strength applies to reinforcing steel and/or concrete. It was observed that strength of concrete under cyclic loading can be drastically reduced. The limit state function for fatigue can be expressed in terms of two variables, the number of cycles to failure under given stress history, and the number of applied cycles. Both are random variables and they can be described by their cumulative distribution functions.

For reinforced concrete T-beams, a procedure is presented for prediction of the remaining life with regard to fatigue. Presented model assumes degradation of concrete in the compression zone caused by repeated loading. Accumulated damage in a composite material such as concrete results in micro-cracks and a reduced ultimate strength. Then, the load carrying capacity of flexural members can be governed by the ability of the compression zone to carry the load (as in case of an over-reinforced beam).

Limit state function

Limit state function for fatigue can be written as a function of time (the time to failure should be greater than the time of desired service), or as a percentage of the remaining life (damage function reaches value 1 at the failure point).

The limit state function for fatigue in steel girder bridges can be expressed in terms of two variables

$$N_{\mathrm{f}} - N_{\mathrm{n}} = 0 \tag{1}$$

where N_{f} is the number of cycles to failure under given stress history, and N_{n} denotes the number of applied cycles. Both N_{f} and N_{n} are random variables, and they can be described by their cumulative distribution functions (CDFs).

Load model

Amplitude and frequency of loading as two most important fatigue parameters can be obtained from WIM measurements.[1] For bridges especially loaded with heavy trucks, WIM can be used to calculate stress parameters for girders.

Field measurements were conducted on steel girder bridges. Strain transducers were attached to all girders at the lower, mid span flanges. Dynamic strain cycles were measured under normal traffic using the rainflow algorithm of cycle counting. The data was collected and recorded including stress histograms for the girders and other components. The rainflow method counts the number, n, of cycles in each predetermined stress range, S_{i}, for a given stress history.[2]

Typical cumulative distribution functions (CDFs) for a steel girder bridge are shown in Fig. 1, on a normal probability scale. In this example, strains were measured on US-23/Saline Rd Steel Bridge in Michigan, USA, in 1995. Strain histories were collected continuously and reduced using the rainflow algorithm. The data presented here represents strain cycles due to seven

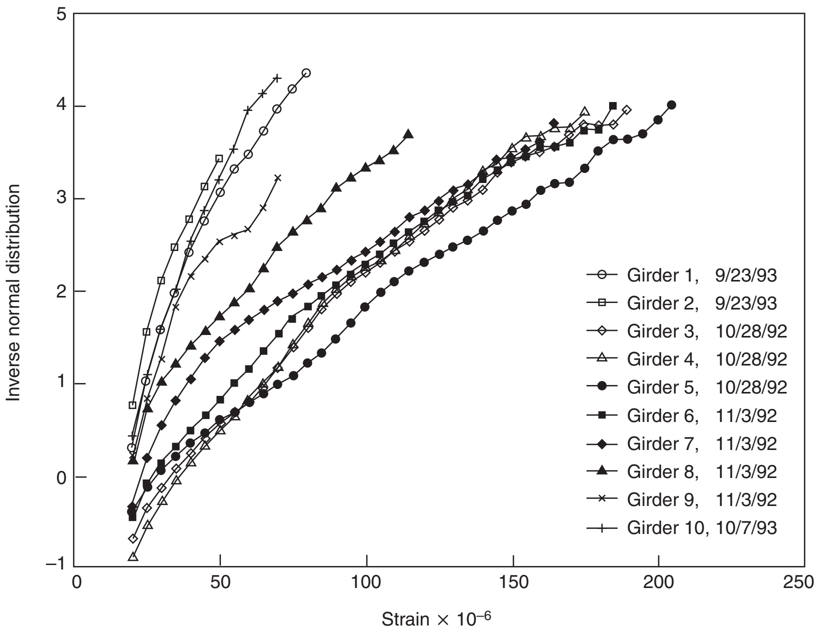

Fig. 1. Cumulative distribution functions of measured girder strains

days of normal traffic. The girders were numbered from G1 (exterior), through G10. The maximum strain was in the interior girder, G5. Stress history (S_i ranges) calculation is based on strain measurements and modulus of elasticity of steel. The observation of extreme loaded girder is important for focusing inspection on potential fatigue prone details and fatigue design of components near the location of maximum equivalent stress.

The equivalent stress, S_{eq}, which corresponds to a constant load amplitude is calculated for each girder using the following root mean cube (RMC) formula[3]

$$S_{eq} = \sqrt[3]{\sum (p_i \times S_i^3)} \tag{2}$$

where S_i is the midpoint of the stress interval *i*, and p_i is the relative frequency of cycle counts for interval i. This formula is based on the assumption that the damage in metal (steel) accumulates in a linear way due to applied loading cycles.[4] The equivalent stress values for strain spectra given in Fig. 1 are shown in Fig. 2. The number of applied load cycles, N_n, in Equation 1, is associated with the frequency of cycle counts for given strain history. The uncertainty in N_n is expressed in terms of the bias factor, 1·0, and a coefficient of variation, 0·15.

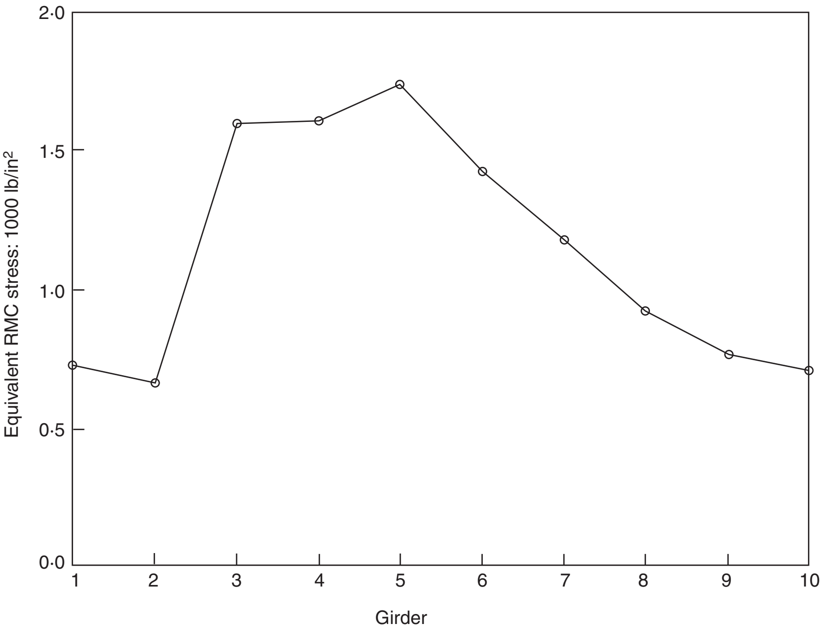

Fig. 2. Equivalent girder stresses

Resistance model

Resistance in fatigue analysis is the ability of the structure to resist cyclic loads. For steel girders, number of cycles to failure, N_f, in Equation 1 can be determined from the S–N curves. For each girder, the load amplitude varies as shown in Fig. 2, so analysis should be done separately. Most of the available material tests were performed for a constant load amplitude. For this reason, equivalent stress, S_{eq}, is used to estimate N_f. An example of a CDF of N_f is shown in Fig. 3 on the normal probability paper. The calculations are based on the S–N regression line for A441 welded girder, with stress range of 220 MPa, obtained from Fisher *et al.*[5]

In concrete structures, cyclic loading can also result in a higher damage rate compared with sustained load. This effect should be taken into account in prediction of the remaining life of a concrete bridge. Fatigue changes in concrete may be difficult to monitor because they are inside of material, but they can lead to a decrease in strength of concrete and reduction of the modulus of elasticity. Such changes obviously affect the load carrying capacity and deflection. In particular, this applies to girders which support the slow lane traffic.

Kinematically irreversible microscopic deformations which can occur in material under cyclic loading may lead to so called mechanical fatigue in concrete. Fatigue damage can arise in macroscopic and microscopic scales. The macroscopic damage can be caused by cyclic slips of the interfaces

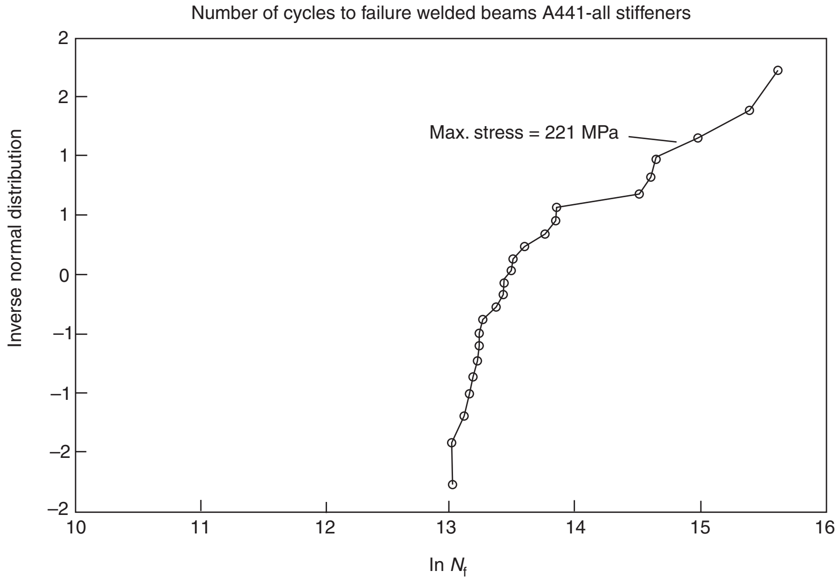

Fig. 3. Cumulative distribution function of number of cycles to failure for welded plate girder of A441 steel and stress range of 220 MPa

between the matrix and gravel, flaws and wedging of the mating surfaces. This damage appears as a visible cracking and is called *cyclic damage*. The microscopic changes occur in the matrix, they include frictional sliding of the mating faces of microcracks. Also existing microcracking due to shrinkage can be increased under cyclic loading and lead to permanent transformation strain as a result of intrinsic changes in material. The fatigue microscopic damage can be compared with rheological changes in concrete with time, under sustained load, but the rate of this damage is much faster, especially in the last stage of fatigue life of the sample.

Based on these observations, the degradation of concrete can be considered as a cyclic damage resulting from microcracking, cumulating in a linear way, and a time dependent damage with assumed viscous behaviour of material. The governing type of damage depends on the loading level.

Models describing the behaviour of the constitutive materials are required to predict the strength of concrete structures subjected to traffic loading. Based on the available literature and second co-author's test results, an analytical approach has been proposed by Szerszen *et al.*[6] that allows for evaluation of both time-dependent and cycle-dependent damage. The theory of viscoplastic damage is used to set up a constitutive law that accounts for the decrease of the longitudinal modulus and for the irrecoverable strain which develops up to failure.

The number of cycles to failure can be considered as a normal random variable, with the coefficient of variation decreasing for decreasing stress levels. However, this observation is based on a limited number of specimens and there is a need for further verification of the results.

Prediction of service life of concrete girders

The proposed way to estimate the remaining life of concrete T-beams assumes degradation of concrete in compression zone, caused by repeated loading. The load carrying capacity of the flexural beam can be governed by the ability of the compression zone to carry the load. The degradation of concrete in this zone is described by a damage function changing in time due to applied loading cycles.

Miner's rule[4] provides a simple way to allow for the rate of cycle-dependent damage. The damage factor D_{n} accumulates in a linear way in terms of cycle ratio to failure

$$\mathrm{d}D_{\mathrm{n}} = \frac{\mathrm{d}n}{\mathrm{N_f}} \tag{3}$$

where N_{f} is the number of cycles corresponding to pure fatigue failure. For higher levels of loading, the number of cycles to failure proves to be more sensitive to the parameters of the cyclic stress. A convenient way to express the rate of creep damage, D_{t}, at any time t is

$$\dot{D}_{\mathrm{t}}(t) = \frac{\beta}{\mathrm{r}+1}\left(\frac{\sigma}{f'_{\mathrm{c}}}\right)^{\mathrm{k}}(1 - D_{\mathrm{t}})^{-\mathrm{r}} \tag{4}$$

where β, k and r are material coefficients, and σ is the stress range and the dot represents d/dt.

One way to express that the behaviour of concrete is both cycle and time-dependent is to assume the rate of total damage to be a linear combination of pure fatigue damage and creep damage

$$\dot{D} = \phi_{\mathrm{n}}\dot{D}_{\mathrm{n}} + \phi_{\mathrm{t}}\dot{D}_{\mathrm{t}} \tag{5}$$

where ϕ_{t} and ϕ_{n} are two coupling coefficients and $t \geq 0$.

The rate of pure fatigue damage stays constant for identically repeated cycles, but the rate of creep damage at a given stress depends on r (constant rate for $r = 0$).

Longitudinal modulus decreases with the growing of a plastic strain in material.[6] The total strain, $\varepsilon(t)$, can be expressed at any time as the sum of an elastic strain and a creep strain, $\varepsilon_{\mathrm{p}}(t)$

$$\varepsilon(t) = \frac{\sigma(t)}{(1 - D(t))E_0} + \varepsilon_{\mathrm{p}}(t) \tag{6}$$

where E_0 is the initial value of the longitudinal modulus of elasticity. The damage factor D(t) was determined from Equation 4, assuming the existence of a possible initial damage, D_0.

$$D(t) = 1 - \left[(1 - D_0)^{r+1} - \beta\Gamma^{(\mathrm{k})}t\right]^{1/(\mathrm{r}+1)} \tag{7}$$

$$\Gamma^{(\mathrm{k})} = \frac{1}{T}\int_0^{\mathrm{T}} \left[\frac{\sigma(t)}{f_{\mathrm{c}}}\right]^k \mathrm{d}t \tag{8}$$

where T is the cycle period, and D_0 is the initial, existing damage. The influence of the damage level on creep rate is accounted for by considering the effective stress, σ'

$$\dot{\varepsilon}_{\mathrm{p}}(t) = \left(\frac{\sigma'(t)}{B}\right)^{\mathrm{m}} \tag{9}$$

$$\sigma'(t) = \frac{\sigma(t)}{(1 - D(t))} \tag{10}$$

where B and m are material coefficients.

By integrating Equation 9, with the initial strain, ε_0, creep function is

$$\varepsilon_{\mathrm{p}}(t) = \varepsilon_0 + \left(\frac{B}{f_{\mathrm{c}}}\right)^{-\mathrm{m}} \Gamma^{\mathrm{m}}\delta(t) \tag{11}$$

where

$$\delta(t) = \frac{1}{T}\int_0^{\mathrm{t}} (1 - D(\tau))^{-\mathrm{m}} d\tau \tag{12}$$

$$\Gamma^{(\mathrm{m})} = \frac{1}{T}\int_0^{\mathrm{T}} \left(\frac{\sigma(t)}{f_{\mathrm{c}}}\right)^{\mathrm{m}} \mathrm{d}t \tag{13}$$

All model parameters were calibrated using test results.[6] Equations 6, 7 and 11 describe the constitutive law of plain concrete subjected to compressive cyclic loading.

According to AASHTO,[7] the flexural load carrying capacity of a concrete T-beam depends on dimensions of the section (d, h_{f}, A_{s}, b and b_{w}) and the strength of materials (f_{y} and f'_{c}). Under live load, concrete degrades in time losing its strength. To estimate the actual strength of concrete (after a certain duration of time under service conditions) the following data is needed

- dimensions and material properties of the beam
- load history: live load levels and duration of loading
- amplitude and frequency of loading (rainflow method)
- stress amplitude in the compressive zone of the section.

The presented equations can be used to calculate the damage function for concrete under compression, effective stress and moment carrying capacity of the section. Flexural behaviour of the beam can be considered by analysis of the total strain in the compressive zone of the section. Fatigue degradation of the modulus of elasticity allows for a better prediction of bending stiffness and for span deflection calculated in more realistic conditions.

Reliability analysis

In recent studies of the ultimate limit states, the structural performance was measured in terms of the reliability index.[8,9] It is further assumed that the resistance and load parameters (N_f and N_n) are lognormal random variables. Therefore, the reliability index, β, can be calculated as follows

$$\beta = \ln(m_{\mathrm{Nf}}/m_{\mathrm{Nn}})/(V_{\mathrm{Nf}}^2 + V_{\mathrm{Nn}}^2)^{1/2} \tag{14}$$

where m_{Nf} is the mean number of cycles to failure; V_{Nf} is the coefficient of variation of the number of cycles to failure; m_{Nn} is the mean number of applied cycles; and V_{Nn} is the coefficient of variation of the number of applied cycles.

The reliability analysis for steel bridges was performed for several values of the effective stress ranges and different categories of the steel beam cross-sections.[10] For example, classification of details according to BS 5400, Part 10, gives nine detail classes for welded and non-welded details.[11] Presented analysis includes different slope coefficients in the S–N curves for particular detail classes. The results for a case of applied loading obtained from WIM measurements are presented in Fig. 4.

Conclusions

The paper presents a practical way to estimate the remaining life of reinforced concrete beams exposed to repeated dynamic loading. The fatigue model for concrete is based on the theory of viscoplasticity for composite materials and reflects the real behaviour of concrete subjected to cyclic compression. It is a convenient way to predict the remaining life of existing concrete bridges, especially those which were designed without regard to cyclic character of live load and rheological changes in material.

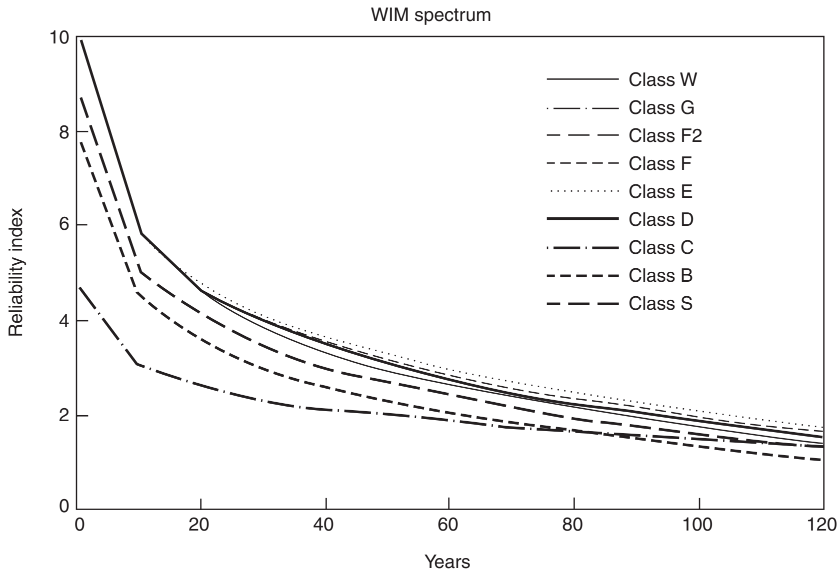

Fig. 4. Reliability indices against time

Reliability analysis is performed for fatigue limit state in steel bridge girders. The limit state function for fatigue is formulated in terms of number of load cycles. The parameters of load (number of cycles applied) and resistance (number of cycles to failure) are derived from the available test data. However, the available data is rather limited and there is a need for further verification.

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On-going issues in time-dependent reliability of deteriorating concrete bridges

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Introduction

Reliability-based design and assessment of bridges and other highway structures and associated deterioration issues are of increasing interest to researchers and the profession. Recent developments in these areas are being used as management tools for design and assessment. However, for chloride-induced corrosion, structural deterioration time-dependent reliabilities are probably of limited accuracy. For example, existing bridge reliability studies tend to ignore the interaction between cracking, diffusion of chlorides and corrosion initiation; reduction of bond; influence of design specifications on corrosion initiation and propagation; and the influence of serviceability limit states (e.g. longitudinal cracking or spalling). As such, management decisions based upon some existing time-dependent reliability models may not produce optimal outcomes.

The present paper will address these and other issues that relate to the accuracy of time-dependent reliability models for bridges — issues that need to be resolved before time-dependent reliabilities can be used for bridge design and assessment with any confidence. The scope of the discussion is restricted to chloride-induced corrosion of reinforced concrete (RC) bridges. To illustrate potential benefits and possible improvements, time-dependent probabilities of structural and serviceability failure (flexure and spalling limit states) are shown for a typical reinforced concrete bridge. The advantages of considering serviceability limit states are considered also, such as, how spalling is a precursor to collapse.

Life-cycle performance and reliability-based design/assessment

Current design and assessment reliability procedures tend to consider present loads, capacities and safety levels. This may well optimise short term performance (say, over several years), but of more relevance should be the minimisation of life-cycle costs associated with the 50–100 years service life expected of bridges. Thus, it is often more meaningful to consider life-cycle performance and optimum expenditure when making decisions related to design, construction and maintenance specifications and

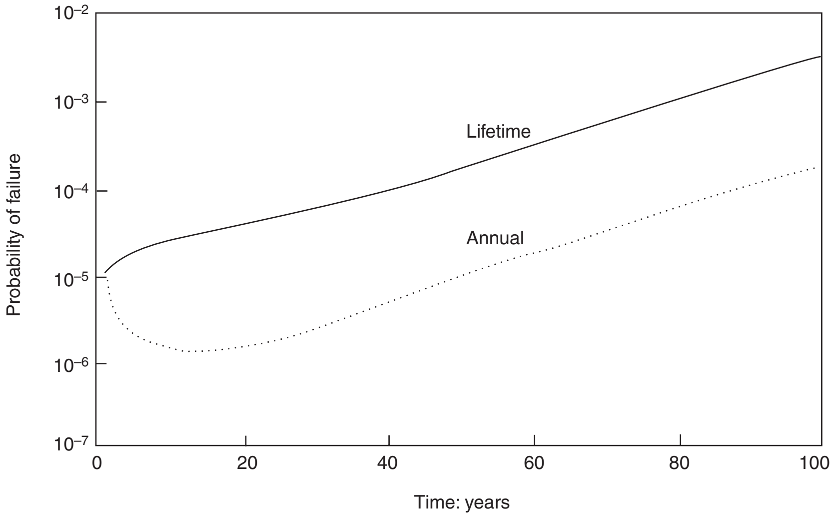

Fig. 1. Annual and cumulative time-dependent probabilities of failure[4]

strategies since such decisions consider longer term as well as immediate benefits and consequences.[1, 2]

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. 1 shows an example of annual and cumulative reliabilities considering a flexure limit state (collapse) for a bridge exposed to de-icing salts. The annual probability of failure is the probability of failure in year t, while a cumulative (or lifetime) probability of failure is the probability that the structure will have failed anytime up to year t.

However, reliabilities may increase if the bridge has survived a proof load test. Service loads on a bridge constitutes quasi proof loads, so service proven (older) bridges may have a higher bridge reliability than newer bridges.[3] For example, the probability that a bridge will fail (in flexure) in t subsequent years given that it has survived T years of service loads is denoted as $p_f(t|T)$ and is shown in Fig. 2 for the cases of deterioration (de-icing salts) and no deterioration. The calculation of $p_f(t|T)$ is given later in the paper, see Equation 2.

Therefore, life-cycle performance is best measured in a time-dependent reliability format in which structural reliabilities, p_f, or expected lifetime costs (e.g. $C_I + p_f C_F$) may be used to

- optimise design and construction specifications (cover, protective measures)
- compare with reliability-based acceptance criteria for bridge assessment

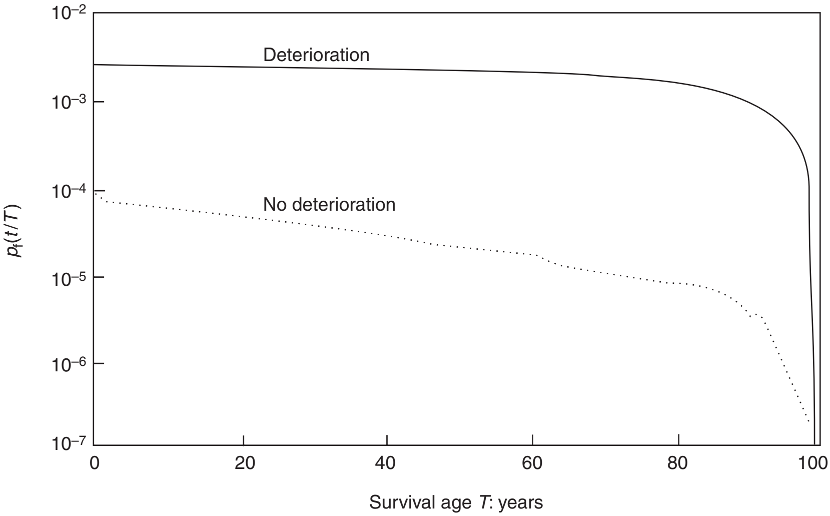

Fig. 2. Time-dependent probability of structural failure for a service proven bridge[4]

- ☐ predict magnitude of proof load and use test results to update bridge assessment
- ☐ prioritise maintenance or repair by ranking reliabilities or expected lifetime costs
- ☐ identify the most likely failure mode within one structure (and design against it).

These and other decisions are often based on a comparison of risks (costs) against benefits, such as minimising the expected cost (e.g. risk–cost–benefit analysis) or maximising the expected utility. Thus a decision analysis may quantify the expected cost of a decision.

Corrosion and fatigue are deterioration processes that significantly influence the long term performance of bridges. Changes in traffic volume and loads will also be experienced over the life of the bridge, although such changes are subject to less uncertainty and may be ameliorated by appropriate traffic controls. Consequently, it is likely that life-cycle performance is influenced most by deterioration processes and so the effect of deterioration processes on time-dependent reliability is of particular interest. Corrosion of reinforcement bars in reinforced concrete structures is the primary cause of structural deterioration of bridge decks.

Time-dependent reliability of deteriorating bridges

A time-dependent reliability analysis is a predictive analysis that requires the following information

- predictive probabilistic models for corrosion initiation and rate of propagation
- effect of corrosion propagation on structural and serviceability performance
- structural analysis model
- definition of ultimate limit states — flexural failure, shear failure, collapse, etc
- definition of serviceability limit states — cracking, durability, deflection, vibration
- accurate and efficient computational procedure for time-dependent reliability analysis.

Existing time-dependent reliability analyses of deteriorating RC bridges consider each of these aspects with varying degrees of completeness and success. Research in this area is proceeding at an increasing rate, although it is still relatively limited.[5, 6]

Review of existing time-dependent reliability models

Hoffman and Weyers[7] have developed a probabilistic model for bridge deck chloride diffusion due to the application of de-icing salts. The probabilistic models for chloride diffusion are based on extensive US data; however, this reliability model assumes that failure occurs when corrosion is initiated; a very conservative assumption.

Thoft-Christensen[8] has developed what appears to be the first reliability-based expert system that calculates structural reliabilities considering both collapse (system) and flexural cracking limit states based on initiation (chlorides, carbonation) and propagation of corrosion of the reinforcement. Inspection and maintenance data may be used to update reliabilities. Cost-benefit decision models incorporating updated time-dependent reliabilities then provide a structural assessment which can then be used to select optimal maintenance and repair work; namely, anticipated type of repair, the time of first repair and the number of repairs in the remaining life of the bridge.

Val *et al.*[9] considered reduction in area of steel and loss of bond using a non-linear finite element model that considered collapse and deflection limit states, immediate corrosion initiation (diffusion not considered), and homogeneous and localised corrosion propagation.

Frangopol *et al.*[10] have developed a useful practical application of probabilistic corrosion modelling; namely, a reliability-based design approach based on the minimisation of expected lifetime costs. The reliability analysis considers diffusion of chlorides, influence of design specifications (f'_c, cover) on corrosion initiation, and flexural and shear limit states.

Stewart and Rosowsky[4,11] have developed a structural deterioration reliability model that includes corrosion initiation due to transverse cracking (shrinkage, flexural) and chloride diffusion, the influence of design specifications (f'_c, cover) on corrosion initiation and propagation, serviceability limit states considering longitudinal cracking and spalling, and demonstrates how known exceedence of a serviceability limit state (spalling) can be used to update the probability of structural failure. The reliability model calculates annual and cumulative probabilities of structural and serviceability failures for flexure and spalling limit states.

These and other studies[12–14] have their relative merits and deficiencies. For instance, all of the above models assume uniform corrosion rates (except Teply *et al.*)[14] and ignore spatial effects and the interaction of carbonation.

Ongoing issues

Cumulative probabilities of failure for 100 years of service (strength limit state) for a bridge subject to de-icing salts vary from 3×10^{-3} for a three-span continuous RC slab bridge[4] to 1×10^{-25} for a single span RC slab bridge.[15] However, changing concrete compressive strength from a normal to a lognormal distribution changes the probability of failure for a single span bridge[9] from 2×10^{-14} to 4×10^{-20}, although it is recognised that such sensitivities are considerably higher at higher reliabilities. These results, though not directly comparable because of differing structural and corrosion models, demonstrate the difficulty in using reliability-based criteria for decision making since absolute values of risk may be sensitive to modelling assumptions and so are inherently uncertain. Such an observation is not restricted to bridges, it is equally applicable to reliability analyses of chemical and process plants, nuclear power plants and other engineering systems.[16] There are limitations and uncertainties associated with bridge behaviour, traffic volume and loads, deterioration process, system representation, selection of models and parameter values, human error and workmanship. They also draw into question how risks of 10^{-15}, 10^{-20} or 10^{-25} can be interpreted for management decisions since such values are perhaps too low to be meaningful.

It follows that the absolute precision of so-called realistic bridge reliabilities is in considerable doubt; 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) and calibration with calculated bridge reliabilities of similar bridge-types. Further effort is needed to improve existing time-dependent reliability models — some on-going issues are now described.

Material behaviour models: corrosion initiation and propagation
The deterioration process associated with the corrosion of reinforcement typically consists of

- *Initiation*: time to commencement of reinforcement corrosion. Corrosion is initiated mainly by chloride contamination, often in conjunction with reduced concrete cover, low quality concretes, and poor compaction and curing. Chlorides may penetrate through the protective concrete cover and corrosion is initiated once the chloride concentration exceeds a critical threshold value. Corrosion may be initiated also if shrinkage, flexural, thermal or other crack widths are sufficiently large to allow the direct ingress of chlorides, oxygen and moisture. Carbonation (exposure to atmospheric CO_2) will free chlorides bound to the concrete matrix, thus making the reinforcement more susceptible to chloride-induced corrosion.
- *Propagation*: Loss of area (metal loss) for reinforcement. The increased volume of corrosion products (rust) causes concrete tensile stresses that may be sufficiently large to cause internal microcracking, external longitudinal cracking and eventually spalling. This may lead to an acceleration in corrosion rate and/or reduction of bond — leading to serviceability failure and/or a loss of structural integrity.

This deterioration process is well accepted in a qualitative sense. However, for a predictive analysis, problems arise when attempts are made to quantify this process and assess its influence on structural performance; for example, what is the rate of deterioration after transverse cracking? or longitudinal cracking? Is the rate of corrosion constant? What is the influence of cover depth? Fig. 3 shows loss of bar diameter with time and how different material behaviour assumptions can lead to drastically different outcomes.

The basis for most deterioration models used in reliability studies is that corrosion of reinforcement will lead to a uniform reduction in the bar diameter of the reinforcing steel, that this corrosion rate is not influenced by cover depth or concrete material properties and that all reinforcement corrodes uniformly across the entire structural element — this approach is recognised as being overly simplistic. Material behaviour models that predict corrosion initiation and propagation need to be developed or refined for

- chloride penetration in uncracked/cracked concrete for continuous and intermittent exposure
- drying shrinkage, flexural (surface) and longitudinal (bond) cracking
- loss of steel area and reduction of concrete/steel bond strength
- chloride concentration and crack width(s) for corrosion initiation

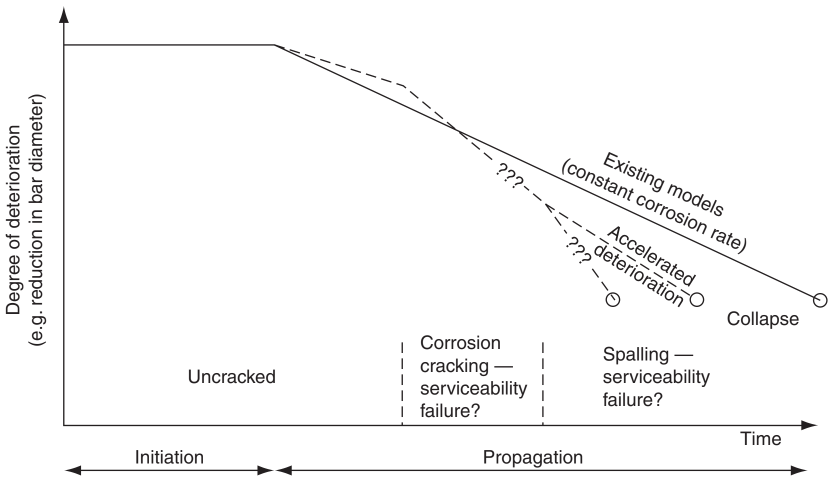

Fig. 3. Deterioration process for reduction of reinforcing steel diameter

- ☐ interaction between carbonation and chloride-induced corrosion
- ☐ corrosion rates and influence of concrete quality, cover, oxygen diffusion, cracking, etc
- ☐ spatial effects when material behaviour properties vary spatially over the structural element.

Deterministic models exist for only a few deterioration processes; for example, penetration of chlorides into concrete is usually given empirically by Fick's second law of diffusion. Yet the accuracy (and, hence, uncertainty) associated with these and other deterministic models is often unknown, and the influence of other variables such as concrete mix design (e.g. water–cement ratio, cement type and content, pozzolanic replacement materials, air entrainment, etc.), cover depth, cracking, exposure conditions, corrosion initiators or interactions with loads are ignored. Existing deterministic (although empirical) predictive models are often based on a specific duration of exposure, material and environmental conditions which may not be applicable to more general use. Clearly, such phenomena will influence structural and serviceability performance.

For example, existing studies suggest that structural reliabilities are most sensitive to the corrosion rate.[11] The corrosion rate is governed by the availability of water and oxygen at the steel surface, and so is probably a function of mix design, cover and degree of cracking. Available information on the effects of such phenomena are limited; however, it is essential that reliability analyses consider the influence of these variables when predicting corrosion rates.

Time-dependent reliability analysis — time variant plotted against time invariant analyses

Current bridge reliability analyses often ignore the influence of service loads on bridge resistance. This is quite conservative. For bridge assessment, the observation that the bridge has actually survived enables information about structural resistance to be updated and consequently the bridge reliability also, resulting in the increase of bridge reliability for service proven (older) bridges (Fig. 2).

A corrosion-induced deterioration process will reduce structural resistance (or capacity) and so structural resistance is time-dependent — denoted herein as $R(t)$. Further, structural loads may occur randomly in time and/or in intensity (Fig. 4). 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 or proof loaded bridges (strength limit state) anytime during this time interval is

$$p_f(0,\ t_L) = 1 - \Pr[R(t_1) > S_1 \cap R(t_2) > S_2 \cap \ldots \cap R(t_n) > S_n]$$
$$t_1 < t_2 < \ldots < t_n \leq t_L \tag{1}$$

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 updated on survival of the previous load events.

The probability of failure for any time increment t_{n-1} to t_n is simply $p_f(t_n) = p_f(0,\ t_n) - p_f(0,\ t_{n-1})$ — for annual time increments this is known as an annual probability of failure. Clearly, the probability of failure is dependent upon the prior load and resistance histories — this is referred to herein as a time variant reliability.

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

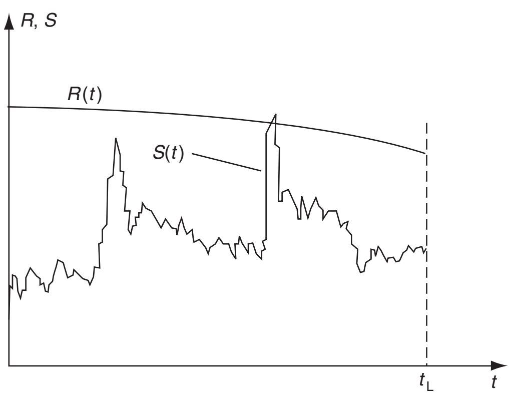

Fig. 4. Typical realisations of load effect S(t) and resistance R(t)

$$p_f(t|T) = \frac{p_f(0,\, T+t) - p_f(0,\, T)}{1 - p_f(0,\, T)} \tag{2}$$

where $p_f(0, T+t)$ and $p_f(0, T)$ are obtained from Equation 1, see Fig. 2.

The above time-dependent reliability formulation can be extended to consider more realistic load realisations; namely, load combinations and stochastic load processes.[17,18] Only under fairly simple conditions can the solution to the above equation be obtained in closed-form. For example, the problem can be cast as a first-passage or upcrossing problem under some very generalised conditions. However, this time-dependent reliability analysis is complicated by the necessary inclusion of highly nonlinear limit state functions, non-normal random variables, stochastic load process models, time-varying resistance quantities (due to corrosion), and correlated resistances and loads (e.g., corrosion and flexural cracking). This approach has been adopted by Stewart and Rosowsky[4,11] in their studies, although the computational effort associated with Monte Carlo simulation is considerable and is not yet suitable for immediate practical implementation.

However, if loads and resistance are independent quantities then analytical approaches used by Mori and Ellingwood[17] and Val *et al.*[9] may be just as appropriate.

The computational effort may be reduced considerably (at the expense of simplifying assumptions) if it is assumed that the probability of failure is dependent only on the maximum lifetime load up to time t_L and resistance at time t_L, represented as

$$p_f(t_L) = \Pr[\max(S_1,\, S_2, \ldots, S_n) > R(t_L)] \tag{3}$$

This is often termed a *time-invariant* or *point-in-time* reliability. Fig. 5 shows a comparison of the time variant and time-invariant reliabilities calculated for a three-span continuous RC slab bridge. It is evident that the time-invariant probability of failure over-estimates the actual (time variant) probability of failure and so in this case is quite conservative.

In all such cases, the calculation of time-dependent reliabilities requires the use of sophisticated computational procedures. Consequently, to date time-dependent reliability analyses have found little immediate practical implementation, although software is becoming available that includes the effect of some types of deterioration on time-dependent reliability.

Limit states — serviceability

To date, reliability analyses have concentrated on the loss of flexural strength being the main consequence of deterioration. However, punching shear and serviceability limit states such as deflection, longitudinal cracking and spalling are also influenced by corrosion. For example, bond performance (reduced by corrosion) has a significant influence on deflection and crack development and the spalling of concrete cover from bridges is a

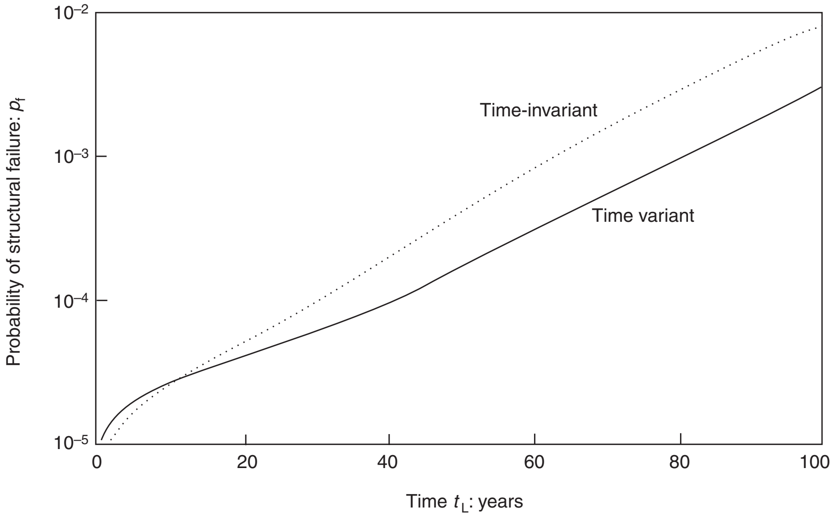

Fig. 5. Comparison of time variant and time-invariant reliabilities

common problem, that in some cases constitutes a life-safety hazard since falling concrete has been known to cause damage to vehicles and death. Moreover, for strength limit states, serviceability failure may well be a precursor to strength failure and may also be a more useful indicator of structural deterioration, particularly with respect to optimising inspection and maintenance schedules.

For instance, the probability that a bridge will fail in t subsequent years given that longitudinal cracking or spalling has just occurred at T years ($t + T = 100$) is denoted as $p_{\mathrm{f}}(t|\text{spalling at } T)$ and is shown in Fig. 6. It is assumed that longitudinal cracking, spalling or low concrete cover doubles corrosion rate. As expected, the conditional probability of structural failure decreases as the time of detection of first spalling increases since there is less time for corrosion propagation and so less time for failure to occur.

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.

Figure 7 shows the probability of longitudinal cracking and spalling for a RC bridge subject to repeated applications of de-icing salts. The information contained in Fig. 7 may be used to help prioritise bridge inspection strategies. For instance, an existing bridge with known (measured) 75 mm cover will need little, if any, inspections in the first 20

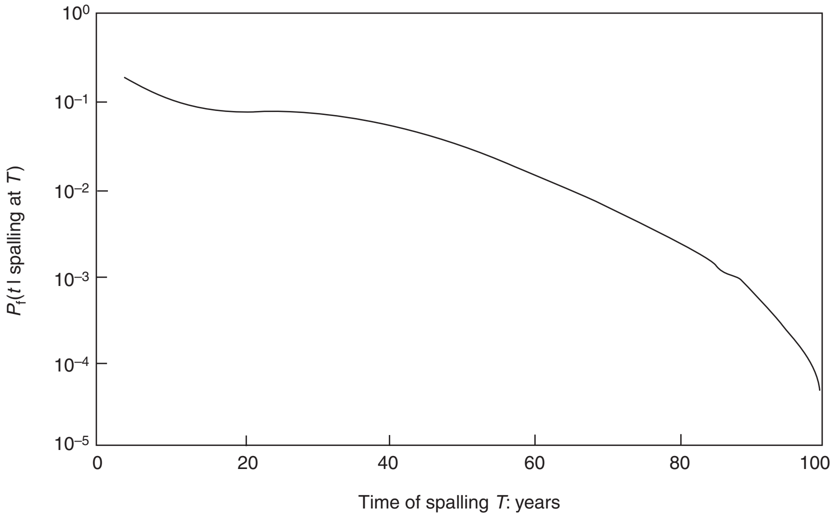

Fig. 6. Probability that structure will fail given that spalling occurs at year T[4]

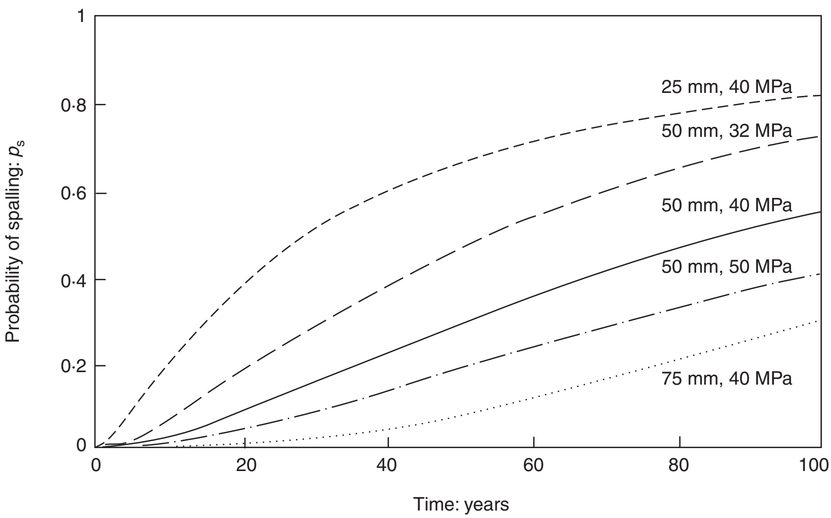

Fig. 7. Influence of cover and concrete compressive strength on probabilty of spalling[4]

years since Fig. 7 suggests that it is highly unlikely that inspections before this period will reveal any evidence of damage or deterioration. However, an existing 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. These reliabilities can be updated also when 'new' information becomes available;

for example, if after ten years the bridge with 25 mm cover shows no sign of damage, then reliability calculations can be updated. In this case, resulting in lower predicted probabilities of spalling. The approach described herein may thus be used to optimise inspection strategies by focusing inspection resources on those bridges most likely to experience damage or deterioration.

Accuracy of long-term predictions

It should be borne in mind that long-term predictions of bridge performance are inherently uncertain. In the case studies presented herein, bridge reliabilities are calculated for intervals or reference periods of up to 100 years. In practice, predictions of bridge performance can only be relatively accurate for much shorter time periods — about 5 to 10 years. This applies to deterioration processes and also to load modelling. It is very difficult to predict with any degree of confidence what the actual traffic loads will be in ten or more years time since historically traffic loads have continually been increasing. As such, it is strongly advised that reliability analyses (or other predictive analyses) be restricted to relatively short reference periods.

Sensitivity of decision analyses

Bridge management decisions may be based on reliabilities or expected life-cycle costs. Issues such as the reality of computed reliabilities, economic or other value of a human life, who bears the risk? (damage costs normally paid by society through insurance), who benefits? who is the decision-maker? (contractor, highways agency or society) are yet to be fully resolved. The outcome of the decision analysis might well be sensitive to these uncertainties and so should be treated with some caution. Clearly, the decision analysis should be subject to a sensitivity analysis to ensure that decisions are not unduly influenced by damage, construction or other costs or by the estimates of bridge reliability.

Conclusion

Estimates of time-dependent reliability provides a powerful decision-making tool for the management of highway structures and other infrastructure. However, existing methods used to calculate time-dependent bridge reliabilities are by no means accurate — modelling of corrosion initiation and propagation needs to be more realistic and the interaction between serviceability and strength limit states needs to be included in future analyses. Existing models are most likely of limited accuracy and so decisions based upon such time-dependent reliability models may not produce optimal outcomes.

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Whole life costing of maintenance options

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Introduction

Bridges deteriorate progressively with age for a variety of reasons and this deterioration may lead to a reduction in load carrying capacity. Maintenance work is normally carried out periodically to extend the life of a bridge. This paper appraises the economics of a range of maintenance strategies for concrete bridges in order to find the strategy which provides the best value for money. The appraisal was carried out using whole life costing with discounted cash flow. The costs considered were for materials, labour, access and traffic management and for traffic delays resulting from the maintenance work.

Maintenance strategies

There are three main types of maintenance strategy

- preventative maintenance to slow down the rate of deterioration and hence to increase the age of the bridge when repairs or strengthening is needed
- remedial work to repair the damage caused by deterioration and to slow down the rate of future deterioration
- strengthening work to recover the lost load carrying capacity caused by deterioration.

Examples of preventative maintenance are surface protection by silane impregnants or waterproofing membranes. Silane is generally applied to all exposed surfaces of a bridge and provides a hydrophobic layer that largely prevents the ingress of aqueous salt solution. Waterproofing membranes are always used on the upper surface of bridge decks but they can also be effective when applied to substructure elements such as piers, crossheads and abutment or bearing shelves.

Examples of repair work are cathodic protection and concrete repairs. Concrete repair involves cutting out the chloride contaminated and damaged concrete, cleaning the reinforcement and backfilling with a suitable repair concrete or mortar. Cathodic protection involves impressing a small electric current in the reinforcement using an external anode and a dc power source in order to counteract the corrosion current, thereby

Table 1. Maintenance strategies for calculating discounted whole life costs

	1st Maintenance			2nd Maintenance			3rd Maintenance		
Option	Age	L/S/A*	Super/Sub†	Age	L/S/A*	Super/Sub†	Age	L/S/A*	Super/Sub†
Waterproofing	20	L	Super + Sub	40	A	Super	60	A	Super + Sub
				40	L	Sub			
Surface treatment	20	A	Super + Sub	40	A	Super + Sub	60	A	Super + Sub
CP min.	20	L	Super + Sub	45	A	Super	75	A	Super
				45	L	Sub	75	S	Sub
CP all	20	A	Super + Sub	50	A	Super + Sub	80	A	Super + Sub
Cut out and repair (COR)	20	L, 20%	Super + Sub	40	A, 20%	Super	60	A, 50A, 2	Super
				40	L, 50%	Sub	60	0	Sub

* L/S/A: Leakage area/spray area/exposed area
† Super/sub superstructure (deck)/substructure (piers and abutments)

stopping the corrosion reaction. Damaged concrete, but not contaminated concrete, must be repaired before applying cathodic protection.

An example of strengthening work is to replace elements such as decks, piers and crossheads or columns. Abutments are not normally replaced and one of the other maintenance options should be used for this type of element.

In addition to the type of maintenance option, the timing of maintenance is also important and this depends on the maintenance free life of the element and the life of the maintenance option selected. Maintenance treatments can be applied to the entire element or just to those parts exposed to leaks from expansion joints or spray from traffic. Details of the timing and extent of maintenance for the different elements and options considered in this paper are shown in Table 1.

The whole life cost model

For each maintenance operation identified in Table 1 the engineering costs were calculated using a bill of quantities format based on normal quantity surveying procedures for a typical dual 2 trunk road reinforced concrete overbridge. The traffic delay costs arising from each maintenance operation were estimated using the QUADRO[1] ready reckoner tables in the *Trunk road maintenance manual*[2] assuming the traffic included 10% HGV and the closure was 1 km in length. A range of traffic flow rates (*vpd*) was considered and the type and duration of lane or carriageway closures was based on typical values for the type of maintenance and element maintained. Maintenance was assumed to be repeated at a frequency dependent on the life of the maintenance treatment until the design life (120

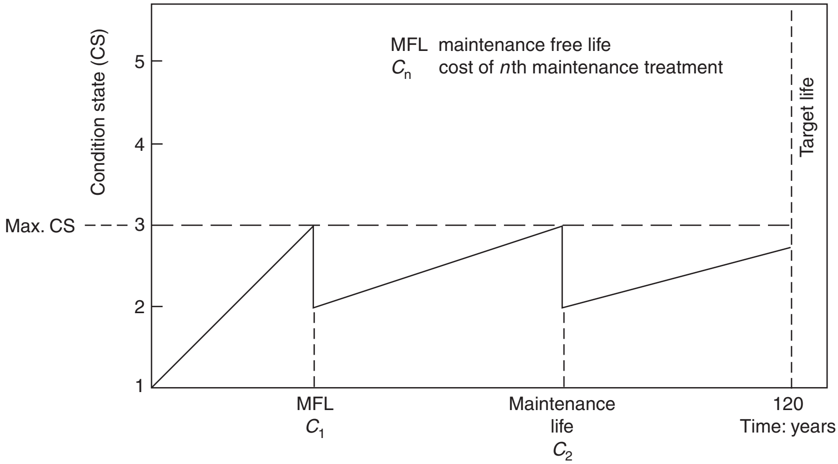

Fig. 1. Whole life costing process

years) was reached. All the costs for a particular strategy were discounted at a rate of 6% per year based on the age of the bridge when the maintenance operation was completed. The individual discounted maintenance and traffic delay costs were finally summed to give the whole life cost. This procedure is summarised in Fig. 1.

Traffic delay costs

There are two schools of thought about whether traffic delay costs should be included in a whole life cost analysis. Traffic delay costs are not of direct relevance to the bridge owner who funds the maintenance work since the delay costs are dispersed among all the users and do not, therefore, act as a cost to the owner, unless tolls are the source of income. However, the delay costs associated with some types of maintenance on a busy road can be substantial and to ignore them would not provide a satisfactory appraisal of the value for money. On balance it is better to include traffic delay costs, but to record them separately from the engineering costs.

Traffic delay costs are particularly sensitive to the value of the traffic flow rate. Maintenance operations take place at different times and the flow rate will vary with time due to the effects of traffic growth, therefore the delay cost will vary depending on the age of the bridge when the maintenance option is undertaken. As a consequence it is important to take account of traffic growth when calculating the delay cost associated with future maintenance operations.

Discounting

In this work two discounting procedures have been adopted. The first and most commonly used procedure is that used to calculate whole life costs where all future maintenance and delay costs are discounted back to the date of construction. In general the base date for discounting should be the date at which the investment decisions are made. Thus for a new bridge the decisions needed to calculate the whole life cost, such as the type of construction and the expected maintenance strategy, are made just before construction begins. For an existing bridge, however, the investment decision about which maintenance strategy to use is usually made some time after construction when deterioration is initially encountered. Thus, the second discounting procedure assumes that the decision about which strategy to use is made at an age of ten years when any of the three maintenance strategies (preventative, repair, strengthening) would be viable. The base date for discounting using this procedure would be ten years. Another important feature of this procedure is that different maintenance operations take place at different levels of deterioration; for example, in order to be effective preventative maintenance must be carried out at an earlier stage of deterioration than strengthening work. This approach is more realistic than the traditional method of applying maintenance at a fixed level of deterioration which is somewhat biased in favour of the preventative methods. For the second discounting procedure it has been assumed that the first preventative maintenance operation takes place at age 10, the first repair operation takes place at age 20 and the first strengthening operation takes place at age 30 years. Thus, the first preventative maintenance is not discounted, the first repair work is discounted by ten years and the first strengthening work is discounted by 20 years.

Results

Deterministic model

The deterministic model was run with fixed inputs for maintenance free life, life of maintenance treatment, construction and maintenance costs, initial traffic flow rate, traffic growth rate and discount rate. The whole life costs for a range of traffic flow rates and growth rates using the first discounting procedure are shown in Table 2. It can be seen that waterproofing of exposed concrete in decks and substructures has the lowest whole life cost at all values of initial traffic flow rate and growth rate. Cut out and repair has by far the highest whole life cost for all values of initial flow rate and growth rate.

*Table 2. Discounted whole life cost for different maintenance options (The costs are ranked in order of increasing cost)**

Initial flow rate (vpd) Traffic growth (per year)	30 000.00 0	WLC	30 000.00 0·01	WLC	30 000.00 0·02	WLC
Maintenance type	Construction type		Construction type		Construction type	
Waterproofing	Default construction	£346 926	Default construction	£428 690	Default construction	£580 885
Surface treatment	Default construction	£360 891	Default construction	£513 019	Default construction	£819 371
CPM	Default construction	£380 821	Default construction	£670 205	Default construction	£1 172 157
CPA	Default construction	£478 196	Default construction	£975 553	Default construction	£1 936 925
COR	Default construction	£680 912	Default construction	£3 841 662	Default construction	£9 626 411
Initial flow rate (vpd) Traffic growth (per year)	40 000.00 0	WLC	40 000.00 0·01	WLC	40 000.00 0·02	WLC
Maintenance type	Construction type		Construction type		Construction type	
Waterproofing	Default construction	£384 134	Default construction	£609 210	Default construction	£783 600
Surface treatment	Default construction	£472 963	Default construction	£989 695	Default construction	£1 427 654
CPM	Default construction	£502 819	Default construction	£1 258 252	Default construction	£1 827 917
CPA	Default construction	£863 397	Default construction	£2 564 828	Default construction	£4 014 675
COR	Default construction	£2 093 610	Default construction	£10 682 340	Default construction	£17 285 999
Initial flow rate (vpd) Traffic growth (per year)	70 000.00 0	WLC	70 000.00 0·01	WLC	70 000.00 0·02	WLC
Maintenance type	Construction type		Construction type		Construction type	
Waterproofing	Default construction	£777 729	Default construction	£957 761	Default construction	£973 374
Surface treatment	Default construction	£1 657 140	Default construction	£2 161 509	Default construction	£2 247 792
CPM	Default construction	£1 791 861	Default construction	£2 380 016	Default construction	£2 434 819
CPA	Default construction	£4 933 501	Default construction	£6 626 665	Default construction	£6 963 616
COR	Default construction	£17 020 415	Default construction	£23 840 805	Default construction	£24 465 714

* Deterministic model used
Discount Rate: 6%

Probabilistic model

Two versions of the probabilistic model were used. In the first version probability distributions were entered for the maintenance free life, the life of the maintenance treatment and the traffic growth rate; fixed values were entered for the construction and maintenance costs, the initial flow rate and the discount rate. Typical input distributions are shown in Figs 2 and 3. In the second version a further input distribution for initial traffic flow was used as shown in Fig. 4. The results of the first version are shown in Table 3. The whole life costs for a range of initial traffic flow rates are broken down into engineering maintenance costs and traffic delay costs. It can be seen that waterproofing has the lowest whole life cost and cut out and repair the highest whole life cost at each initial flow rate. The results of the second version are shown in Table 4 and the same

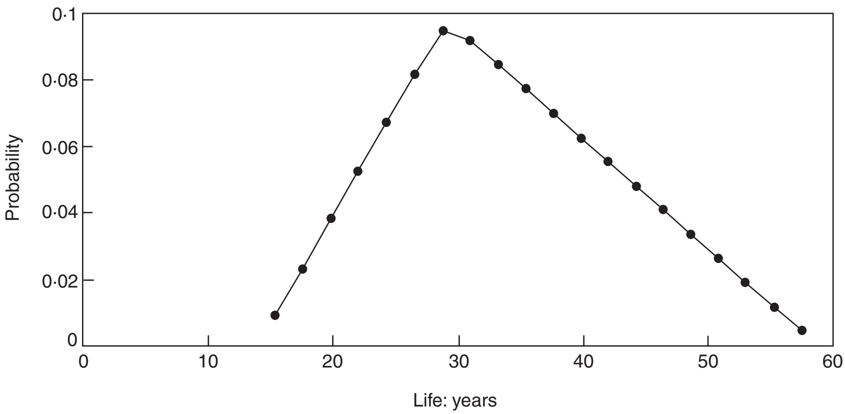

Fig. 2. Input maintenance life distribution for RC slab deck for CP minimum

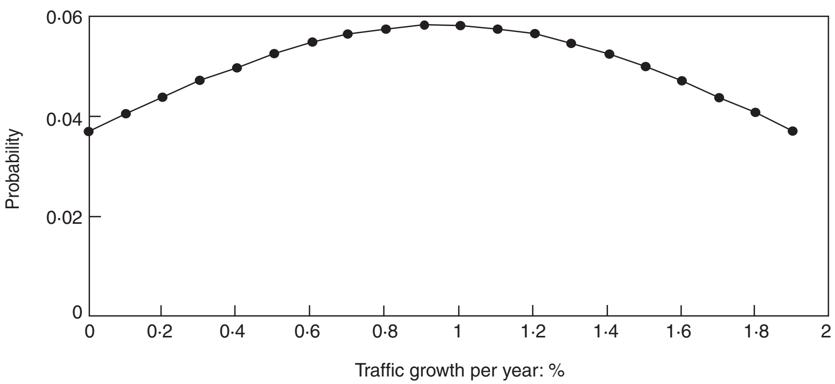

Fig. 3. Input distribution for traffic growth per year

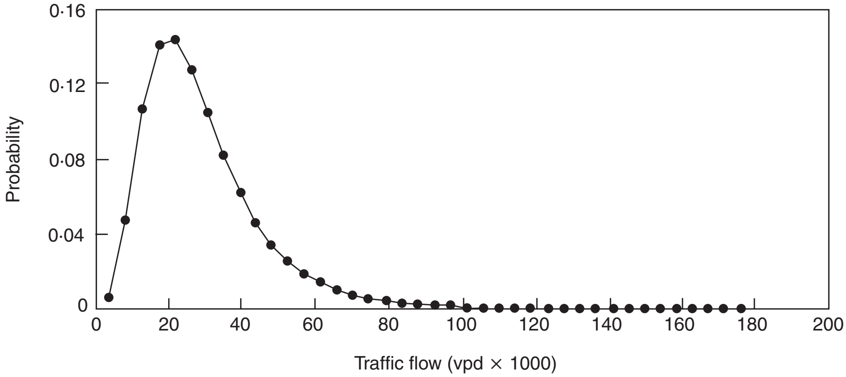

Fig. 4. Input probability distribution for initial traffic flow

*Table 3. Summary of costs for different maintenance options at different initial traffic flows using the probabilistic model, version 1**

Initial flow rate		30 000	40 000	70 000
Construction type	Maintenance type	WLC	WLC	WLC
Default construction	Waterproofing	£423 474	£555 967	£827 861
Default construction	Surface treatment	£540 237	£950 696	£1 958 511
Default construction	CPM	£672 088	£1 168 230	£2 201 891
Default construction	CPA	£1 004 136	£2 332 923	£5 726 205
Default construction	COR	£3 306 549	£7 968 898	£17 724 200
Construction type	Maintenance type	Total maintenace cost	Total maintenace cost	Total maintenace cost
Default construction	Waterproofing	£5 282	£5 264	£5 250
Default construction	Surface treatment	£10 361	£10 363	£10 358
Default construction	CPM	£27 910	£27 871	£27 846
Default construction	CPA	£85 092	£85 107	£85 101
Default construction	COR	£147 782	£147 457	£147 314
Construction type	Maintenance type	Total traffic delay cost	Total traffic delay cost	Total traffic delay cost
Default construction	Waterproofing	£81 838	£214 350	£486 257
Default construction	Surface treatment	£193 522	£603 979	£1 611 799
Default construction	CPM	£307 825	£804 005	£1 837 691
Default construction	CPA	£582 691	£1 911 462	£5 304 750
Default construction	COR	£2 822 413	£7 485 086	£17 240 532

* Risk parameters
Traffic growth: TNormal (0·01, 0·01, 0, 0·02)
Leakage life, spray life, maintenance life: Triang (default life × 0·5, default life, default life × 2
Discount rate: 6% per year

*Table 4. Summary of costs for different maintenance options using the probablistic model, version 2 (Results are given as the mean value of the distribution for WLC, total maintenance cost and traffic delay cost)**

	Total maintenance cost	
Construction type	Maintenance type	Mean
Default construction	Waterproofing	£5 254
Default construction	Surface treatment	£10 363
Default construction	CPM	£27 876
Default construction	CPA	£85 128
Default construction	COR	£147 705
	Total traffic delay cost	
Construction type	Maintenance type	Mean
Default construction	Waterproofing	£120 608
Default construction	Surface treatment	£352 837
Default construction	CPM	£454 893
Default construction	CPA	£1 127 867
Default construction	COR	£4 257 338
	WLC	
Construction type	Maintenance type	Mean
Default construction	Waterproofing	£462 216
Default construction	Surface treatment	£699 554
Default construction	CPM	£819 123
Default construction	CPA	£1 549 349
Default construction	COR	£4 741 398

* Risk parameters
Initial traffic flow: Lognorm2 (31142, 16388)
Traffic growth: TNormal (0·01, 0·01, 0, 0·02)
Leakage life, spray life, maintenance life, Triang (default life × 0·5, default life, default life × 2)
Discount rate: 6% per year

ranking of whole life costs was obtained as for the first version of the probabilistic model and the deterministic model. It should be noted that the values of whole life cost, maintenance cost and traffic delay cost are the mean values of the output probability density functions for these parameters. These output distributions for the CP min. maintenance option using the second version of the probabilistic model are shown in Figs 5 to 7 where it can be seen that they are all skewed towards higher costs indicating that there is a significant probability that the whole life cost could be greater than the mean.

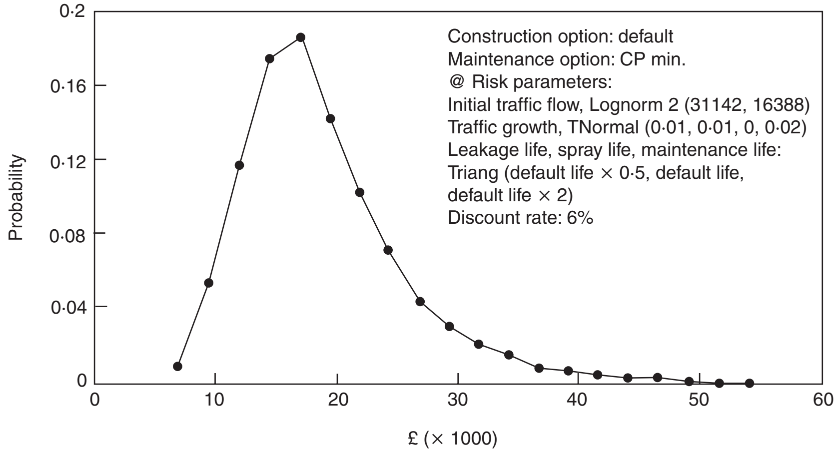

Fig. 5. Results of risk analysis expressed as probability function for maintenance cost

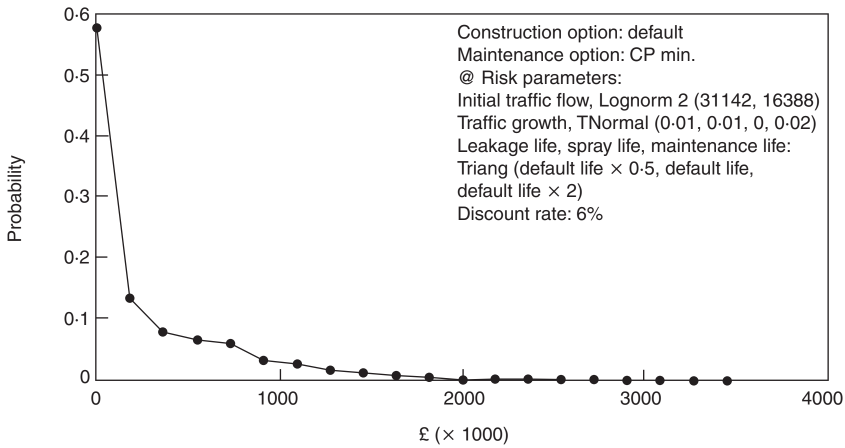

Fig. 6. Results of risk analysis expressed as a probability function for traffic delay cost

The results for each maintenance option using the second version of the probabilistic model are plotted together in Fig. 8 as cumulative frequency distributions. This figure shows graphically that, even after taking account of all the uncertainties in the values of the input parameters, waterproofing of substructures has the lowest whole life cost and cut out and repair has the highest whole life cost because the cumulative distribution for waterproofing lies to the left of all the other distributions whereas the cumulative distribution for cut out and repair lies to the right of all the other distributions.

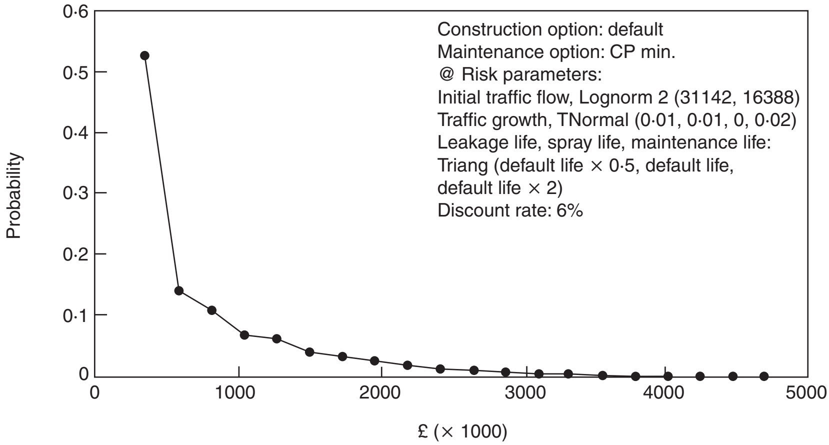

Fig. 7. Results of risk analysis expressed as a probability function for whole life costs

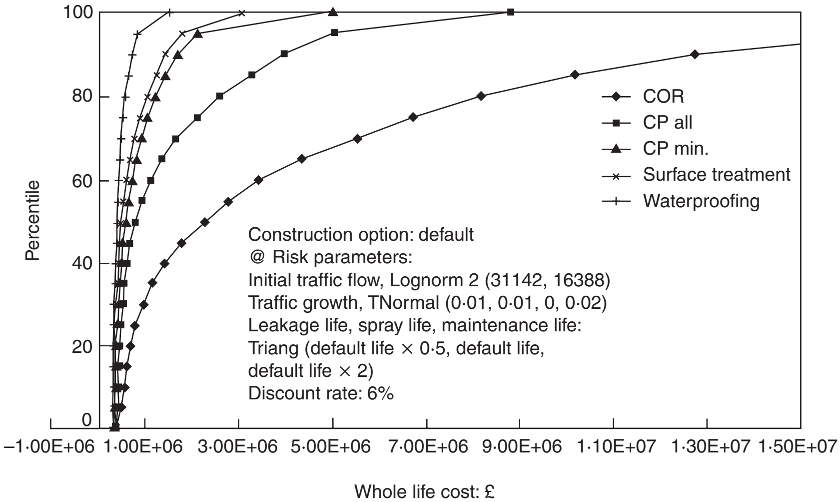

Fig. 8. Cumulative frequency distribution for different maintenance options

Deterministic model — second discounting procedure

The maintenance strategy details are shown in Table 5 and the results are shown in Table 6. The results show that the whole life cost values have the same ranking as those shown in Tables 2 to 4, although the whole life cost values are generally higher in Table 6 owing to the shorter period of discounting. It can also be seen that the whole life cost for the replacement option, although substantially less than that for the cut out and repair, is significantly greater than the values for the preventative maintenance options and cathodic protection.

Table 5. Maintenance strategies for calculating discounted future maintenance costs using the second discounting procedure

Option	1st Maintenance Age	L/S/A*	Super/Sub†	2nd Maintenance Age	L/S/A*	Super/Sub†	3rd Maintenance Age	L/S/A*	Super/Sub†
Waterproofing	10	L	Super + Sub	30	A	Super	50	A	Super + Sub
				30	L	Sub	50		
Surface treatment	10	A	Super + Sub	30	A	Super + Sub	50	A	Super + Sub
CP min.	20	L	Super + Sub	45	A	Super	75	A	Super
				45	L	Sub	75	S	Sub
CP all	20	A	Super + Sub	50	A	Super + Sub	80	A	Super + Sub
Cut out and repair (COR)	20	L, 20%	Super + Sub	40	A, 20%	Super	60	A, 50%	Super
				40	L, 50%	Sub	60	A, 20%	Sub
Replacement	–	–	–	30	A	Deck + Pier	–	–	–
Surface treatment	10	A	Abutment	30	A	Abutment	50	A	Super + Sub

* L/S/A: Leakage area/spray area/exposed area
† Super/sub superstructure (deck)/substructure (piers and abutments)

Table 6. Future maintenance costs for different maintenance options using the second discounting procedure

Option	Maintenance cost (£)	Traffic delay cost (£)	Future lifetime cost (£)
Waterproofing	26 580	565 070	591 650
Surface Treatment	28 214	643 144	671 358
CP min.	40 010	963 665	1 003 675
CP all	126 431	1 599 704	1 726 135
Cut out and repair	316 124	15 896 843	16 212 967
Replacement/Surface Treatment	79 392	4 901 020	4 990 412

Traffic flow rate at age = 0 is 30 000 vpd

Conclusions

The results clearly indicate that the preventative maintenance options provide the best value for money. Although they are applied initially at a younger age and thereafter more frequently than cathodic protection and replacement of elements, the preventative maintenance options cost less to carry out and more significantly cause less traffic disruption. The greater age at which replacement of elements is undertaken leads to a larger reduction in cost due to discounting although the effect of discounting on the traffic delay cost is counterbalanced by the increased traffic flows caused by traffic growth. The traffic delay cost is the major component of the whole life cost for most maintenance options except at the lowest traffic flow rates.

References

1. Department of Transport (1982). *Departmental Publication QUADRO 2 manual*.
2. Highways Agency (1995). *Trunk road maintenance manual*, Vol. 1, HMSO, London.