DELIVERABLE D14
FINAL REPORT

BRIME
Contract No.: RO-97-SC.2220

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Project Duration: 01 January 1998 to 31 December 1999

Date: March 2001

PROJECT FUNDED BY THE EUROPEAN COMMISSION UNDER THE TRANSPORT RTD PROGRAMME OF THE 4th FRAMEWORK PROGRAMME

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Date	30/03/01
Version	4
Classification	P
Distribution	Publicly available

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EXECUTIVE SUMMARY

The report describes a project to develop a framework for a bridge management system (BMS) for the European road network that would enable the bridge stock to be managed on a rational basis and enable bridge maintenance to be optimised taking account of all factors affecting bridge management. These include: condition of the structure, load carrying capacity, rate of deterioration, effect on traffic, life of repairs and the residual life of the structure. The project, known as BRIME (Bridge Management in Europe), was undertaken by the national highway research laboratories in the United Kingdom, France, Germany, Norway, Slovenia and Spain. It was 50% funded by the European Commission Directorate General for Transport, with balancing funds provided by the authorities responsible for the national road networks in the participating countries. Further details concerning the project are given in the other project deliverables and a brief overview is given on the TRL web-site (http://www.trl.co.uk/brime/index.htm).

Over the last 50 years there have been major road building programmes in Western Europe to cope with the increasing growth in traffic. However in most countries the main motorway construction programme is now coming to an end and attention is switching to maintenance of the existing stock. Many of the bridges built in these road programmes are showing signs of deterioration after only a few decades in service, as a result of the increasing volume of traffic and increases in the weights of individual vehicles. Deterioration is exacerbated because many modern structures are more prone to chemical degradation than their forerunners. The effects of alkali silica reaction, chloride ingress and carbonation are worsened by low cover and poor quality materials, and are causing progressive deterioration of the bridge stock. As bridges age, deterioration caused by heavy traffic and an aggressive environment becomes increasingly significant resulting in a higher frequency of repairs and possibly a reduced load carrying capacity. By contrast highway structures in Eastern Europe tend to be older, have been neglected and are in need of major rehabilitation.

In both cases a systematic approach to maintenance is required to ensure that structures remain safe and serviceable. Numerous management systems have been developed throughout Europe to assist engineers in deciding what maintenance is required and when it should be carried out. The simplest systems consist of a database that holds all the relevant information on each structure – for example structural details, records of inspections, previous maintenance history – that the engineer needs to reach a decision. More complex systems contain algorithms that manipulate the data to produce optimum maintenance strategies at both project level and network level, taking into account constraints such as inadequate funding.

Most systems have been developed either for a particular stock of bridges, or to meet the requirements of a particular bridge owner and then developed further to meet the needs of a wider market. The project was aimed at developing a framework for a system that specifically met the needs of the European Road Network.

Work Programme

The research was undertaken in seven workpackages, six of which focused on the modules required to create a bridge management system. Each of these six workpackages was split into two stages; the first stage involved a review of the state of the art and an identification of the requirements for a BMS. The second stage involved developing guidelines for the various

modules for the system. Under the seventh workpackage, existing systems were reviewed and the findings, together with the results from other workpackages, were used to develop the framework for a BMS.

The first workpackage comprised a review of current methods used in Europe and North America for inspecting and assessing bridge condition. Three basic types of bridge inspection were identified: superficial, general, and major. A fourth type of inspection – an in-depth or special inspection – is carried out on structures where there is a particular problem or cause for concern either found during an inspection or already discovered on other similar bridges. These inspections are also carried out for a variety of other reasons, for example on bridge foundations after flooding, and on structures after earthquakes. They involve extensive measurements on site and may include laboratory testing. Other types of inspection such as an ‘acceptance inspection’, which is carried out before a structure is opened to traffic and a ‘guarantee inspection’ are used in some countries.

The results of an inspection are used to provide a measure of the structure’s condition. Two approaches have been used. The first is based on a cumulative condition rating obtained from a weighted sum of all the assessments of the condition of each element. The second gives the assessed condition of the bridge as the highest condition rating of the bridge elements.

Artificial intelligence methods were investigated as a means of improving condition assessment and a review was undertaken of neural network, fuzzy logic and genetic algorithms. A neural network model was then developed for categorising the condition of corroded areas on reinforced concrete bridges. Whilst this gave promising results it could only give the condition at the time the measurement was made: further work is required to evaluate the change in condition with time.

The second workpackage developed recommendations for methods to assess the load-carrying capacity of highway bridge structures. These methods were based on a review of current assessment procedures used in the countries participating in BRIME. This included details of the characteristics of existing structures, the standards used in design and assessment, and experimental methods used in the assessment of bridge structures. The purpose of this work was to illustrate how assumptions for material and structural properties, and traffic loads can be obtained and used in structural assessment.

To assess whether structural elements are capable of carrying modern day traffic loads, models for both element resistance and applied loads are required. Load models that take account of the extreme traffic loads applied to structures were developed. Material strengths were obtained from statistical data for reinforced and prestressed concrete, steel, masonry and timber structures.

Bridge assessment in the partner countries is based either on a deterministic or a semi-probabilistic (ie with the use of partial safety factors) approach. In both cases the load effects are determined by structural analysis, using design standards that can be amended to take account of information from measurements on the structure. These methods are sometimes considered to be conservative, and a new approach using reliability methods to take account of uncertainties in variables is emerging. Reliability calculations are beginning to be introduced, with the target reliability index becoming the governing factor for assessment.

The workpackage partners recommendation is based on assessment procedures adopted in the UK in which several levels of assessment of increasing sophistication are available. The first level requires a site inspection and uses a basic load model, codified resistance models and simple analysis to produce a conservative assessment. If this does not prove the structure satisfactory, the analysis and site data are refined eventually working up to a full reliability analysis in a minority of cases. The load model and codified resistance models have been developed specifically for assessment avoiding over-conservative requirements. Without this provision, some existing structures would unnecessarily be assessed as unsafe. The implication is that countries cannot adopt the recommended assessment procedures without transitional arrangements, because it will take time to provide a realistic capacity for all bridges on which to base the management of funding. In the UK, a programme of assessment and rehabilitation has been undertaken in which some bridges were assessed as unsatisfactory and were strengthened or replaced.

The objective of the third workpackage was to quantify the structural effects of material deterioration so that they could be incorporated into the assessment of load carrying capacity of bridges. The common forms of deterioration in European bridges and their respective causes were investigated – results showed that corrosion of steel due to carbonation and chloride contamination is the most frequently occurring problem, although deterioration due to alkali-silica reaction, sulfate attack and freeze-thaw action are also common.

Existing methods of dealing with deterioration in assessment (eg reduced cross-sectional area, modified stress-strain relationship, and modified bond properties) were evaluated. Guidelines for taking account of deterioration were produced but these models are based on experiments using laboratory specimens, and calibration with site measurements is required. Such measurements need to be carried out over a long period of time to give realistic results. This is also complicated by the fact that the deterioration and its affect on load carrying capacity is not linear with time and the actual rate will depend on site specific conditions.

The fourth workpackage was concerned with predicting the rate of deterioration for the various processes, for which there are currently two approaches. The first uses historical data to predict future performance while the second attempts to model the various deterioration processes. This is an enormous subject, so the BRIME project focussed on modelling the ingress of chlorides into concrete, which can be used to provide data for forecasting maintenance actions, but not for assessing deterioration and structural capacity.

Further research is needed to develop models to predict the initiation and rate of corrosion once the chloride ions reach the reinforcement. Similar models are also required for other forms of deterioration, so that the future condition of the structure can be predicted and input into the assessment calculations. The development of deterioration models and their use to determine load carrying capacity of concrete structures in general is being carried out under a separate European project – known as CONTECVET – which will produce a validated users' manual for assessing the residual life of concrete structures.

Under the fifth workpackage, the objective was to develop a methodology for selecting the best maintenance option for a given bridge, considering safety, durability, functionality and socio-economic issues. A method was developed which is based on a global cost analysis which took account of all the costs involved in construction, inspection, maintenance, repair, failure, road usage and replacement. The method minimised the global cost while keeping the lifetime reliability of the bridge above a minimum allowable value.

This was taken further in workpackage six with the development of methods for determining an optimum maintenance strategy. This workpackage included a review of current methods for prioritisation and optimisation of bridge maintenance at both project and network level. Simple procedures were developed for selecting bridge structures for inclusion in the maintenance programme and for ranking bridges with respect to the impact of their location in the road network. Costs that had to be taken into account when examining different maintenance strategies were identified. The need to model deterioration rates, to enable future deterioration of structural elements to be predicted when the maintenance work is deferred, was identified.

The seventh workpackage was to produce a framework for a bridge management system. A review was undertaken of the requirements for a bridge management system for the European Road Network. This was then used in conjunction with the results from workpackages 1 to 6 to produce a framework for a bridge management system that would operate at both project level and network level.

Project level information is related to individual bridges, elements or components and is important for specifying the maintenance requirements and retrieving data about particular bridges. Network level information relates to the entire bridge stock or to subsets of the stock, such as all the bridges in a given region. Network level information is important for determining whether the average condition of bridges in the stock is improving or deteriorating and for estimating the value of the budget needed in order to maintain the condition of the network at an acceptable level.

As an example of how the findings were combined, the assessment of bridge strength is required for the evaluation of maintenance strategies and for the decision making process. It is also a significant input for priority ranking, for the routing of abnormal vehicles and for the management of safety measures. The philosophy adopted in the development of the framework was to identify the outputs required by the engineer and then determine the inputs required to produce those outputs.

Conclusions

This project has shown how results from the main bridge management activities such as inspections, assessments, testing, maintenance, prioritisation and replacement, described in Chapters 3 to 8 of this report, can be combined to produce a framework for a computerised bridge management system, that will provide both project and network level information. The types of project level information generated include:

- measures of the condition of each structural element and component of a bridge and for the complete bridge
- the load carrying capacity of a bridge and its most structurally vulnerable parts
- the current extent and rate of deterioration of elements and components of a bridge enabling their future condition to be predicted
- predictions of when a bridge will become substandard in terms of the load carrying capacity
- identification of the maintenance requirements of a bridge
- guidance on effective maintenance strategies and methods
- programmes of maintenance work indicating the timing of specified maintenance methods needed in order to minimise the whole life cost of a bridge.

The types of network level information generated include:

- prioritised programmes of maintenance when the optimisation of the programme is constrained by factors such as a maintenance budget that is insufficient to enable all the work in the optimal programme to be carried out
- values of policy target parameters such as (a) the number of bridges with load restrictions at a given date, (b) the number of bridge replacements each year and (c) the average condition of bridges in the stock at a given date
- degree of compliance of measured policy target parameters with set benchmark values
- size of maintenance budget needed to achieve a specified degree of compliance.

Whilst the system was developed for the European road network, it could also be applied to national and local road networks.

Ultimately it should be possible to combine management systems for pavements, earthworks, highway structures (eg bridges, culverts, retaining walls and tunnels) and street furniture to achieve a route management system.

The results from this project will be of interest to organisations responsible for the management of bridges at both network and local level, national railway authorities, and owners of other infrastructure such as waterways. Other organisations such as consultants employed to assess the load carrying capacity of bridges and test houses responsible for determining structural condition will also benefit from the outputs of workpackages 1, 2, and 3.

Future developments are likely to include further application of the use of artificial intelligence methods for various aspects of bridge management and increased use of reliability techniques. Life cycle assessment is also likely to be used to minimise the environmental impacts of bridge management. Finally, the management of the highway network as a whole will mean that bridge management will become a part of a much larger asset management system that ensures that society gets maximum benefit from its investment in the highway infrastructure.

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

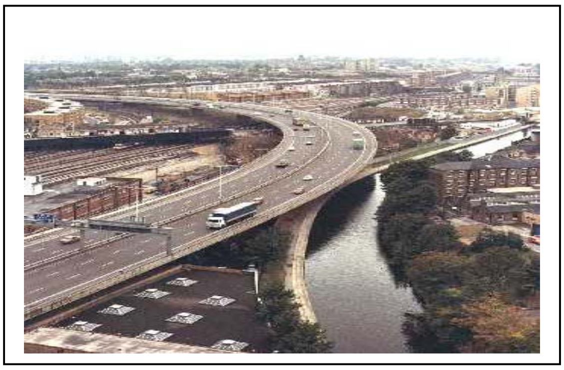
The transport network is extremely important to Europe's economic and social development. It has been a crucial factor in economic growth and prosperity and it plays an important role in the everyday life of the citizens of Europe, by allowing the quick, easy and safe movement of people and goods. It has been estimated that the movement of goods and people around the European Union costs 500 billion Euros per annum which is about 15 percent of the income of all European citizens. Most of this mobility is provided by the road infrastructure because of its high quality of service and flexibility. The high usage is causing congestion the costs of which are currently estimated at 120 billion Euros annually. This is likely to increase as road traffic is increasing at a rate of four to five per cent each year, leading to an expected growth from now to the year 2020 of fifty to sixty percent.

The capital investment in the road network is enormous and bridges are the most vulnerable and expensive element. The value of bridges on the national networks of the countries participating in the BRIME project is estimated at 12 billion Euros in France, 23 billion Euros in the UK, 4.1 billion Euros in Spain and 30 billion Euros for Germany. Typically they comprise about 2% of its length and about 30 % of its value [PIARC, 1996]. They allow roads to cross rivers, estuaries, canals, railways, valleys and other obstacles, both man made and natural, improve traffic flow at intersections and provide access to remote communities (Figures 1.1 and 1.2).

Most bridges on the national road networks in the European Union have been built during the last 50 years although some are much older. However the increasing volume of traffic and maximum weights of individual vehicles mean that for many structures the loads to which bridges are being subjected are far higher than those envisaged when they were designed (Figure 1.3). Deterioration is exacerbated because many modern structures are subject to a more aggressive environment than their forerunners. The effects of chlorides, either in a marine environment or from de-icing salts, alkali silica reaction, carbonation and inadequate corrosion protection are causing progressive deterioration of the bridge stock (Figure 1.4). This is resulting in a higher frequency of repairs and possibly a reduced load carrying capacity.

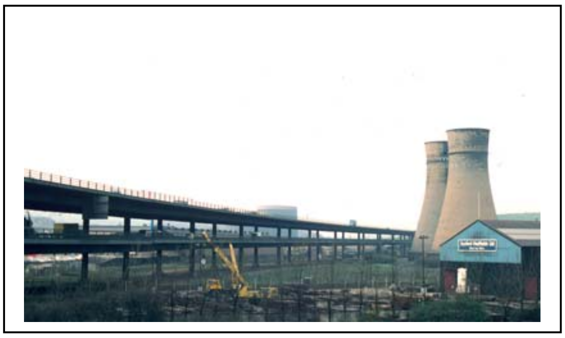
Maintaining structures in a serviceable condition is complicated by the wide variety of structural types. Whilst the majority of modern structures are of reinforced or prestressed concrete construction, there are also a large number of composite bridges with steel beams supporting a concrete deck and a smaller number of steel bridges. The majority of the older structures are of masonry arch construction (Figure 1.5). Each type of structure behaves differently, suffers from different types of deterioration and has different maintenance needs. All this adds to the difficulty of ensuring that bridges are properly maintained.

River Soèa Bridges, Slovenia **Estuary Crossing, Normandy, France**

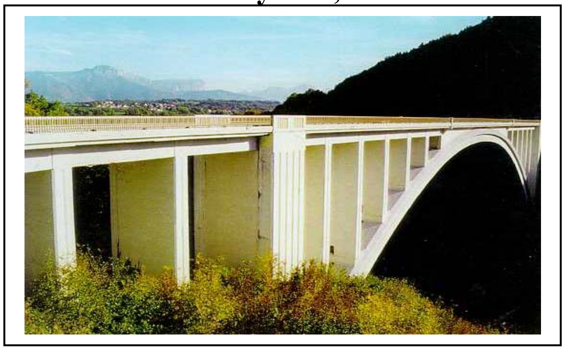


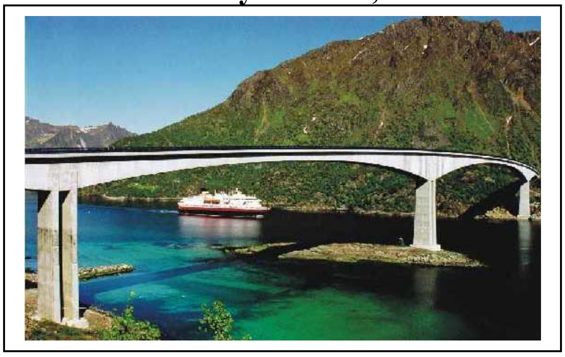
Grade separated junction, UK **Elevated motorway, UK**

Figure 1.1: Functions of a bridge (river crossing, estuary crossing, grade separated junction and elevated motorway



Urban Flyover,UK **Tinsley Viaduct, UK**

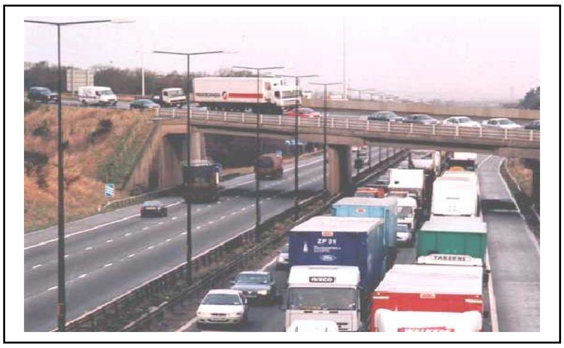




Pont de la Caille sur le ravin des Usses, France **Raftsundet Bridge, Norway**

Figure 1.2: Bridges in different environments (urban, industrial, rural, mountainous)

Abnormal load - UK



Congested traffic - UK

Heavy goods vehicle - Germany

Heavy traffic, Aquitaine bridge - France

Figure 1.3: Examples of heavy loads and heavily trafficked bridges

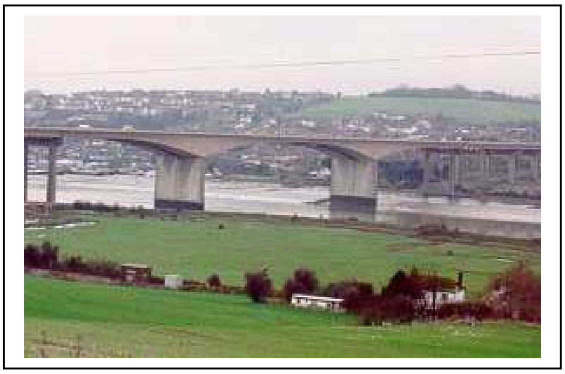
Corrosion of prestressing tendons

Reinforcement corrosion

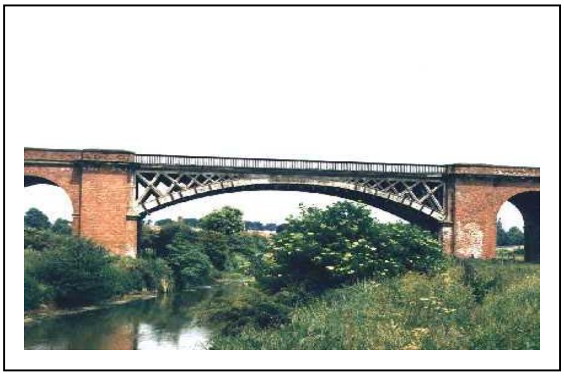
Alkali silica reaction

Corrosion of steel beam

Figure 1.4: Examples of deterioration

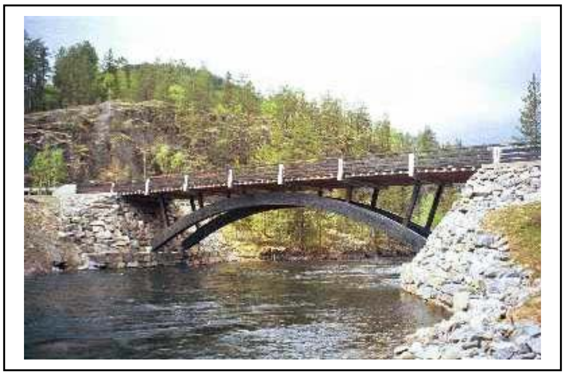


Concrete



Steel

Masonry arch



Timber

Figure 1.5: Different types of bridge

The direct cost of the engineering work necessary to maintain a satisfactory road network is high. The annual expenditure on maintenance and repair on national bridges in England is of the order of 180 million Euros, in France the figure is 50 million Euros, in Norway 30 million Euros and in Spain 13 million Euros. Furthermore national bridges only represent a small proportion of the total population of bridges. It is about 10% in England and France, and about half in Norway. These costs are likely to increase as the large number of bridges built during the 1960s and 1970s begin to deteriorate. In addition the traffic congestion and disruption that result from repair work carry a severe economic penalty particularly on the increasing number of roads where traffic flows are reaching saturation.

1.2 FUNCTIONS OF A BRIDGE

Bridges are designed to carry traffic across an obstacle (Figure 1.1). The minimum length that has to be traversed before the structure can be classified as a bridge varies between countries but generally it is around 2m although in some countries it may be higher, for example in Slovenia it is 5m. In performing their function they must resist loads from a number of sources including:

- the weight of the structure plus any superimposed dead load such as the road surfacing
- traffic
- horizontal forces due to braking

- impact
- wind
- scour
- temperature
- earthquake
- settlement.

Bridges must also be durable. They are often situated in harsh environments with severe local micro-climates and must resist the action of the environment on the structure. As indicated above these severe conditions are taking their toll and many structures are beginning to deteriorate. Whilst some deterioration is inevitable bridges are required to remain serviceable throughout their service life. This means that deflections, deformations and cracking must remain within acceptable limits both from the point of view of appearance and safety.

During their service life bridges are likely to suffer a loss of strength as a result of structural damage or material degradation and at some point, a minimum acceptable level of performance is reached and this defines the end of the service life. The minimum acceptable level of performance is not easily defined and needs to take account of a number of factors including the consequences of failure in terms of both costs and potential loss of life.

Bridges therefore need an effective maintenance strategy to increase their service life at minimum cost. For example, routine maintenance such as painting, cleaning and minor cosmetic repairs can be used to slow down the rate of deterioration. Repair or rehabilitation work can be used to eliminate the source of the deterioration or to restore lost capacity. However individual structures cannot be treated in isolation and the maintenance strategy needs to take account of other factors such as current and future usage of the bridge, its position on the network and the environmental, social or political impact of any maintenance works.

There is therefore a need for a set of rational criteria that ensure that bridges are maintained in a safe and serviceable condition, with the required load carrying capacity. This must be done throughout their design life at a minimum life-time cost whilst causing the least possible disruption to traffic.

1.3 BRIDGE MANAGEMENT SYSTEMS (BMS)

Bridge management addresses all activities throughout the life of a bridge from design and construction to replacement and is aimed at ensuring their safety and functionality. The OECD report on bridge management defines a bridge management system as a tool for assisting “highway and bridge agencies in their choice of optimum improvements to the bridge network that are consistent with the agency’s policies, long-term objectives, and budgetary constraints.”

This requires procedures that ensure that bridges are regularly inspected and assessed, and that appropriate maintenance is carried out to achieve a required standard of condition throughout their service life. To do this efficiently and effectively essential information is needed in a readily accessible form. In the past this has been done using manual filing systems. These are acceptable but have a number of disadvantages, eg, data is less accessible and secure, data processing is more difficult, trends are less easy to detect, and the interconnections between different components of the transport network are less apparent and cannot be quantified. The growth of the bridge stock and the advent of personal computers have led to the development of automated systems for managing the maintenance of bridges.

Initially BMSs were little more than computerised inventories of basic bridge information such as age, owner, etc. They were then developed to include the scheduling of inspections and storage of data arising from the inspections, and remedial work. Subsequently procedures for the prioritising maintenance on a network of bridges were introduced so that those bridges most urgently in need of remedial treatment were repaired first.

Over the last few years as the number of bridges requiring maintenance has increased there have been reductions in public expenditure. This has meant that it has become essential to appraise the maintenance of bridges in economic terms. Economic appraisals are usually made by comparing the costs and benefits of proposed maintenance work. The use of personal computers makes the prioritisation of maintenance based on a cost-benefit analysis feasible.

The analysis of the costs and benefits of alternative maintenance procedures highlights the need to quantify such factors as the cost of traffic delays, the deterioration rate of bridges, the effective life of repair systems, the time value of money and the benefits accruing from improvements such as bridge widening. When the quantification of items such as these has been achieved it is possible to put forward a programme of maintenance optimised to achieve a set standard condition at minimum long-term cost. If this optimised maintenance programme costs more than the annual budget for maintenance work, the BMS would re-analyse the problem incorporating the budget constraint and produce a revised maintenance programme. The increase in long-term costs and the divergence from the target condition standard of the bridge stock resulting from the budget constraint would also be evaluated by the BMS.

1.4 FACTORS AFFECTING BRIDGE MANAGEMENT

The most appropriate maintenance strategy for a stock of bridges is a complex subject and there are a wide range of issues that determine the most economic approach. These include:

- condition of the structure
- load carrying capacity
- rate of deterioration
- maintenance treatments available and their effectiveness, lifetime and cost
- traffic management costs
- traffic flow rates and the associated delay costs
- cost of working in the future discounted to present day values
- implications for safety and traffic flow if the work is not carried out immediately.

A schematic diagram showing the basic building blocks of a BMS is shown in Figure 1.6. The inputs are the information required to determine optimised maintenance programmes (eg condition, load carrying capacity) and the outputs provide the basis for developing optimum maintenance programmes within the available budget (eg deterioration rates, cost of maintenance options). Each of the elements shown in the figure represents the modules that make up a BMS.

Inventory

Condition assessment

Structural assessment

Comparison of maintenance options

Optimal maintenance programme

Prioritised maintenance programme

Figure 1.6: Schematic diagram for a bridge management system

BMSs are being developed in several European countries, but some of the data needed to provide models for optimisation are not readily available. Further information is required on techniques for assessing the condition and load carrying capacity of a structure. These assessments require the effects of deterioration, strategies for repairing or replacing structures, models for deterioration and methodologies for prioritising maintenance, repair and/or replacement to be taken into account.

1.5 ECOMONIC AND SOCIAL IMPACTS

Proper management that ensures that maintenance work is carried out at the optimum time will keep costs down and avoid the build up of backlogs of work and the consequent disruption to traffic. The provision of a well managed European highway network that provides free access across Europe with a minimum of disruption will assist the carriage of freight and encourage trade between member states. It will also assist communications between all areas of the EC and help the economic development of the poorer regions.

Good communications also bring social benefits in terms of improved access for individuals between countries. Planning bridge maintenance to minimise traffic disruption reduces the pollution generated by long queues of traffic.

The storage of maintenance information can be used to provide feedback on designs, materials, components, construction practices, Quality Assurance and Quality Control procedures and maintenance strategies, and their effectiveness in different environments. For example, the knowledge gained from improved methods for determining the condition of a structure could help provide a better understanding of deterioration mechanisms and be used to improve standards for the design of new structures. Advantage can be taken of these lessons when the time comes to renew or repair components and would result in more durable repairs and the construction of more durable bridges. This would be especially beneficial for the less developed parts of Europe where the road network is still expanding. This would bring the economic benefits of lower maintenance costs and less disruption to traffic.

At present decisions on when to maintain or whether to repair or replace a structure are largely based on technical factors and availability of funds. What is required is a rational procedure that takes account of all relevant parameters both local and global and this would include historical, social and environmental factors.

Getting the best use out of existing bridges and ensuring that they are properly maintained means that structures are not replaced unnecessarily. This reduces demands on the scarce resources needed for the construction of new structures.

1.6 DEVELOPMENT OF A FRAMEWORK FOR A BMS.

To address the problems described above a project was set up under the fourth Framework to develop the tools required for a BMS for the European road network. The organisations participating in the project were the national highway research laboratories in France, Germany, Norway, Slovenia, Spain and the UK. The partners represented countries from all parts of Western Europe (Figure 1.7, Appendix I).

The objective of the project was to develop a framework for a BMS for the European highway network that can be used at both the project and network levels. Project level information is related to individual bridges, elements or components. It is important for specifying the maintenance requirements and retrieving data about particular bridges. Network level information relates to the entire bridge stock or to subsets of the stock such as all the bridges in a given region. Network level information is important for determining whether the average condition of bridges in the stock is improving or deteriorating and for estimating the value of the budget needed in order to maintain the condition of the network at an acceptable level. To evaluate the effectiveness of a bridge maintenance programme, it is necessary for the BMS to have in-built targets related to benchmark values. These could be for the average condition of the stock, the replacement rate for bridges, the percentage of the stock with traffic restrictions and the disruption to users arising from traffic restrictions at different times. An assessment of how closely such targets are met will establish the sufficiency of the budget and the consequences associated with particular budget levels.

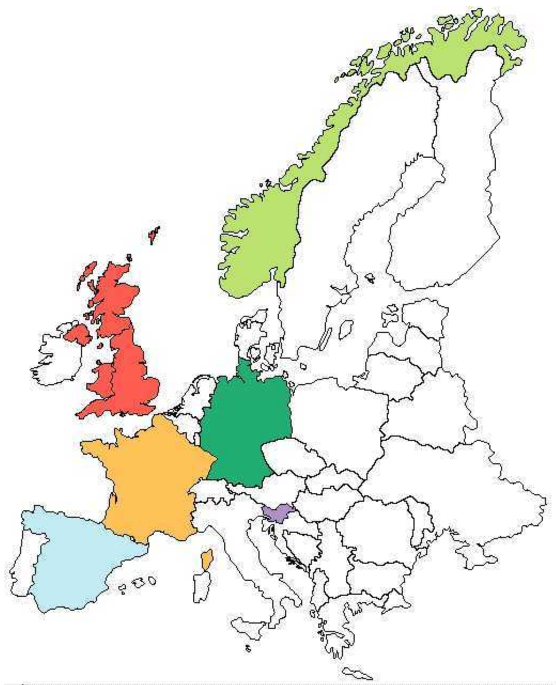


Figure 1.7: Map showing the participating countries

The research was undertaken in seven workpackages, six of which focused on the modules required for the BMS. Each of these six workpackages was split into two stages. The first stage comprised a review of the state of the art and an identification of the requirements for a BMS. The second stage was for the work required to develop detailed guidelines for the various modules for the system. The seventh workpackage comprised a review of existing bridge management systems and the findings were used to develop the framework for a BMS. The seven workpackages were as follows:

Workpackage 1:	Classification of the condition of a structure.
Workpackage 2:	Assessing the load carrying capacity of existing bridges, including the use of risk based methods.
Workpackage 3:	Modelling of deteriorated structures and effect of deterioration on load carrying capacity.
Workpackage 4:	Modelling of deterioration rates of corroding structures.
Workpackage 5:	Deciding whether a sub-standard or deteriorated structure should be repaired, strengthened or replaced.
Workpackage 6:	Prioritising bridges in terms of their need for repair, rehabilitation or improvement.
Workpackage 7:	Review of systems for bridge management and development of a framework for a bridge management system.

In addition to the production of a framework for a BMS the outputs from this research include guidelines for recognising susceptibility to the various forms of deterioration and inspection techniques to identify and quantify deterioration. Methods of taking account of deterioration in the assessment of existing bridges, deterioration rates for different types of deterioration, strategies for repair and replacement of bridges and a method for prioritising maintenance, repair and/or replacement needs have also been established.

The work undertaken to develop the modules required for a BMS is described in the following chapters. Chapter 2 describes the requirements for a BMS and the remaining chapters cover each module in turn that broadly corresponds to one of the workpackages. They also outline any further work that is required in order to complete the module. A more detailed description of the work is described in the deliverables that were produced during the project. These are listed in Appendix II.

1.7 BENEFITS

The major target audiences for the outputs from this project are bridge owners across Europe. These include:

- national organisations responsible for management of national bridges
- local authorities responsible for local roads
- national railway authorities
- other infrastructure owners eg waterways.

The results will both assist these organisations in the day to day management of bridges and help strategically in formulation of policy.

Other organisations such as consultants who are employed to assess the load carrying capacity of bridges and test houses responsible for determining structural condition will also benefit from the outputs of Workpackages 1, 2 and 3.

Finally the results will add to scientific knowledge and be of interest to scientists and researchers in organisations such as the Forum of European Highway Research Laboratories, other research establishments and universities who may be carrying out research on related topics.

1.8 REFERENCES

AASHTO, 1993. *Guidelines for Bridge Management Systems*. American Association of State Highway and Transportation Officials. Washington D C, USA.

OECD Road Research, 1992. *Bridge Management.* Paris.

PIARC Committee on Road Bridges, 1996. *Towards an Indicator of the Health Condition of Bridges.* World Road Association.

PIARC Committee on Road Bridges, 1999. *Reliability-Based Assessment of Highway Bridges.* World Road Association.

CHAPTER 2

CONTEXT AND PRINCIPLES OF A BMS

2.1 INTRODUCTION

Bridges located on major road networks have been subjected to a substantial increase in the volume of traffic (in particular heavy goods vehicles) and to an increase in the number of the vehicles exceeding the authorised maximum weight limit. In addition there are the indirect effects of traffic growth such as the increase in the number of vehicular impacts with supports and decks. Moreover, in many countries, the intensive use of deicing salt reduces the durability of the structures by causing corrosion of reinforcing steel and spalling of concrete.

The management of structures thus becomes an important issue for the economy of all countries. It requires the design of a bridge management system (BMS) which has the following principal objectives, classified in order of importance:

- to guarantee the safety of the users and third parties
- to ensure a targeted level of service (variable according to the routes)
- to ensure the conservation of the heritage in the long-term.

A management system comprises a set of procedures intended to ensure adequate maintenance of all structures. It includes the methods, analytical models, data-processing tools, organisational processes, and databases necessary for its implementation. As discussed in Chapter 1, it generally acts on two levels. The project level which is primarily concerned with the technical management of individual structures and the network level which is primarily concerned with the management of a stock of bridges and where there is more emphasis on economics and political management. Strong interactions exist between these two levels of management

2.2 CONTEXT OF THE BRIDGE MANAGEMENT SYSTEM

The following important aspects need to be examined before setting up a bridge maintenance management system:

- A stock of bridges is in general a collection of individual and distinct objects although it is possible to distinguish some families of structures, such as motorway bridges. This diversity poses problems for management related to the difficulty in deducing general laws of deterioration based on observations on particular bridges. The heterogeneity of the behaviour of bridges partly explains the difficulty in formulating general laws for the rate of deterioration.

- A BMS must take account of site specific aspects because different individual and types of bridge may require different levels of management. For example families of bridges having a good robustness may require less inspection, testing and maintenance while some exceptional structures, such as large suspension bridges, may need to be managed independently of the remainder of the stock with their own specific management plan.

- The required life of a bridge plays a critical role in its management. Normally a life in excess of 100 years is required and most bridge stocks now contain an increasing proportion of older bridges. The management of bridges thus covers several human generations, and this fact unquestionably poses problems for the continuity of socio-economic approaches with time.

- A BMS is dependent on the administrative organisation of the owner, and in particular on the distribution and the qualifications of personnel; this is particularly true for the policies adopted for the inspection and evaluation of bridge condition.

- Finally, certain bridges are classified as historic buildings or have special architectural value, necessitating a different management approach from other bridges in the stock.

In general, additional short term funding becomes necessary if the decision is taken to apply preventive maintenance in order to reduce life-time costs. Preventative maintenance must be designed to reduce or postpone the amount of essential maintenance in future years. For example, failure to replace a defective waterproofing membrane on a prestressed concrete bridge can result in corrosion of prestressing tendons (Figure 2.1).

Figure 2.1: Corrosion of a prestressing tendon

2.3 REVIEW OF EXISTING SYSTEMS FOR BRIDGE MANAGEMENT

In order to produce an outline framework for the management of bridges on road networks across Europe, a review was undertaken of existing BMSs used in Europe and abroad. The purpose was to identify the best features existing in the various countries, and to specify the requirements of a bridge management system that could realistically be achieved without the need for excessive resources.

To obtain information on the BMSs currently in use a questionnaire was developed and sent to the partners in the BRIME project, and to other European countries, as well as countries outside Europe known to be well advanced in terms of bridge management (Canada, Japan, USA). The questionnaires were sent to the national highway authorities in each country and to the New York City and California State Roads Departments. Replies were received from most of the countries approached.

2.3.1 Global description of bridge management systems

Of the sixteen countries that replied eleven use a computerised bridge management system; three do not use a BMS but are in the process of developing one, and two countries use a partially automated system. The ages of the computerised BMSs vary from 2 to 22 years.

2.3.2 Documentation

In most countries the procedures for using the BMS are given in various documents such as maintenance manuals, management instructions and user manuals. In Slovenia there is no official user manual or guidelines; the available documentation consists of a three volume report from a research project. Three countries do not have any special documentation on their BMS.

Most countries use the BMS to manage bridges on the national highway network ie motorways and trunk roads. No country has its BMS linked to a road management system. However the Norwegian system has an automatic link to the road network for route number and location. In Sweden, the integration of pavement and bridge management systems is under discussion.

2.3.3 Database

All countries, except the UK and Slovenia, use commercial database software; the most popular is ORACLE although ACCESS, DELPHI, POWER BUILDER and Structured Query Language are also used. Most countries use WINDOWS based systems.

In most countries the database is used to manage both individual structures and the bridge stock. In Spain and Portugal the database is used mainly for the management of the bridge stock, and in France a different database is used for the management of individual structures and for the management of the whole bridge stock.

The BMSs are used at all the different levels that have a role in maintenance ie national, regional or county authorities and maintaining agents. They are also used by consultants in Norway, Sweden and Denmark. The responsibility for maintenance is always at the national level.

Information is updated at varying intervals; for some it is done daily, some occasionally and for others annually or every 2 years. The interval depends on the types of data and the BMS.

The number of datafields varies enormously; for example the Norwegian database contains 1228 fields in 147 different tables. The Finnish database contains 250 datafields. Some databases have the facility to add user-defined fields.

2.3.4 Bridge condition

There are 3 or 4 levels of inspection (routine, general, detailed and special). The results of general and detailed inspections are usually stored in the database and in Norway the results of measurements and investigations are also stored. In general the condition is stored for both individual elements and the whole bridge, except in Germany and Ireland where the condition is stored only for the whole bridge and the UK where it is only stored for individual elements. The condition is mostly based on a 3 to 5 point rating scale.

2.3.5 Other information recorded on BMS

The date, type, cost and location of maintenance work are recorded in every country. In France, it is only stored for maintenance work which costs more than 300kF and in Slovenia, type and cost are stored separately. The condition rating immediately before and after is stored in five countries but not in the remainder.

2.3.6 Prediction

Most countries do not use past condition data or a deterioration model to predict future condition. The exceptions are:

- Finland where probabilistic Markovian models are used at the network level, and deterministic models at the project level.
- New York, Belgium and Sweden where past condition data and degradation of materials with time are used.
- California where past condition data is used.
- France and Slovenia where previous condition ratings are used.

2.3.7 Costs

Most countries store maintenance, repair and, in some cases, inspection costs on their BMS; the exceptions are Belgium, Germany, Ireland, Norway and Slovenia.

The BMSs used in most countries do not calculate the financial consequences of traffic disruption caused by maintenance work and the associated traffic management. In the UK, delay costs are calculated using either the computer programme QUADRO or look up tables derived from the programme. Ireland also uses QUADRO, and Sweden has its own model for the calculation of user costs.

2.3.8 Decisions on maintenance and repair

Most countries do not use the BMS to make decisions on maintenance and repair. The exceptions are: Denmark where a prioritisation programme is used, Sweden where a planning module is used which allows the study of alternative strategies associated with current value costs, Finland where a repair index is used and California where long-term least cost optimal strategies from the PONTIS management system are used. In Belgium, France, Germany and Ireland decisions are based on engineering judgement. In the UK whole life costing and cost benefit analysis are used. In Spain decisions are based on the cost of the repair as a percentage of replacement costs taking into account the traffic disruption costs

Most countries decide when maintenance work is needed on the basis of inspections and engineering judgement. In Slovenia the decision is based on increased traffic flows and the importance of the bridge to the region. In California it is based on safety and an analysis of the economic benefits. (In general it is based on technical rather than economic requirements.)

Most countries decide which is the best maintenance option to use on the basis of engineering judgement. In the UK it depends on the solutions available, a whole life cost appraisal, the cost of traffic management and the traffic disruption to the network.

2.3.9 Prioritisation

Belgium, Croatia, Germany, Ireland, France, UK, Norway, Portugal and Slovenia do not have a BMS module for generating an optimal (minimum cost) maintenance strategy subject to constraints such as a lowest acceptable level of condition. However a system is currently being developed in the UK. Such a module is used in Denmark (for repair) and Spain, Finland, Sweden, New York and California.

In the optimisation process, other constraints are often applied such as cost and policy in the UK, and the lowest long-term cost that avoids failure in California. In Sweden, the calculation of the profitability of an action in year 1 is compared with deferring that action 1, 2, 3 or 5 years. Budget and bridge condition are also used as a constraint in several countries.

The BMS used in Croatia, France, Germany, Ireland, UK, Norway, Portugal and Slovenia do not produce a prioritised maintenance strategy for the bridge stock when the maintenance budget is insufficient. However prioritisation is used in Belgium, California, Denmark, New York, Sweden and Spain, although in Denmark this is only done for repair. Only Denmark, Sweden and New York quantify the economic consequences of carrying out a sub-optimal maintenance strategy.

Each country uses different criteria for prioritisation. For most countries, the responsibility for prioritisation of bridge maintenance is at the national level. The exceptions are Norway, Finland and Sweden where the responsibility is at the local level.

2.3.10 Quality control

For all countries, there is no quality control of the management of bridges, except for Finland and Sweden where some internal procedures are applied.

2.3.11 Some global data

From the answers to the questionnaires, some global data relating to management of bridges in Europe are presented in Table 2.1.

2.3.12 A particular BMS : PONTIS

In the United States of America the problems arising from the number of deteriorated bridges and their evaluation, repair and strengthening constitute an increasing concern. Indeed, according to criteria of the Federal Highway Administration (FHWA), approximately a third of the 570,000 bridges on the American network are classified as defective and require rehabilitation or replacement. The total cost of the programme of replacement and rehabilitation of bridges is estimated to be approximately 70 billion dollars.

In the last ten years, the FHWA has developed a software package called PONTIS, which allows a choice of optimisation policies at the network level while being based on minimising life-cycle costs. It recommends maintenance for each structure by carrying out a cost-benefit analysis where the benefit is calculated from the saving made from maintaining the bridge immediately compared to postponing the maintenance for one or more years. Currently, PONTIS is probably the most advanced BMS; it is used by about forty states in the US, notable exceptions being New York and Pennsylvania. A particular feature of PONTIS is its statistical approach to the condition of bridge elements, each element of a bridge is considered as part of a family of elements isolated from the individual bridges. The software uses a simple form of Markov chain to model the progress of deterioration, and transition probabilities are applied to model the change of the condition rating of each element. According to studies conducted by the Highways Agency [Das 1996], it appears that the influence of defects on the reliability of the bridge is ignored and that the assessment of load carrying capacity is not involved. Moreover, the bridge elements are considered as being totally independent of the bridge, and global data from a large number of bridges to determine deterioration rates of bridge components can lead to erroneous results if specific or undocumented factors are not taken into account. A too sophisticated approach may lead to simple factors, which can have a profound influence on bridge management, being overlooked.

The survey has given the current position regarding bridge management in various countries. The following sections (2.4 – 2.7) discuss some important features of a BMS and an architectural framework of a BMS is presented in Section 2.8.

2.4 IMPORTANCE OF THE EVALUATION OF SAFETY

The first objective of a BMS is to guarantee the safety of the users. Certain bridges (such as masonry bridges) often possess reserves of strength such that they can tolerate significant deterioration without the safety of the users being affected. For these bridges a visual evaluation of their condition can be used as a basis for their management. On the other hand, some bridges require a more formal approach to their safety. This could be because visual inspection or a simple investigation cannot always detect internal disorders, for example the corrosion of internal prestressing tendons in post-tensioned concrete. Also minor disorders localised to critical parts of a structure could endanger it, for example the propagation of a fatigue crack in a critical steel connection. For these bridges, the evaluation of load carrying capacity constitutes a fundamental element of their management. When assumptions made in assessment calculations are subjected to significant uncertainty, a probabilistic approach to their safety is essential and reliability methods then provide a decision-making aid for engineers [Cremona, 1996].

Table 2.1: Global data on bridge management in some European countries showing the relationship between the cost of annual maintenance and the replacement value.

Owner	Number of bridges	Maintenance : annual cost (MEuro)	Replacement value of the stock (MEuro	**Ratio (%)**
Belgium Roads of Wallonnie	5 000	10	3 800	**0.3**
Finland Road Network	15 000	30	2 900	**1.0**
France National Road Network	22 000	50	10 800	**0.5**
France National conceded motorways	6 000	23	4 100	**0.6**
Germany National Road Network	34 600	318	30 000	**1.0**
Great Britain National Road Network	9 500	225	22 500	**1.0**
Ireland National Road Network	> 1800	2.5	450	**0.6**
Norway Road Network	17 000	37	6 000	**0.6**
Spain National Road Network	13 600	13	4 100	**0.3**
Sweden National Road Network	15 000	92	5 300	**1.7**

The phases of the service life of a bridge are dictated primarily by loss of structural performance, although loss of serviceability (for example, due to defective non structural components) can be just as important. During its whole life, a bridge subjected to various repairs or strengthening work may reach a minimum acceptable safety level (Figure 2.2), this point corresponds to the end of the service life if no other rehabilitation action is conducted. The convex form of the curve is due to the deterioration process that transfers the load supported by the deteriorated areas towards the sound parts of the bridge; this transfer is in general irreversible since repairs do not restore the initial stress state within the entire structure. The objective of an effective maintenance strategy is therefore to increase the service life of the bridge at minimum cost; the longer the service life without incurring substantial maintenance costs, the more durable the bridge may be considered to be.

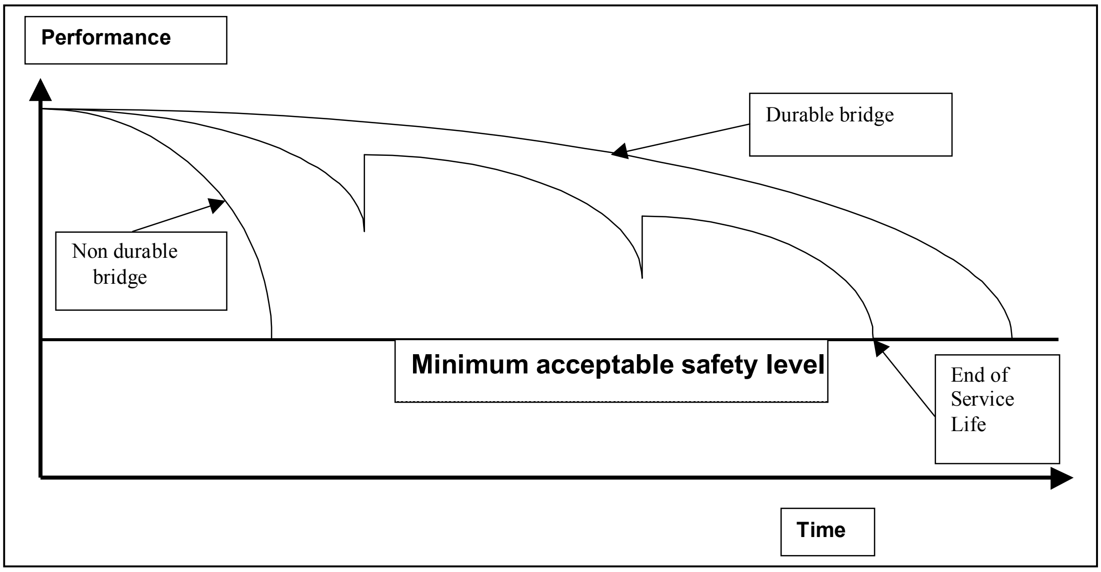


Figure 2.2: Performance of a bridge as a function of time. (the vertical steps correspond to maintenance actions). The performance may be represented by the condition rating or the load carrying capacity.

2.5 CONCEPT OF LIFE CYCLE

The concept of life cycle cost is fundamental to bridge management. The cost of maintenance must indeed take into account not only the original costs, but also the future costs and these are a function of the maintenance strategy adopted. Thus, if a provisional repair is carried out today in place of a final repair, it will be necessary to carry out further work later.

To be able to apply this concept of life cycle, the costs and benefits obtained over the course of time must be able to be evaluated. Time is taken into account by using the discount rate which measures the preference that society has for the present rather than for the future. The most profitable policy is the one which maximises the difference between the discounted benefits and costs (costs should include both the expenditure on maintenance carried out by the manager and the social costs carried by society).

Although it appears paramount to compare various strategies for maintenance on the basis of life cycle cost, it is however important to note that this approach has some limitations:

- To be able to use life cycle costing, the costs must be discounted, but the choice of the discount rate constitutes a real difficulty. A solution may be found by using upper and lower bound calculations for several different values of the discount rate [Llanos 1992]. Indeed, it is mainly used to compare the profitability of different investments in infrastructures over a 20 or 30 year term, and it should be changed when used for structures such as bridges which have a required life of more than 100 years. Regarding the management of bridges on a long-term basis, it is considered that the discount rate should have a reasonably low value, such as 1 or 2 % (Figure 2.3). Using a discount rate means that after several decades, the future costs become negligible compared to current costs. The OECD report on bridge management [OECD 1992] states that the costs occurring in the remote future, for example beyond 50 years, are not significant in terms of present monetary values. The temptation is therefore to postpone major repairs. However, the accumulation of deferred maintenance work can generate, in the long-term, a particularly expensive full rehabilitation. It is therefore essential to consider the long-term consequences of a maintenance strategy.

- Although the use of the concept of life cycle cost is probably useful for individual bridges, it has not been established that the sum of the life cycle cost of each bridge gives an optimised cost for the network as a whole. The application, without understanding, of this method could, therefore, lead to the simultaneous repair of a large number of bridges. In practice it might not be feasible to undertake this work for reasons of traffic management of the network, the capacity of the industry to carry out the work, and for simple budgetary constraints.

- The notion of life cycle cost must integrate the social costs. On a macro-economic level, these social costs can be defined to be the loss of productivity of a region as a consequence of the faulty operation of its road network. On a more pragmatic level, they represent the costs of disruption and delay to the user, ie the loss of time due to reductions in speed or detours, increased wear on vehicles, increased risk of accidents, etc. All these social costs are very difficult to estimate with any degree of accuracy.

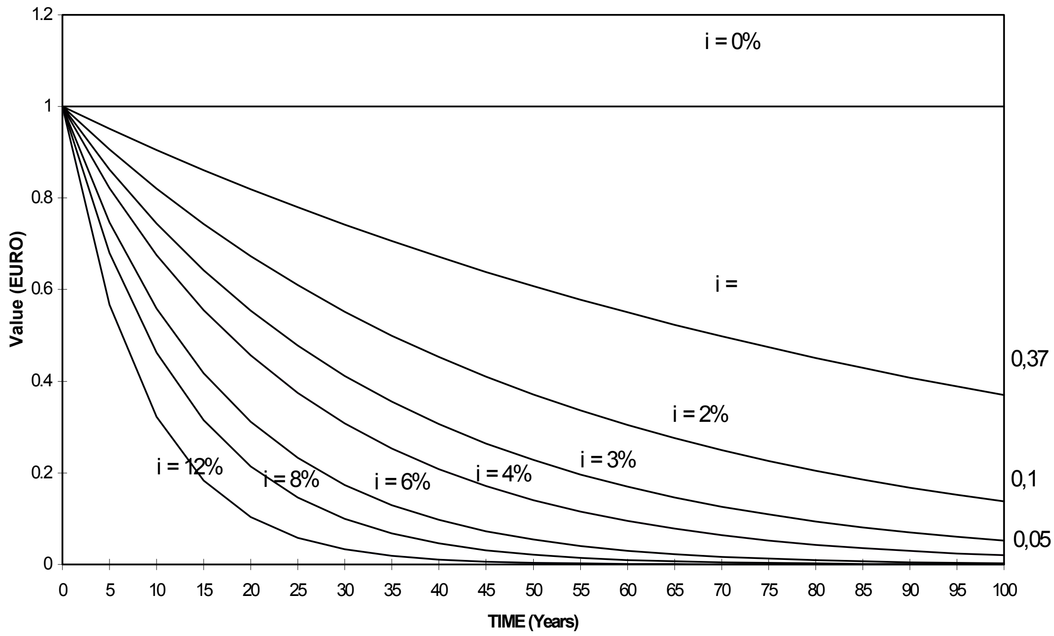


Figure 2.3: Depreciation of 1 EURO over a long time using various discount rates, ranging between 0 and 12 %. (Comment: in order for a bridge to keep a value at 100 years, the discount rate should be below 4 %)

2.6 ESTABLISHMENT OF PRIORITIES

The establishment of priorities is one of the most difficult aspects of management. The simplest systems are generally founded on the coupling of a condition index and a strategic index* which at least makes it possible to obtain a set of priorities for the maintenance of a group of bridges.

This approach makes it possible to select bridges (or groups of bridges) which are in need of maintenance and to put them in order of priority to develop a maintenance programme. This policy applies classically to developing countries, which often have a degraded network. It also applies to communities of developed countries which have neglected the maintenance of their bridges over many years are now being confronted with the necessity of carrying out extensive repairs or rehabilitation to a significant part of their stock.

In countries where maintenance has been regularly applied and where the condition of the stock is generally satisfactory, it then becomes possible to establish priorities using the optimisation of resources based on a true socio-economic approach.

- The strategic index can integrate multiple criteria, but the concept of bridge grading which is based on the hierarchy of the road network, the crossing of important obstacles, the difficulty of re-routing traffic, and the concept of single access (at a village, a factory, etc) has an important influence on the index.

2.7 THE PROBLEM OF FORECASTING

As previously indicated, it is impossible to develop maintenance strategies unless the future condition of bridges can be predicted using well established procedures. Currently lack of knowledge makes it very difficult to establish a prognosis on the future behaviour of a bridge and its residual life. The simplest way to make a prediction is to base it on an extrapolation of deterioration curves based on monitoring of the existing bridge stock (a global approach). Nevertheless, for certain material degradations like corrosion or the alkali silica reaction, it appears possible to build models of ageing based on physical degradation laws of materials. In this case, the essential difficulty lies in transferring laws for degradation of material to laws for the development of structural disorders.

2.8 STRUCTURE OF A BMS

On the basis of the preceding discussion, a bridge management system that is able to answer the various objectives of the managers, must be modular and incorporate, at least, the following principal modules:

1. Inventory of the stock
2. Knowledge of bridge and element condition and its variation with age
3. Evaluation of the risks incurred by users (including assessment of load carrying capacity)
4. Management of operational restrictions and the routing of exceptional convoys
5. Evaluation of the costs of the various maintenance strategies
6. Forecast the deterioration of condition and the costs of various maintenance strategies
7. Socio-economic importance of the bridge (evaluation of indirect costs)
8. Optimisation under budgetary constraints
9. Establishment of maintenance priorities
10. Budgetary monitoring on a short and long-term basis

Figure 2.4 presents an architectural framework of a BMS including these principal modules with their main interactions. The framework takes into account the two levels of management (project level and network level), and is organised in order to show the contributions of each BRIME Workpackages (WP 1 to 6). This is discussed in more detail in Chapter 9.

2.9 CONCLUSIONS

The objective of a BMS is to preserve the asset value of the infrastructure by optimising costs over the lifespan of the bridges, while ensuring the safety of users and by offering a sufficient quality of service. It is thus a problem of optimisation under multiple constraints.

Llanos [1992] states that a short-term policy has visible effects in the short run; it makes it possible to improve the condition of the stock at the end of the fixed term, although this policy has a higher cost compared to the economic optimum, which requires long-term management. In addition to the ‘revolution’ needed in the spirit of financiers and managers to whom this policy of long-term management is directed, there is a technical problem namely the absence of prediction models for the behaviour of bridges. Another problem is the limited knowledge of the social costs generated by the degradation of the bridges. As noted by Yanev [1998], it is thus probable in the future that engineering judgement will still be necessary to take decisions in terms of priority and of maintenance option, and that computerised management of bridges will only be used for data storage and retrieval and to inform the decisions made by engineers.

This is why, even if a computerised BMS was able to draw up a list of priorities for maintenance, the results must be studied carefully and the decisions taken by considering factors that cannot be evaluated in terms of cost. Such factors cannot therefore be introduced into the information processing system: it is thus not only the judgement of the engineer, but also the political, aesthetic, or the prestigious character of the bridge that will determine the maintenance constraints strategy for a bridge stock.

2.10 REFERENCES

Cremona C 1996. *Optimisation de la fiabilité et de la maintenance des ouvrages d'art métalliques*. Troisième conférence nationale sur l'intégrité des structures INSTRUC 3, Société Française des Mécaniciens, 27-28 novembre 1996.

Das P 1996. *Bridge management methodologies. - Recent advances in Bridge Engineering.,* Edited by J.R. Casas, F.W. Klaiber and A.R. Mari, CIMNE Barcelona, 1996.

Lauridsen J et al 1998. *Creating a Bridge Management System.* Structural Engineering International, Vol 8, Number 3, pp 216-220, August 1998.

Llanos 1992. *La maintenance des ponts, Approche économique.* Presses de l'ENPC, 176 pages.

OECD 1992. *La gestion des ouvrages d'art.* Rapport de l'OCDE, 132 pages.

Yanev B 1998. *Bridge Management for New York City.* Structural Engineering International, Vol 8, Number 3, pp 211-215, August 1998.

2.11 BIBLIOGRAPHY

Calgaro J A and R Lacroix 1997. *Maintenance et réparation des ponts.* Presses de l'ENPC, 1997, 666 pages.

ENPC 1994. *Gestion des ouvrages d'art.* Actes du colloque organisé par l'ENPC, Paris, 18-20 octobre 1994, Presses de l'ENPC, 596 pages.

Thomas Tefford 1998. *The Management of Highway Structures.* London 22-23 June 1998, The Institution of Civil Engineers and the UK Highway Agency.

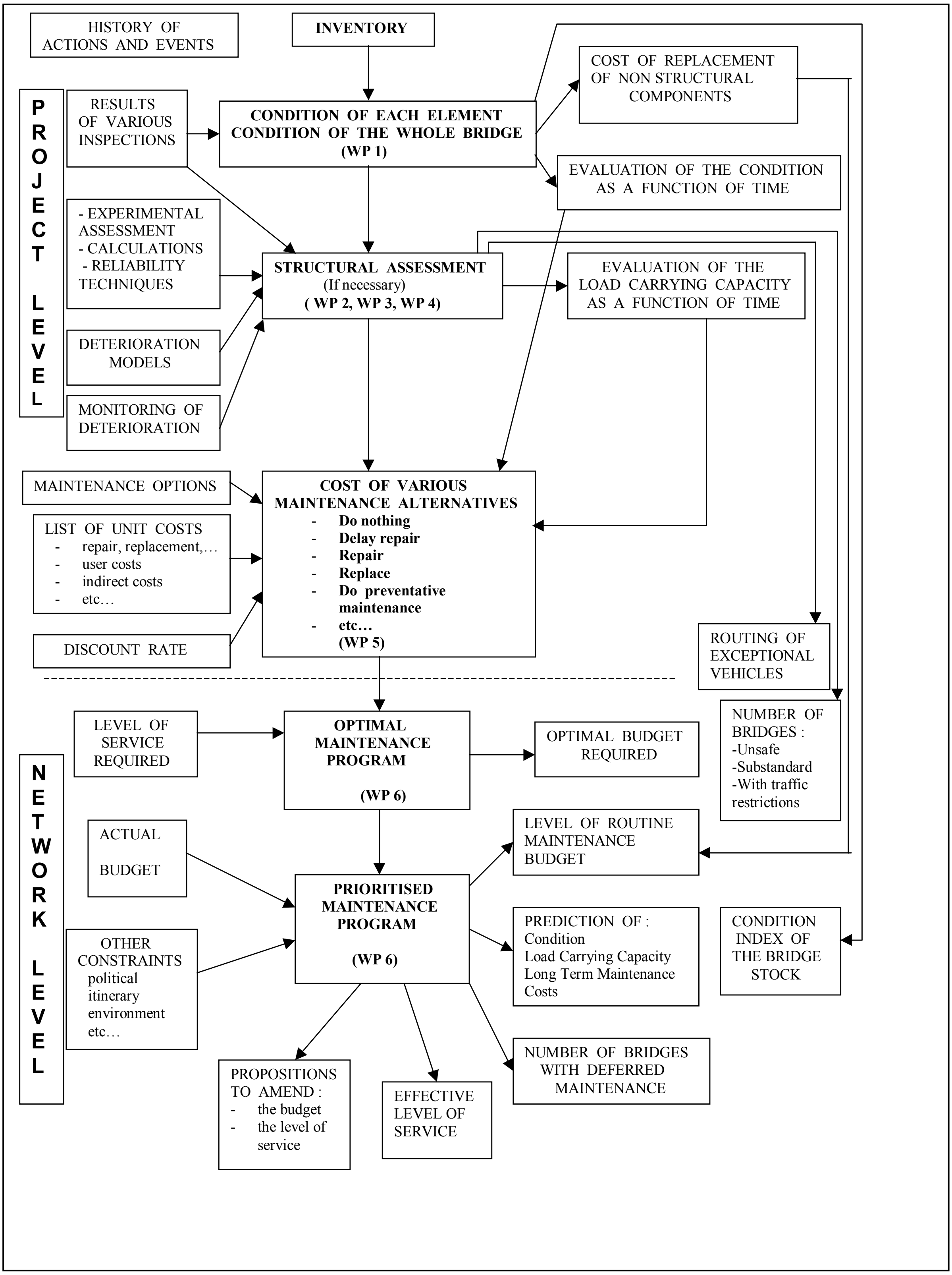


Figure 2.4: A schematic diagram of the architectural framework of a BMS indicating the main interactions between the various modules.

CHAPTER 3

CONDITION ASSESSMENT

3.1 INTRODUCTION

Over the last three decades considerable problems have arisen due to deterioration of the bridge stock. This has occurred for several reasons: ageing of the bridge stock, increases in traffic load, environmental attack and sometimes due to poor design, detailing and construction of the structure. The most common form of deterioration on concrete bridges is corrosion of the reinforcement caused by the ingress of carbon dioxide or chloride ions. The latter penetrate structures in marine environments or in countries with harsh winters, where de-icing salts are used on roads during winter. De-icing salts either penetrate through the deck or are sprayed onto the substructure of overbridges by passing vehicles. Concrete structures also deteriorate due to cyclic freezing and thawing during the winter. Chemical reactions within concrete caused by sulfate attack or alkali silica reaction can also cause severe degradation of the concrete surface or structural element itself. A more detailed description of the causes of deterioration is given in Chapter 5.

To determine which structures require maintenance, it is necessary to undertake a systematic programme of inspections. One of the main purposes of these inspections is to provide data on those structures that are in a poor or critical condition and in need of repair, strengthening or rehabilitation. The results of these periodic inspections are used to provide an assessment of the condition of both the structural elements and the structure itself.

This chapter describes the research carried out within Workpackage 1 (WP1), Condition Assessment of Bridge Structures. The work focused on:

i) a review of methods currently in use for condition assessment in Europe and the United States

ii) a review of the use of artificial intelligence methods for condition assessment

iii) development of a simple model for categorising damaged areas on a bridge deck.

Current methods of condition assessment use two different approaches for both the assessment of individual elements and the assessment of the structure as a whole. The first uses a cumulative condition rating which is derived from the condition of individual elements and the second uses the condition rating of the bridge element in the worst condition as the condition rating of the structure itself. The condition assessment is based on bridge inspection, and similar procedures are used in different countries although the frequency with which the different types of inspection are carried out do differ.

Artificial intelligence methods are being increasingly used in civil and structural engineering and a review was made of a number of different methods including neural networks, fuzzy logic and genetic algorithms with particular emphasis placed on their use in bridge management.

A simple model was developed for categorising damaged locations in concrete structures. This was based on a visual assessment of the damaged area and the results of tests both on site and in the laboratory. The model used the neural network hybrid model and it can be used for assessing the type of repair work required on structures with a large number of deteriorated areas.

3.2 BRIDGE INSPECTION

Degradation of bridge structures is a world-wide problem. It usually starts at the weakest point of the structure, for example on the sub-structure underneath expansion joints and construction joints, around drainage systems and at damaged waterproofing membranes. It is therefore necessary to look for visual evidence of distress, for example cracking in the surfacing (figure 3.1) and damp patches on the underside of the deck. Deterioration of exposed bridge members can be observed during a visual inspection, but deterioration of elements which are hidden from view, such as the bridge deck underneath a waterproofing membrane, can be observed only after the removal of the pavement and waterproofing membrane (figure 3.2).

To deal with all these problems, different procedures for bridge inspection have been applied in different countries. In general, the procedures used are similar. The main differences are: the length of the bridge which is inspected, as in some countries structures with a span greater than 2m are defined as a bridge whereas in others it is 5m (Chapter 1), and the intensity and frequency of the inspections. A review of bridge inspection procedures used in different countries is described in Deliverable D2. It identifies three basic types of inspection.

Figure 3.1: Cracks in the pavement of carriageway

Figure 3.2: Deterioration of the bridge deck after the removal of pavement and waterproofing membrane

3.2.1 Superficial inspections

Superficial inspections are carried out by maintenance personnel who are familiar with the safety procedures for working on the highway but do not have specialist knowledge of bridge pathology. The aim is to observe major defects (for example damaged safety barriers (figure 3.3) or broken drainage systems) on and under the bridge. This is done continuously and a note is made of any observations ie date of inspection and details of any defects.

Figure 3.3: Damaged steel Guard Rail

3.2.2 General inspections

General inspections are a visual examination of all the accessible parts of a bridge but without the use of special access equipment. Normally, they are undertaken by technicians who have received some training on bridge inspection. A more qualified inspection team is required for more complex bridge structures. The aim of the inspection is to detect all defects that can be seen from the ground and to evaluate the condition of the structure (figure 3.4). The recommended frequency of general inspections is one to three years. An inspection report must be prepared that should, if required, give a recommendation for a more detailed inspection. If a general inspection is carried out after a major inspection (see below), and no repair or maintenance work has been undertaken since the previous inspection, only the observed defects are assessed in the inspection report. The evaluation of other defects observed during the previous major inspection but which cannot be properly evaluated during general inspections, are not changed.

Figure 3.4: Deteriorated surface of the concrete sidewalk

3.2.3 Major inspections

Major inspections are a visual inspection of all parts of the bridge structure. They are carried out by qualified bridge engineers with experience in bridge maintenance. The aim is to get within touching distance of all parts of the bridge and make a visual assessment of the condition (figures 3.5 and 3.6). Therefore access must be provided and specialised equipment may be required.

The inspector should identify and record poor construction details as well as defects. The recommended frequency for major inspections is at least five to ten years, although they could be undertaken more frequently depending on the condition of the structure and its load carrying capacity. For example, more frequent inspections would be required on structures with excessive deflections and settlement, or where joints opened under load. This type of inspection may include some measurements eg vertical displacement, settlement, chloride content at most critical elements of the structure, the scope of which depends on the condition and complexity of the structure. A full report is required giving a description of the defects, an assessment of the condition of the structure and recommendations for special - or detailed

- inspections and urgent repairs. The extent and severity of any defects should be described in sufficient detail to enable a reasonable estimate to be made of the cost of the repair work.

Figure 3.5: Damaged girder, access by movable platform

Figure 3.6: **Close view of corroded tendons from the platform at major inspection**

A special type of major inspection is an **Acceptance inspection**, which, in some countries, is carried out before a structure is opened to traffic. Another type of inspection is a **Guarantee inspection**, which is carried out before the end of the guarantee period.

3.2.4 In-depth inspections

In-depth inspections are performed on bridge structures that are undergoing repair. They are usually carried out on complex structures and may cover the whole structure or be restricted to the components or elements that are likely to be affected by the repair (figure 3.7). Usually the inspection includes extensive measurements both on site and in the laboratory which are undertaken to determine the cause and extent of the damage or deterioration and provide data to ensure an effective repair.

3.2.5 Special inspections

Special Inspections are carried out where there is a particular problem or cause for concern either found during an inspection or already discovered on other similar bridges. They are also carried out for a variety of other reasons, for example: structures strengthened by the use of bonded steel plates, bridge foundations after flooding, and structures after earthquakes.

Two ways in which the results obtained during an inspection can be used are presented graphically in figures 3.8 and 3.9.

If it becomes obvious during an inspection that deterioration of critical parts of the bridge structure might threaten the safety of users, then an inspector can recommend one of a number of interim measures. These include: temporary load restrictions, propping or closure of the bridge, until the results of an in-depth or detailed inspection of the structure and an assessment of load carrying capacity reveal whether or not there is a need for such measures. In cases where deterioration does not affect the safety of the structure, an inspector should recommend a detailed inspection and assessment of load carrying capacity to determine what further action is required.

Another purpose of periodic bridge inspections is to obtain data on the condition of the bridge stock. Inspection data can be used to rate the condition of the bridge. The methods available can either be used to rate the condition of the bridge as a whole or of its individual members or elements. Inspection data can also be used to identify those bridges that are in need of repair. Based on the results of an inspection, the decision may be taken to monitor a bridge for certain period of time.

Figure 3.7: **In-depth inspection of piers by abseiling**

3.3 CONDITION ASSESSMENT OF STRUCTURES

The main objective of assessing the condition of bridge structures is to monitor the extent and severity of any defects or deterioration that is present and to determine the optimum time for intervention. That is, to determine the appropriate time for any repair or maintenance that is required to preserve the condition of the structure within acceptable limits. An additional objective is to evaluate the efficiency of different repair techniques, the suitability of different materials used in repair work and their application. The results of the inspection can also be used for verification of the different measurement techniques used on site and in the laboratory. For example, measurement of chloride contents using different technique for obtaining dust samples (eg continuous drilling, obtaining the samples from several holes, taking cores) and different measuring methods (eg titration, photometry).

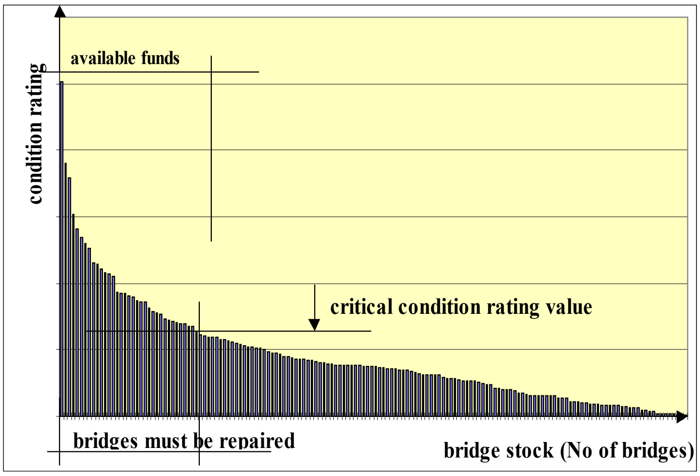


Figure 3.8: **Prioritisation of bridge structures based on condition rating**

For the purposes of bridge management, the most important use of information on the condition of a large number of bridges in a bridge stock is to identify those that are most deteriorated and in need of repair work. Different methods have been developed for evaluating bridge inspection data to give the bridge a condition rating. A short description of some methods that have been developed in Europe and the USA, is presented in Deliverable D2.

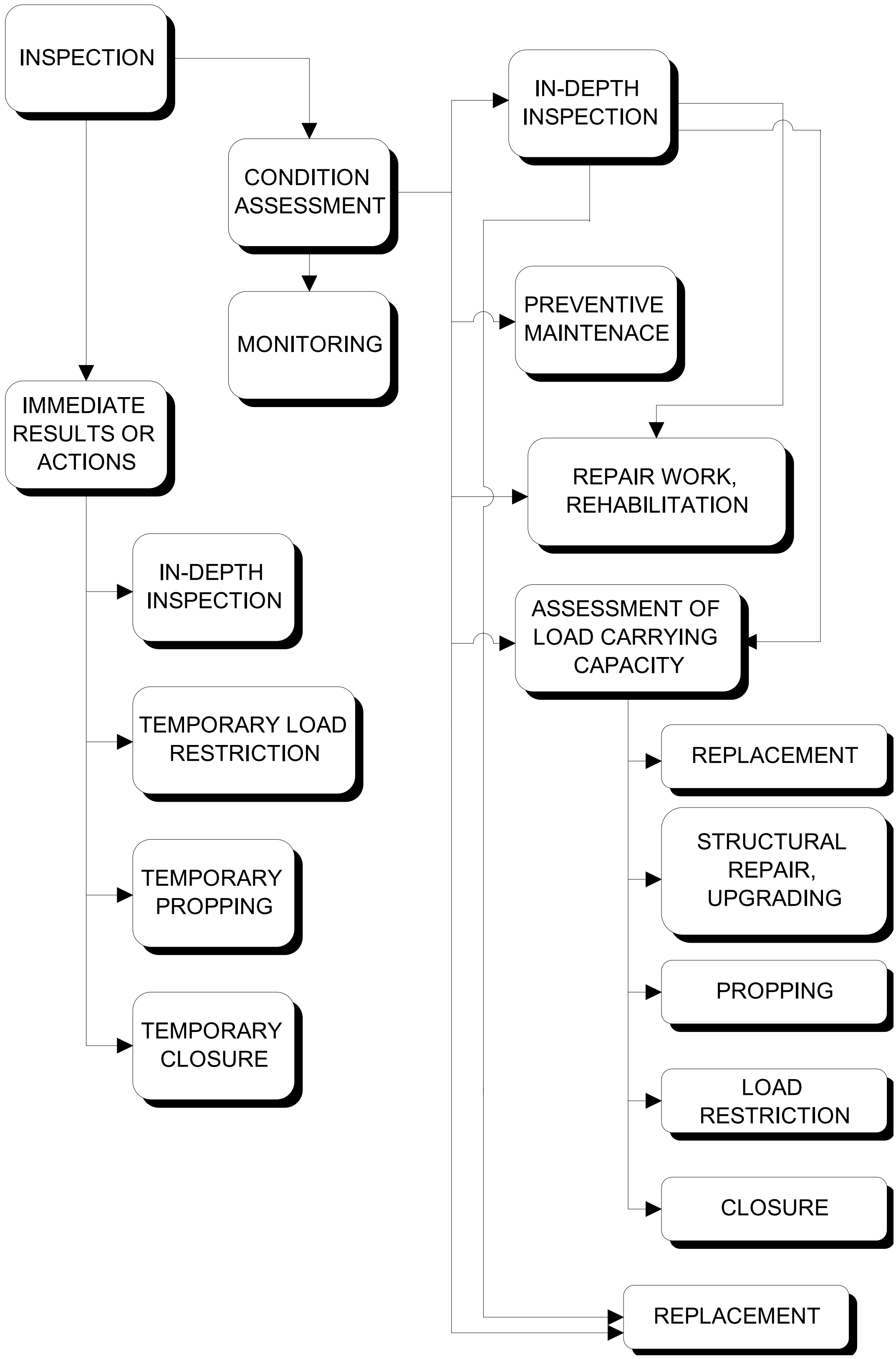


Figure 3.9: Flowchart showing use of data on condition assessment

The report on condition assessment should include all the data needed for a cost optimisation to determine whether any preventative maintenance or repair work should be undertaken. These data are:

- location of the bridge
- the condition of all elements of the bridge structure
- type of damage observed
- severity of the damage
- extent of the damage (length, surface, volume eg the amount of embankment that has been washed away or foundation that has deteriorated or been lost due to erosion, quantity eg number of bearings, lamp posts, expansion joints that have been damaged)
- the urgency of the repair work (urgent, very urgent)
- location of any defects (where it is not possible to give a precise location an approximate indication should be given for example for defects observed from the ground during a general inspection of high or long bridges, or where there are a large number of deteriorated areas it may only be practicable to locate the largest)
- the need for further detailed or underwater inspections
- the need for temporary closure, propping or load restrictions (this would require a structural assessment).
- whether any maintenance recommended after previous inspections has been carried out
- whether maintenance recommended after the previous inspection was carried out
- photographs of the most severely damaged locations on the bridge structure.

3.4 INTELLIGENT SYSTEMS IN STRUCTURAL ENGINEERING

The treatment of natural phenomena is usually based on the derivation of a relationship between measurements of variables that relate to the phenomena. Theoretically, the relationships are most appropriately specified in terms of abstract mathematical models representing mathematical laws. However from the practical point of view, simulated analogue models based on electronic devices are sometimes more convenient. A neural network and neural network-like systems are examples of such analogue models.

In recent years there has been a large increase in the development and application of intelligent systems to problems in structural engineering. They are used to handle data obtained from observations or measurements both in the field and in the laboratory. A review of the literature has shown that there are four methods that are most commonly used. These are expert systems, neural networks, fuzzy logic and genetic algorithms. Examples of their use include: the design optimisation of reinforced concrete members and frames, analysis of bridge condition rating data and optimisation of bridge deck rehabilitation and pavement rehabilitation. A short review of some applications of artificial intelligence systems in civil and structural engineering is presented in Deliverable D2.

3.4.1 Why use neural networks and intelligent, neural network-like systems in bridge management?

The field of bridge management is large and complex. Some aspects can be effectively handled and solved using classical mathematical tools. However, some aspects such as the degradation of concrete structures are very complex and decisions are usually based on engineering judgement. The advent of modern computers capable of handling large amounts of data, has made it possible to develop alternative approaches for manipulating the data. Over the last decade intelligent systems, sometimes known as expert systems and neural networks, have offered a promising way to combine the complex information contained in an expert's knowledge and measured data that describe different phenomena.

To develop models that could be used to replace or supplement expert knowledge from the huge amount of knowledge that experts possess is a difficult task. The purpose of using neural networks in this project is to demonstrate that modern techniques are able to deal effectively with the large volume of measured data and engineering judgement that is used in bridge management. The examples presented in this report to illustrate the use of neural networks are relatively simple – but they demonstrate the possibilities for further development.

3.4.1.1 Conditional Average Estimator (CAE)

CAE is an hybrid neural network model. The basic idea of CAE comes from artificial intelligence, or more precisely, from its specialist area - neural networks. Whilst such tools work as "black boxes", some supplementary tools have been added to improve the efficiency of CAE. These tools provide a different mathematical measure, which correlates with basic statistical measures. Therefore the efficiency of the model can be defined mathematically, the efficiency of different models can be compared and the reliability and/or the quality of the solution can be compared with the existing solutions. Finally, there are also some ideas that allow a simple explanation of the predicted results.

It is expected that CAE could solve most of the problems in areas where there is sufficient data and/or sufficiently strict descriptions (ie the knowledge is not too vague) of phenomena. Using CAE it is possible to avoid problems concerning the selection of different coefficients, which define the learning of normal artificial neural networks and also the problems of local minima. CAE can also be used as a filter in data preparation for other neural networks. Furthermore it can be used for the analysis of the problem itself, and for many other purposes.

A more detailed theoretical description of the method is given by Grabec and Sachse (1996), Grabec (1990), Peruš et al (1994), Znidaric and Perus (1997) and in the Appendix to Deliverable D9. A short description of how it works is given in Chapter 8. When the computation time[1] is important, CAE also needs a learning process – it is called self-organisation and represents compression of information based on the maximum entropy principle [Grabec, 1990]. However, most applications do not require such a learning process, which makes the method more applicable. It is a very powerful method compared with other neural networks.

3.4.1.2 Algorithm Inductive Decision Tree (ID3)

ID3 was proposed by Quinlan [1979]. It is a training system in which individuals are divided into several classes according to their characteristics. ID3 methods output shapes acquaintance in arboraceous (tree) form from which IF - THEN rules can be constructed. This tree-like form is founded on the theory of information transmission. When the causality rule cannot be established, incorrect data is added to the database. If the neural network

[1] CAE predicts by using conditional average and for this reason the whole datebase is examined. Therefore, if the database is very large, the computation time is also long.

system is a black box then ID3 is a transparent box. It is easy to introduce fuzzy logic and the theory of possibility into the ID3 method so that it can be adapted for different bridge maintenance procedures in different countries. It is also easy to classify bridges in the field immediately by using minimum attributes.

3.4.1.3 Back Propagation Neural Network (BPNN)

A neural network is a parallel distributed information processing structure in the form of a decision tree where the nodes are called processing elements (or neurons), and the links are called connections. Each processing element can have a local memory.

A back propagation (BP) neural network is one of the most important neural networks. It is a very powerful mapping network that has been successfully applied to a wide variety of problems. A detailed description of neural networks in general, and some powerful neural network paradigms, including BPNN, can be found in Hecht-Nielsen [1990]. A typical BPNN is a three-layer network with an input layer, one hidden layer and an output layer (of neurons).

3.4.2 Damage categorisation

During the inspection of a deteriorated structure a considerable amount of information is obtained based on visual observations and on measurements on site or on samples in the laboratory. A great deal of this information is concerned with deterioration of concrete structures due to reinforcement corrosion. These measurements (half-cell potentials, depth of concrete cover, resistivity, chloride concentrations, carbonation depth) are used to provide an indication of the condition of the reinforcement [Mallett, 1994]. The results of each method are interpreted in terms of the probability of corrosion (ie unlikely, probable, almost certain) or in terms of the corrosion process. To determine whether the reinforcement is corroding and if so by how much is difficult, especially when there are no visible traces of corrosion and the result of a single test or measurement is not usually sufficient. Therefore a combination of several tests is usually required.

In the past some methods were developed for categorising deteriorated bridge decks. One such method is used in the Pennsylvanian Bridge Management System, [Pennsylvania Department of Transportation, 1987] and is based on visible spalls and delaminations, and measured electrochemical potentials. Based on the prescribed criteria, a deterioration classification category can be made. Another approach is to classify damage and recommend repair procedures with respect to the type of structure [Söderquvist, 1998]

One of the objectives of this project was to investigate the use of neural networks for categorising deterioration on large bridge structures with many areas of deterioration. The basic requirement for using neural networks is to obtain data from in-depth inspections on a large number of damaged areas.

The first step is to map all the damaged areas on a structure and make a visual categorisation of each area. Five categories are used based on a visual assessment of: the intensity of wetting, the depth of delaminated areas ie whether they are to the depth of the stirrups, main reinforcement or tendons, spalling of the concrete cover, crack widths, width of joint openings of pre-cast elements, corrosion of the reinforcement and/or tendons and surface imperfections. General repair procedures are then given for each deterioration category. If the structure has a large number of damaged areas, field measurements and laboratory test are performed on representative areas of each category. Tests include measurement of electrochemical potentials, carbonation depth, pH at the level of the reinforcement, concrete permeability, chloride profile and concrete cover. In the future, data from other types of test could be added. A new categorisation is then made which is based on the results of the tests. The two categorisations for each damaged area are compared and the degree of agreement between the two assessed.

Ideally damage categorisation should not be subjective but should be based on the results of tests carried out on deteriorated areas. However as it would be impractical to carry out tests on all the damaged areas, a neural network model has been developed which relates the visual categorisation to that obtained from the results of tests. It includes all of the possible combinations of the measured and/or observed parameters.

3.4.3 Comparison of CAE, ID3 and three layer BPNN

A comparison was made to show how the same problem can be solved (modelled) using three different artificial intelligence methods although it was not intended to show which approach is best. For the sake of simplicity and to illustrate the application of the three methods described above (CAE, ID3 and three layer BPNN), a simple example of damage categorisation is given below. To simplify the approach, the database for the ID3 method has been divided into three groups and damage classification is based only on these three input parameters which are:

1. *Cl content at the depth of reinforcement (Cl^-),*
2. *depth of carbonisation (crbn) and*
3. *cracks (crks)*

The database comprises 10 samples (ie 10 sample vectors or model vectors). Each input parameter has value in the range 0 - 4, while the final categorisation has a value in the range 1 - 5.

Sample database:

No.	Cl^- *(% by weight of cement)*	Crbn *(depth in mm)*	crks *(class)* *	Category
1.	0.5	5	1	1
2.	1.2	0	3	5
3.	0.1	1	2	2
4.	0.1	10	1	2
5.	0.6	3	1	2
6.	0.0	15	3	3
7.	0.6	0	4	4
8.	0.2	3	4	5
9.	0.2	5	2	2
10.	0.1	2	1	1

*description of crack classification:
1. cracks at distances approx. 30 cm, crack width less than 0.3 mm, dry
2. cracks at distances approx. 30 cm, crack width less than 0.3 mm, wet

3. flexural cracks, crack width more than 0.3 mm, dry
4. flexural cracks, crack width more than 0.3 mm, wet

3.4.3.1 Results

As has already been described, the CAE method does not need any training. However, for comparison, the results of filtration[2] can be presented as training results for smoothing parameter[3] (*w*) = 0.1,0.15,0.2. These are given below:

No.	Cl^- *(% at cement weight)*	**Crbn** *(depth in mm)*	**Crks** *(class)*	Category	**CAE** w=0.10	**CAE** w=0.15	**CAE** w=0.20	**BP NN**	**I.D.3**
1.	0.5	5	1	1	1.23	1.37	1.45	1.16	A
2.	1.2	0	3	5	5.00	5.00	4.99	4.93	C
3.	0.1	1	2	2	2.00	1.94	1.85	2.09	B
4.	0.1	10	1	2	2.00	1.99	1.92	2.02	B
5.	0.6	3	1	2	1.77	1.63	1.56	1.87	A
6.	0.0	15	3	3	3.00	3.00	3.00	3.01	B
7.	0.6	0	4	4	4.00	4.03	4.14	4.02	C
8.	0.2	3	4	5	5.00	4.97	4.85	4.94	C
9.	0.2	5	2	2	2.00	1.96	1.87	2.08	B
10.	0.1	2	1	1	1.00	1.11	1.31	1.02	A

The categories are defined as:

- category 1: 0.5 - 1.5
- category 2: 1.5 - 2.5
- category 3: 2.5 - 3.5
- category 4: 3.5 - 4.5
- category 5: above or equal 4.5.

The category is allocated by the expert and, as can be seen, the training results indicate a good correlation for all methods.

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[2] The most basic tool in the modelling process is filtration. It gives the most straight forward estimate of the noise in the data, and shows how well the CAE neural network was trained for different *w* values. In most cases, low values of *w*, which give the best results, produce an "over-trained" model (poor prediction capabilities).

Procedure: Values of each output variable for each model vector from the database are predicted using all model vectors from the database. A small value of *w* allows an estimate of the noise in the data with one of the measures of global error.

[3] For application of CAE, only one single parameter must be defined, compared with many when using other types of neural network. The smoothing parameter has an indirect relationship with the learning error (and/or learning threshold) in classical neural networks - both influence the final solution. The smoothing parameter determines the shape of the curve in two-dimensional problems, and the shape of the hyper-plane in multi-dimensional problems. While the smoothing parameter influences the accuracy and/or the efficiency of the model, the determination of its optimal value represents the key point in a modelling process. A few methods (i.e. filtration and verification) can help the users find the right value of *w*. For modelling the phenomena in the BRIME project, the most simple variant of the smoothing parameter was used ie *w* has a constant value over the whole problem space.

3.4.4 CAE model for damage categorisation

A generalised interpretation of the phenomena arising in the field of assessment of existing concrete structures using the neural network-like method (CAE), which can be applied using a personal computer, is presented and discussed for the problem of damage categorisation. When assessing a deteriorated structure the damaged areas on structural components are usually classified into deterioration categories. As stated above, this categorisation should not be subjective, but should be based on the results of tests carried out as part of an in-depth inspection and/or parameter from visual observations.

3.4.4.1 Modelling the deterioration category

Input variables

Experimental data

The CAE experimental model for categorising deterioration takes into account a number of measured parameters that in mathematical terms correspond to the components of a model vector. One model vector comprises variables representing one distinct deterioration category. In the case of reinforcement corrosion, the following test results were selected and processed to become input parameters describing the deterioration category:

- gas permeability
- content of chloride-ions at the reinforcement level
- alkalinity
- depth of carbonation/carbonation front
- electrochemical-potentials.

Visual data

The CAE visual model for categorising deterioration takes into account a number of visual parameters that in mathematical terms correspond to the components of a model vector, similar to that used for the experimental model. In the case of reinforcement corrosion, the following visual parameters were selected and processed to become input parameters for describing the deterioration category:

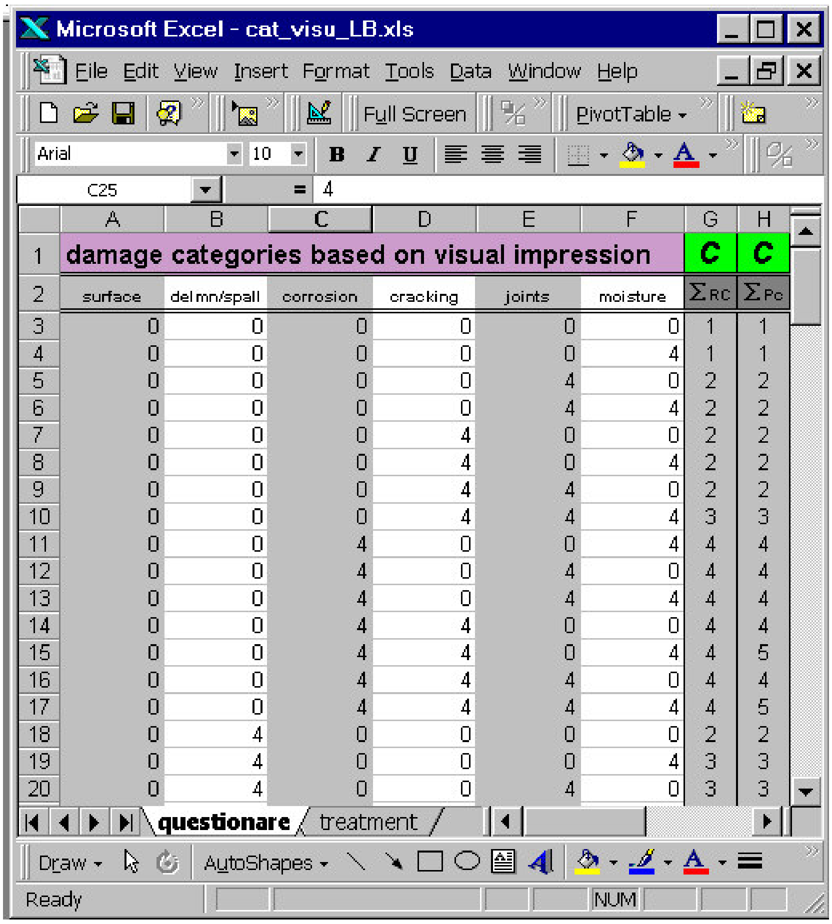
- surface condition
- delamination / spalling of concrete
- reinforcement corrosion
- cracking
- condition of joints
- moisture.

Output parameter

The output parameter is the deterioration category, denoted *DC*. This is a uniform variable that can have any value between 0.5 and 5.5. Individual deterioration categories have been determined by convention as described in Section 3.4.2.

Database

A database was obtained from a questionnaire, which was sent to experts as an MS Excel file. The questionnaire was prepared using a random number generator. The typical form of such a questionnaire for visual categorisation is shown in figure 3.10. A similar questionnaire was prepared for experimental data. Experts from Slovenia (the green columns in figure 3.10) then completed the questionnaire although it could easily be extended to include data from other experts. The experts entered the category for each damaged area on a reinforced concrete structure (RC) and prestressed concrete structure (PC) taking into account the values of each of the randomly generated parameters.



	A	B	C	D	E	F	G	H
1	damage categories based on visual impression						C	C
2	surface	delmn/spall	corrosion	cracking	joints	moisture	ΣRC	ΣPC
3	0	0	0	0	0	0	1	1
4	0	0	0	0	0	4	1	1
5	0	0	0	0	4	0	2	2
6	0	0	0	0	4	4	2	2
7	0	0	0	4	0	0	2	2
8	0	0	0	4	0	4	2	2
9	0	0	0	4	4	0	2	2
10	0	0	0	4	4	4	3	3
11	0	0	4	0	0	4	4	4
12	0	0	4	0	4	0	4	4
13	0	0	4	0	4	4	4	4
14	0	0	4	4	0	0	4	4
15	0	0	4	4	0	4	4	5
16	0	0	4	4	4	0	4	4
17	0	0	4	4	4	4	4	5
18	0	4	0	0	0	0	2	2
19	0	4	0	0	0	4	3	3
20	0	4	0	0	4	0	3	3

Figure 3.10: Questionnaire for visual model - example.

3.4.4.2 Results

The results from the different experts can be presented either in a set of tables or on a set of simple diagrams, showing the iso-lines of equal deterioration or the boundaries between two deterioration categories. Alternatively the results can be presented as 3D-surface graphs.

The following figures present typical results for each deterioration category based on visual data (figure 3.11) and experimental data (figure 3.12). The colours in the plots represent the vertical scale and are automatically generated for each point. Dark blue is the lowest category (good condition) and dark red is the highest category (the worst condition).

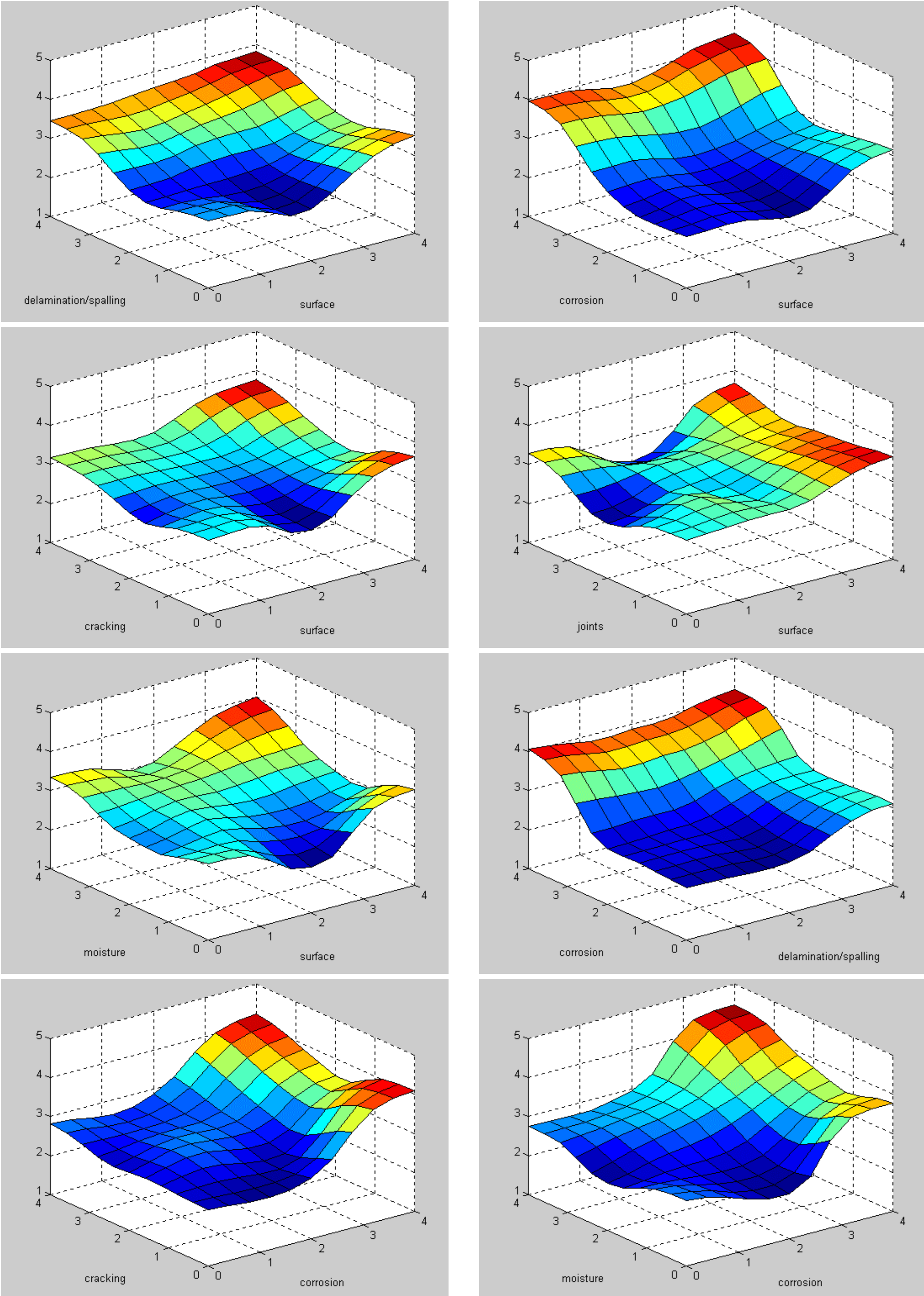


Figure 3.11: 3D presentations of deterioration category as a functions of two parameters –"visual model".

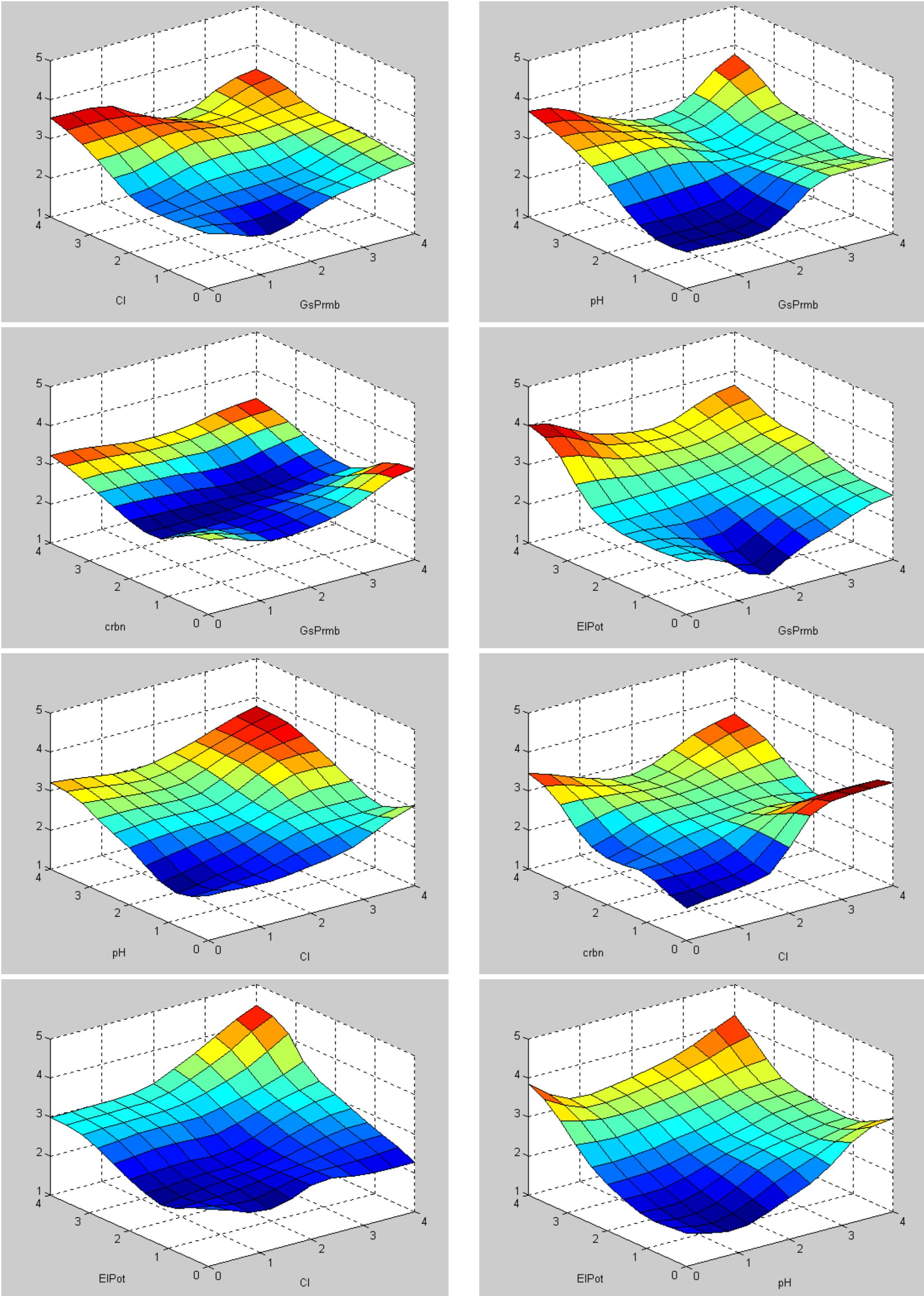


Figure 3.12: 3D presentations of deterioration category as a functions of two parameters – "experimental model".

A more detailed description of the model and a case study on a smaller sample of data is given in Deliverable D9.

3.4.4.3 How can this method be used in practice and for what purposes?

The main purpose of this method is to assess the damage categories of a large number of damaged areas on large structures. The first step is to map all the damaged areas and provide a categorisation for each area based on visual inspection. A representative sample of each category is then selected for more detailed testing both on site and on samples in the laboratory. These results are used to provide a categorisation of each of the areas tested. The results of the experimental and visual categorisations are then compared and the results of this comparison are used to adjust the visual categorisations of the remaining damaged areas. The amended categories can then be used to obtain a more reliable cost estimate for any repair work that is required.

3.5 CONDITION RATING

Condition rating is an effective means of quantifying the general deterioration of a structure. Methods have been developed for bridge management purposes to identify the most damaged structures for further in-depth inspection and examination, and to establish preliminary priorities for further rehabilitation.

Condition assessment should be based on a simple scoring method either for the inspected members or for the whole structure. The evaluation of any deterioration should take into account all types of defect revealed during an inspection, whose character, severity and extent might have a substantial impact on the safety and durability of the structural member or structural component.

Therefore the evaluation of every damage type should account for:

- the type of damage and its affect on the safety and/or durability of the affected structural member
- effect of the affected structural member on the safety and durability of the whole structure (eg bridge) or structural component (eg span structure of a bridge)
- maximum severity of any defects on the inspected members
- extent and expected propagation of the damage on the observed members within a component.

A review of methods for condition assessment used in Europe and the USA showed that there are basically two approaches to the evaluation of the condition of the whole structure:

- ➢ The first one is based on a cumulative condition rating, where the most severe damage on each element is summed for each span of the superstructure, each part of the substructure, the carriageway and accessories. The final result is the condition rating for the structure, which can be used for a preliminary prioritisation of the structure (Chapter 8). The result of condition rating is shown in figure 3.8.

- The second method uses the highest condition rating of the bridge components as the condition rating for the structure itself. An example of this method is illustrated in figure 3.13 for the French inspection method and in figure 3.14 for the German inspection method. The results in these two examples do not represent real conditions but show graphically the results of the two methods.

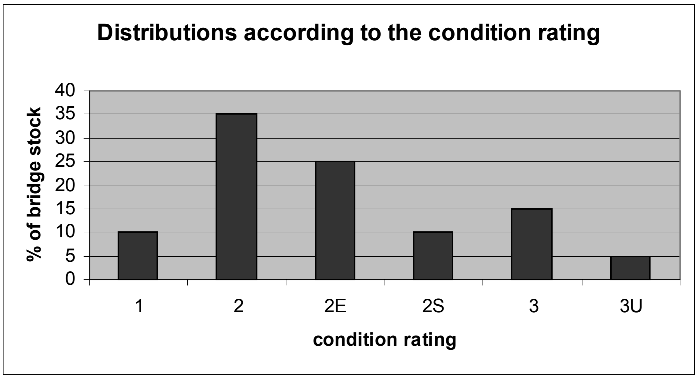


Figure 3.13: Distribution of bridges - French

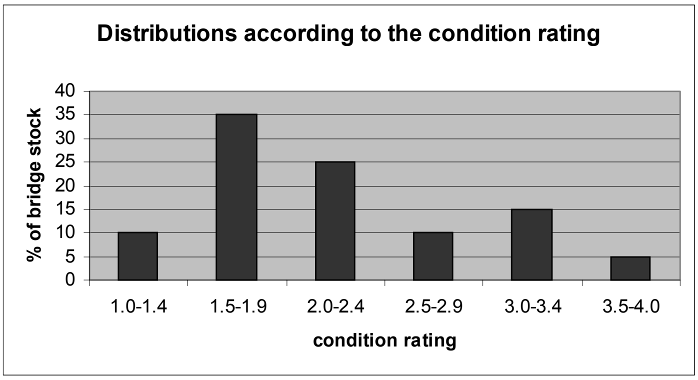


Figure 3.14: Distribution of bridges - German method

More detailed information on the methods used for condition assessment of individual elements, bridge components and the bridge structure as a whole is presented in Deliverable D2.

3.6 CONCLUSIONS

Condition assessment of bridge structures as well as other engineering structures on the road network, ie tunnels, retaining walls and culverts, has become increasingly important as structures age and begin to deteriorate. Methods for condition assessment require further development so that they can be used to monitor the deterioration of structural elements as well as the structure itself. More systematic collection of data is needed. In addition to traditional methods, new methods such as neural networks [Yun et al, 1998 and Hecht-Nielson, 1990] are being used for classifying the condition and assessing any deterioration or damage that has occurred. Further research is needed to improve the methods developed for categorising damaged areas. This requires the development of a larger database and the addition of new parameters. It is also necessary to ensure that inspection data is stored systematically.

3.7 REFERENCES

Grabec I (1990). *Self-organization of neurons described by the maximum entropy principle,* Biol. Cybern., Vol. 63, pp. 403-409.

Grabec I and W Sachse (1996). *Synergetics of Measurement, Prediction and Control,* Springer-Verlag, Heidelberg.

Hecht-Nielsen R (1990). *Neurocomputing,* Addison-Wesley Publishing Company Inc.

Mallett G P (1994) *Repair of Concrete Structures, State of the Art Review,* Thomas Telford.

Nakamura M, Masri S F, Anastassios G. Chassiakos A G and T Caughey (1998). *A Method for Non-parametric Damage Detection Through the use of Neural Networks*, Earthquake Engineering and Structural Dynamics.

Pennsylvania Department of Transportation (1987). *The Pennsylvania Bridge Management System, Final Report*, February 1987, Commonwealth of Pennsylvania, Department of Transportation.

Peruš, I., Fajfar, P and I Grabec (1994) *Prediction of the seismic capacity of RC structural walls by non-parametric multidimensional regression,* Earthquake Enfineering and Structural Dynamics, Vol. 23, pp. 1139-1155.

Quinlan J.R. (1979), *Discovering Rules by Induction from Large Collections of Examples,* Expert Systems in the micro-Electronic Age, pp.168-201.

Söderquvist M K (1998). *The Finnish Practice and Experience Regarding Bridge Inspection and Management*, The management of Highway structures, Highways Agency, 22-23 June 1998.

Yun C B, Bahng E Y and J H Yi (1998). *Neural Network Approach to Damage Assessment of Civil Structures*, Paper T131-2, Elsevier Science Ltd.

Znidaric J and I Perus (1997). *Condition rating methods for concrete structures*, CEB working group WP V/4.1, Final Report.

CHAPTER 4

ASSESSMENT OF LOAD CARRYING CAPACITY

4.1 SCOPE

The European road network is called on to carry steadily increasing heavy goods traffic, and, from time to time, increases in legal vehicle or axle loads. It contains many bridges built before modern design standards were established and consequently some would not comply if checked against those documents. Furthermore, as bridges grow older, deterioration caused by heavy traffic and an aggressive environment becomes increasingly significant resulting in a higher frequency of repairs and in a reduction in load carrying capacity. Insufficient load carrying capacity of structures may affect safety, traffic flows, environment and transportation costs. Because of these effects, it is necessary to have reliable but not over-conservative methods for the assessment of existing bridges.

The techniques used at present vary between countries. Most use a deterministic procedure and it is known that some aspects of the standards used are conservative and some structures possess reserves of strength that are not taken into account in the standards. Risk based procedures that take into account the risk of failure using bridge specific information such as actual traffic density and composition, the degree of redundancy as well as the consequences of failure are being developed in some countries.

Workpackage 2 'Structural Assessment' deals with the structural assessment input to the bridge management system, i. e. the calculation of the safe load-carrying capacity of bridges. The objective is to derive general guidelines based on current knowledge for the assessment of the load carrying capacity and to identify areas where improvements are required. To achieve this, the following tasks have been undertaken:

Task 1: Review of current assessment procedures including details on the characteristics of existing structures, standards used in design and experimental assessment methods.

Task 2: Review of different hypotheses for the use of material properties in assessment.

Task 3: Development of traffic models on the basis of existing traffic data.

Task 4: Introduction of reliability methods based on a probabilistic approach for bridge assessment including the use of measurements for updating the reliability of structural elements.

Task 5: Review of current experimental assessment methods, e.g. the use of load tests for bridge assessment.

Task 6: Provision of recommendations for methods and procedures that can be adopted for the assessment module of the management framework highlighting where further development will be beneficial.

In the following the main results of Workpackage 2 ”Structural Assessment” are presented. More detailed information is given in four background reports:

Deliverable D1: Review of current procedures for assessing load carrying capacity

Deliverable D5: Development of models (traffic and material strength)

Deliverable D6: Experimental methods and use of reliability techniques

Deliverable D10: Guidelines for assessing load carrying capacity

The reports have an important impact on the other components of BRIME. The review of current practice for assessing load carrying capacity presented in Deliverable D1 defined the starting point for Workpackage 3: Modelling of deteriorated structures and its Deliverable D11 Assessment of deteriorated bridges. Subsequently, the output from WP3 was used in the final output from WP2. The assessment of bridge strength and calculation of structural safety represents a significant parameter for priority ranking (as investigated by Workpackage 6: ”Priority ranking and prioritisation”) and the decision-making process (as investigated by Workpackage 5: ”Decision making, repair, strengthening, replacement”). Therefore the output of Workpackage 2 plays an important role in the development of an overall bridge management system (Workpackage 7: ”Systems for bridge management”).

The symbols used in this chapter are as follows:

S	stress
R	strength
P_f	probability
R_d	characterictic value of R
S_d	characterictic value of S
γ_R, γ_S	partial safety factors
HGV	heavy goods vehicle
ADT	average daily traffic
T	reference period
R_T	return period
α	fractile value
a, u, x, y, z	satistic parameters
Fx(x), Fy(y), Fz(z)	distribution functions
fx(x), fy(y), fz(z)	density functions
Mf1	bending moment at midspan (one span beam)
Mf2	maximum field moment (two span beam)
Ms	bending moment at the inner support
ΔS	difference of load effects.

4.2 REVIEW OF CURRENT PROCEDURES

4.2.1 Introduction

An initial step to achieve the objectives of Workpackage 2 involved taking stock of the procedures and techniques currently used by countries participating in the EU Bridge Management in Europe (BRIME) project. On the basis of a survey carried out among the participating countries, fundamental information was first obtained on the types of structures found in the national bridge stocks, the structural condition and types of deterioration present, and the national codes (Standards, Advice Notes, Guidelines, etc) used for determining load carrying capacity. The survey included information on the tasks and objectives relating to the determination of load carrying capacity, as well as the principles and subsequent computation processes. In addition, clarification was required as to which experimental methods, including non-destructive testing (NDT) are employed on site and what laboratory tests are used to support the analytical processes. The questionnaire form used for this survey is included in Deliverable D1.

The material gathered and supplied by the participating countries was evaluated and organised in terms of common features, differences, and national characteristics. The results are presented in detail in Deliverable D1. A summary of the results is provided in sections 4.2.2 and 4.2.3.

4.2.2 Existing bridge stock

The bridge inventory across Europe forms the objective basis for developing relevant techniques for structural assessment, which can be standardised for future implementation on an international scale. It is expected that differences in climate, environmental effects, construction practices, etc, in the different countries will give rise to different problems, which will have an effect on how bridges are managed. It is useful therefore to examine information on the composition of the bridge stock, and the condition of the bridges in terms of structure type, geometric relationships etc. The data listed in Table 4.1 refer in general to national highway and trunk road networks. In most cases, this is only a proportion of the overall bridge stock, as bridges managed by local authorities are not included.

Table 4.1: Bridge stock of national highway networks in the participating countries

Country	France	Germany	Norway	Slovenia	Spain	UK
Number	21549	34824 1)	9163	1761	13600 2)	9515 3)
Area [1000m²]	7878	24349 1)	2300	660	5526	5708 3)

1) Number of bridges recorded until 1997, highway and trunk road network only
2) Number of bridges recorded until 1996
3) Bridges owned by the Highways Agency, i.e., in England only.

In France and Germany, there is a predominance of reinforced concrete over prestressed concrete for highway and trunk road bridges as far the number of structures is concerned, although this order is reversed when bridge areas are considered. This is because the longer span concrete structures tend to be prestressed. The proportion of steel and steel composite bridges in all countries is notably lower. An evaluation of the number of structures showed that bridges less than 20 years old predominate except in the UK. With the exception of Slovenia, bridges more than 40 years old are rarest. It should be remembered that the large number of short-span bridges found on rural and urban roads are not considered in this statistics.

Division in accordance with bridge length reveals a nearly equal distribution among all countries. The proportion is more than 80 per cent for lengths of up to 50m, and considerably more than 90 per cent for lengths of up to 100m. These common features prove favourable for determining marginal conditions and the validity of the traffic-load simulation model developed in Task 3: Traffic loads. For detailed information refer to Deliverable D5.

The basis of all maintenance strategies is the acquisition of information on the current condition of structures. For this purpose, structures are inspected in accordance with specific standards at regular intervals. The occurrence and extent of damage and defects found at successive inspections are used to determine deterioration rates. These, in turn, serve as criteria for making decisions concerning maintenance and rehabilitation measures (refer to Deliverable D2 Review of current practice for assessment of structural condition). For the purpose of avoiding damage, however, it is even more important to obtain conclusions concerning the causes of different types of deterioration. These findings can be used to update and supplement existing sets of technical rules for design, construction, repair and rehabilitation of bridges. Deterioration can be subdivided into 3 different types:

- deterioration arising from faults in design, building materials or components
- defects due to the construction method or occurring during the production process
- deterioration caused by external influences.

The most common defects found in highway bridges are cracking, staining and spalling of concrete resulting from the corrosion of the reinforcing steel. These defects can occur even in areas that appear to be protected from detrimental environmental conditions. The absence, or failure, of a waterproofing membrane may allow the migration of salt-laden water to even the most protected areas. Spray from passing vehicles can result in deterioration of bridge elements well above the road surface. Bridge expansion joints contribute seriously to the problem, particularly where poor detailing, inadequate joints or lack of proper maintenance allows the run-off to flow over the sub-structure. As a result, few bridge components are safe from chloride attack. A particular problem has been the corrosion of prestressing tendons in post-tensioned structures.

In Workpackage 2 only a first view on the problem of deteriorated bridges was undertaken. It was dealt with in more detail in Workpackage 3: Modelling of deteriorated structures. Information on the effect of deterioration on strength and how it can be taken into account in assessment is presented in Deliverable D11: Assessment of deteriorated bridges.

4.2.3 Assessment procedures and standards

The review confirmed that there are significant differences in procedures and methods used for bridge assessment by the partner countries. This includes the reasons for initiating a bridge assessment, which can be summarised as:

- when there is a need to carry an exceptional heavy load
- where the bridge has been subjected to change such as deterioration, mechanical damage, repair or change of use
- where a bridge is of an older type built to outdated design standards or loading and has not been assessed to current standards.

Bridge assessment in the partner countries generally relies on orthodox structural calculations in which the load effects are determined by structural analysis and the corresponding resistances are determined by code-type calculations. Reliability calculations are beginning to be introduced in which a target reliability index is the governing factor.

Currently, the rules used in bridge assessment are provided mainly by design standards with additional standards relating to testing methods including load testing. In some countries, the design standards used can be either the current standards or those that were current at the time of construction. Current design loading specifications can be used, although in some cases these may modified specifically for assessment and can include a reduced load level based on restricted traffic conditions. Additional requirements can be given regarding exceptional traffic loading.

Design standards are mainly based on two alternative approaches: the allowable stress design as prescribed in the German codes and the partial safety factor design as prescribed in the French, UK and Eurocode documents.

In the UK, assessment standards have been developed by modifying the design standards. The modifications provide more realistic formulae for member resistance, allowances for non-conforming details and imperfections, and methods for incorporating in situ material strengths in calculations. Five levels of assessment are provided of increasing sophistication that may be applied when a simple assessment (Level 1) indicates that the bridge is sub-standard [Highways Agency 1997 and 1998].

In other partner countries, there is minimal official documentation for assessment although Germany has an assessment standard for bridges in the former states of East Germany. Norway has provision for assessment loading and Slovenia has been developing a reliability method for assessment. Spain uses the design documents for assessment, as does France, but in the latter case there is flexibility to reduce partial factors or improve structural or resistance models with the help of laboratory or site measurements.

For more information on the structural principles of assessment, limits in using design methods for structural assessment and assessment steps when using a probabilistic approach, refer to Deliverable D1.

4.3 USE OF RELIABILITY TECHNIQUES

For many centuries, the builder was left to his own intuition, to his professional ability, to his experience and to that of his predecessors (the limits often being determined by the observed accidents or collapses) for designing structures. Such empiricism however did not allow the design of new structures with new materials. The emergence of the science of building, with the mechanics of structures and the strength of materials, occurred only much later and very gradually. The disappearance of empiricism to the benefit of engineering sciences was largely served by the development of steel construction. However, even at that stage, the concept of "structural safety" was not yet mentioned in the technical literature and the use of reduction factors applied to strength appeared to be the true expression of safety. The adopted safety principle consisted in verifying that the maximum stresses calculated in any section of any part of a structure, and under worst case loading, remained lower than a so-called allowable stress. The design method based on the principle of allowable stresses was used in the first part of this century without the definition of these allowable stresses really being considered. Their values were set arbitrarily on the basis of the mechanical properties of the materials used. Allowance for improvements in the production of steel, as well as in the design and construction of structures led to the raising of the allowable design stresses. Attempts to improve the design rules based upon the allowable stress principle to obtain a better definition of loads and strengths revealed the scattered nature of the data and of the results.

The need to use tools dealing with these variabilities became obvious. Furthermore, failure stresses were not necessary the most appropriate quantities. In fact, two problems were identified by using the allowable stress principle for assessing structural safety:

- to replace the criteria of allowable stresses by other criteria such as limit states
- to rationalise the way to introduce safety.

For this reason, many engineers have tried to approach the problem from a different point of view by defining safety by means of a probability threshold. Under the stimulus of some engineers and scientists, the concept of probabilistic safety of structures was born. However, it was not until the Nineteen-Sixties and Seventies that mathematical tools were developed for studying the reliability of structures.

In a probabilistic approach, the stress *S* applied to a structural element, and the variable characteristic of the strength *R* of this element, are randomly described because their values are not perfectly known. If the verification of the criterion related to the limit state results in the inequality:

$$S \leq R \tag{4.1}$$

the failure of the component being related to the fact that this limit state is exceeded. The probability P_f of the event $S \leq R$ will characterise the reliability level of the component with regard to the considered limit state:

$$P_f = \mathrm{Prob}(R \leq S) \tag{4.2}$$

The semi-probabilistic approach used in many design codes schematically replaces this probability calculation by the verification of a criterion involving characteristic values of *R* and *S*, noted R_d and S_d, and partial safety factors γ_R and γ_S which may be represented in the following form:

$$\gamma_S S_d \leq \frac{R_d}{\gamma_R} \tag{4.3}$$

The partial safety approach is described as semi-probabilistic, considering the application of statistics and probability in the evaluation of the input data, the formulation of assessment criteria, and the determination of load and resistance factors. However, from the designer's point of view, the application of the partial safety approach in specifications is still deterministic. The partial factors approach does not provide relationships or methods that would allow the designer to assess the actual risk or reserves in carrying capacity of structural members resulting from the semi-probabilistic procedure.

Partial safety factors are designed to cover a large number of uncertainties and may thus not be highly representative of the real need for evaluating the safety of a particular structure. For exceptional or damaged structures, the evaluation of reliability may be overestimated or underestimated. The introduction of uncertainties appear to be needed to rationalise the evaluation of safety. This is motivated by various reasons:

- the evolution of loads with time is often not handled
- the properties of materials are also liable to evolve in an unfavourable direction, for example through corrosion, loss of durability or fatigue

- the combination of multi-component load effects is badly introduced (such as the combination of normal and moment effects)
- real elements are often different from the specimens on which their performance was measured
- studies on sensitivity to errors in modelling the behaviour of structures are generally omitted
- poor workmanship is unfortunately statistically inevitable
- construction requirements discovered when the works are being carried out may lead to alternative solutions which bring about an overall behaviour of the structure slightly different from the one provided for in the design.

A method taking into account uncertainties on variables appears to be a realistic safety assessment criterion. Therefore, probabilistic methods today constitute an alternative to semi-probabilistic approaches. They are based on:

- the identification of all variables influencing the expression of the limit state criterion,
- studying statistically the variability of each of these variables often considered to be stochastically independent
- deriving their probability function
- calculating the probability that the limit state criterion is not satisfied
- comparing the probability obtained to a limit probability previously accepted.

These methods are generally grouped under the name of *reliability theory*. Although extremely attractive, the probabilistic reliability theory is limited by many factors:

- some data are difficult to measure
- required statistical data often do not exist
- probability calculations quickly become difficult to manage because of their amount.

These considerations are decisive for determining what may be expected from the limits of probability. They imply in particular that the probabilities suffer from the fact that they are only estimates of frequencies (sometimes not observable) based upon an evolving set of partial data. They also result from hypotheses (choice of type of distribution, for example) which make them conventional. Consequently, the outcome of a probabilistic approach depends strongly on the assumptions which are made about the uncertainties associated with variables. If these assumptions are not founded on adequate data, estimates of safety will be misleading. Indeed, probabilistic methods are often abused when variables are not carefully modelled. It is therefore essential that the quality of data and validity of assumptions are borne in mind when using a probabilistic approach to make decisions about the apparent safety of a structure.

The widely used methods of bridge assessments are sometimes considered to be unduly conservative. New more sophisticated methods are therefore needed, and the reliability theory has so far been used only in a limited manner although the potential benefits are considerable. It has remained a method for experts and most of the applications, at least for bridges, have been in the context of design code calibration. However the method is being used increasingly for the assessment of existing bridges, particularly for investigating optimal maintenance strategies. It is why Deliverable D6 provides general bases and examples for the use of reliability techniques in bridge assessment. Such an approach is also used as level 5 assessment in the set of recommendations from Workpackage 2 (see section 4.5).

4.4 DEVELOPMENT OF MODELS

4.4.1 Introduction

For the structural assessment of existing bridges, several procedures are practicable, which differ with regard to amount and complexity. First of all, the true loads and present structural strength have to be clarified. Beside this the format expressing those variables will differ according to the assessment approach (i.e. deterministic, semi-probabilistic, probabilistic).

To limit the amount of work and complexity of an assessment it is obviously sensible to start with simple assumptions and methods and to refine the investigation by steps in case of need. Clearly, the simple methods adopted should be conservative, and consequently it may be assumed that the more refined methods will yield a higher assessed capacity. Simple and moderately refined methods will use deterministic or semi-probabilistic methods. These may be insufficient, if they deliver too low a carrying capacity, or if the characteristics of the structure give rise to concerns that these methods may be non-conservative. In that case, probabilistic methods become more attractive in spite of the extra complexity and amount of work.

In that case, the essential parameters that characterise structural resistance or applied loads, cannot be defined solely in terms of characteristic values reduced by partial safety factors. They must be defined in terms of random variables characterised by means and moments. Indeed, if a full reliability analysis has to be performed, one has to express them in terms of statistical distributions (refer to Deliverable D6).

To describe adequately the resistance properties of structural elements, following information is required [Melchers 1999]:

- statistical properties for material strength and stiffness
- statistical properties for dimensions
- rules for the combination of various properties (as in reinforced concrete members)
- influence of time (e. g. size changes, strength changes, deterioration mechanisms such as fatigue, corrosion, erosion, weathering)
- effect of "proof loading", i.e. the increase in confidence resulting from prior successful loading
- influence of fabrication methods on element and structural strength and stiffness (and perhaps other properties)
- influence of quality control measures such as construction inspection and in-service inspection
- correlation effects between different properties and between different locations of members and structure.

Relatively little information is available in statistical terms, mostly for the first three items. Useful summaries of time-independent statistical properties for reinforced and prestressed concrete members, metal members and components, masonry and heavy timber structures are given in the literature (see Deliverable D2). To illustrate the underlying thought processes, Deliverable D5 provides a review of the statistical properties of structural steel and concrete.

Concerning loads the activities of Workpackage 2 only cover traffic loads, because this type of action is particularly important for bridges. Carrying this load is the primary function of a bridge and when it is deficient, the capacity of the bridge is reduced and traffic restrictions or other remedial measures are required. Traffic loads are subjected to changes in time. Therefore they differ from permanent loads, which remain constant during the service life, if no substantial intervention on the structure was performed. The increase of traffic loads has to be taken into account in the load model, because bridges may reach a service life over 100 years. This is done by the application of extreme traffic situations and the definition of a sufficient safety level. From time to time it should be checked to ensure that the load standard covers the actual traffic. When this occasion arises, the code must be revised. Revision must be considered in due course when the EC1 load model is introduced in the individual European countries.

Not every existing bridge is exposed to extreme traffic loads and the frequency of high traffic loads can be lower during the remaining or planned service life. These points should be taken into account in developing a load model for the structural assessment of existing bridges. For bridges that are to be used for 10 or 20 years, it is not appropriate to calculate loading on a basis of a 1000-year return period for extreme loads, the basis for the load model of EC1.

One aim of Workpackage 2 was to show how appropriate and realistic assumptions for material and structural properties and traffic loads can be obtained at project level and used for structural assessment.

4.4.2 Resistance modelling

The choice of stochastic models for resistance variables such as yield strength and modulus of elasticity can be based on information from a number of sources:

- experimental results/measurements: based on such data statistical methods can be used to fit probability density functions, see below. One main problem with fitting probability density functions on the basis of experimental results is that usually most of the data are obtained in the central part of the density function whereas the most interesting parts from a reliability point of view are the tails. For a resistance variable, the lower tail is of interest and for a load variable the upper tail.

- physical reasoning: in some cases it is possible on the basis of the physical origin of a quantity modelled as a stochastic variable to identify which stochastic model in theory should be used. Three examples of this are described below, namely the normal, the lognormal and the Weibull distributions. When a stochastic model can be based on physical reasoning the above mentioned tail sensitivity problem is avoided.

- subjective reasoning: In many cases there are not sufficient data to determine a reasonable distribution function for a stochastic variable and it is not possible on the basis of physical reasoning to identify the underlying distribution function. In such situations subjective reasoning may be the only way to select a distribution function. Especially for this type of stochastic modelling it can in the future be expected that for the most often stochastic variables there will be established code based rules for which distribution types to use.

The probabilistic description for the strength or other properties of structural members ends on the probabilistic description of component properties for the member(s), such as cross-sectional dimensions and material strengths. When probabilistic properties for the members are derived using mathematical relationships, differences between the derived result(s) and field or experimental results would be expected. In part this is due to inherent variability in experimental techniques and observations. The greater part of the difference. however, is the result of the simplification(s) introduced by the mathematical model which relates material and geometric parameters to structural element behaviour. For example, in deriving an expression for the ultimate moment capacity of a reinforced concrete beam section, it is well known that assumptions are made about the concrete compressive stress distribution, about the form of the stress-strain relationships for the reinforcement, about the concrete tensile strength, etc. These assumptions usually are conservative. However, they add a degree of uncertainty to the transition from individual parameters to member strength. This variability is known variously as the 'modelling' uncertainty or the 'professional factor'. It does not arise, if statistical properties of a structural member are obtained directly from 'extensive' experimental observations on the member itself. However, such tests are not always practical and recourse may have to be made to modelling the member behaviour mathematically and using as input data information about the material and geometric probabilistic properties.

4.4.3 Assessment of traffic loads and load effects

4.4.3.1 Design and assessment codes

Following the above-mentioned multi-level process of structural assessment, it is suggested to apply specific traffic load models of actual design codes for the basic level of assessment. The advantage is that in most cases these load models are simple and practicable. Frequently the results of load effect calculations from the design stage are in hand. Thereby assessment calculations can be simplified and shortened. In this way a first orientation value of the existing structural safety can easily be obtained.

The disadvantage of the performance of a design code load model is, that possibly the assumptions are far to unfavourable for the structure which is in question. Therefore it depends on the assessment results if more precise load assumptions and refined methods must follow.
Actually in the countries of the BRIME-consortium (with the exception of UK) the load models of national design codes are usually applied for assessment. Nevertheless in future the main load model 1 of EC1 [Eurocode 1 1995] will be standard for design, when the Eurocodes are obligatorily introduced by the EU-countries in combination with National Application documents (NAD).

The UK is the only one of the countries participating in BRIME to have an established procedure for bridge assessment supported by a comprehensive set of documents. The assessment standards for each type of structure are based on the corresponding design codes for steel, concrete and composite bridges. The principles are identical, except that the bridge engineer can expect to be able to produce a more realistic strength evaluation by taking advantage of information which was not available at the design stage.

Many conservative measures built into the design codes have been modified for assessment purposes. Bridge design loading takes account of all possible uses of the bridge, irrespective of local conditions and includes the effects of impact, lateral bunching of traffic, overloaded vehicles and various load combinations. It also includes an allowance for future development and an increase in vehicle weights.

For assessment this allowance is not applied. In addition bridge-specific loading can be used as defined in BD21 [Highways Agency 1997] which allows reductions in loading for low traffic flow and good road surface condition. To take advantage of this provision, road surface and traffic conditions must be determined.

4.4.3.2 Stochastic simulation - methodology

Unfortunately simple load models fixed in existing codes disregard the real actual traffic which is required by higher level methods of structural assessment. An artificial traffic, which represents actual conditions with sufficient accuracy as a basis for simulation of load effects can be described by using the stochastic simulation method, well known as Monte Carlo Method, if sufficient traffic and structural data is available.

Beside structural data (static system, transverse load distribution, dynamic parameters of the structure) the following parameters describing the heavy goods vehicle (HGV) traffic are needed:

- frequencies of gross weight and axle loads for each type of vehicle
- frequencies of vehicle types
- characteristics of traffic flow (flowing traffic, traffic jam)
- distances of vehicles
- average daily HGV-traffic

On the basis of the given parameters, the simulation results in a load time function which is evaluated by specific algorithms. For simulation of load effects the extreme values for a given reference period T are of major interest. The repeated simulation supplies a random test with n extreme values, from which the parameters of an appropriate frequency distribution can be calculated.

The distribution of extremes belongs to the type I, well known as Gumbel distribution, if the starting distributions are of a normal – or exponential type:

$$F_X(x)=\exp(-\exp(-a(x-u))) \qquad (4.4)$$

This assumption is justified because the gross weight of vehicles is bi- or trimodal normal distributed (refer to Figure 4.1) respectively it can be approximated by such distributions. Parameters a and u must be determined by the random test.

The basis of structural assessment should be the determination of characteristic values due to traffic loading for a given reference period T. Generally these are fractiles of distribution of extremes for traffic loads or load effects. The mean return period R_T represents the period in which a given level is exceeded once in a mean (e. g. service life of the structure)

$$R_T \approx T / \alpha \qquad 0 < \alpha << 1 \qquad (4.5)$$

Where α represents a fractile value (e. g. 0,02).

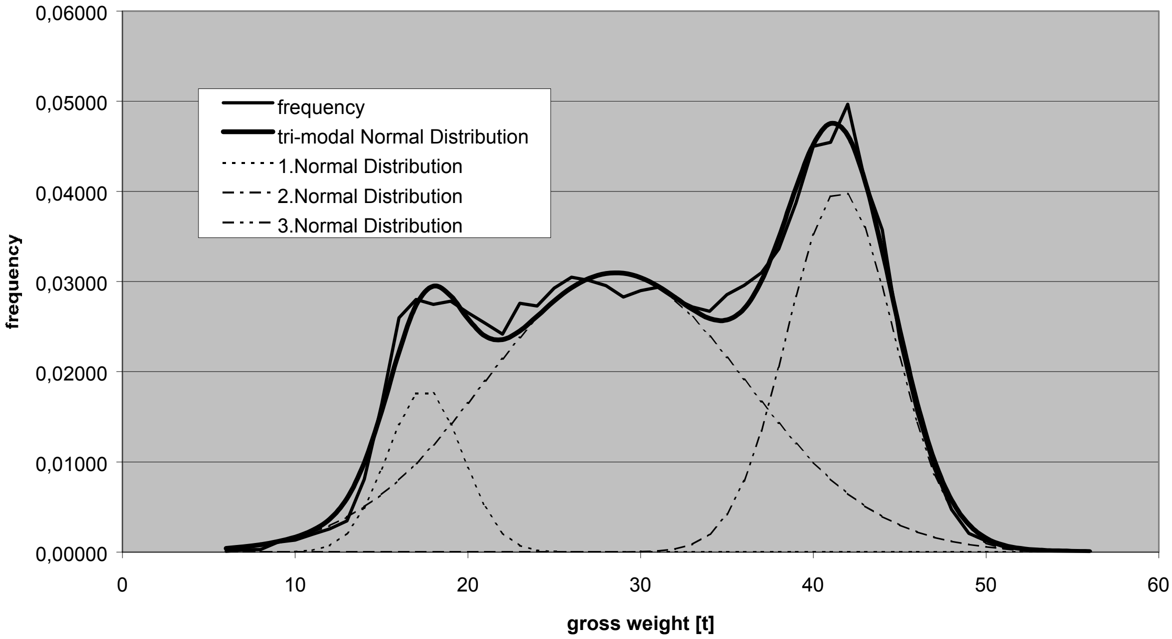


Figure 4.1: Tri-modal normal distribution of vehicle gross weight

Likewise the method of stochastic simulation can be used to determine distributions of extremes for gross weight of individual vehicle types. The knowledge of distribution functions of gross weight, which can be determined e. g. from data given by weight in motion – measurements is prerequisite. The simulation results in an artificial random test of extreme values for a given reference period T. Because of the normal distribution of gross weight, the distribution of extremes can be determined according to equation (4.4) by regression from the random test. The resulting distribution of extremes can also be extrapolated for longer periods with the help of equation (4.5).

Usually stochastic problems are solved by application of analytical methods. There are different methods available. Especially the method of asymptotic distribution of extremes is suitable to solve the above mentioned problems. This method is described in detail in Deliverable D5 where also additional information on the stochastic simulation method is given.

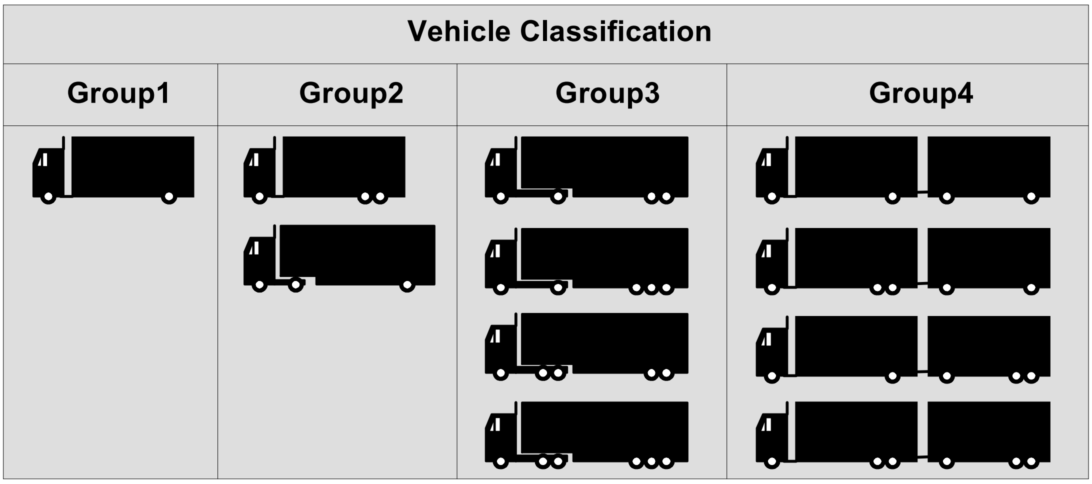
4.4.3.3 Stochastic simulation - application

The traffic loading applied in the performed investigations is based on actual axle load measurements performed in the German highway network [Federal Highway Research Institute 1998]. The composition of heavy goods vehicle traffic for French and UK-conditions was developed on the basis of condensed published data [Cremona and Jacob 1995, Ricketts and Page 1997]. In the French data-files the different vehicle types were combined to specific groups. To guarantee comparable conditions, this classification was also used for the German and the UK data. For each vehicle group a representative vehicle type was determined. Results for long distance traffic are given in Table 4.2 as percentage of the whole traffic flow.

Table 4.2: Frequencies of vehicle groups

	Group1		Group2		Group3		Group4	
FRANCE	25	(20-25)	0	(<5)	65	(65)	10	(10-15)
GERMANY	23	(21)	0	(4)	47	(46)	30	(29)
UK	35	(32)	0	(4)	65	(61)	0	(3)

generalized values



In Deliverable D5 the calculation method and the results of extensive investigations for different traffic and structural data are described in detail. Simulation of load effects (bending moments at midspan, bending moment at the support, shear force at the support) was carried out with the help of the computer program REB [Geiβler] on one and two span slab and beam bridges for different span lengths. The distribution of daily extremes result from 900 random tests by an adaptation to an asymptotic distribution of extremes type I (Gumbel) according to equation (4.4).

The application of (4.4) results in a forecast of annual extremes. The 98%-fractile of load effects, which is the basis of the following investigations, corresponds to a average return period of 50 years. This period is used in a lot of national codes for the characteristic load value.

Table 4.3: Fractile value of bending moment at midspan for different traffic combinations

Szenario	98%-Fraktil [kNm]
FRANCE	2404
GERMANY	2430
UK	2324

From Table 4.3 follows that the influence of different combinations of HGV-traffic on the fractile values of load effects is comparatively low. The given bending moments at midspan were calculated for a single span beam with a span length of 20m and a HGV frequency of 10.000 vehicles per day.

Figure 4.2 shows a comparison of simulated load effects for different span lengths to load effects calculated by taking the EC1 load model into account. Mf1 is the bending moment at midspan (one span beam), Mf2 is the maximum field moment (two span beam) and Ms is the bending moment at the inner support. The situation ”traffic jam” was calculated for a probability of 1% of the HGV-traffic. Whereas the simulated field moments show distinct reserves in any case, the bending moments at the support may exceed the EC1-values. That applies especially to the ”traffic jam” situation for span lengths of more than 40 m.

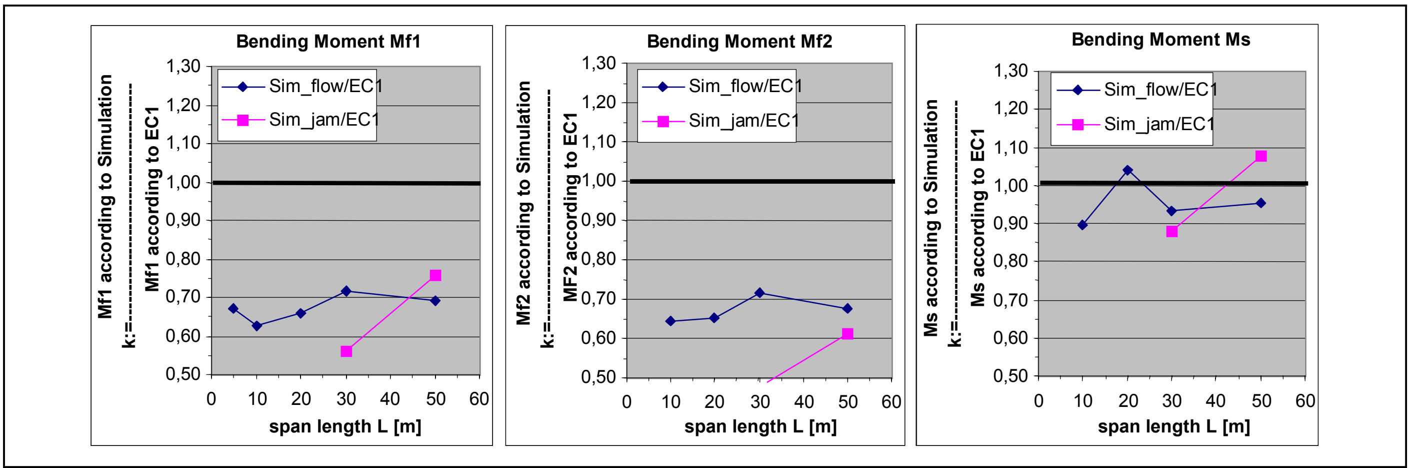


Figure 4.2: Comparison between simulation and EC1

The influence of different object-related HGV-frequencies can be considered in an easy way by taking equation (4.6) into account where ΔS represents the difference of load effects (e. g. ΔMf1):

$$\Delta S = [\ \ln(N_1) - \ln(N_2)\ ] / a \qquad (4.6)$$

With:
- a — parameter of distributions of extremes for annual extreme values
- N_1 — number of HGV per year
- N_2 — $365 * 10^4$ (number of HGV per year used in simulation)

4.4.3.4 Approximation of extreme load effects

As an alternative to the method of stochastic simulation the fractile values of relevant load effects can be approximated. The method is described in detail in Deliverable D5. The basic idea is that for those static systems where only one HGV on every lane has to be positioned due to the shape of the influence line and the geometric extent the extreme loading effects can be determined by using the extreme vehicle gross weights. For example that applies for the bending moment at midspan of a one or multi span beam with a span length up to 30m, because for this the situation ”traffic flow” is authoritative. It is presupposed that HGVs in lane 1 and 2 are of the same type and are positioned as a ”vehicle packet”. Then gross weights for vehicles on two lanes can approximated by taking the distribution of loads in transverse direction into account. For this a fractile value can be calculated from the vehicle gross weights by taking into account formula (4.7):

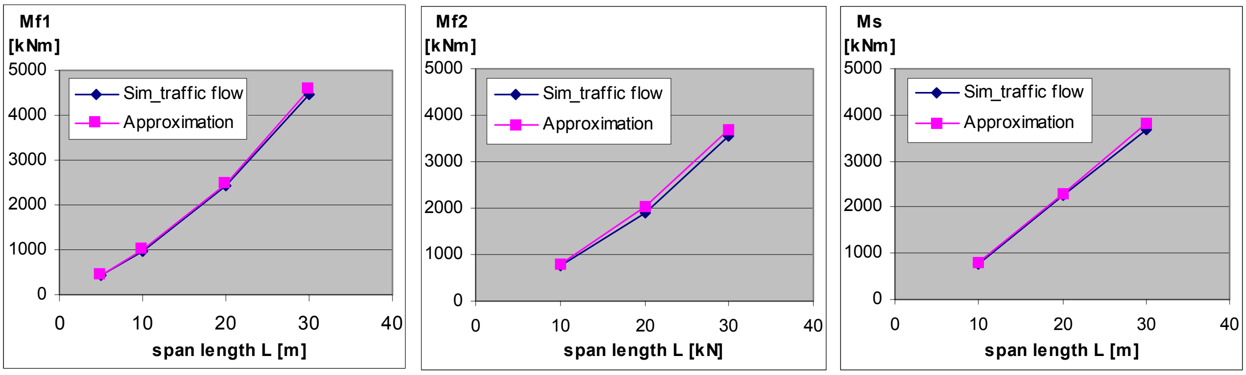
$$P(Z < z) = F_Z(z) = \int_0^z F_X(z - y) * f_Y(y) * dy \qquad (4.7)$$

$F_X(x)$, $F_Y(y)$ and $f_X(x)$, $f_Y(y)$ are accompanying distribution and density functions (in this connec-tion distribution of extremes of gross weight for both lanes) and $F_Z(z)$ is the distribution of the "vehicle packet". 10000 HGV/day, a simplified German traffic composition and a 10% proportion of HGV in lane 2 result in the loads according to Table 4.4. Beside this Table 4.4 gives the fractiles of gross weight for a single vehicle (group 3).

Table 4.4: Authoritative extreme values of gross weight [kN]

Span length L [m]	5		10		20		30		50	
Distribution of loads in transverse direction (bending moment)	η1	η2	η1	η2	η1	η2	η1	η2	η1	η2
	0,90	0,10	0,85	0,15	0,75	0,25	0,70	0,30	0,65	0,35
98% fractile of gross weight [kN] ”vehicle package”	612,4		607,9		603,4		601,5		599,8	
98% fractile of gross weight [kN] single vehicle, lane 1	600,3		567,0		500,3		466,9		433,6	

Figure 4.3: Comparison of results from simulation and approximation



By taking into account these rates and actual vehicle data the bending moments at midspan and at the support are given in Figure 4.3 for one and two span beams and span length L up to 30 m. As a basis for the approximation for bending moments in the case of L = 20 and 30 m it was assumed that an additional ”extreme” HGV is positioned in the neighbour span of lane 1. The authoritative weight of a single vehicle results from the multiplication of the transverse distribution index and the fractile value of the gross weight. As Figure 4.3 shows load effects given from simulation and approximation correspond sufficiently.

For span lengths up to 30m the given approximation method makes it possible to calculate load effects as a result of traffic, which are necessary for structural assessment, without complex individual simulation, provided that fractile values of vehicle gross weight are well known

Depending on transverse load distribution the 98% - fractile values (α=0,02) for gross weight of the vehicle packet (lane 1 + 2) and a single vehicle in lane 1, which are given in Deliverable D5, result from the distribution of gross weight produced for the actual long distance traffic. The load effects have to be calculated by taking these values into account.

The influence of lower vehicle densities may be considered by formula (4.5). However only the number of HGV of the authoritative vehicle group has to be used for determining the fractile value of gross weight. Values equivalent to those given in Table 4.4 can be produced e. g. from formula (4.6), if the distribution of gross weight differ.

4.4.3.5 Conception of a static load model

As shown by the evaluation of results obtained from simulation, it is convenient, to take into account the two individual traffic situations ”flowing traffic” and ”traffic jam” separately. However this means for a general concept of presumed traffic loads, which covers both situations, a subdivision into two separate load models. For this the following assumptions are taken:

A two lane traffic is assumed in principle. Overtaking traffic by HGV in lane 2 shall take place occasionally (i.e. 10% probability). In the most unfavourable case two HGV are positioned side by side as a packet. Only HGV of group 3 according to Table 4.4 are used as traffic load, because they produce the highest fractile values of gross weight and show the highest load concentrations.

For flowing traffic only in lane 1 further HGV are positioned in front of and behind the vehicle packet in a distance of 12 m, which represents about one vehicle length. The real loading occurs by axle loads in the case of the vehicle packet and by distributed load for the other vehicles considering a load distribution of 12 m according to the length of the vehicles. All traffic loads are multiplied with a dynamic factor according to appropriate national codes, if no structure related rates are given.

In the case of traffic jam the chain of vehicles is condensed in lane 1, so that in front of and behind the vehicle packet a continuously distributed load arises in a distance of 0,5 m. Load distribution of single vehicles occurs at vehicle length plus 2 $*$ 0,5 m $\approx$ 13 m.

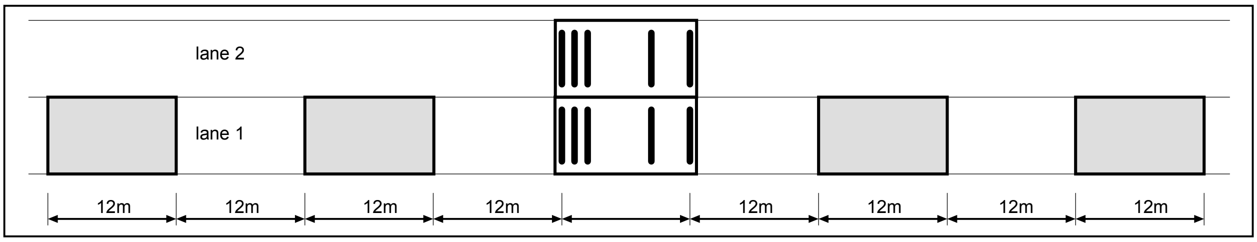


Figure 4.4: Load concept for flowing traffic

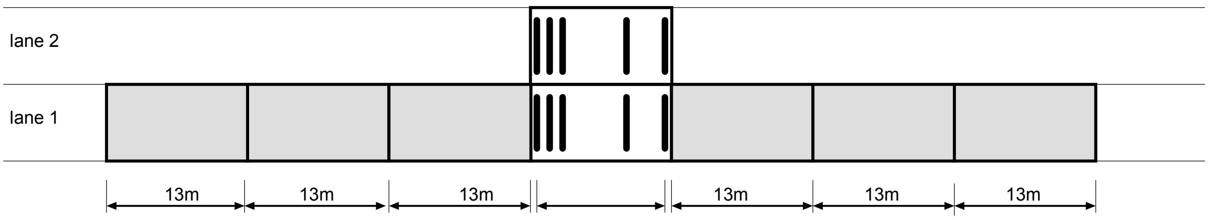


Figure 4.5: Load concept for traffic jam

Bending moments at midspan for the single span beam and bending moments at the support for the two span beam were calculated with the help of the above mentioned load concept for span length of 5 m to 50 m and compared with the results obtained from simulation. The comparison of load effects given in Figure 4.6 shows a good correspondence for span length up to 30 m. For increasing span length, the bending moments differ for the two situations flowing traffic and traffic jam. The reason is that gross weights of vehicles and their distances were chosen uniformly, but actually they are random. However it is guaranteed that for each span length the approximate values are higher than the simulated rates. It is obvious that the bending moments at midspan show the same good results in the case of the two span beam, because there is nearly no difference between the shape of the influence line for the one span and the two span beam in the authoritative field section. With regard to the exactness of the method the same assessment can be given for the shear force.

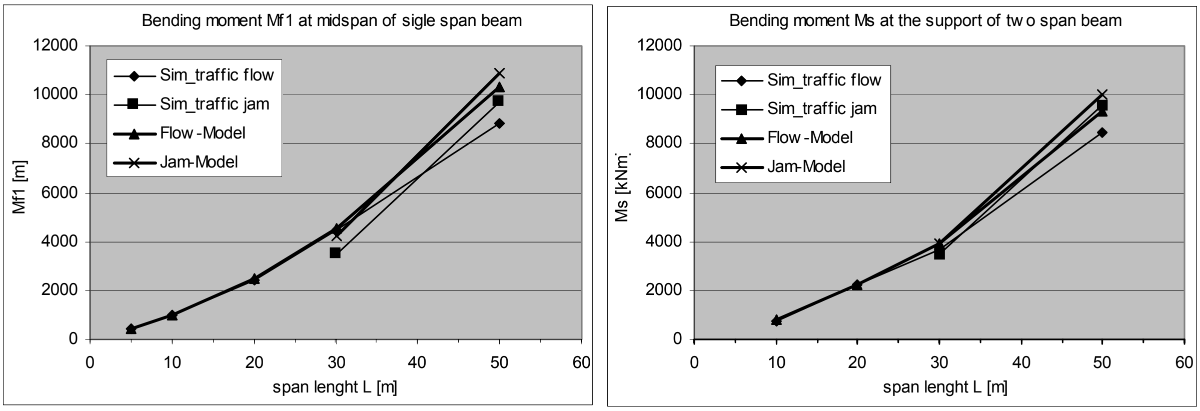


Figure 4.6: Comparison of results from simulation and load conception

The given load concept represents an useful approach for further investigations, in which a number of parameters must be evaluated, e g:

- further cross section types
- additional load effects
- changed distribution of HGV traffic on the individual lanes
- distributed loading outside the lanes and on the sidewalk

Additionally it has to be proved if the load configuration can be further simplified.

4.5 GUIDELINES FOR ASSESSMENT

4.5.1 Introduction

Deliverable D10 describes the proposed Guidelines for assessment that constitute the final output of Workpackage 2. The D10 report contains the Guidelines and its Appendix contains background material and a discussion of the provisions. A brief summary of D10 is given in this section.

All countries carry out bridge assessments but their methods and procedures differ. In general terms, the process is well known, and many of the principles and objectives identified are self-evident. The Guidelines have been compiled using the best practices and procedures found in the BRIME partner countries. They are written in the form of proposals recommended to the relevant Technical Authorities in the countries concerned. They are not intended to be a manual for use by engineers to assess bridges.

Bridge assessment can be a stand-alone process for a single bridge or an integral part of a bridge management system. The requirements are very similar, but differences arise from the greater need for uniformity, efficiency and information recording when a system is applied to a network of roads and bridges.

D10 contains details of the proposed objectives, principles and methodology that are recommended to BRIME partners. Some of the terminology defined in D10 is given below in simplified form.

Technical Authority is the authority responsible for technical standards for the bridge or bridge stock in question.

Assessment is a set of activities used to determine the safe load-carrying capacity of an existing structure and *Universal Assessment* refers to the principle of assessing or providing a proxy for an assessment for all bridges within the bridge stock. Assessment can be carried out at one of a number of *Assessment Levels* of increasing refinement that can be applied successively in the assessment of a bridge, normally when the lower levels are insufficient.

The *Load Carrying Capacity* of a bridge is the traffic load that can be carried in combination with other loads/actions where appropriate. A traffic-loading model used for bridge assessment is referred to as *Assessment Live Loading* which may be less onerous than design loading. *Bridge-Specific Live Loading* is a traffic-loading model devised for a specific bridge using measured site-specific data, e.g. weigh in motion or traffic flow. *Reduced Live Loading* is a reduced traffic-loading model to cater for the assessment of bridges that are to be restricted in use.

A bridge is *Sub-Standard* when it has failed an assessment. The exact definition of the term will depend on national practice but it may include the use of a *Capacity Index*, which is a ratio quantifying the extent of the deficiency or reserve of strength in a member (or whole structure). *Monitoring* is the process of observing a Sub-Standard bridge in a manner sufficient to detect any change in performance that could affect its safe operation.

4.5.2 Main provisions

The Guidelines are recommended to Technical Authorities for applying or developing bridge assessment methods in their area of jurisdiction, whether or not a comprehensive optimised prioritising bridge management system is in place. However, it is a matter for individual Technical Authorities to decide the methodology and procedures to adopt for assessment and its use in the bridge management process. Policy decisions that have to be considered include the adoption of Universal Assessment and the prioritisation of bridge management expenditure based on current and predicted inadequacies in carrying capacity.

Suggestions are made for developing assessment standards and Assessment Live Loading using design standards and loading as the starting points. It is proposed that wherever possible bridges should be assessed using versions of national design codes and loading adapted for assessment. In the longer term, it may be possible to move towards European standards but although this should be possible for loading, structural standards may present longer-term problems.

Level I reliability methods with limit states and partial factors should be used in assessment, or the Technical Authority should move towards this position if it is not currently the case. Assessment should be carried out using simple methods initially but with the option to use increasingly refined methods as necessary. The final optional stage of refinement is the direct application of Level III reliability methods.

The introduction of an assessment programme and revised assessment methods can usefully be staged to take account of the evolution of the methods and an increasing knowledge of the load carrying characteristics of the bridge stock. Throughout the process, a clear policy is needed for dealing with theoretically Sub-Standard bridges that are still serviceable. This is particularly important where brides may only be provisionally Sub-Standard, while better methods of assessment are developed.

The primary objective of bridge assessment is to ensure that bridges are operated safely within their carrying capacity. It follows that for effective bridge management this should be accomplished efficiently in terms of the cost of bridge-works and the maintenance of public utility. The Technical Authority should seek to minimise over-conservatism consistent with safety and provide short-term solutions for Sub-Standard bridges (such as Monitoring) that avoid traffic restrictions wherever possible. The cost of bridge assessments must also be considered particularly when Universal Assessment is introduced. This is a reason for adopting Assessment Levels of increasing refinement, rather than start with Level III reliability methods from the outset.

A basic Bridge Management System should include as a minimum a record of the safe capacity of the bridge in the form of a Capacity Index. Arguably, it should also contain several values of the index relating to other near-critical parts of the structure. Where progressive assessment is adopted, the bridge management system should also contain the relevant Assessment Levels.

Deterioration, current or future and its timing can be reflected in values of the Capacity Index derived by the engineer independently of the Bridge Management System and calculated before data are entered into the BMS. Later, if a suitable bridge management system is available, it could be accounted for internally by the system algorithms.

For use within a bridge management system, bridge assessment methods should be regulated over the network so that they can be applied uniformly and hence facilitate the processes of prioritisation and optimisation of expenditure on bridge management. It would seem that a high level of detail is required in the system database to compensate for a potential loss of judgement applied at the project level. This will be the case if it is decided to assess all bridges that are not known to comply with current standards and allow Capacity Index to take precedence in prioritising expenditure on bridge management. The alternative is to allow the observed condition from inspection to govern but this may reduce reliability against failure.[4]

[4] The opposing argument is that methods of assessment at the ULS are too conservative (because structural models are not known well enough) and the relationship between assessed capacity and true ultimate load varies between bridge types. This argument extends to the proposition that it is more effective to maintain the bridges in good condition and act only if structural distress begins.

The weight given to these two factors in a bridge management system is a matter for the Technical Authority to consider using recommendations from Workpackage 7.

4.5.3 Framework for assessment

The Technical Authority should establish a framework for assessment to be used by engineers and managers within its jurisdiction. It is suggested that documentation should cover the following:

- Primary documents to define and implement the assessment policy of the Technical Authority, and specify the methods and standards to be used. Assessment standards equivalent to design standards for all common structure types tailored to the requirements of assessment, avoiding over-conservatism, and prescriptive methods and details.

- Assessment live loading including a standard loading model to be used in the general case, Bridge Specific Live Loading, and Reduced Live Loading when weight restrictions are to be imposed.

- Approved methods of determining deterioration and its effect on current and future assessed load carrying capacities.

- Actions required in case of an assessment failure including progressive assessment with increasing levels of refinement, identification of low-risk Sub-Standard bridges that can remain in service provisionally, Monitoring and strengthening.

- The requirements for recording assessment conclusions and interaction with the management system, costing, funding etc.

Experience in the BRIME partner countries shows that it is impractical to launch a complete assessment package at the outset. Interim rules will allow the process to commence (or continue) in a systematic way, and improvements can be made as more information about the bridge stock and its apparent deficiencies becomes available. The main proviso is that avoidable repetition of bridge assessments should be carefully excluded where possible.

4.5.4 Assessment levels

It is proposed that assessments are initially carried out using simple methods but more refined methods are used if the required capacity is higher than the assessed capacity. Five Assessment Levels are proposed, with a sixth, Level 0, as an option (a proxy for assessment) that some Technical Authorities may wish to consider. The Assessment Levels described below and shown diagrammatically in Figure 4.7 are illustrative rather than prescriptive. It is assumed that the Technical Authority would adopt a set of levels that suits the national circumstances and priorities.

Level 0 describes a state in which a structure is accepted into the management system without a formal assessment. Placing it into this category implies that records have been consulted that permit this level to be assigned, and its condition is not giving cause for concern.

Levels 1 and 2 entail carrying out a formal assessment using available records of the bridge to determine dimensions, structural details and material properties. It is highly desirable to use standards for loading and structural analysis that have been developed specifically for assessment. The difference between the two levels is that Level 1 uses simple methods of structural analysis to determine load effects and Level 2 uses more refined methods. Unless there are recent inspection results that confirm the construction and condition of the bridge, the site will have to be visited to supplement inspection records.

Levels 3 and 4 make use of tests or surveys to obtain current bridge-specific data - Level 3 - and allow adjustments to the standard safety factors - Level 4. If load testing is to be carried out, it is likely to be a part of Level 3. In Level 4, normal semi-probabilistic methods are used but with revised partial factors to account for bridge specific information. Preferably, the changes in partial factors will be obtained from reliability assessments of typical bridges of the type and specific conditions and tabulated in the assessment standard.

It is envisaged that Level 4 could consider various factors. Element specific target reliability could be reviewed when assessment criteria have been primarily devised for longitudinal effects on main deck members. The whole life reliability of a structure, in the absence of any significant deterioration, increases from the day it is constructed to the end of its functional life. This effect may not have been taken into account in the current criteria. A bridge over a very small watercourse has different associated risk than the average bridge, because of its much lower consequence of failure.

Level 4 stops short of a full reliability assessment based on statistical data for loading, material resistances and loading models – which requires Level 5.

Level 5 is a full reliability analysis using Reliability Level III methods. Although this is sufficiently well developed for practical application, there is a need to base the rules on a standard and rational footing. Important matters to settle are model uncertainties, the target reliability and factors affecting these quantities. Level 5 assessments require specialist knowledge and expertise and are likely to be worthwhile only in exceptional cases. If this form of analysis is used, the Technical Authority may require be consultation on the methods and criteria to be used.

Reasons for levels of assessment. There are several reasons for proposing a formal set of Assessment Levels. Bridges without a capacity problem can be assessed quickly and cheaply whereas those with a potential capacity problem can be assessed in a standard manner with increasing refinement. Contracts or instructions to assess a bridge can be defined in scope. A record of the assessed Capacity Index, and the Assessment Level used to obtain it can be entered into the BMS. This may be helpful in predicting priorities for bridge repairs, as the Capacity Index will depend on Assessment Level – in general, higher levels producing higher assessed capacities.

4.5.5 Resistance calculations

In changing from design standards to assessment standards a number of points should be taken into account.

Opportunities should be sought to relax the design rules based on more up to date information and research. It is advantageous to use the same format for the assessment rules as used in the corresponding design standard. Mistakes can be reduced and the engineer can identify differences more easily.

Minimum and maximum provisions (e.g. reinforcement area, lap length) should be relaxed or removed provided its effect on resistance can be calculated explicitly. The same applies to non-compliances that relate to ease of construction, robustness or durability which are likely to be inappropriate for assessment provided the as-built structure appears to be satisfactory: e.g. cover or bar spacing in reinforced concrete

Underlying formulas should be provided for nominal resistances rather than using tabulated values, the range should be extended when justified by research and safety factors should be given explicitly to facilitate the use of alternative values. Formulas should be given for calculating capacities using measured imperfections, and non-standard detailing, sizes, sections, elements, connections and assemblies that do not comply with design assumptions.

Where a code provision is believed to be very conservative, new testing combined with a review of the technical literature may lead to a less conservative method of assessment. Bear in mind that it is sometimes justifiable to adopt a more complex method of resistance calculation for assessment than for design. The balance between complexity and higher resistance is different in assessment.

Safety factors: although it may not be the intention to reduce the safety of bridges in assessment, a review of safety factors may be appropriate in the assessment standard. This may be treated as a separate issue from Assessment Level 4 provisions.

4.5.6 Adoption of guidelines

Implementation of an assessment programme or standards and a full BMS are not necessarily inseparable. Either could be implemented without the other. The Technical Authority must decide its policy with respect to providing Assessment Loading, Universal Assessment, Reliability analysis etc, and transitional arrangements.

Bridge assessment is an on-going process. It is not suggested that BRIME partner countries interrupt their current assessment programmes and abandon the methods and procedures they are currently using. It is proposed, however, that they consider what can be done to improve their current practices and how best to move towards a compliant assessment programme suitable for a full BMS.

Initially it is acceptable to assess bridges only when the need arises – for instance when there is deterioration or a change of loading - but the Guidelines are written assuming that all bridges will eventually be assessed (Universal Assessment) and data entered into the BMS for prioritising maintenance and strengthening work

Bridge assessment should be carried out using loading and structural standards devised for the purpose and not design standards, as these will generally be too conservative. Countries currently using design standards should aim to improve their methods.

4.6 CURRENT POSITION

In most of the BRIME countries, and especially in France, an assessment is initiated primarily if an inspection shows that the condition of the bridge has deteriorated to the point that the carrying capacity needs to be checked. On other occasions the need for assessment may arise if there is expected to be a change in loading – for instance if more lanes are required on the bridge deck or the passage of an exceptionally high load is expected.

The main exception to this is the UK, which is currently in the later stages of a 15-year programme of bridge rehabilitation and strengthening that was initiated in 1987. In this programme, all bridges that were not designed to certain specified standards were to be assessed and appropriate action taken thereafter depending on the assessed capacity. In all cases, the assessments have been preceded by an inspection to check the condition, but the assessment has proceeded even when the condition is perfect. It is proposed in the UK to adopt a management system in which the primary need for maintenance, strengthening or replacement is the level of safety remaining in the structure following an inspection and an assessment. At another level, the stages in assessment depend on the results obtained for the bridge. If it is shown to have a satisfactory capacity there may be nothing remaining to do – apart, perhaps, for preventative maintenance. When the assessed capacity is not satisfactory, further action is required.

In 1998, an Advice Note was issued in the UK (BA 79) that sought to regularise these procedures. There are two provisions in the Advice Note. The first defines five levels of assessment of increasing complexity that may be used in an attempt to prove that the bridge is not sub-standard – the assumption being that the assessment will commence at a simple level and progress further if necessary. The second provision consists of recommendations for actions in the meantime, while the structure is still provisionally sub-standard. In the case of a bridge defined as presenting an 'immediate risk', actions to secure safety must be put in place without delay. In other cases the bridge may be left in service with monitoring provided it falls within a class of bridge defined as "monitoring appropriate".

In Germany, a starting classification of load carrying capacity of a bridge could be performed on the basis of a level 0 assessment. The German DIN 1072 for example defines bridge classes for different vehicle gross weights. These vehicle gross weights represent an input to the static load model of DIN 1072, eg, if a bridge is classified as '16/16 tonnes', it must be guaranteed that the bridge is only used by vehicles up to 16 tonnes. This information can be used for the bridge management as well as the HGV-routing.

Workpackage 2 proposes to follow this Assessment Levels methodology as described in Section 4.5.

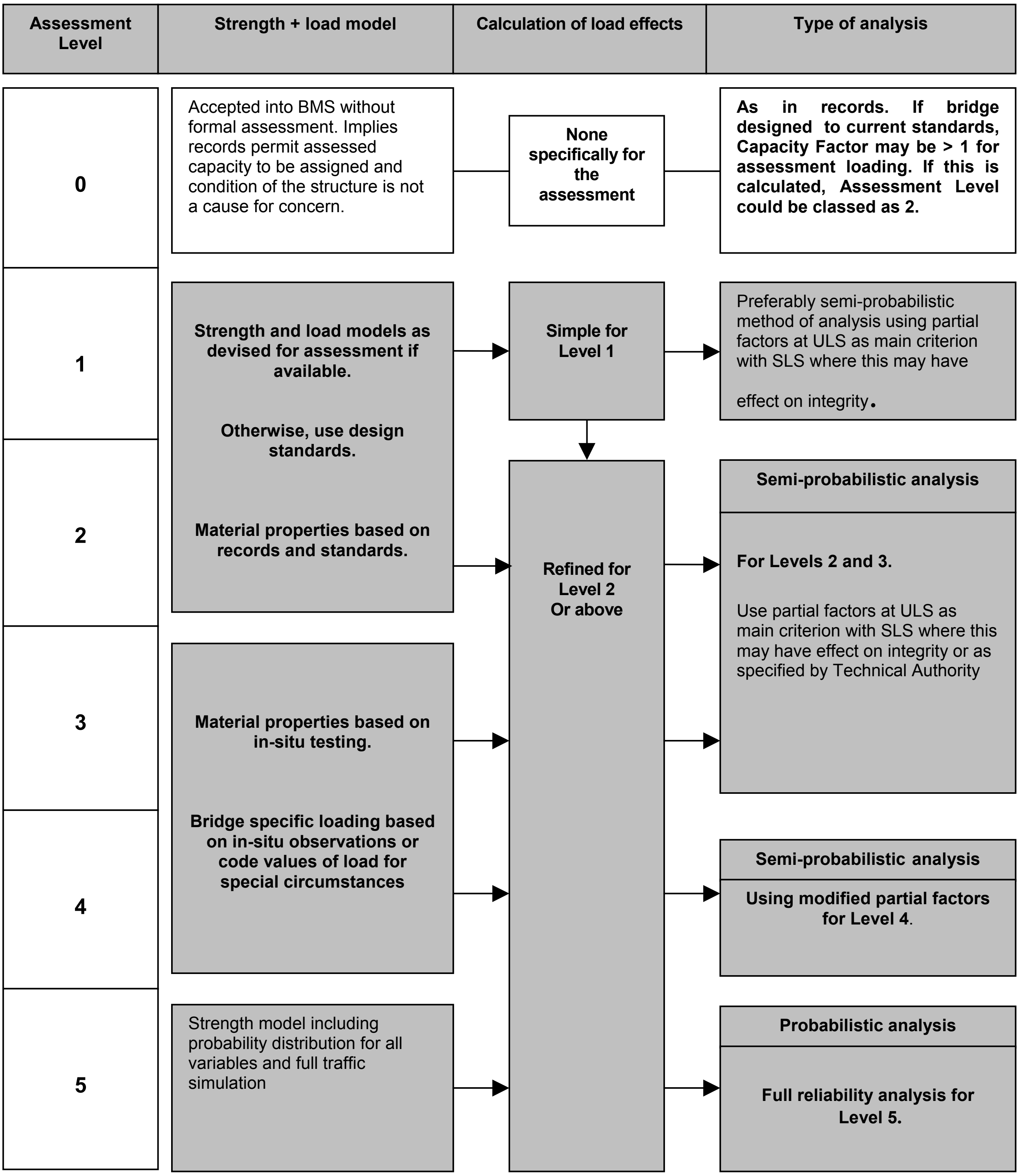


Figure 4.7: Load models and calculation principles

4.7 IMPLEMENTATION

Structural assessment plays a major role in bridge management at project level together with an economic analysis of possible maintenance activities based on condition assessment determined from regular inspection. This applies whether Universal Assessment is adopted or structural assessment performed only when a specific need is identified.

The input necessary for structural assessment in bridge management is a comprehensive set of information from the inventory e.g. dimensions, materials used, results from inspection especially defects/deterioration and the condition values of elements and the whole bridge. Additional and more specific information on design and construction, real traffic and other loads and also experimental investigations (in-situ tests or laboratory investigations) has to be taken into account if available.

A very important point is that in the case of damaged or deteriorated structures, the results of investigations on specific deterioration and damage models or of deterioration monitoring have to be considered. For more information please refer to Chapter 5.

The structural assessment module developed for the BRIME project results in information on the load carrying capacity of bridges. If performed consequently (if necessary or systematically) information on history of load carrying capacity for a specific bridge but also for bridges in the network or in the course of a road might be available.

Still a problem is the prediction of load carrying capacity. In fact it depends on the quality of deterioration models which are not sufficiently developed at present for predictions at project level. For more information refer to Chapter 5 and 6. Another point is that reliable predictions for traffic loading are still missing.

As mentioned before structural assessment plays a major role in the bridge management at project level. Information on load carrying capacity is important for determining maintenance needs and when essential maintenance is required (refer to Chapter 7). It is also important for evaluation of a condition index (refer to Chapter 3) as a basis of prioritisation at the network level (refer to Chapter 8). Beside this load carrying capacity gives information on substandard bridges or bridges with traffic restrictions. In this context load carrying capacity serves as a basis for routing activities, especially for routing of exceptional vehicles.

4.8 CONCLUDING REMARKS

The investigations presented deal only with load carrying capacity. However, for some structures, especially steel bridges but also prestressed concrete bridges, fatigue could be an important issue. If fatigue cracks are discovered in a bridge structure, a rational maintenance strategy must be developed. It is likely that further inspection will reveal more cracks, and that continued use of the structure will result in crack growth at other locations. Replacement of the structure is rarely desirable due to costs. On the other hand, if nothing is done, a critical situation may occur. Therefore identification of the causes of fatigue cracking and evaluation of the possibilities of solving the problem are priorities. Moreover the evaluation of the remaining fatigue life is still a very important economic problem and also interesting from the engineering point of view.

This is also true for the estimation of the remaining service life of corroding structures (refer to chapter 5). Reliable methods have to be developed.

In the context of bridge management, it seems that the ”Structural Assessment” module has to be widened into ”Refined Project Related Analysis”. This covers not only the Load Carrying Capacity but also the consequences of every type of deterioration/damage e.g. due to corrosion, fatigue and other problems that influence the condition of structures. In this context advanced NDT-methods for evaluating deterioration processes must be evaluated.

The structural resistance is only one side of the problem; the other side is the load situation. The static traffic load concept given in Deliverable D5 represents a useful approach for further investigations, in which a number of parameters must be evaluated (refer to Deliverable D5):

- further cross section types,
- additional load effects
- changed distribution of HGV traffic on individual lanes
- distributed loading outside the lanes and on the sidewalk.

Additional reliable load models, for example for fatigue assessment, are still missing and need to be developed.

4.9 REFERENCES

Cremona C and B Jacob (1995) *Une Base de Donnees d'Enregistrements de Trafic; Un Outil d'Aide a la Conception et a la Gestion des Ouvrages Routiers*, 1st European Conferenc on WIM of Road Vehicles, March.

Eurocode 1 (1995) *Basis of design and actions on structures - Part 3: Traffic loads on bridges*. German Version ENV1991-3: 1995

Federal Highway Research Institute (1998). *Auswertung von gemessenen Achslastdatensätzen an fünf Autobahnquerschnitten der BRD.* Internal analysis report, Bergisch Gladbach (not published)

Geißler, K., *REB - Rechenprogramm zur Restnutzungsdaueranalyse von Bahn- und Straßenbrücken, modifizierte Version mit Berücksichtigung der Extremwertauswertung*

Highways Agency (1997) *The Assessment of Highway Bridges and Structures.* Design manual for roads and bridges BD 21/97. London

Highways Agency (1998) *The management of sub-standard highway structures.* Design manual for roads and bridges, BD 79/98. London.

Melchers, R. E (1999) *Structural reliability analysis and prediction.* Wiley

Ricketts N J and Page J (1997) *Traffic Data for Highway Bridge Loading*, TRL Report 251.

CHAPTER 5

MODELLING OF DETERIORATED STRUCTURES

5.1 INTRODUCTION

The increasing age of the bridge stock throughout Europe has highlighted the problems associated with deterioration in existing structures. Current assessment methods do not normally include reliable techniques for the evaluation of the structural consequences of deterioration. This part of the BRIME project focused on the common forms of deterioration found in bridges and the effects they have on assessed capacity. The ultimate aim was to identify models for the deterioration process which can be used as part of the general assessment procedure developed under Workpackage 2, *Bridge assessment*, to enable a reliable estimate of load carrying capacity to be produced and used in an overall bridge management system.

Figure 5.1: Corrosion in UK bridge.

Bridges are not immune to the ageing process, and damage arising from poor maintenance, improper use and from the adverse effects of the environment needs to be taken into account using appropriate techniques. A prerequisite here is the development of appropriate methods of quantifying the extent of deterioration in a rational and practical way. The determination of residual life is often required, particularly where future management strategies need to be defined in terms of financial programming and setting work priorities. Most of the existing research into deterioration has concentrated on methods of preventing corrosion from taking place or of dealing with the problem as it arises by repair and replacement. This Workpackage was devoted to identifying methods of modelling the structural effects of deterioration and is concerned primarily with the strength of deteriorated bridges in relation to the loads that they are required to carry.

Workpackage 3 has been divided into a number of tasks as follows:

- a survey to determine the forms of deterioration present in the European bridge stock
- collation of information obtained from site investigations of bridges to evaluate the structural effects of the various common forms deterioration
- a desk study to identify and evaluate existing methods of taking deterioration into account in the assessment of bridge capacity
- a literature review to investigate the performance of deteriorated bridges and bridge components and to identify areas where further work is required

- production of guidelines for the assessment of deteriorated bridges.

5.2 CONDITION OF BRIDGES IN EUROPE

The first stage in the project was to identify the common forms of deterioration present in the European bridge stock and to determine the main causes. It is clear from previously published work that corrosion of steel due to chloride contamination and carbonation of concrete is a serious problem in bridges. Other forms of deterioration such as alkali-silica reaction, freeze-thaw action and sulphate attack are also common. As part of this project, a questionnaire was circulated to the participating BRIME countries (France, Germany, Norway, Slovenia, Spain, UK) to obtain general information on the number and type of bridges in the national bridges stock, the condition of the bridges, and the main forms of deterioration present. Specific information was also obtained on the procedures used to assess bridges and the methods used to take account of deterioration.

The responses to the questionnaire are summarised in Table 5.1; a more detailed breakdown is given in Deliverable D11. The table gives the estimated number of bridges and the proportion of bridges affected by deterioration for each of the partner countries. The intention is to indicate the proportion of bridges having defects that might affect performance now or at some time in the future. The information obtained was very variable since each country is at a different stage in terms of bridge assessment and has different priorities. One problem is that different countries categorise “defects” in different ways. In most countries, existing data pertained only to the bridges on the national routes (ie, roads managed directly by government departments) and thus the questionnaire tended to focus on these. It is recognised that, in some countries, this represents only a small proportion of the national bridge stock. Other bridges are the responsibility of local authorities and private owners such as toll road concessionaires, railway operators, regional transport systems, national river authorities, etc. Thus the statistics may be biased towards the more recent, longer span structures with better maintenance regimes. In some cases, statistics are only available from a survey of a relatively small number of bridges.

It was clear from the analysis of the responses that the same problems that are found in all countries in spite of the different traffic conditions and climate. The sources of deterioration can be sub-divided into three different groups:

- deterioration or defects arising from faults in design and construction: these include low cover, reinforcement congestion, badly located joints, poor drainage system, ASR susceptible aggregates, insufficient foundation capacity

- defects arising during construction: poor quality concrete, bad compaction, inadequate curing, poorly fixed reinforcement, faulty ducting for post-tensioning systems, inadequate grouting, inadequate painting or coating

- deterioration from external influences: overloading, vehicle impact, chloride attack, carbonation, poor maintenance, freeze-thaw action, dynamic loading.

Table 5.1: Estimated number of bridges and % with defects.

Country	Est. number of bridges	Number on national roads	% with defects	Main causes of deterioration
France	233,500	21,500	39% [1]	Corrosion of reinforcement Inadequate compaction Corrosion of prestressing tendons Defective grouting Inadequate water-proofing Inadequate design for thermal effects Alkali-silica reaction
Germany	80,000	34,800	37% [2]	Corrosion of reinforcement Design/construction faults Faulty bearings, joints, drainage, etc Overloading Vehicle impact Fire, flooding.
Norway	21,500	9,173	26% [3]	Corrosion of reinforcement Freeze-thaw damage Alkali-silica reaction Deterioration of paint, etc Corrosion of steel Construction faults, Shrinkage Use of sea-water in mix Settlement of foundations, Scour
Slovenia	N/A	1,762	N/A	Corrosion of reinforcement Corrosion of prestressing tendons Failure of waterproofing Corrosion at abutments Freeze-thaw damage Corrosion of steel Defective expansion joints
Spain	N/A	12,380	N/A	Corrosion of reinforcement Corrosion of steel Inadequate waterproofing Defective expansion joints Impact from high-sided vehicles
UK	155,000	10,987	30% [4]	Corrosion of reinforcement Corrosion of prestressing tendons Impact damage Shrinkage cracking Freeze-thaw Alkali-silica reaction Carbonation

[1] Based on survey of bridges on national roads only, based on an IQOA as follows:

- 2E: bridges with minor defects, but which require immediate attention to prevent rapid progression (25%)
- 3: structurally impaired, requiring non-urgent repair work (11%)
- 3U: structurally impaired, capacity already inadequate (3%)

[2] Based on inspection a condition rating of >2.5 from a survey of 750 bridges on federal highway and trunk road network.

[3] Based on a survey of 149 concrete bridges only: a further 17% contained repaired corrosion damage.

[4] Based on visual survey of random samples of bridges of all types.

Unfortunately bridge deterioration is rarely the result of just one of the above factors and serious deterioration often involves a combination of them. This confuses the identification of the primary cause and complicates the models for predicting the development of damage and determining the structural implications. This is particularly true for the most serious and widespread problem which is that of corrosion of reinforcement and prestressing tendons due to chloride contamination (either from sea-water or de-icing salts) and (to a lesser extent) carbonation. Contributing factors include inadequate detailing (insufficient cover, etc), ineffective drainage systems, leaking joints and failed (or absence of) waterproofing systems.

In all countries, serious attempts have been made to eliminate these problems in new construction and these have proved to be very effective. In particular, the role played by well-compacted concrete with adequate cover to reinforcement in preventing corrosion has been recognised. Other techniques, such as use of protective coating, have also been implemented. Some attempts have also been made to take account of the deterioration in assessing structural strength in a rational way, although in most cases, it has been attempted in a simplistic way using some form of “condition factor”. Current practice is discussed in the next section.

5.3 CURRENT ASSESSMENT PROCEDURES

In the questionnaire, countries were asked to supply information on general assessment procedures used and, in particular, how deterioration in bridges is taken into account. A description of the different procedures used is given in Deliverable D1. There are clear differences in approach. The UK has adopted standards specific for the assessment of existing bridges and these are used in a formalised assessment programme. In most other countries, the appropriate design codes are used, although in some cases these are modified occasionally according to the specific requirements and according to the expertise and experience of the assessment engineer.

The current practice in all countries is to take account of deterioration in some way. In general, this means using actual sound section dimensions as measured on site or assumed, and modifying the material properties based on material tests or NDT methods. Except for the UK, taking account of deterioration is, in general, carried out on an *ad hoc* basis and depends on the knowledge and experience of the assessment engineer. In the case of the UK, there are assessment documents relating to deterioration arising from chloride induced corrosion and alkali-silica reaction. These documents tend to be general in nature and contain little quantitative guidelines.

The main conclusion is that, in general, while many countries have adopted general rules for investigating deterioration as part of condition assessment, there are few procedures available for taking deterioration into account in a structural assessment. Generally, the emphasis is on determining the presence of deterioration, and determining the condition of the bridge in terms of the extent, and possibly the rate, of progress of damage. This information is then used in formulating maintenance strategies and prioritising repair or rehabilitation works. An exception is the UK, where the current documents used for bridge assessment contain some guidelines on how specific forms of deterioration can be taken into account in determining bridge load carrying capacity. These documents were produced because of the different strategy adopted in the UK in relation to bridge assessment.

5.4 ASSESSMENT PROCEDURES FOR DETERIORATED BRIDGES

Bridge owners and managers are required to ensure that the structures for which they are responsible serve the purpose for which they were built in a safe and maintainable manner. They are also required to ensure that appropriate maintenance strategies are implemented in a cost effective way. For an ageing bridge stock, and this is the current state in most countries, maintenance of the existing structures is becoming more and more important in terms of the commitment of resources. As a result, the requirement to be able to identify the presence of deterioration and to quantify it in terms of its effect on serviceability and load carrying capacity is increasing.

It is important to differentiate between the condition of a deteriorated bridge and the effect the deterioration has on load carrying capacity. Workpackage 1, described in Chapter 3, was concerned with the condition assessment of bridges. In this Workpackage, the focus is on the effect that this condition has on strength. It is important to highlight this difference as the condition of a bridge can be considered to be poor, but the effect on structural performance may be slight, or insufficient to bring the structure below the minimum acceptable performance level. On the other hand, the converse may be true and bridges that are classified as being in good condition, based on visual examination, may still be under-strength. An example is the local corrosion in the tendons of segmental post-tensioned members due to the ingress of chlorides through defective joints. In such a situation, a small amount of localised corrosion can cause serious loss of strength even though a visual examination might suggest only minor corrosion. In the case of Ynys-y-Gwas bridge in Wales, the result was a dramatic and completely unexpected collapse in 1986 [Woodward and Williams, 1988]. This is not to understate the importance of condition assessment: bridge users must have confidence in the structure and a structure will be considered a failure if its appearance prevents normal use. Similarly, the present and future condition of a bridge should be such that normal maintenance is sufficient to keep the bridge serviceable.

Figure 5.2: Corrosion in post-tensioning tendons.

5.4.1 Minimum acceptable level of performance

The phases of the service life of a structure are dictated primarily by loss of strength, although loss of serviceability can be just as important. At some point in a deteriorating structure, a minimum acceptable level of performance may be reached and this defines the end of the service life. This is illustrated schematically in Figure 5.3, which gives an example of a structure that is deteriorating due to corrosion of reinforcement resulting from chloride ingress. In this example, the chloride concentration at the reinforcement is used as a measure of the condition of the structure. When the chloride level reaches some critical value, then corrosion will be initiated and loss of section will result. The objective of an effective maintenance strategy is to increase the service life at minimum cost. For example, routine maintenance such as painting, cleaning and minor cosmetic repairs can be used to slow down the rate of corrosion. Repair or rehabilitation work can be used to restore lost capacity as shown in the figure. This ensures that the structure does not go below the minimum

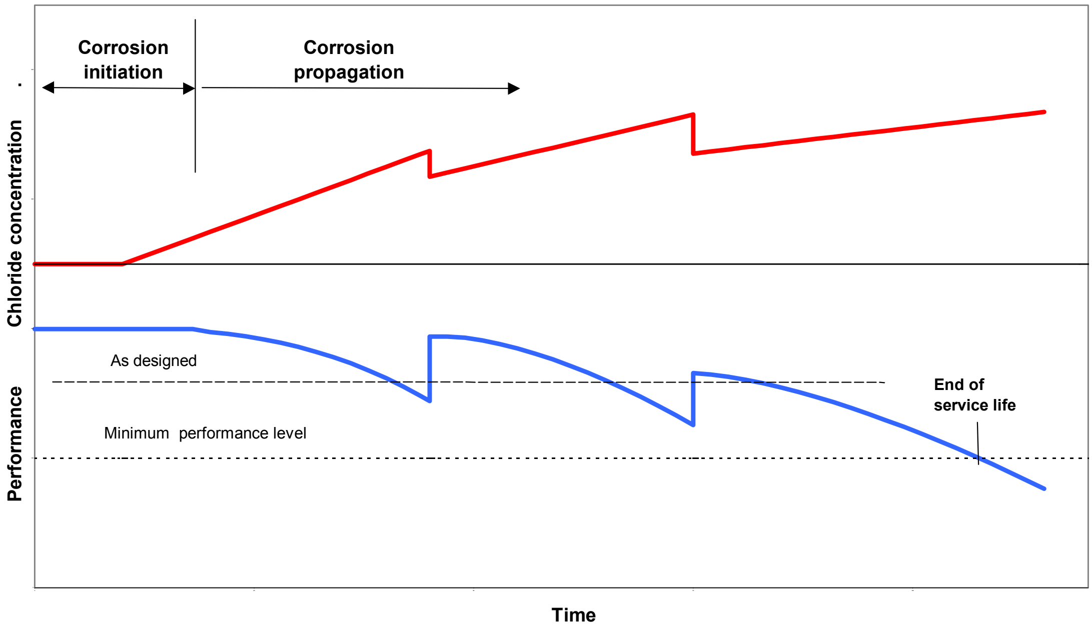


Figure 5.3 Corrosion loss and performance as a function of time.

acceptable level of performance - the level below which the structure must not be allowed to reach – as defined by the appropriate technical authority.

For new construction, this level is defined by national codes and standards and it is likely that these will continue to be used for assessment, albeit with appropriate modifications. For a bridge, several levels of performance must be considered including:

- Serviceability: appearance and maintainability; deflection and deformation; crack widths, safety of bridge users;
- Ultimate limit state: using full highway loading, or restricted traffic if appropriate
- Future state: future progress of deterioration, at possible increased rate due to increasing damage;
- Consequences of failure: in terms of both costs and potential loss of life.

In general, it can be argued that serviceability need not be considered in bridge assessment. This is because, if a bridge has already seen many years of service without serviceability problems, it can be assumed that the serviceability requirements have been satisfied. This approach has been generally accepted in the UK. Bridge owners may be willing to accept lower levels of serviceability based on the previous acceptable performance of the structure. Greater deflections or crack widths may be acceptable where aesthetics are of less importance because of the location and use of the bridge. There are other implications in accepting lower levels of serviceability and these should be fully considered and taken into account within an overall management strategy before being accepted. It may mean that the need for rehabilitation work is postponed which might increase the cost. Risks to user safety should also be considered: for example, spalling concrete could fall onto a carriageway or a member of the public. In structural assessment, attention is focussed on the ultimate limit state for the current condition of the bridge, and for the condition at some time in the future. It is envisaged that, in assessing bridge capacity, the engineer should make some allowance for future deterioration, ideally presenting an indication of the residual service life. Attention should be given to possible modes of failure particularly those unique to the deterioration process.

5.5 GENERAL METHODS OF DEALING WITH DETERIORATION

There are a number of general methods that can be used to take account of the modified behaviour of a structure as a result of deterioration. These can be applied to most forms of deterioration and are described in the following paragraphs.

5.5.1 Reduced cross-sectional area

In determining the strength of a deteriorated or damaged structure the most common approach is to take account of the material loss by direct measurement of the remaining sound material. Measurements can easily be made of external concrete dimensions or the steel thickness of a steel beam to take account of section loss due to spalling or surface corrosion. It is not quite so straight-forward when deterioration of reinforcing steel or delamination within the concrete is suspected. In some cases, breaking out portions of the structure can be justified in order to determine the extent of material loss. A wide range of non-destructive testing (NDT) techniques are available which can be used to determine residual steel thickness, presence of corrosion in concrete structures, delamination of concrete and presence of faults as well as concrete quality. The information provided by these techniques is largely qualitative, which is useful but is unlikely to completely satisfy the requirements of practical strength assessment. However NDT techniques are continuously being developed and improved.

5.5.2 Condition factor

Where measurements are not possible or where there are other uncertainties in the determination of resistance, some codes suggest the use of a *condition factor* to take account of any deficiencies that are noted in an inspection but not allowed for in the determination of member resistance. The factor should be based on engineering judgement and should represent an estimate of any deficiency in the integrity of the structure or member. It may be applied to a member, a part of the structure or the structure as a whole. While this is an imprecise and subjective method of allowing for defects in the structure, it is often the only approach available due to the absence of data on the strength of deteriorated structures. In general, there is a lack of specific guidance and it is left to the experience and judgement of the assessment engineer to make an appropriate choice of condition factor.

5.5.3 Modified concrete properties

Knowledge of the in-situ material properties is required if a reasonable estimate of the capacity of a structure is to be made. The most direct method of determining in-situ concrete strength is to take core samples from the structure and carry out compression tests. NDT techniques can also be used but careful calibration is required for a particular structure.

The assessment of the strength of a bridge entails calculations pertaining to a number of different load effects such as flexure, shear, bond, bearing and deflection. A number of concrete properties are required, such as compressive and tensile strength, bearing strength, and elastic modulus. Most design codes use the compressive strength (cube or cylinder) as a reference for these concrete properties. This is a convenience which may bear little resemblance to the actual behaviour of concrete in a structure [Clark, 1989]. The equations were developed to correlate reasonably well with test data for good quality concrete. For deteriorated concrete these relationships may not be appropriate, since the various concrete properties may be affected differently by the deterioration. The solution is to avoid using the cube strength as a reference where possible and measure the properties with appropriate tests. These properties can then be used directly in the assessment.

5.5.4 Modified steel properties

The strength of corroded steel members can be taken into account by reducing the cross-sectional area to allow for the material loss. There is no evidence in the literature to suggest that the stress-strain relationship of structural steel is greatly modified by normal forms of deterioration. It is known that corrosion in concrete can have an effect on the properties of the reinforcement. Ductility can be reduced and fatigue properties can be affected.

5.5.5 Modified bond properties

For reinforced concrete to behave as a composite material, adequate bond between the concrete and reinforcement must be maintained. Bond is developed by chemical adhesion and mechanical interlock. For deformed bars, the main component of bond is the interlock between the deformations and the surrounding concrete. When cover is low and where no transverse steel is present, failure occurs by splitting of the concrete and bond strength depends on the tensile strength of the concrete. Where splitting of the concrete is prevented, either by adequate cover or the provision of transverse steel, the concrete between the bar deformations shears from the surrounding concrete and the bond strength is a function of the strength of the concrete in direct shear. Most codes simplify bond strength by presenting it as a function only of concrete strength and bar type.

Small amounts of surface corrosion can enhance bond strength by increasing the confinement of the reinforcing bars. In general, however, corrosion disrupts the interface between the steel and concrete thus reducing the bond strength and ductility of the element. This can be the most significant effect of corrosion and various equations have been proposed to model the relationship between bond strength and extent of corrosion. While these are based almost exclusively on pull-out tests on laboratory specimens, they provide a useful means of determining the effectiveness of reinforcement in naturally corroded members.

5.5.6 Modified structural behaviour

The approaches so far assume that the structural behaviour of the components is not altered by the deterioration. Where substantial amounts of deterioration have occurred the mechanisms by which a structure resists load may be modified. For example, a concrete column extensively corroded on one side will develop bending moments when subject to an apparently axial load. Complete breakdown of the bond between steel and concrete would result in a beam being incapable of behaving as a conventional beam. In fact, such a member could carry the load by acting as a tied arch provided there is adequate mechanical anchorage at the bar ends and the member is not too long. Shear strength can also be affected since dowel action could be lost. In most instances, impairment of performance is likely to be sensitive to detailing but no specific guidance can be given.

Where serious corrosion exists, there is always the possibility that the integrity of the structure is impaired. In such cases, it would be necessary for the assessment engineer to demonstrate that the assumed structural behaviour can actually be achieved. For deteriorated structures, advantage can be gained by using alternative methods which take account of restraints which are generally ignored in design. One example is compressive membrane action, which can considerably enhance the strength of slabs. Another example of possible modified structural behaviour is the additional restraining forces and moments, which could develop as a result of corrosion in metallic bearings. The implication is that, if traditional methods result in slabs which are over-designed, then the loss of steel due to corrosion may not be such a serious problem.

5.5.7 Additional stress

Some forms of deterioration result in additional stresses being imposed in the structure. For example, when ASR expansion is restrained either by internal reinforcement or external restraints, additional stresses will develop in the concrete and the beam is effectively prestressed [Clark, 1989]. For a reinforced concrete section, these stresses can be taken into account by treating it as a prestressed beam, using some proportion of the additional stress. For prestressed beams, the additional ASR prestress can be included in the assessment. In principle, these stresses can be estimated from a knowledge of the free expansion due to ASR and the reinforcement details. In practice, however, complications arise since the stresses due to restrained ASR expansion depends on the stress history and on the expansion rate.

5.6 MODELLING DIFFERENT FORMS OF DETERIORATION

The paragraphs above present straight-forward methods of taking account of deterioration in the determination of the structural strength of a member. How they can be applied to different forms of deterioration depends on the type of deterioration and how it affects structural behaviour. The following sections presents specific guidelines on how they can be applied to the main forms of deterioration found in bridges. It is important to note that the assessment of the effects of deterioration and the determination of residual life depends on the correct diagnosis of the deterioration and the conditions causing it.

5.6.1 Corrosion

Strength loss as a result of corrosion can be due to one or more of the following:

- reduced steel cross-section due to corrosion of the bar;
- reduced concrete cross-section due to cracking or spalling of concrete cover;
- reduced bond between the reinforcement and the concrete due to presence of corrosion products

- change in material properties (strength, modulus) due to corrosion process.

In order to take account of reinforcement corrosion correctly all of these must be considered in the assessment of load carrying capacity and it is necessary to understand the physical processes involved. The service life of a structure affected by reinforcement corrosion can be divided into two phases, an initiation phase and a propagation phase. The initiation phase is the time before any loss of steel section. The propagation phase is the period of actual material loss due to corrosion. The steel cross-section is reduced and corrosion products are produced at the steel concrete interface. This generates stresses within the concrete, which can disrupt the cover concrete by cracking and/or delamination so that the effective concrete cross-section is also reduced. This can continue until the strength loss is such that the minimum acceptable level of performance has been reached. In some cases, failure might result for reasons other than loss of load carrying capacity. For example, the poor condition due to corrosion might render the structure bridge unserviceable from the point of view of safety of the users (from, for example, spalled concrete dropping onto a carriageway), appearance or maintainability. Reinforcement corrosion can also reduce the ductility of concrete members, which can affect structural performance.

In order for the corrosion of reinforcement to proceed the normal alkaline environment of the concrete (pH > 13) which provides a natural protection against reinforcement corrosion must be broken down. There are generally two different mechanisms to cause this, carbonation or chloride contamination. Carbonation is the natural process by which carbon dioxide from the atmosphere is gradually absorbed into concrete neutralising the alkali environment. It has little detrimental effect on concrete properties: in fact it tends to make the concrete stronger, although this depends on type of cement used. If the carbonation front reaches the level of the reinforcement then corrosion will follow. Chlorides comes either from de-icing salts, or some other source such as sea-water or contaminated water used in the concrete mix. This provides a source of chloride ions, which can depassivate the alkaline environment within the concrete.

Once corrosion has been initiated, either due to carbonation or chloride contamination, loss of steel section proceeds at a rate that is dependent on a number of different parameters. The diffusion of these two contaminants is different and they result in different forms of corrosion. Carbonation produces generalised corrosion while chlorides result in localised or pitting corrosion. Because of these differences, deterioration models need to be developed separately. These have been investigated by some researchers but the testing has invariably been carried out on laboratory specimens in controlled environments. Various models have been proposed by different researchers and these are described in Deliverable D11.

From the point of view of loss of member strength, the most important information of direct use in determining member strength is that relating to the amount of material loss due to corrosion. The steel lost due to corrosion can only be accurately determined directly by exposing the bars and measuring the remaining cross-section. The average corrosion rate, in terms of mm/year loss of steel section, can then be determined provided the time of corrosion is known. This can be deduced from the age of the structure making some allowance for the time required for the corrosion process to initiate. If section losses are measured over a period of time, then an average corrosion rate can be determined more accurately. These measurements are complicated by the fact that corrosion will probably not proceed at a uniform linear rate due to changes in conditions. Even in closely adjacent sections corrosion may differ considerably and it is not appropriate to carry out such intrusive investigations at too many locations or too close to critical areas.

Numerous researchers have proposed models to predict the rate of reinforcement corrosion. In most cases, the models are based on experiments on laboratory specimens using various parameters, such as concrete properties (cement content, water cement ratio, etc) and exposure conditions (temperature, humidity, chloride level, etc). The results can be calibrated by comparison with site measurements and non-destructive tests. Other researchers have investigated the results of corrosion in terms of cracking and loss of bond and have produced equations for quantifying these in terms of corrosion level.

These models can then be used within the assessment procedure so that the deterioration is taken into account in determining bridge capacity. In determining member resistance, the equations used must be modified. This includes flexural capacity, shear capacity, axial compression capacity, bond strength, anchorage bond, anchorage and lap requirements and fatigue capacity. In addition, the method of analysis might also need to be modified. Each of these is dealt with in the following sections.

5.6.1.1 Flexural strength

Using the methods described in the previous sections, along with appropriate site measurements and tests, it is possible to determine the flexural capacity of a corroded member. Generally, the normal equations or methods used in design or assessment as defined in national codes and standards can be used, but the section properties are modified to take account of material lost due to corrosion. Loss of concrete section can be measured directly from careful crack and delamination surveys. For steel loss, as previously mentioned, it is necessary to make assumptions regarding the distribution of corrosion. It is unlikely that intrusive measurements will be possible at all critical sections, so reliance will have to be placed on non-destructive techniques such as half cell or linear polarisation measurements. Such measurements can be calibrated against similar measurements and intrusive investigations at non-critical sections. It is clear that a great deal will depend on the experience of the assessment engineer.

For the generalised corrosion of reinforcement in concrete, the loss of static strength of the steel is insignificant since the material loss at any section is small. With localised corrosion, however, the severe loss of metal at a section can result in serious loss of static strength. Pritchard and Chubb [1987] report that the loss of strength is much less than can be attributed to material loss. They suggest that strain hardening at the corrosion site enables the bar force to be increased above its yield value. Their test results indicate that bars with up to 30% section loss did not suffer any loss of yield strength, while those with 60% section loss showed only 10% strength loss.

The assessment engineer must be assured through site inspection and special investigations if required that this approach is appropriate. Where serious corrosion has occurred, disruption of the section can be such that the normal assumptions for beam behaviour do not apply. For example, if longitudinal cracking along the cover concrete or spalling has occurred allowance must be made for loss of bond and the engineer must decide whether the steel is appropriately anchored and therefore effective in resisting flexure. It is likely, however, that structures in this state would not be allowed to continue in service from the point of view of serviceability.

Material properties can also be affected. If there is any doubt about appropriate values of concrete properties, cores should be taken to determine in-situ values of strength and modulus. Similarly, steel properties can be affected and samples should be taken to determine strength and modulus. Tilly [1988] reports that the ductility of reinforcing bars is likely to be reduced where the rate of loading is high. Rodriguez *et al* [1996] report reductions of 30% and 50% in the elongation at maximum load for corrosion losses of 15% and 28% respectively. This would have an effect on design methods, eg, where moment re-distribution or plastic methods of analysis are used. The engineer must satisfy himself that sufficient ductility exists in order to justify particular methods of design. In any case the maximum level of re-distribution quoted by codes of practice should be treated with care where corrosion has occurred.

The guidelines outlined above apply equally to the assessment of prestressed concrete. In addition, however, sensitivity to stress corrosion must be considered and allowance must be made for loss of prestressing force if appropriate. Where individual strands are corroded, the effective prestress can be determined by ignoring their contribution. Where strands have completely corroded through locally, re-bonding can occur so that the local prestress loss may not be lost throughout the length of the strand. With post-tensioned construction, this is very dependent on the quality of the grouting. Various non-destructive testing techniques have been developed and are the subject of continuing research.

5.6.1.2 Shear strength

Shear capacity is determined using different methodologies depending on which code of practice is used. However, the effects of corrosion can be described in general terms and applied whichever method is used. Corrosion is taken into account in much the same way as outlined above for flexure primarily through making allowances for loss of concrete, main longitudinal reinforcement and vertical shear reinforcement, ensuring that all steel is appropriately anchored in order to be effective. It should be noted that shear links are particularly prone to corrosion as they generally pass around the main steel and therefore have less cover. In addition, the corrosion of links is usually concentrated at the bends where 100% losses have been recorded. Particular care should be taken to ensure that shear links are effectively anchored.

5.6.1.3 Bond

The process of corrosion dissolves the surface of the reinforcement and this can have a very disruptive effect on bond. Cracking of the concrete cover to the reinforcement can weaken the confinement of the reinforcement and this can reduce bond capacity significantly. The available data on bond strength cannot easily be related quantitatively to the amount of corrosion. Laboratory tests have shown that a small amount of corrosion can increase bond strength, but once corrosion has developed to the extent that the cover concrete is cracked then bond strength reduces significantly. Loss of bond means that the reinforced member cannot behave as a composite section. However, provided the anchorage is not affected then the element can continue to carry load.

As well as rendering reinforcement ineffective, loss of bond can result in a significant reduction in ductility and the ability to redistribute load in redundant systems. This must be taken into account particularly where non-linear analysis methods are used

5.6.1.4 Axial compression capacity

The capacity of bridge piers and members subject to axial load can be calculated by making due allowances for loss of steel and concrete section as described above. In addition, spalling of concrete may increase the eccentricity of the applied load and this should be allowed for in the analysis. The possibility of buckling failure should also be considered. Where cover concrete has spalled, or where stirrups have corroded at bends, loss of restraint along the reinforcement will increase the likelihood of buckling in compression steel. In extreme cases, it may be necessary to consider particular bars ineffective in carrying compression.

5.6.1.5 Method of analysis

In general, the method of analysis used for deteriorated structures will be the same as that used in conventional design as outlines in national codes and standards. It is envisaged that linear elastic methods will generally be used as they will produce a safe assessment. Where more sophisticated methods of analysis are used, such as yield line analysis, non-linear finite element analysis, etc, the engineer must be assured that all assumptions are consistent with the existing condition of the structure, particularly those relating to durability.

Where the engineer judges that the level of deterioration is such that the behaviour of the structure is altered, then this should be taken into account in the structural analysis. For example, existing corrosion cracks would indicate that cracked section properties should be used rather than gross section properties. The level of corrosion at fixed or partially fixed supports may be such that some or all moment restraint is lost. The required modifications can be summarised as follows:

- section properties modified to allow for material lost due to corrosion;
- support properties modified if appropriate;
- ductility to be checked if plastic methods or moment re-distribution are used.

Specific guidelines cannot be given and each structure should be judged on its own merits. It is likely that insufficient information will be available to completely define the structural behaviour and some assumptions may have to be made. It may be necessary in some cases to carry out a series of analyses to determine the effects of the assumptions used and the sensitivity of the assessment results to them.

5.6.1.6 Fatigue strength

Fatigue strength may be affected if local discontinuities (pitting, local cracks, etc) form due to non-uniform corrosion. For bars having localised corrosion, taken from a bridge deck after 20 years service, the loss of fatigue strength is greater than can be attributed to the reduction in cross section and secondary bending Tilly [1988]. The reduction factors were reported as:

- 1.35 for level 1 pitting (up to 25% reduction in area),
- 1.70 for level 2 pitting (more than 25% reduction in area).

The fatigue properties of reinforcing bars in the form of S-N curves have been derived from tests and are available in BA 38/93 [Highways Agency, 1993]. This document can be used to determine the fatigue life of two classes of reinforcing bars for bars with minor or serious corrosion. The method used is similar to that presented in BS 5400: Part 10 [British Standards Institution, 1980]. There are no data available on the effects of corrosion on the impact strength of bars.

5.6.2 Alkali-silica reaction

Alkali-silica reaction (ASR, also called alkali-aggregate reaction) is a reaction between the hydroxyl ions present in the pore water of concrete and certain forms of silica which may be present in the aggregate. The reaction occurs on the surface of the reactive silica and produces a highly complex alkali silica hydrate in the form of a gel. This gel is expansive in nature and the volume increase can initially be absorbed within the pore structure of the concrete. However, when sufficient quantities of gel are produced, the expansiveness generates internal forces. If these forces are greater than the local confinement can resist, micro-cracking of the aggregate particles and surrounding past matrix will occur.

Figure 5.4: ASR affected bridge.

ASR normally develops slowly and is affected by the temperature and the availability of water. Thus the progress of the reaction is highly variable. Deterioration of concrete only occurs when the following three conditions are met:

1. Sufficient alkalinity of the pore water in the concrete;
2. The aggregate contains silica which is susceptible to attack;
3. Sufficient supply of water.

ASR has been recognised as a potential problem in concrete construction since the 1940s but only came into prominence in relation to UK bridges in 1971, and in relation to French bridges in 1987, when the first cases were discovered. ASR and its structural implications have been described in detail by a number of authors [Hobbs, 1990; Clark, 1989; McLeish, 1990; Larive, 1998] and much work has been carried out on methods of detecting and quantifying the resulting deterioration [Smith and Crook, 1989; IStructE, 1992]. Much of the research has been, quite rightly, directed towards the prevention of ASR in new construction by quantifying the risks in terms of materials and environmental. This is equally applicable to existing structures and normally the first step in the diagnosis of ASR derives from an investigation of the aggregate used and its susceptibility to ASR. This section deals with how the strength of a bridge is assessed once ASR has been correctly diagnosed.

World-wide, ASR is recognised as a serious form of deterioration affecting all concrete structures including buildings, bridges and dams. In the UK, some 300 bridges are thought to be affected by ASR, while in France, the number is 400. This has lead to the publication of a number of documents for assessing the strength of ASR affected structures, for example the UK Departmental Standard BD 52/94 [Highways Agency, 1994], British Cement Association [1992] and Godart *et al*, [1992].

5.6.2.4 Expansion due to ASR

The production of ASR gel and its expansion into the concrete pores can generate internal stresses, which are sufficient to cause the concrete to crack. ASR often produces a characteristic "crazed" cracking pattern, the exact form depending on the geometry of the concrete member and any restraint present in the concrete. Cracks can be between 0.1mm and 10mm in width, generally located within 25mm to 50mm of the concrete surface and lying perpendicular to it. In some cases cracks can go deeper into the concrete, up to 300 or 400 mm, and in the worst cases they go through the entire member. The cracking is very sensitive to restraint and the cracks tend to align themselves in a direction parallel to the direction of the restraint. The restraint can be provided by a combination of reinforcement, prestress, support system, imposed stresses, adjacent unaffected concrete, etc.

The extent of the current level of ASR present in a structure is normally represented by measuring overall in-situ restrained expansion of the concrete member. This can be determined by making a careful record of the position and geometry of the cracks present: see for example Fasseu [1997]. Any distortion of the structure should also be noted: this includes closing of joints, distortion of the member, bulging of the concrete surface, etc. The expansion can be quantified by measuring the widths of cracks crossing a straight line on the concrete surface, the sum of the widths divided by the length of the line giving the expansion in terms of an overall strain. However, this method is inaccurate for a number of reasons:

- the effect of restraints are unknown;
- the effects of reinforcement detailing are difficult to eliminate;
- the concrete strain between cracks is ignored;
- effects of other cracking phenomenon are included, eg, shrinkage, settlement, thermal contraction, other deterioration processes;
- ASR expansion can vary considerably through a concrete section.

Expansion can be estimated from knowledge of similar aggregates and concrete, examination of differential movements within the structure, etc. Actual ASR expansions can vary by ±50% from that calculated from the crack summation. Clark and Jones [1996] found that estimation of expansion can be improved by taking account of the angle of the crack and that this gave a lower bound to the actual expansion.

A better way to estimate the expansion is to use a distancemeter which measures the displacement between two studs which are fixed to the edges of the structure [Godart *et al*, 1996]. This device uses a wire made of INVAR steel or an infrared optical system to record the swelling in one direction. However, the device cannot be used to determine deformations prior to installation.

5.6.2.5 Loss of concrete strength due to ASR

When ASR was initially found in bridge structures in the UK it was assumed that it would lead to substantial reductions in strength. However, extensive research in affected bridges has shown that the structural affects of ASR is much less than was originally thought, even where extensive cracking has occurred. Nevertheless, structures suspected of being affected by ASR should be the subject of a detailed inspection with special attention given to the possibility of delamination and excessive cracking particularly in areas of high shear force or bond stress. However, even if the characteristic "crazed" cracking pattern is observed, it does not necessarily mean that ASR is the primary cause and the presence of ASR should only be accepted after all other explanations have been eliminated. Guidance on the correct diagnosis of ASR is given in many publications [eg, British Cement Association, 1992]. The primary parameter to be used is the amount of free expansion determined from the accelerated laboratory testing of cores taken from the structure. Experience has shown that strength reductions due to mild amounts of ASR are not great and expansions of less than about 0.7mm/m, based on core expansion tests at 20°C and 100% relative humidity, do not normally cause any significant loss of strength [Highways Agency, 1994].

The strength of concrete can be affected by ASR and it is necessary to rely on a combination of core results and engineering judgement to determine an appropriate value to be used in a strength assessment. Because of the cracking induced by ASR, it is often difficult to take and test cores and they often give very variable results. The UK experience is that the most appropriate values of concrete strength to use is obtained from the relatively intact cores: even where excessively cracked cores were rejected, the “good” cores still under-estimated the strength [Chana and Korobokis, 1991a; 1991b]. This suggests that a good deal of reliance must be placed on engineering judgement.

As far as other concrete properties (modulus, tensile strength, etc) are concerned, the normal assumption is to use the compressive strength as a reference: this is the basis of the equations in most codes of practice. For ASR affected concrete, these relationships are not appropriate as the various concrete properties are affected to different extents. The French experience is that the modulus is first affected and then tensile strength is reduced: only later is compressive strength affected [Godart, 1993]. It is recommended that the required properties be measured where possible using with appropriate tests and used directly in the assessment where possible. As for compressive strength, realistic values should be used in the assessment calculations. Table 5.2, taken from IStructE [1992], indicates the effect ASR has on concrete properties, as a function of free expansion.

5.6.2.6 Loss of bond

Bond strength is more closely related to tensile strength than compressive strength. However, it is difficult to determine an appropriate value for the tensile strength of ASR affected concrete due to the high variability of the test results. Clark [1989] reports that ASR reduces the bond strength of bars with low cover, but has no significant effect on bars with large cover. He proposes formulae which can be used to take account of depth of cover, restraint from transverse steel and lateral pressure, in addition to concrete strength. More details are given in Deliverable D11.

Table 5.2: Properties of ASR affected concrete as function of free expansion [IStructE, 1992].

Property	Percentage compared with unaffected concrete				
	0.5mm/m	1.0mm/m	2.5mm/m	5.0mm/m	10.0mm/m
Compressive strength	100	85	80	75	70
Uniaxial compressive strength	95	80	60	60	-
Tensile strength	85	75	55	40	-
Elastic modulus	100	70	50	35	30

5.6.2.7 Assessment methodology

The assessment methodology for ASR affected structures should follow the same procedures as for other forms of deterioration. Appropriate inspection is vital if ASR is to be correctly diagnosed and a safe assessment result is to be obtained. In-situ measurements are required to make a realistic estimation of the existing level of ASR expansion in the structure. These should be backed up by appropriate laboratory tests to determine the expected free expansion and hence the long term prognosis for the structure including residual service life.

Member resistance can then be calculated using normal assessment methods. Concrete properties used should be based on appropriate tests (cores, non-destructive tests) backed up by existing research information and engineering judgement as previously described. The only other modification to the procedure is the inclusion of additional stress induced by the ASR expansion. This will introduce tensile stresses into any reinforcement that is present and effectively prestress the section. This will not affect flexural capacity but can have a beneficial effect on shear capacity. While it is conservative to ignore such beneficial effects, it can partly offsetting any loss of capacity due to a reduction in concrete strength. Even a small amount of prestress may be enough to show adequate structural capacity. Clark and Jones [1996] found that the strain distribution along a bar in an ASR affected specimen is not constant, and as failure can occur in an area of least prestress, they suggest that it would be imprudent to allow for more than about 50% of the theoretical prestress. This should be sufficient to take account of any reduction in prestress with time.

Where compressive reinforcement is present, ASR expansion will tend to reduce the compressive strain in the reinforcement and this may result in concrete crushing failure prior to yielding of the reinforcement. Clark [1989] showed that this will occur at 1200 microstrain for high strength bars and 2500 microstrain for mild steel bars. In this case, the section should be considered to be over-reinforced. The UK bridge assessment code requires that such sections be over-designed in flexure by 15% to make allowance for the fact that the steel has not yielded and a brittle mode of failure may result.

Conventional methods of analysis such as a grillage analysis can be used as they lead to a safe assessed capacity. For statically indeterminate structures with cracked members, it is logical to use cracked section properties. As for any assessment, the section dimensions should be based on sound material as confirmed by site inspection, with due account taken of section loss due to corrosion, delamination and spalling of cover concrete. Reinforcement that is not considered to be effectively bonded to the concrete should be ignored. Consistency is required, and the same section should be used in the analysis and in the calculation of member resistance. Where uncertainty exists it may be necessary to re-analyse using different section properties to determine the sensitivity of the structure to the assumptions used.

5.6.2.8 Future deterioration

The assessment analysis described in the previous section should be carried out for two levels of deterioration, ie, the current level, and the long term level. The present capacity of a bridge is determined by carrying out the assessment analysis using the current level of ASR expansion as estimated from in-situ measurements. In most cases, this provides the most useful information to the bridge manager, as it enables immediate priority actions to be decided, including any requirements to restrict, rehabilitate or even close the bridge. However, even if the assessment shows that the structure in its current state is adequate, the bridge manager will be concerned with determining the long term capacity and, in particular, the remaining service life of the structure.

This can be tackled in a number of ways. The expected long term expansion can be determined from accelerated free expansion tests. This, along with assumptions regarding the restraints within the structure, can be used to calculate future levels of stress and the assessment analysis modified appropriately. In addition, long terms concrete properties (strength, modulus, etc) can be determined from the same cores on completion of the expansion tests. This approach is very approximate as future expansion depends on a large number of parameters which are difficult to take into account. In particular, the free expansion measured in cores may over-estimate the expansion in the original member because of the removal of the original restraints and the additional take-up of water which occurs with cores.

The expansion of individual members or components of the structure can be determined by putting a long term monitoring in place. This can consist of period measurements using, for example, DEMEC studs or other strain measuring devices. However, it will be necessary to carry out measurements over a long period of time, probably years, in order to determine clear trends. The measurements must be taken regularly so that other effects can be extracted from the movements, eg, those due to temperature. From this point of view a continuous logging system provides the best information, as the effect of daily and seasonal variations in environment can be removed from the measurements. This is the only reliable method of determining expansion rates and limits on future expansion.

Alternatively, the future development of ASR expansion can be assumed on the basis of information on similar concrete in a similar environment. It is important that the mix materials and proportions are identical, as even different cement batches can have a significant effect on ASR expansion.

5.6.3 Freeze-thaw action

Wet concrete exposed to cycles of freezing and thawing is one of the major causes of loss of durability and can cause early loss of strength even in good quality, well placed concrete. The actual mechanism of damage has been the subject of much debate. The principle mechanism is a consequence of the expansiveness of water as it freezes within the confines of concrete pores. This is due to the 9% increase in volume as water turns to ice [Neville, 1995]. It was originally thought that as crystals form within concrete, unfrozen water is expelled away from areas of freezing. This generates hydraulic pressure in the pores which exposes the pore walls to stress which can cause severe disruption to the concrete structure.

This mechanism was found to be inconsistent with some experimental studies which determined that the movement of water was towards, and not away from, freezing sites. This can be explained by considering the osmotic forces developed in freezing concrete [ACI, 1991]. The water in the cement past is a weak alkali. When the temperature drops, there is an initial period of super-cooling after which ice crystals start to form in the large capillaries. The result is an increase in the alkalinity of the unfrozen water in these areas. This creates an osmotic potential that causes unfrozen water in neighbouring capillaries to diffuse towards the site of freezing. This in turn dilutes the high alkali water, which allows more ice crystals to form. When the capillary is full of ice, further ice formation can cause the cement paste to fail. The presence of chlorides (or any of the common de-icers) increases the osmotic potentials created and thus accelerate the disintegration of concrete. The use of air-entrainment reduces the distance between air bubbles in the paste and the air bubbles compete with the capillaries for the unfrozen water. This prevents the diffusion of water towards the freezing site and the resulting build-up of ice crystals. The mechanism is different in the aggregate particles where the pore sizes are much larger and the hydraulic pressures developed on freezing causes most of the damage.

5.6.3.4 Type of damage

The type of damage and the susceptibility of concrete to them have been investigated by Fagerlund [1995] and Webster [1995] as part of BRITE/EURAM European Union funded project which examined the residual life of structures subjected to deterioration. Two types of damage were identified:

- internal damage
- surface scaling.

These have different mechanisms and often occur independently of one another. The type of damage depends primarily on whether the concrete surface is exposed to salt laden water.

Internal damage is caused by water freezing inside the concrete. This can occur where water is present in capillary pores in the cement matrix and aggregate, or in voids in the concrete. Damage can be induced in one freeze cycle if the water content is above a critical value. The freezing causes internal cracking, either in the cement paste, or the aggregate particles, or both. The cracks in the heart of the concrete are random but cracks also form parallel to the exposed surface. These are of particular concern as they are also close to, and parallel to, the main reinforcement. The damage results in loss of cohesion of the concrete, which can reduce compressive and tensile strength as well as the bond between the steel and concrete.

Surface scaling is caused by freezing of the concrete surface in contact with water. Surface scaling is only likely to be a problem when chlorides are present, either from de-icing salts or sea-water. The cement matrix is gradually broken up by this process with the eventual loss of sand and aggregate particles. The main result is that the concrete surface is gradually eroded away which affects the strength and stiffness as well as the appearance and durability. Surface scaling is a progressive problem, with each freeze-thaw cycle producing further loss of material. The extent of scaling is dependent on the severity of the environment, the rat of cooling and the chloride concentration. The main parameter is minimum temperature, and scaling at –20°C is often five times worst than at –10°C. Unlike internal damage, surface scaling is a progressive problem, with each freeze-thaw cycle producing a similar amount of lost material. Scaling can be assumed to be linear with time so that the loss of concrete cover can be estimated.

5.6.3.5 Susceptibility to freeze-thaw damage

The potential for freeze-thaw damage can be divided into different sets of parameters as follows:

- parameters associated with the external environment in which the concrete is placed;
- parameters associated with environment within the concrete;
- parameters associates with the concrete itself
- geometry and orientation of the structural member.

The likelihood of frost damage is clearly dependent on the external environment to which the concrete is exposed. The most important factor is the lowest temperature which occurs in the cycle. The rate of temperature change also has an influence as it affects the ability of the concrete to resist the induces strains. The number of freeze-thaw cycles is an important parameter for surface scaling as each cycles causes progressive damage. It has less effect on internal damage. However, once saturated concrete has been subjected to a freeze-thaw cycle, additional cycles will cause additional damage. The availability of water and the effect of drainage away from the concrete determine the degree of saturation of the concrete. The presence of chlorides, either from de-icing salts or from marine environments, accelerates the occurrence of surface scaling but has little effect on internal damage.

The internal environment is also important, the primary effect being availability of freezable water. Water can be present in the capillary pores of the cement paste or the aggregate particles, voids, cracks or interface zones between the aggregate and cement paste. Some of this water is not freezable, for example, water present in small size pores will not freeze even at very low temperatures. Susceptibility to frost damage is also a function of the chemical composition and physical properties of the concrete. Resistance can be increased by considering the type of cement used: for example, cements with lower C_3A and alkali content give better resistance to surface scaling. Highly porous aggregates absorb more water and can produce more damage if saturated. The porosity of aggregates must be limited to avoid excessive frost damage.

Even if freezable water is present, it does not necessarily mean that damage will occur. Air-entrainment agents are very effective in providing resistance to frost damage and are now generally used in all susceptible locations in a concrete structure. They work by increasing the air content and creating small well-distributed air bubbles within the concrete: this provided an escape path for the excess water expelled by freezing. However, the volume of capillary pores must be minimised since otherwise the amount of freezable water could exceed that which can be accommodated by the entrained air voids [Neville, 1995]. Thus for frost resistant concrete there is a requirement for a low water-cement ratio: this also ensures a high strength which can better resist the internal disruptive forces. The occurrence and extent of damage is also dependent on other properties of concrete. For example, creep can help to dissipate the strains developed.

Water reducing agents and plasticisers tend to reduce frost resistance and must be compatible with the cement and air entraining agents to avoid producing unsuitable air voids [Webster, 1995]. The use of cement replacement materials such as pulverised fuel ash and silica fume have little effect on frost resistance air content of the concrete.

The location, size, and shape of members are also of importance as these control the exposure to water and chlorides, the rate at which the concrete cools, the rate at which water is removed from the concrete, and the capacity for drying out of the concrete between wetting cycles. Reinforcement detailing can mitigate the effects of freeze-thaw damage: for example, the presence of transverse reinforcement helps to maintain bond strength even when cover concrete is damaged.

5.6.3.6 Effects of freeze-thaw action

The effects of freeze-thaw action on the structural performance of concrete needs to be understood in order to carry out a proper assessment of an affected structure. However, while the problem is widespread among northern hemisphere countries, there is very little qualitative information available on the effects of freeze-thaw damage on concrete properties. Most of the research effort has focused on avoiding the problem through the use of appropriate aggregate and air entrainment and how to avoid problems when concrete placing in cold weather. The following sections describe the structural consequences with a view to quantifying the effects in terms of strength assessment.

The most obvious symptom of freeze-thaw damage is the loss of effective concrete cross-section. This can be due to surface scaling, or internal damage resulting in delamination, or cracking of the concrete. Losses due to surface scaling can easily be identified by the appearance of the concrete surface, ie, the concrete paste is gradually eroded away leaving coarse aggregate particles partially imbedded in the remaining concrete. These aggregate particles can eventually be undermined and loosened. It is most commonly found on horizontal surfaces where inadequate drainage allows water to pond. Cracking and delamination of concrete due to internal freeze-thaw damage are more difficult to diagnose, but can cause significant loss of strength and stiffness. The main problem is the direct loss of cross-section, which can reduce stiffness and strength. However, the reduced cover can also result in lower bond strength, lower protection of reinforcement from corrosion and a poor general appearance.

The internal cracking induced by freeze-thaw action has a significant effect on the mechanical properties of concrete. In spite of the extent of the problem, there is relatively little quantitative data available on its affects. Fagerlund [1995] carried out a detailed investigation and found that assuming a loss of compressive strength of 35% is a reasonable lower bound. The cracking has a much greater affect on tensile strength than on compressive strength. The limited test data available has shown that a reduction of 70% in tensile capacity provides a reasonable lower bound for concrete affected by frost damaged concrete.

Bond strength may be the most critical parameter in determining the effects of freeze-thaw damage and should be given careful consideration. There is a lack of quantitative experimental results but the general approach used for other forms of deterioration is applicable. This entails expressing the bond strength in terms of the tensile strength of concrete and using this as the main parameter, base on either measured or assumed values, rather than the cube strength. Most codes (eg, EC2, BS 5400) simplify the expression for bond strength and ignore the beneficial effects of cover, generally by assuming a cover value of 2 times the bar diameter. They also assume a direct relationship between tensile strength and compressive strength based on the properties of good quality concrete. To determine the bond strength of freeze-thaw damaged concrete, it is useful to use a more comprehensive equations for bond strength which includes these parameters as input variables. Appropriate expressions are given in Deliverable D11. Good value for bond strength can then be obtained by making appropriate adjustments to the input variables based on site measurements, experience, engineering judgement, etc.

5.6.3.7 Assessment

The assessment methodology for frost damaged structures should follow the same procedures as for other forms of deterioration as summarised in Section 5. It is necessary to quantify the effects in terms of how they modify both the effective cross-section and the properties of the concrete. Appropriate inspection is vital if the damage is to be correctly diagnosed and a safe assessment result is to be obtained. The current level of damage must be measured in a systematic way and quantified in terms of its effect on concrete cross-section and properties. Member resistance for flexure, shear, etc, can then be calculated using normal assessment methods.

In determining effective cross-section, locations of potential losses should be identified for further examination. Where losses are thought to be significant, measurements should be made of the remaining sound material for use in a strength assessment. For surface scaling, this is usually straightforward. The position of original surfaces can normally be identified and losses measured relative to this. Design or as-built drawing can be used to help determine original dimensions. Areas of cracking should be examined using cores to determine the depth of affected concrete. Areas of delamination can be identified by tapping with a hammer. Where loss of concrete has occurred over large areas, the residual cover to reinforcement should be measured or estimated. This damage also reduces the level of protection against corrosion, and consideration should be given to the possibility of loss of steel cross-section. The extent of damage should be estimated. Small sections are likely to be more damaged than larger sections where the core might be protected. Cover concrete is more likely to be damaged on more exposed surfaces: this should be identified in the site inspection. All assumed values used in the assessment should be confirmed using material samples where possible.

Where possible, compressive and tensile strength should be obtained from cores taken from the structure. These can be used to determine the depth and extent of the damage. As for ASR damage, the results of core tests require careful interpretation and a good deal of experience and engineering judgement will be required to determine a strength value appropriate for use in a strength assessment. Cores from undamaged section should also be taken for comparison.

The tensile capacity of the concrete should be determined using appropriate tests and an appropriate value used to determine bond strength. Even where the cover concrete appears sound, losses of up to 90% can occur due to internal cracking parallel to the exposed surface. Where cover concrete has completely spalled away, it is unlikely that any significant bond strength exists and no local bond capacity should be assumed. This may result in the requirement to ignore any reinforcement when determining shear strength.

The methods of analysis used to assess frost damaged structures are likely to be the same as those used for conventional assessment. For most bridges this will consist of a line beam or strip analysis for most cases. Grillage analysis can also be used. It may be appropriate to consider different structural actions. For example, loss of bond due to cracking and spalling over central parts of beams may make a member behave as a ties arch, provided the reinforcement is adequately anchored. However, the engineer must be satisfied that any assumptions made are properly justified. Use of non-linear analysis such as yield line should only be used where sufficient ductility can be shown to exist.

5.7 IMPLEMENTATION

This chapter outlines general methods of taking account of deterioration in the determination of the carrying capacity of bridges. More details of the available models to be used for particular forms of deterioration are given in Deliverable D11. The models have been developed by various researchers and it should be noted that they are continuously being refined as more information becomes available. The methods described here should be used in conjunction with the general assessment procedures proposed by Workpackage 2. These are summarised in Chapter 4 and described in more detail in Deliverable 10. Additional background information is supplied in Deliverable D1.

Structural assessment as described in Chapter 4 is an important component of a practical bridge management system. Methods of determining the current carrying capacity of bridges are required if effective maintenance strategy both at project and network level are to be implemented. Deterioration must be taken into account if accurate results are to be derived from structural assessment and appropriate cost-effective maintenance strategies adopted. For example, the deterioration in a bridge may have reached such a level that a bridge manager might wish to replace it eventually due to high maintenance costs or because of the poor visual impact. However, knowledge of existing capacity may enable him to schedule the replacement in a more routine way over a longer term without having to take emergency steps such as traffic restrictions or immediate closure. Keeping bridges in service as a temporary measure pending rehabilitation through normal programming is a useful and cost effective approach.

Application of these models to particular structure is very dependent on the experience on the assessment engineer and relies heavily on his subjective judgement. This is because of the difficulties in quantifying the level of deterioration present and in relating this to degradation of structural performance. The assessment conclusion is only as good as input parameters used and appropriate information must be obtained through site investigation. Depending on the form of deterioration, various techniques are available for quantifying the structural effects: these are described in more detail in Deliverable D11. The assessment engineer must use his judgement, along with any information recorded as part of the site investigations, to determine whether the deterioration has a significant effect.

Modelling of deterioration for bridges assessment would generally be carried out as part of a Level 3 assessment (see Chapter 4). This is because many of the parameters needed to quantify the extent of deterioration and its affect on structural behaviour often require detailed site investigation. There is very little information available on the statistical characteristics of deterioration and currently use of these model within a full probabilistic approach is not currently possible.

5.8 FURTHER RESEARCH

This Workpackage highlighted a number of areas where further work is required, or would be useful, in the application of deterioration models to an effective bridge management system. These are as follows:

- Some of the models available are based on limited test data often on small laboratory specimens in controlled environments. Further development of these models is required and it is expected that they will improve as more information becomes available. Difficulties with field equipment and measurements, and in particular their dependence on the interpretation of readings, have resulted in a lack of reliable field data and problems of calibration with practical situations.

- A further problem is how to take account of the location of deterioration damage. In many cases, intrusive investigation requiring the removal of material for direct examination is the only accurate method of quantifying deterioration damage. An example of this is the corrosion losses in grouted post-tensioning tendons. These types of investigation can weaken the structure further and may also increase the future deterioration rate. A similar problem occurs where test specimens need to be taken from the structure to determine in-situ material properties. As a result, tests are normally only carried out at non-critical sections. Assumptions regarding how these results relate to critical areas are often arbitrary. Development of non-destructive techniques are required.

- If deterioration modelling is to be carried out properly, then reliable methods of collecting appropriate data through routine inspections is required. At present, inspection methods focus on condition monitoring with a view to determining maintenance and repair options. It is often not possible to use the results directly in a strength assessment. There is a need to modify bridge inspection procedures so that the results can be used more directly in determining load carrying capacity.

- Interaction between different forms of deterioration is often complex and difficult to deal with from a modelling point of view. Different forms of deterioration often occur together. For example, ASR and freeze-thaw can reduce corrosion protection and may lead to accelerated corrosion rates. The processes cannot be dealt with separately and appropriate models are not available.

- At present, it is difficult to incorporate deterioration models into a probabilistic strength assessment, as there is not enough statistical information available on the deterioration processes. There is a need to determine how these models can be used along with reliability techniques.

- Rate of deterioration depends on a wide variety of parameters and these need to be examined if residual life calculations are to be carried out accurately. In particular, repair and rehabilitation affect deterioration rates and these need to be investigated further.
- This Workpackage has focussed on the load carrying capacity at the ultimate limit state. The effect of deterioration on other limit states need to be examined, eg, serviceability, fatigue, etc.

5.9 REFERENCES

ACI (1991). *Guide to durable concrete*. ACI 201.2R-92, Report by ACI Committee 201, American Concrete Institute, USA.

British Cement Association (1992). *The diagnosis of alkali silica reaction*, Second edition, British Cement Association, Crowthorne.

British Standards Institution (1980). BS 5400: Part 10: 1980: *Steel, concrete and composite bridges: Code of practice for fatigue*. British Standards Institution, London.

Chana, P S and G A Korobokis (1991a). *Structural performance of reinforced concrete affected by alkali silica reaction: Phase I*. TRL Contractor Report 267, Transport Research Laboratory, Crowthorne.

Chana, P S and G A Korobokis (1991b). *Bond strength of reinforcement in concrete affected by alkali silica reaction: Phase II*. TRL Contractor Report 233, Transport Research Laboratory, Crowthorne.

Clark, L A (1989). *Critical review of the structural implications of the alkali silica reaction in concrete*. Contractor Report 169, Transport Research Laboratory, Crowthorne.

Clark, L A and A E K Jones (1996). *The effect of restrained expansion on the assessment of structures with ASR*. TRL Report 161, Transport Research Laboratory, Crowthorne.

Fagerlund, G (1995). *Freeze-thaw resistance of concrete – A survey of destructive mechanisms, technological factors and test methods*. BRIE/EURAM Project BREU-CT92-0591, Deliverable 30.4: Part 1.

Fasseu, P, and M Michel (1997). *Détermination de l'indice de fissuration d'un parement en béton*. Méthode d'essai LPC no 47, LCPC, Octobre 1997.

Godart, B (1993). *Progression dans les connaissances sur les phénomènes d'alcalis-réactions: Evaluation et surveillance des ouvrages*. Annales de l'ITBTP No 517, Octobre 1993, pp 134-162.

Godart, B, M Michel and P Fasseu. (1996). *Treatment of structures by waterproofing coating*. 10th International Conference on Alkali-Aggregate Reaction in Concrete, Melbourne, 19-23 August 1996.

Godart, B, G Kittel and P Fasseu (1999). *Manuel d'identification des réactions de dégradation interne du béton dans les ouvarges d'art*. Guide LCPC , Février 1999.

Highways Agency (1993). BA 38/93: Assessment of the fatigue life of corroded or damaged reinforcing bars. *Design manual for roads and bridges*, Volume 3, Section 4, Part 5, Highways Agency, Department of Transport, London.

Highways Agency (1994). BA 52/94: The assessment of concrete structures affected by alkali silica reaction. *Design manual for roads and bridges*, Volume 3, Section 4, Part 10, Highways Agency, Department of Transport, London.

Hobbs, D W (1990). *Alkali-silica reaction: its effects on concrete members and structures*. Developments in structural engineering, edited by B H V Topping, Chapman and Hall, London.

IStructE (1992). *Structural effect of alkali-silica reaction: Technical guidance on the appraisal of existing structures*. Institution of Structural Engineers, London.

Larive, C (1998). *Apports combines de l'experimentation et de la modelisation a la comprehension de l'alcali-reaction et de ses effets mecaniques.* Laboratoire Central des Ponts et Chaussées, Paris, France.

McLeish, A (1990). *Structural implications of the alkali silica reaction in concrete*. TRL Contractor Report 177, Transport Research Laboratory, Crowthorne, UK.

Neville, A M (1995). *Properties of concrete*. 4th edition, Longman Ltd, UK.

Pritchard, B P and M S Chubb (1987). Concrete bridge integrity assessment. *Structural assessment: The use of full and large scale testing*, edited by F K Garas, J L Clarke and G S T Armer, Butterworth, pp. 9-19.

Rodriguez, J, L M Ortega, J Casal and J M Diez (1996). Corrosion of reinforcement and service life of concrete structures. *Durability of building materials and components 7: Volume 1*, edited by C Sjostrom. E&FN Spon, London.

Smith, R L, and M J Crook (1989). *The non-destructive detection and mapping of ASR cracking in concrete*. TRL Contractor Report 156, Transport Research Laboratory, Crowthorne, UK.

Tilly, G P (1988). Durability of concrete bridges. *Journal of the Institution of Highways and Transportation*, Vol. 35, No. 2 (February), pp. 10-19.

Webster, M A (1995). *The structural implications of frost damage*. The residual life of concrete structures, Workshop for owners of structures, Imperial College, London, 6/7 April 1995.

Woodward, R J and F W Williams (1988). *Collapse of Ynys-y-Gwas bridge, West Glamorgan*. Proceedings ICE, Part 1, Volume 84, August, pp 635-669.

CHAPTER 6

MODELLING DETERIORATION RATES

6.1 INTRODUCTION

This chapter presents the results of Workpackage 4 (WP4) which are described in more detail in Deliverable D8. The workpackage has limited itself to dealing with modelling chloride ingress (initiation phase only) and monitoring corrosion within a bridge management system (BMS). This was due to the fact that the project had a limited time scale and that corrosion of reinforcement is the most serious and widespread type of damage for bridges. As such the workpackage has not investigated other types of damage e.g. freeze/thaw, carbonation, alkali silica reaction, sulfate attack which were described in Workpackage 3 (Chapter 5). Most deterioration types are described by one or more models [Sarja and Vesikari 1996] and the majority are under continuous development. The models differ in their accuracy and complexity and frequently do not take more than one damage mechanism into account. In fact, real damage to the structure or reduction in the bearing capacity is usually an extrapolation, based on engineering judgement, from the results of the models rather than from the models themselves.

The benefits of being able to predict the future condition of an element, or of the entire structure, are discussed in Deliverable D13 and can be briefly summarised as:

- allow the right maintenance/repair operation to be performed at the right moment in time thereby optimising the maintenance budget for that structure
- optimise the long term budget of the bridge stock
- forecast the safety index of individual structures and of the bridge stock
- determine the effect of non-optimal budget strategies.

Prediction of deterioration is an important aspect of bridge management for estimation of remaining service life and planning future maintenance tasks. The objective of the workpackage is to consolidate and improve existing knowledge concerning the modelling and surveillance of chloride penetration in concrete. Chloride ions are considered the primary cause of corrosion in concrete bridges. The results of this workpackage will help public authorities establish investigative procedures to monitor the danger of, and predict corrosion of their concrete structures. As such it will be an important tool in:

- increasing the durability of new concrete structures by allowing the identification and ranking in order of importance, the predominate factors affecting corrosion
- deciding the optimal time to carry out preventative maintenance or repair
- assisting in long term budget planning.

Workpackage 4 had the following three main tasks:

1. Creation of a database of condition parameters for several concrete bridges including local exposure conditions (micro-climates). Extra investigations are performed to confirm/confute the predictions made by the models selected in Task 2.

2. Selection, use and assessment of several chloride ingress models. The models investigated are: Fick's 2nd law (used as reference model); Selmer - Poulsen model (LightCon model) with improvements by Mejlbro (Hetek model). The use of neural network models is also evaluated.

3. Investigate the requirements of a bridge management system that incorporates prediction models, condition surveys and monitoring. Assessment of residual service life and a probabilistic approach is also addressed.

The symbols used the following sections are as follows::

Cl^-	chloride ion
w	water / smoothing parameter
c	cement / variable describing concrete cracks
t	Time
C	chloride content
D	diffusion coefficient
x	Depth
h	Height
wt	Wetting
o	Orientation

in	Inspection
ex	Exposure
s	Surface
i	Initial
a	Achieved

6.2 NETWORK / PROJECT DETERIORATION RATES

Bridges deteriorate naturally as they age due primarily to degradation of the construction materials by physical processes, such as corrosion of steel, alkali silica reaction of concrete aggregates, and sometimes due to overloading. A certain amount of deterioration is, therefore, to be expected, but there has been a tendency during the last few decades towards rapid rates of deterioration leading to large increases in repair costs and traffic restrictions, and a reduction in serviceable life. It is for these reasons that it is important to be able to differentiate those bridges with a high rate of deterioration from those deteriorating at a more normal rate. In particular a knowledge of the rate of deterioration enables the condition of the bridge at times in the future to be predicted and maintenance work to be planned and budgeted.

The deterioration of bridges can lead to a number of deleterious effects:

- loss of serviceability
- loss of load carrying capacity
- reduction in safety

- increase in traffic restrictions
- loss of aesthetic value

For example deterioration can result in a restriction to the number of lanes or to the maximum weight of vehicles that can be used. Safety of users can be affected by spalling concrete and aesthetics reduced by rust staining and cracking of concrete.

It is difficult to generalise about the rate at which bridge elements deteriorate because different bridges and even different parts of bridges are exposed to different macro and micro climates and even bridge elements of nominally similar construction and materials can have variations in concrete mix, cover depth and latent defects which can significantly influence the deterioration rate.

The term ‘rate of deterioration of a bridge’ appears to be straight-forward but in practice it hides many complexities. A bridge consists of several elements and components that are likely to deteriorate at different rates for a variety of reasons. In addition, the rate of deterioration will be affected by the frequency and extent of maintenance and repair operations.

To obtain a value for the rate of deterioration of a bridge would inevitably involve some sort of weighted average of the values for its different parts with the result that rapid deterioration of one element could easily be unintentionally concealed. The value of rate of deterioration for a bridge is only useful for network level bridge management; in this case the average value for a group of bridges can be used to evaluate the effectiveness of the maintenance strategy, that has been employed, by comparing measured values of deterioration rate with the target value. In order to evaluate the significance of deterioration for a particular bridge, in project level management, it is necessary to consider each part of the bridge separately before attempting to assess the effect of deterioration on the bridge as a whole. It is difficult enough to measure the deterioration of one component of a bridge, but it is even more difficult to assess how the increased stresses resulting from deterioration in one part influence the structure as a whole. It is clear that the increased stresses are often redistributed satisfactorily so that the strength of the structure is not compromised although it is difficult to define the circumstances when this will occur. In project level management of a bridge it is necessary to know the rate of deterioration of each element and component so that the optimal timing and type of maintenance can be adopted to ensure that the design life for a bridge is achieved at a minimum cost and with minimum disruption to traffic.

There are two main approaches to determining the rate of deterioration of a bridge element or component. Each approach is in an early stage of its development and has a number of limitations. The two approaches are by physical and stochastic modelling. Probabilistic models using Monte Carlo simulation are only appropriate for network level management. In general network level management is easier to carry out than project level management. Network level management is satisfactory for estimating the number of bridges requiring different types of maintenance (essential/ preventative) in a given year, but is limited by its inability to predict the timing and type of maintenance for particular bridge elements. The remainder of this chapter will concentrate on rates of deterioration for bridge elements since it is concerned primarily with project level bridge management.

The variable associated with deterioration rate is the condition of a bridge element. Condition can be measured by a physical parameter, such as chloride content of concrete or cross section of steel remaining at a given age, or by a stochastic parameter such as a condition state awarded by a bridge inspector. Physical parameters are usually continuous functions whereas condition states are discrete functions. The rate of deterioration of a continuous function can be expressed as a derivative in the normal way by measuring the slope of the tangent to the curve of the variation of the parameter with time, such as the chloride content at a specific depth from the concrete surface against time. A deterioration rate determined in this way can be used to predict the values of the parameter at future times if:

- there is a physical or empirical law relating the value of the parameter to time i.e. to the age of the bridge

- measurements indicate there is a linear relationship with time in which case the rate is constant and there is justification for a limited degree of extrapolation.

The rate of deterioration of a discrete function cannot be expressed in the same way as a continuous function since the deterioration time curve consists of a number of steps and is therefore discontinuous. The deterioration of discrete functions is normally expressed in terms of the probability of changing from one condition state to another in a specified interval of time. For example where condition is assessed on a five point scale, from state 1 to state 5, the probability of an element changing from state 1 to state 2 during the interval between inspections can be regarded as a deterioration rate.

Transition probabilities between two states are usually determined by Markov chain modelling which involves a number of stages:

- sub-divide the whole stock of elements into a number of groups of similar form of construction, construction material and environment

- for each group, using the condition states awarded during bridge inspections, determine the best fit function for the variation of average condition state with time; call this A

- using a Markov chain model determine the average condition state at different ages as a function of transition state probability; call this B

- using optimisation techniques find the set of transition probabilities that minimise the difference between functions A and B over specified age intervals.

The optimal transition probabilities represent the most accurate deterioration rates for the group of bridge elements, based on the results of inspections made during the lives of the constituent bridge elements. A knowledge of the condition state of an element at a given age and the optimal transition probabilities relating to that age can be used to predict the future condition of particular bridge elements. In essence this form of stochastic modelling determines the deterioration rate of a group of elements of a given type and age and applies this to a particular element of the group, with known current condition state, to predict its change in condition state as it ages.

This approach takes account of the known variability in condition of nominally similar bridge elements, but is subject to a number of limitations:

- for new materials or forms of construction there is little historical evidence on which to establish transition probabilities

- the Markov chain assumption that future condition depends only on the current condition and not on how the condition has varied earlier in the life of the element

- the reliability reduces the further into the future that predictions are made.

This approach has two useful practical advantages:

- the condition state data are readily available from bridge inspections and do not necessitate large amounts of testing work

- the condition state range covers all conditions from new to unserviceable and each state is linked to an appropriate type of maintenance – preventative, repair or strengthening.

The approach that is studied in some depth in this chapter is physical modelling applied to a particular deterioration process, namely the ingress of chloride ions into concrete bridge elements. When chloride ions penetrate to the depth of reinforcing steel in sufficient quantities they cause the steel to corrode resulting in cracking and spalling of the concrete and loss of steel section. The general approach is to analyse the chloride content of a number of concrete bridge elements as a function of age and depth from the concrete surface, to find how closely the data fit solutions of Fick's Law of Diffusion or empirical laws.

If a good fit can be found for concrete bridges over a sufficient age range then this approach could be used to predict the chloride – depth profile at different ages. This approach has the benefit that it is based on physical principles that are usually better understood by engineers than statistical approaches and it also takes account of the variability in condition observed in nominally similar bridges. There are a number of limitations however:

- It may not be possible to find a law that covers a sufficient number of bridges to be practically useful.

- The data required would necessitate a significant amount of testing.

- Physical models usually cover only a part of the deterioration experienced over the life of a bridge element; for example for a bridge element at risk from reinforcement corrosion the deterioration consists of a number of processes – chloride ingress, corrosion initiation, corrosion propagation, concrete cracking and spalling.

- New concrete materials may not obey the same deterioration law.

Nevertheless if a physical law relating chloride content and depth could be reliably applied to a sufficient range of concrete mixes, exposure conditions and ages then it could be used to predict the chloride content – depth profile well into the future. This information would be necessary, but not sufficient for the manager to decide the type and timing for maintenance. In particular it would also be necessary for the manager to know the threshold chloride concentration above which corrosion takes place in order to evaluate when corrosion will start or for how long it has been occurring. Section 6.6 gives an indication of the quantity of data required and the effect of microclimate when predicting the likelihood of corrosion.

6.3 CONDITION SURVEY DATA - DATABASE

Data from Slovenia, Norway and France were collected and analysed. Even though chloride analysis has been performed on a large proportion of all concrete bridges, only a limited number of structures could be retained for further analysis. This was done to limit the number of calculations but also because there is only a limited number of bridges where there is a complete set of data. The project was also primarily interested in structures with good chloride profiles (accurate measurements at several depths), taken from several locations on the structure and taken at several ages at the same location.

All the Slovenian structures included in the case studies are placed on highways. The bridges are situated in a continental climate environment with hot summers and cold winters. During the winter, de-icing salts are used to provide suitable traffic conditions. The French bridge is located about 50 km south-west of Paris and has similar conditions. The Norwegian structures are all coastal bridges that cross a fjord or a sound. They all have piers placed in the sea.

The Slovenian bridges are: Ivanje Selo, Slatina, Škedenj2, Preloge (Figure 6.1) and Šepina bridge. The Norwegian bridges are: Gimsøystraumen (bridge chosen for the method of inverse cores), Hadsel and Sandhornøya bridge. The French bridge is: A11 PS12-10.

Figure 6.1: Longitudinal view on the viaduct Preloge, Slovenia. Wetting of the concrete surface of the edge beam under the damaged joint of the deck's pre-cast elements

Table 6.1: Example of chloride data. Note that in this case three profiles were determined for the same location and age.

Norway	**Chlorides at Gimsøystraumen Bridge**						
Height Above Sea level m	Location code	Depth Mm	Cl^- weight % of cement	Factor m_{conc}/m_{cem}	Cl^- weight % of concrete	Age at time of inspection	Location
3.6	Column 3	7.5	2.05	6.4	0.3200	12	West side
	(Cl.3.07)	22.5	2.05	6.4	0.3200		
		40.0	1.38	6.4	0.2150		
		62.5	0.49	6.4	0.0760		
		87.5	0.25	6.4	0.0390		
3.6	Column 3	7.5	1.47	6.4	0.2300	12	West side
	(Cl.3.08)	22.5	1.54	6.4	0.2400		
		40.0	0.93	6.4	0.1450		
		62.5	0.51	6.4	0.0800		
		87.5	0.27	6.4	0.0420		
3.6	Column 3	7.5	1.34	6.4	0.2100	12	West side
	(Cl.3.09)	22.5	0.93	6.4	0.1450		
		40.0	0.58	6.4	0.0900		
		62.5	0.22	6.4	0.0340		
		87.5	0.14	6.4	0.0220		

In addition to chloride data, see Table 6.1, certain information about the structures was also collected. This was based on a Norwegian database [NBI 2000] and categorised as follows:

- General information about the structure: county, commune, name, number, length, etc
- Description of superstructure: type of element, specified cover, specified concrete quality, w/c ratio, quantity of cement, entrained air, curing regime, etc
- Description of columns/piers: method of construction (sliding, climbing, ...)
- In-situ measurements: location, structural element, axis or span, shape (circular, square, ..), distance from land, height above water, orientation (north, south, ..), micro-climate chloride measurements, depth of carbonation, cover, electro chemical potential, relative humidity
- Damage/rating in measurement locations: type of damage, degree of damage, chiselled concrete away to inspect rebar (y/n).

6.4 CHLORIDE INGRESS MODELS

Chloride penetration is mainly due to a combination of chemical and physical processes. The most important processes are:

- Diffusion; a process due to a gradient of chloride concentration in the concrete. Gradient means that the chloride concentration is higher at the concrete surface than in its core. As chloride is dissolved in water, diffusion process occurs only in pore solution inside concrete.

- Capillary suction of chloride contaminated water; a process that takes place in empty or partly filled concrete pores. It means that water (moisture) content and concrete porosity are the main parameters that influence capillary suction.

Some chloride binding reactions occur between cement components (chloroaluminates, etc.) and chloride ions. These reactions are either physical adsorption, chemical reaction, or a combination of the two. Chloride binding is strongly influenced by climatic conditions. All the prediction models, which were applied in the BRIME project, are based on the diffusion process but they include several supplementary assumptions. The models are :

- Fick's law (reference model) describes a pure diffusion process. Any diffusion law is valid only in concrete which is permanently saturated with water. It means that it is not valid in the concrete surface layer which sometimes can be dry. So, climatic conditions and concrete porosity determine how thick this concrete layer is where the diffusion law does not apply.

- LightCon and Hetek models are based on the diffusion process. However, boundary conditions in these models that are constant in Fick’s law may be time dependent, e.g. chloride content on concrete surface. Concrete porosity and cement type are important parameters in this model.

- Conditional Average Estimator - Hybrid Neural Network (CAE-HNN). In this model, determination of the whole chloride profile at a certain location is based on a set of measured data with similar features using neural networks. The ingress of the chlorides is based on diffusion. For this reason a substitute diffusion coefficient is calculated between the measured points. As there is currently not enough data available on chloride profiles at the same locations at different times, Fick's law is used to make time prediction.

It should be noticed that the diffusion coefficient is determined for a given substance (chloride ion, etc.) entering a given material. If this material changes, for example, after ageing, this coefficient also changes.

It was initially planned to use two other models: Vesikari and Steen. However, the analysis tool for the Steen model was not obtained and the time necessary to develop an equivalent tool outweighed the possible benefits of its use. The Vesikari model is based on a feature of diffusion law, which states that a relationship exists between times t and depths L, for which chloride content has a given value ($t = K.L^2$). According to this model, factor K depends on concrete water-cement ratio and on the environment. This model can be used in the design phase for concrete bridge decks, but not with condition survey data. As such it was not pursued in this project.

Extensive investigations on chloride ingress in concrete form the basis for the research on chloride induced rebar corrosion. In this project, observations were taken from real structures, made of different concrete grades, and subjected to different environmental conditions and exposure times.

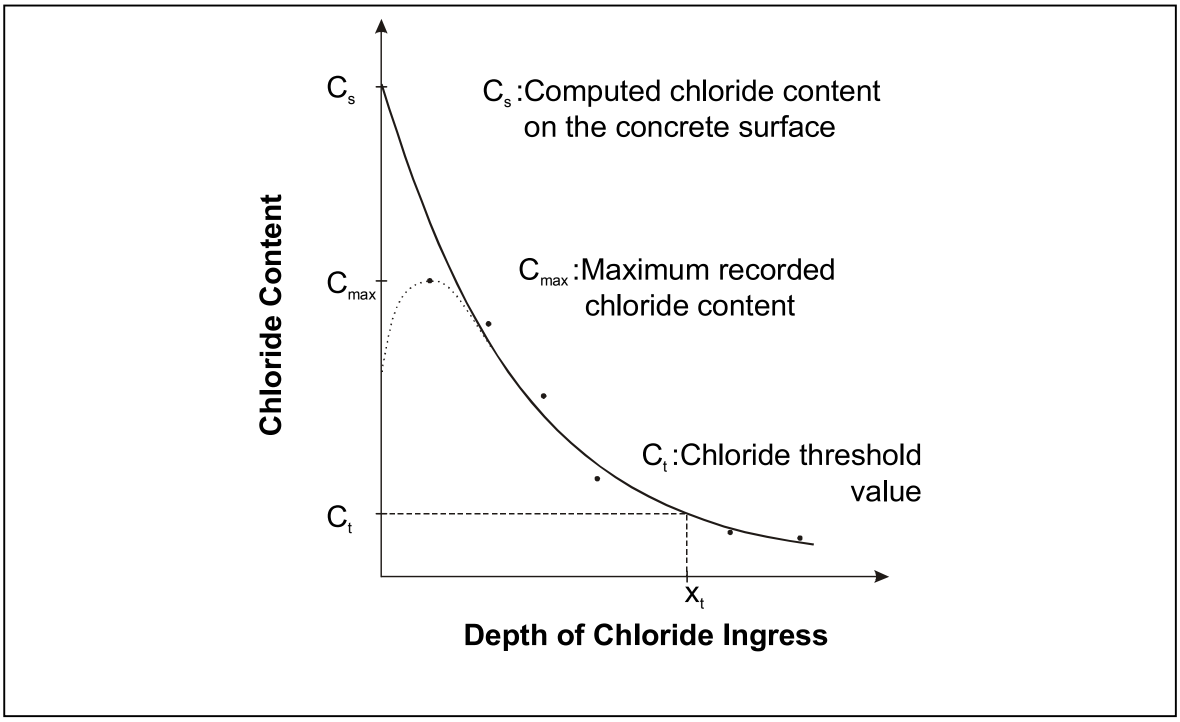


Figure 6.2: Simplified model for chloride ingress in concrete

The chloride profile, figure 6.2, is a simple illustration of some of the essential parameters when modelling chloride ingress in concrete and hence, also the service life of the structure. As a simplification the design service life can be taken as the initiation period with the propagation period conservatively neglected. For the assessment of residual service life, models for both initiation of corrosion and corrosion rate are required.

Fick's second law is adequately described in the literature and is not presented here. The LightCon [Maage et al 1999] and Hetek [Frederiksen et al 1997] models are similar and only the Hetek model and the neural networks are briefly presented here. Due to the similarities between LightCon and Hetek models a special section explaining the differences is given in Deliverable D8.

In order to estimate the future chloride ingress into concrete the Hetek model assumes that at least the following information is available:

- the age t_{in} of the concrete structure at the time of inspection
- the age t_{ex} of the concrete structure at the time of first chloride exposure
- the composition of the concrete, i.e. the type of binder, and the *w/c*-ratio

- the environment of the concrete, i.e. atmospheric, splash or submerged (ATM, SPL and SUB)
- the chloride profile of the concrete obtained at an inspection time $t_{in} >> t_{ex}$, e.g. by the chloride diffusion coefficient and the chloride content of the concrete surface.

When it is not possible to obtain reliable information from the specification of the concrete structure, the inspection must be supplied with a thin section analysis of the concrete in question. Testing the concrete by NT Build 443 [NT Build 443 1995] and/or by the 'method of inverse cores" [Deliverable D8, 2000 and Poulsen and Frederiksen 1998] may strengthen the estimation of the future chloride ingress into the concrete.

The HETEK-model for future chloride ingress into concrete is based on the following assumptions:

- chloride C in concrete is defined as the 'total, acid soluble chloride'
- transport of chloride in concrete takes place by diffusion. There is an equilibrium of the mass of ingress of (free) chloride into each element of the concrete, the accumulation of (free and bound) chloride in the element and an ongoing diffusion of (free) chloride in the element towards a neighbour element, and so on
- the flow of chloride F is proportional to the gradient of chloride $\frac{\partial C}{\partial x}$. The factor of proportionality is the achieved chloride diffusion coefficient D_a
- the achieved chloride diffusion coefficient D_a depends on time, the composition and environment of the concrete
- the boundary condition C_S is a function of time t, and the composition and environment of the concrete
- the initial chloride content of the concrete C_i (per unit element of the concrete) is uniformly distributed at time t_{ex}
- the relations used for the deterministic parameters with respect to the environment (ATM, SPL and SUB), the time and the composition of the concrete are documented at the Träslövsläge Marine Exposure Station on the west coast of Sweden (south of Gothenburg) [Frederiksen et al 1997].

A hybrid neural network-like approach (CAE - HNN) was developed by ZAG and involves an empirical treatment of the phenomena. This is very suitable for problems where models are based on the experimental data. It is shown elsewhere [Grabec and Sachse 1997] that such an approach corresponds to the use of the intelligent systems.

We assume, that the complete phenomenon, in our case in-depth chloride ion ingress, is characterised by a sample of the measurements on N testing specimens that are described by a finite set of so called model vectors:

$$\{\mathbf{X}_1, \mathbf{X}_2, ..., \mathbf{X}_N\} \quad \text{... /Eq. 1/.}$$

Such a finite set of model vectors will be called a database in the subsequent text.

In formulating the model of the phenomenon $Cl^- = Cl^-(x, h, o, wt, c)$ it is further assumed that one particular observation of a phenomenon can be described by a number of variables, which are treated as components of a vector:

$$\mathbf{X} = \{x, h, o, wt, c, Cl^-\} \qquad \text{... /Eq. 2/,}$$

where x is depth, h height above sea level, o orientation, wt wetting, c variable which describes concrete cracks and Cl^- chloride ion concentration at depth x.

Vector **X** can be composed of two truncated vectors:

$$\mathbf{P} = \{x, h, o, wt, c; \#\} \text{ and } \mathbf{R} = \{\#; Cl^-\} \qquad \text{... /Eq. 3/,}$$

where # denotes the missing portion. Vector **P** is complementary to vector **R** and therefore their concatenation yields the complete data vector **X**. The problem now is how an unknown complementary vector **R** can be estimated from a given truncated vector **P** and sample vectors $\{\mathbf{X}_1, \mathbf{X}_2, ..., \mathbf{X}_N\}$. By using the conditional probability function the optimal estimator for the given problem can be expressed as [Grabec and Sachse 1997, Grabec 1990 and Perus et al, 1994]:

$$r_k = \sum_{n=1}^{N} A_k \cdot r_{nk} \qquad \text{... /Eq. 4/}$$

where

$$A_k = \frac{a_n}{\sum_{j=1}^{N} a_j} \qquad \text{and} \qquad a_n = \exp\left[\frac{-\sum_{i=1}^{L}(p_i - p_{ni})^2}{2w^2}\right] \qquad \text{... /Eq. 5/.}$$

r_k is the k-th output variable (e.g. Cl^-; k is equal to 1 in a given problem), r_{nk} is the same output variable corresponding to the n-th model vector in the data base, N is the number of model vectors in the data base, p_{ni} is the i-th input variable of the n-th model vector in the data base (e.g. x, h, o, wt, c), p_i is the i-th input variable corresponding to the model vector under consideration, and L is the number of input variables. w describes the average distance between the specimens in the sample space and is called the smoothing parameter.

A general application of the method does not include any prior information about the phenomenon. Because in some cases there is still a lack of data, *a priori* information is needed to better fit a particular phenomenon. By a relatively simple improvement [Fajfar and Perus 1997], the method can be effectively used for the modelling of many problems in civil engineering. Furthermore, CAE (conditional average estimator) stems from a probabilistic approach and phenomena are not treated as being just deterministic.

For the application of the CAE-HNN a database is needed. It consists of model vectors, which can be presented in general case in matrix form as:

$\mathbf{mv}_1$	=	p_{11}	p_{12}	**...**	p_{1L}	r_1
$\mathbf{mv}_2$	=	p_{21}	p_{22}	**...**	p_{2L}	r_2
...		**...**				**...**
...		**...**				**...**
$\mathbf{mv}_N$	=	p_{N1}	p_{N2}	**...**	p_{NL}	r_N

... /Eq. 6/.

The main task in the first step is therefore to represent the measured data and, if necessary, *a priori* knowledge about the phenomenon in vector or matrix form. Finally, in the second step the choice of appropriate value of smoothing parameter is needed. The parametric study has shown that the appropriate value for modelling chloride ion penetration into concrete is $w = 0.15$. Due to the lack of experimental data on time dependence of chloride ion penetration, Fick's 2nd law is used for time prediction.

6.5 CALCULATIONS AND RESULTS

The models were used to calculate the time to corrosion initiation for three different cover depths and three different threshold values (critical chloride content). These are:

Cover depths: 25, 30 and 50 mm
Threshold values: 0.4, 0.7 and 1.0 % of cement weight.

The results for one location on one bridge are given on the next page (Fig 6.3). It should be noted that the Hetek model is the only model of the four that can take into account two or more chloride profiles. All the other models have based their calculations on the last profile (7.5 years). For the nine bridges, a total of 63 chloride profiles, representing 44 locations, were collected and judged fit for further investigation. Several of these profiles were taken from the same location at different ages of the structure. All 44 locations were calculated with all four models. The results of these calculations are presented in a similar manner to figure 6.3 in Annexe A of Deliverable D8.

Prediction of actual chloride concentration based on a previous inspection was also carried out using the models. In total seven locations on four bridges were investigated. While this test did not show which model was best it did highlight the problem of predicting chloride ingress from a single inspection.

It is difficult to say which of the models gives the best prediction of future chloride ingress. This is primarily due to the lack of data from the same location over an extended time scale. In addition to predicted values, the time, data required and complexity involved in using the models must be taken into account in their evaluation.

6.6 PROBABILISTIC APPROACH TO SERVICE LIFE

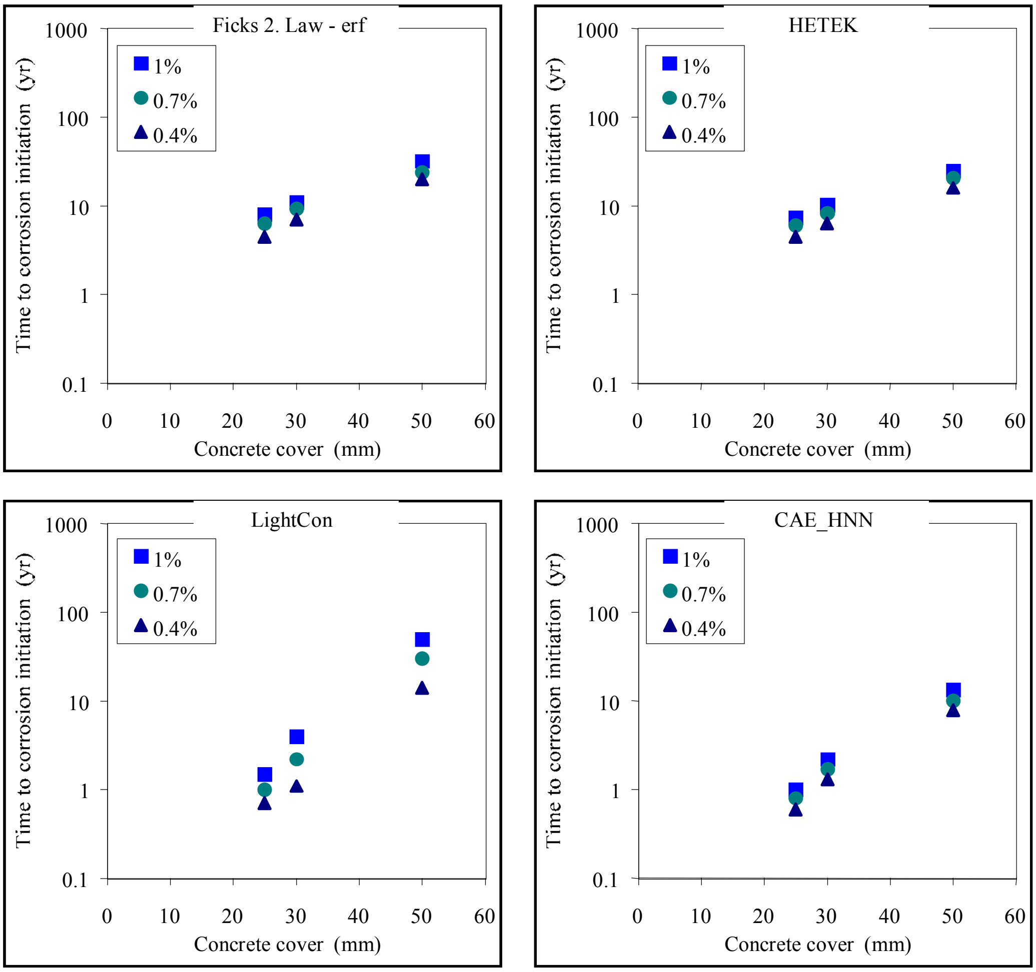
Predictions of chloride ingress at one point on a structure are of little value and a more global approach is needed. The approach described here uses data obtained during the OFU Gimsøystraumen Bridge Repair project [Blankvoll 1997 and OFU 1998] and the Durable Concrete Structures project [Fluge and Jakobsen 1999].

The chloride load for Norwegian coastal bridges is mainly a function of the height above sea level. Figure 6.4 from Fluge and Jakobsen [1999] shows maximum measured chloride content in the concrete, representing 1200 chloride profiles sampled from 30 bridges, all more than 15 years old. The recordings are obtained at different heights above sea level, from all sides of the cross-sections and from bridges exposed to different environmental conditions. On the basis of these findings the exposure conditions, represented by the maximum measured chloride content near the concrete surface, have been classified in four exposure zones, mainly governed by the height above sea level:

I	0-3 m
II	3-12 m
III	12-24 m and
IV	above 24 m.

This kind of classification based on in-situ data is a very important factor in future durability design standards.

Sandhornøya, NP A3, East



Measured chloride profile

Depth (mm)	7.5	22.5	37.5	52.5	67.5				
Cl (% conc.)	0.43	0.032	0.004	0	0.006				3.5 years
Depth (mm)	5	15	22.5	37.5	52.5				
Cl (% conc.)	0.462	0.512	0.106	0.011	0				7.5 years

Cover depth (mm)	Cl_crit (% cement)	Time to corrosion initiation (years) Ficks 2.	HETEK	LightCon	CAE HNN
25	0.4	4.5	4.5	0.7	0.6
25	0.7	6.4	6	1	0.8
25	1	8	7.4	1.5	1
30	0.4	7	6.3	1.1	1.3
30	0.7	9.3	8.3	2.2	1.7
30	1	11	10.2	4	2.2
50	0.4	20	15.9	14	7.8
50	0.7	24	20.5	30	10.1
50	1	32	24.8	50	13.4
Calculated D (mm^2/yr)		17	22.9	10	-
Calculated C_s (% concr.)		1.22	1.15	1.22	-

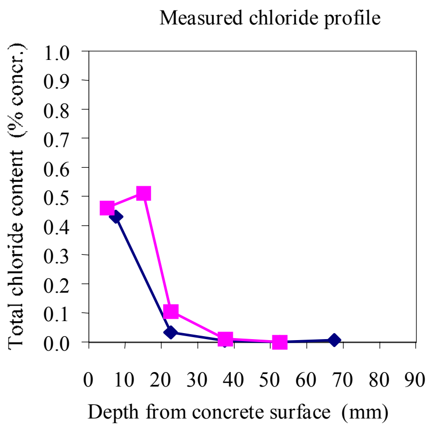


Figure 6.3: Chloride profiles and results from analysis, Sandhornøya Bridge, NP A3, East.

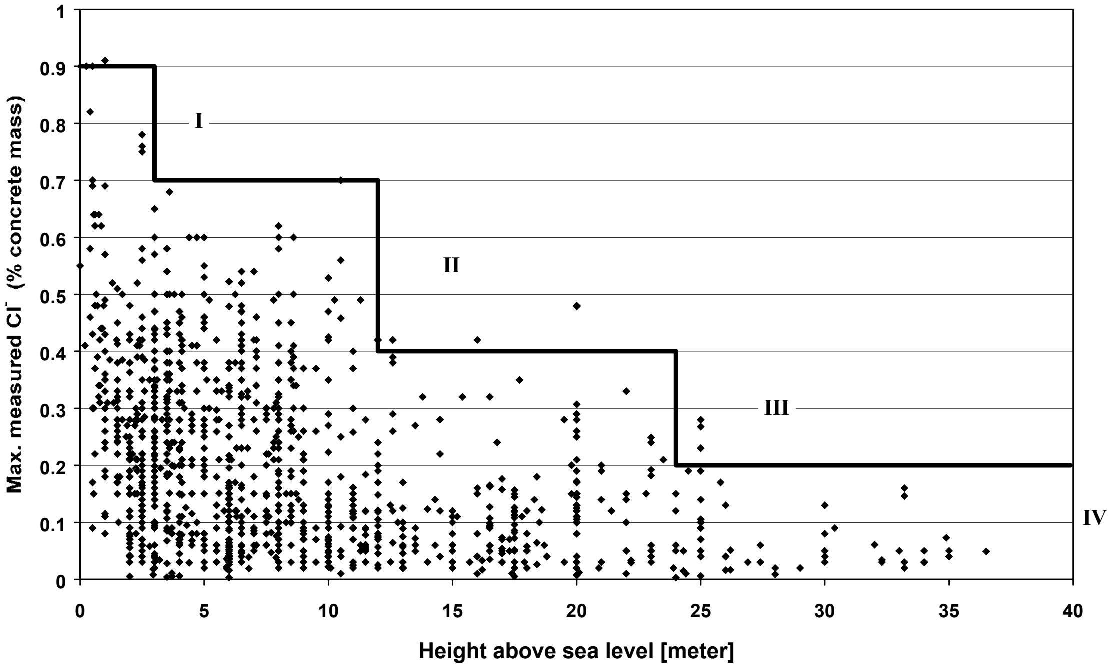


Figure 6.4: Coastal bridges. Max. recorded chloride content in the concrete versus height above sea level

The chloride load is to some degree time and material dependant. However, for structures more than 10-15 years old, there is no indication that the recorded chloride loads are significantly influenced by the time of exposure. Also, any influence of the concrete on the measured chloride loads was in this context negligible. Hence, for all practical purposes the chloride loads as defined above can be taken as a function of the environmental conditions only (i.e. time and material independent). This is the first step in determining a durability load that can be used in design.

The inspections on Gimsøystraumen bridge, performed after 12 years in service also consisted of measurements of concrete cover, chloride analyses and visual observations in more than 200 locations distributed over the 21 investigated cross-sections along one section - a 126 meter long post-tensioned box-girder. The height above sea level of the bridge deck in this section varies from 10.4 to 18.4 meters.

The concrete was grade C40 for the superstructure. The cement content was 375 kg/m^3 OPC and no silica fume was used. Obtained strength varied between 36.5 and 54.0 MPa with a mean of 43.2 MPa. Concrete of grade C40 normally corresponds to a bulk diffusion coefficient of $12\text{-}15 \cdot 10^{-12}$ m^2/s when tested according to NT Build 443 after 28 days.

The concrete cover was specified at 30mm minimum. The average concrete cover was determined at 29 mm with a standard deviation of 5.5 mm. This implies that approximately 50% of the rebars have concrete cover less than the specified 30 mm and 10% less than 22 mm. The statistical distribution of the concrete cover, based on more than 3500 independent readings, is shown in figure 6.6.

The in-situ diffusion coefficient for the bridge section was computed based on chloride analysis of samples of concrete powder drilled from 4 holes at each location. The average in situ diffusion coefficient after 12 years of exposure was determined to be $1.1*10^{-12}$ m^2/s with a standard deviation of $0.25*10^{-12}$ m^2/s.

The bulk diffusion coefficient for a 12 year old "chloride free" concrete sample, drilled from the middle of the superstructure and tested according to NT Build 443, was found to be $7.0*10^{-12}$ m^2/s. Taking the ages of concrete hardening into consideration this value corresponds roughly to a bulk diffusion coefficient of approximately $14*10^{-12}$ m^2/s at 28 days.

The maximum chloride content obtained on the cross-section 11.9 meters above sea level varied between 0.07% Cl^- and 0.38% Cl^- of concrete mass on the windward and the leeward side respectively. Curve fitting of the measured values gave a maximum computed chloride content C_s on the leeward concrete surface of 0.625% Cl^- of concrete mass (figure 6.5) and an in-situ diffusion coefficient of $1.4*10^{-12}$ m^2/s .

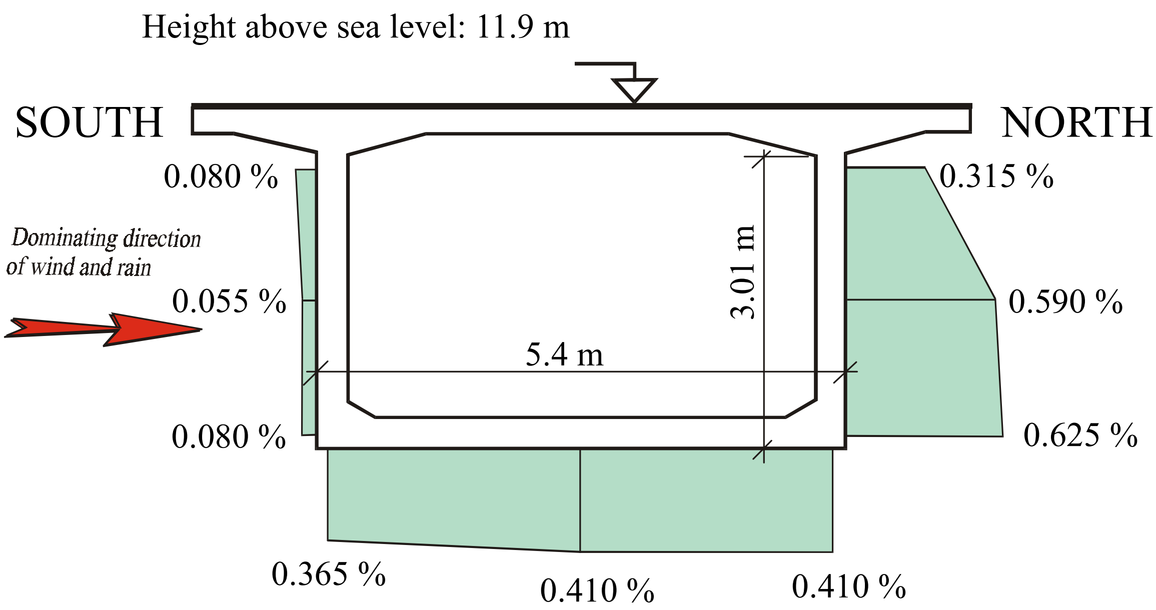


Figure 6.5: Chloride content on the concrete surface C_s, computed from measurements on the cross-section 11.9 m above sea level, Gimsøystraumen bridge.

On the basis of the investigations referred to and summarised below, the chloride ingress and critical depth in the concrete after 10-12 years exposure, i.e. the age of the bridge when inspected, has been computed using the following values.

Chloride load:

- Leeward side $C_s =$ 0.625 % Cl^-
- Windward side $C_s =$ 0.010 % Cl^-

Material resistance:

- In situ diffusion coefficient after 12 years exposure $D = 1.1 \cdot 10^{-12}$ m^2/s
- standard deviation $s = 0.25 \cdot 10^{-12}$ m^2/s

The critical depth was computed on the basis of a threshold value of 0.07% Cl^- of concrete mass (approx. 0.45% mass of cement).

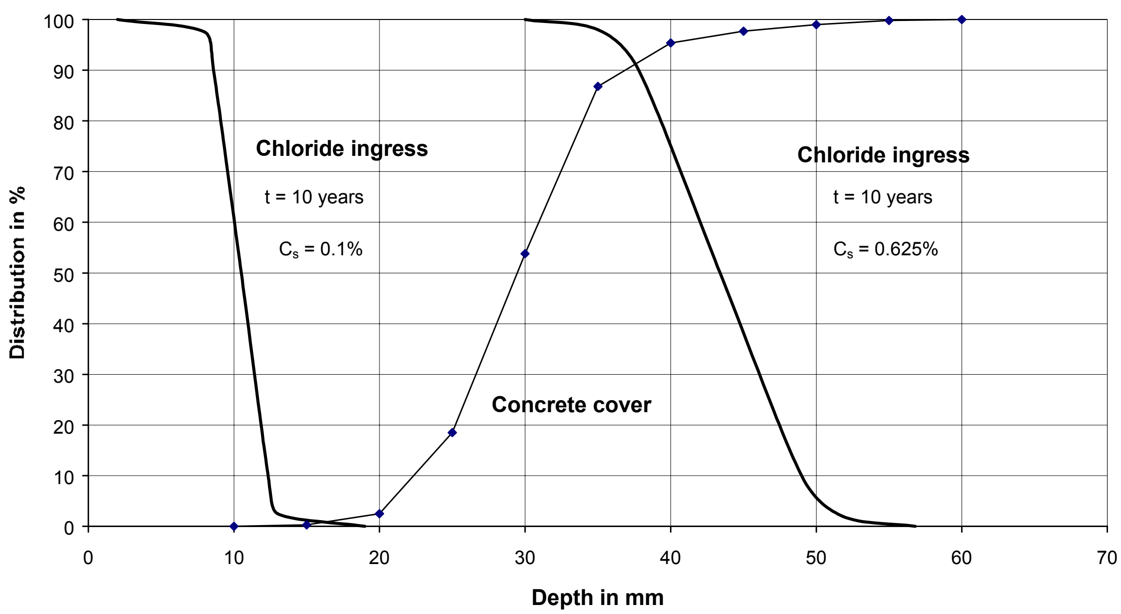


Figure 6.6: Statistical distribution of concrete cover and critical depth.

The most probable critical depths after 10 years of exposure are shown in figure 6.6 for the windward and the leeward side of the structure respectively. Figure 6.6 shows clearly that the probability for rebar corrosion on the windward side is negligible. However, on the leeward side, the probability for depassivation and rebar corrosion exceeds 90 percent. These results concur with the visual observations of no signs of corrosion on the windward side and active corrosion on the leeward side.

6.7 MONITORING

It must be remembered that measurement techniques are only one aspect of a successful monitoring programme. Other aspects of importance are:

- clearly defined objectives
- strategy plan
- installation
- data acquisition
- data processing
- verification and reliability
- documentation
- presentation of results for end-user.

A computerised monitoring system should meet specifically defined objectives and not only be a ‘nice to have’ installation. These objectives can be one or several of the following:

- ensure a structures load bearing capacity and serviceability during its planned lifetime
- optimise repair and maintenance costs
- verify design rules
- research and development.

It is important to note that a computerised monitoring system is not a project or an objective in its own right, but rather a tool within a project. Therefore, in general, all data should be processed and analysed together with results from condition surveys before conclusions about the state of the structure can be drawn.

Several of the above-mentioned objectives may be applicable for a particular monitoring project dependent on the type of construction and the likelihood and consequence of damage. Applications where a computerised monitoring system can be particularly interesting are:

- individual structures that are representative of a section of the bridge stock due to similarities in design, loading and/or construction material
- special or prototype structures
- structural elements that are inaccessible or difficult to access
- structures in a particularly aggressive environment
- structures where damage has been detected and monitoring is used to gather further information before repair is carried out
- individual structures that have been repaired where the type of repair is typical for a large number of bridges
- structures where substantial repair work has been carried out.

In addition, it is important to recognise that a strategy for computerised monitoring will require input from a multi-disciplined task group.

What parameters to measure, which sensors to use and where should these be located are essential questions to answer before any detailed planning can take place. These are not easy questions to answer. In fact, the answer must be firmly based upon the strategy of the monitoring system and on the expected results from the sensors. If, for example, the only guidelines given are: *obtain a warning of impending corrosion,* then this is frequently not sufficient to design a system that will satisfy a client in the long run. It does not indicate which of the following areas of the structure are to be monitored (figure 6.7):

- the most exposed
- the most critical, from a safety point of view, or
- the most expensive to repair.

Figure 6.7: What is the best location for sensors - the most exposed area, the most critical area from a safety point of view, or the area most expensive to repair?

Given that the thickness of concrete cover varies according to the degree of exposure, one should be able to assume that the durability or resistance to reinforcement corrosion should be approximately equal throughout the entire structure. However, a bridge with equal amounts of corrosion throughout its structure has yet to exist. This is a clear indication of our lack of understanding and control of durability. A proposal for a durability surveillance system is presented in Deliverable D8 and NBI [1999] where many of the above elements are taken into account. Figure 6.8 shows the location of the measurements points.

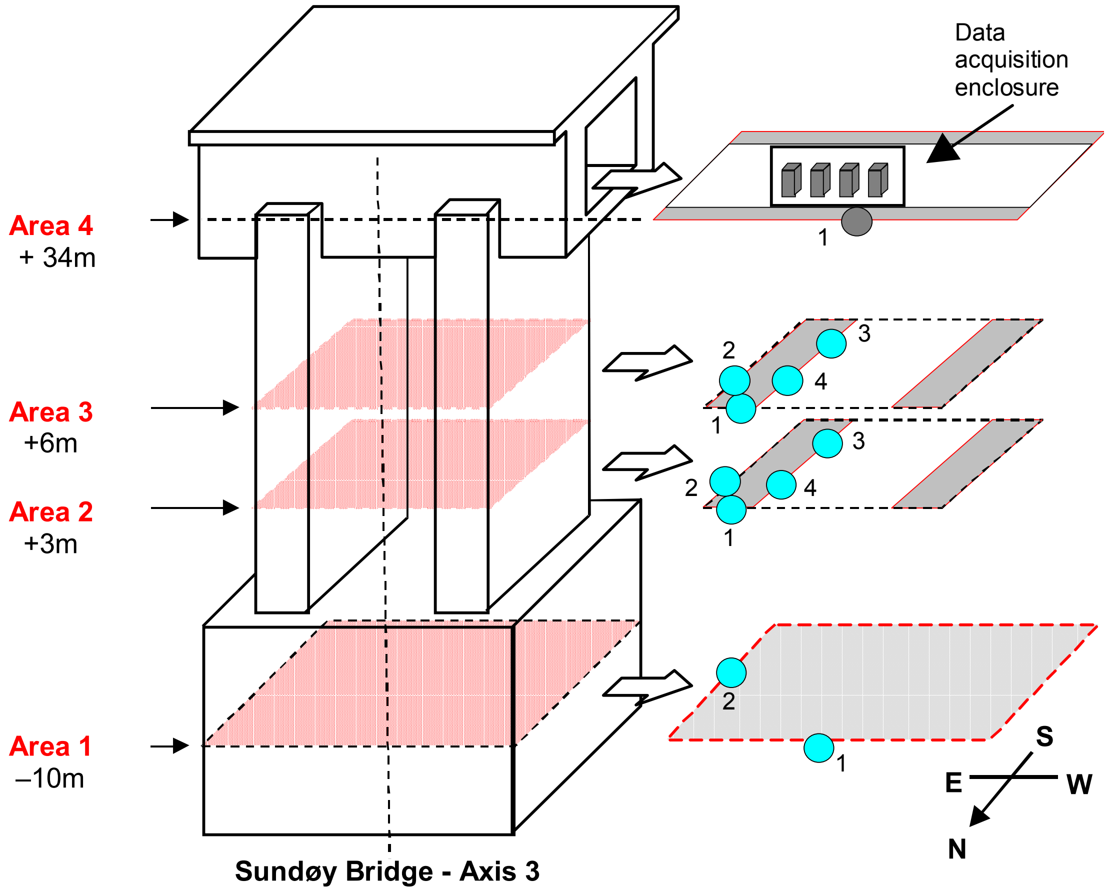


Figure 6.8: Proposal for location of measurement areas and points for durability surveillance. Height above sea-level is also indicated.

6.8 BRIDGE MANAGEMENT SYSTEM

Inspection routines should be modified and reference zones established in order to improve the measured data and thereby increase the reliability of chloride ingress models. In addition, data collection from all bridges should be systematised to facilitate future exploitation.

While it is not realistic to establish reference zones for all bridge structures, structures representative of a large number of similar structures can be identified. In this way, a few structures can be properly investigated and the results cautiously applicable to a large number of structures. Categories for similar structures will vary from country to country and the following list is meant as an example:

- age
- climate
- use of de-icing salts
- distance from the sea
- production methods
- material (silica fume, fly ash, lightweight aggregate, etc).

Once a structure has been selected, section 6.7 gives a good introduction into the problem of selecting the location of these reference zones. The overall objective of these reference zones must be kept in mind, i.e. minimise service life cost.

Before selecting reference zones for a particular bridge, the following must be identified:

- different climatic zones of the structure
- distribution of cover in these zones
- distribution of concrete durability parameters in these zones.

When this information is available, three to five reference zones can be selected. It must be pointed out that factors such as height above sea water, aspect (south facing etc) and changes in structural geometry all contribute to the delimitation of the different climates zones. Reference zones should therefore be defined comfortably within identified climatic zones so as to avoid later discussions of "climatic contamination".

Certain practicalities should also be taken into consideration when selecting the reference zones. Firstly, a zone should be sufficiently large to be able to provide enough sample material for chloride determination for decades. Secondly, due to the increased number of site visits, access to the reference zones should not be too difficult as this will increase costs.

Chloride profiles at these locations should be determined more frequently than for other structures generally. For new structures, profiles after 2, 5 and 10 years of service should give a good basis for any prediction models. These intervals should also give relatively large and measurable changes in chloride concentration. These intervals must be increased for existing structures so as to provide measurable differences.

For bridges where de-icing salts are the predominate source of chlorides, records of salting should be kept and profiles should be taken at approximately the same time of year each time, preferably in the autumn.

Caution must be exercised in determining the correct exposure time when using profiles obtained after a relatively short service interval of, for example less than 10 years, as different parts of the structure may have significantly different ages. This is particularly true for large coastal bridges which frequently have a construction period of 2-4 years. This implies that for a main, in-depth inspection performed after five years of service (ie open to traffic for 5 years), the superstructure may have been exposed to chlorides for 5½ - 6½ years while the columns may have been exposed for 7 - 9 years.

As with any long-term project, documentation of performed activities and decisions taken should be complete so as to provide a solid basis for future interpretation.

For new structures, the selection of reference structures is a good opportunity to install some durability surveillance equipment which can be controlled at the same time as the chloride profiles are determined. This instrumentation may be used to compare "similar reference zones" in addition to providing valuable continuous information allowing seasonal and environmental trends to be quantified.

Test slabs may also be cast and exposed at the structure in order to allow a large quantity of samples to be examined. Micro-climate, concrete and casting conditions for these test slabs should be as similar as possible to the actual structure.

A bridge management system is an important and essential tool for the success of the reference zones as it can:

- provide links to the similar structures
- generate inspection plans which specify chloride profile determination
- provide continuity to the long-term durability surveillance as employees frequently change employer or area of responsibility.

6.9 CONCLUSIONS

Today's chloride ingress models are not sufficiently accurate to automatically initiate a maintenance repair. An understanding of the corrosion process, the limitations of the models and the uncertainty surrounding the measured data are all necessary before any reliable decision can be made. As such, only experienced engineers or corrosion experts should be allowed to act upon the results generated by a chloride ingress model. In addition, engineering judgement will still play an important role in assessing the extent of the damage, the associated maintenance/repair cost and in combining different maintenance tasks from different elements/bridges in order to optimise the limited resources available.

It is important to understand the limitations and possibilities a chloride ingress model can have in a bridge management system (BMS). The model cannot predict how much reinforcement will corrode every year nor can it predict with any certainty when corrosion will initiate. However, it can predict when there will be a certain danger of corrosion initiation. As such, chloride ingress models can be used for planning possible future maintenance. For assessing structural capacity an additional model for predicting the corrosion rate during the propagation period is required.

To fully exploit the possibilities of chloride ingress models, inspection routines should be modified. This will allow reference zones to be established, improve the measured data and increase the reliability of the models. In addition, data collection from all bridges needs to be systematised to facilitate future exploitation. This will greatly benefit neural network models but will also allow new models to be developed.

As a final note, durability surveillance must be based upon and compliment the existing inspection programme of the bridge stock.

6.10 FUTURE WORK

There are currently two main fundamental weaknesses when trying to predict the time to corrosion initiation: the accuracy of the input data and the threshold value for corrosion initiation of real structures.

Based on the results of case studies, further study of different parameters that affect the chloride penetration into the concrete structures is needed. Further improvement of chloride ion ingress models into the concrete structures due to the marine environment, using de-icing salts and air pollution is also needed. For this reason a large amount of data concerning the structure, its quality of construction and environmental load must be collected and put into a database. Further research is also needed on prediction and correlation of chloride ion diffusion coefficient based on measurements on site and in the laboratory. Test methods and models for durability that reflect an actual structure in its environment must also be developed.

Research is needed concerning the application of different types of concrete surface protection coatings against penetration of chlorides, other aggressive ions and gases. Further studies of the effectiveness of these coatings with respect to the time of the first application and period of application are needed. Further research is also needed in on site detecting of the stress corrosion cracking in pre-stressed and post-tension structures. Research on the use of corrosion inhibitors and their application is going on worldwide and further studies are needed to find out the most suitable application and the stability of corrosion inhibitors over time.

Studies of modelling the remaining service life of structures based on available data are needed both at the network and at the project level. Further studies and research to define the appropriate parameters for modelling the expected and remaining service life of different types of structures and/or structural elements are also needed.

6.11 REFERENCES

Blankvoll A (Ed) (1997). *Repair of concrete structures: From theory to practice in a marine environment.* Proceedings Int. Conf. Svolvær, Norway, May 1997.

Brühwiler E and P Mivellaz (1999). *From corrosion of existing to durability of new concrete structures*. Int. Conf.: Structures for the future - The search for quality. IABSE Symposium Rio de Janeiro, Brazil, August 25-27.

Fajfar P and I Peruš. (1997). *A non-parametric approach to attenuation relations*, Journal of Earthquake Engineering, 1 319-340.

Fluge F and B Jakobsen (1999). *Something Concrete about Durability*. 5th International Symposium on Utilization of High Strength / High Performance Concrete, Norway, 20 – 24 June.

Frederiksen J M, (EDT.), Nilsson L-O, Sandberg P, Poulsen E, Tang L and A Andersen (1997). HETEK. *A System for Estimation of Chloride Ingress into Concrete*, Theoretical background. Danish Road Directorate Report No. 83. Denmark.

Grabec I and W Sachse (1997), *Synergetics of Measurement, Prediction and Control*, Springer-Verlag, Heidelberg.

Grabec I. (1990). *Self-organization of neurons described by the maximum entropy principle*, Biol. cybern. 63 (1990) 403-409.

Maage M, Helland S and J Carlsen (1999). *Chloride penetration into concrete with light weight aggregates*. Report FoU Lightcon 3.6, STF22 A98755 SINTEF. Trondheim, Norge.

NBI (1999). *Bestandige betongkonstruksjoner (Durable Concrete Structures) Instrumentert tilstandsovervåking - Ett skritt videre (Project report "Condition monitoring - one step further.* (in Norwegian). Oslo, Report no. 4.1.

NBI (2000). *Bestandige betongkonstruksjoner (Durable Concrete Structures).* Final report (in Norwegian). NBI, Oslo.

NT Build 443 (1995) *Nordtest method: Concrete hardened; accelerated chloride penetration.* Published by Nordtest, ISSN 0283-7153. November.

OFU (1998). *Gimsøystraumen bru. "Climatic loads and condition assessment, final report.* (in Norwegian). Publication no. 85, Norwegian Public Roads Administration (Statens vegvesen), Oslo, Norway. 248 p. 1998.

Peruš I, Fajfar P and I Grabec (1994). *"Prediction of the seismic capacity of RC structural walls by non-parametric multidimensional regression"*, Earthquake Engng & Struct. Dyn. 23 1139-1155.

Poulsen E and J M Frederiksen (1998). *The method of inverse cores*. Private communication 1998. (to be published).

Roelfstra G, Adey B, Hajdin R and E Brühwiler (1999). "*Condition evolution of concrete bridges based on a segmental approach, non-destructive testing and deterioration models"*, Proceedings, 8th International Bridge Management Conference, Transportation Research Board - U.S. Federal Highway Administration, Denver, April 26-28.

Sarja A and E Vesikari (1996). *Durability Design of Concrete Structures*. RILEM report 14. Report of the RILEM Technical Committee 130-CSL. E&FN Spon, 1996.

6.12 BIBLIOGRAPHY

Baroghel-Bouny V, Chaussadent T and A Raharinaivo (1995). *"Experimental investigations upon binding of chloride and combined effects of moisture and chloride in cementitioud materials"*. RILEM International Workshop on chloride penetration into concrete, Saint-Rémy-lés-Chevreuse, Oct. 15 - 18.

Brühwiler E and P Mivellaz (1999). *From corrosion of existing to durability of new concrete structures*. Int. Conf.: Structures for the future - The search for quality. IABSE Symposium Rio de Janeiro, Brazil, August 25-27.

Francy O, Bonnet S, Francois R and B Perrin (1997). *"Modelling of chloride ingress into cement-based materials due to capillary suction"*. Proceedings 10th International Congress of the Chemistry of Cement, H. Justnes, ed., Gothenburg, Sweden, June 1997, Vol. 4, Paper 4iv 078, 8pp.

Mejlbro L (1996). *The complete solution to Fick's second law of diffusion with time-dependent diffusion coefficient and surface concentration.* Proceedings of CEMENTA's Workshop on Durability of Concrete in Saline Environment. Danderyd, Sweden.

Raharinaivo A and G Grimaldi (1995). *"Methodology for monitoring and forecasting the condition of a reinforced concrete structure, under corrosion"*. IABSE Symposium Expanding the lifespan of structures. San Francisco (USA) August 23 - 25.

Roelfstra G, Adey B, Hajdin R and E Brühwiler (1999). "*Condition evolution of concrete bridges based on a segmental approach, non-destructive testing and deterioration models"*, Proceedings, 8th International Bridge Management Conference, Transportation Research Board - U.S. Federal Highway Administration, Denver, April 26-28.

CHAPTER 7

CRITERIA FOR DECISION MAKING

7.1 INTRODUCTION

The decision on whether to repair or replace a bridge has become of great concern to many bridge authorities. This is due to the high rates of deterioration that are occurring on many structures and the subsequent reduction in functionality that has sometimes has occurred.

Any disruption to traffic on a heavily trafficked bridge incurs a high cost for society. Closure of a structure results in traffic being forced to use alternative routes which not only increases journey times but adds to congestion on the network and increases the risk of accidents. Even partial closure of a structure increases journey times and may force some drivers to make detours rather than face congestion on the structure.

There is therefore a need for a set of rational criteria that ensure that bridges are maintained in a safe and serviceable condition, with the required load carrying capacity. This must be done throughout their design life at a minimum life-time cost whilst causing the least possible disruption to traffic.

The analysis of the costs of alternative maintenance procedures highlights the need to quantify such factors as the cost of traffic delays, the deterioration rate of bridges, the effective life of repair systems and the time value of money. By doing this it is possible to put forward a programme of maintenance optimised to achieve a set standard condition at minimum long-term cost.

The main objective of Workpackage 5 is to prepare decision criteria that help to choose the best repair option that takes into consideration safety, durability, functionality and economy. This chapter describes the work undertaken in Workpackage 5 and puts forward a method for decision-making that compares the alternative maintenance options for a deteriorated bridge.

The symbols used in the following sections are as follows:

C	Costs
V_S	Salvage value
P_f	Probability of failure
r	Net discount rate of money
ADT	Average diary traffic flow
t	Time
d	Length of detour
RI	Repair index

7.2 PRELIMINARY WORKS

7.2.1 Literature review

The first stage in the development of a decision system was an extensive literature review of the available documentation on bridge management systems in general and, specifically, on methods for selecting the most appropriate option for repair or replacement of a bridge.

A detailed literature search was carried out on Spanish and international databases, and libraries. In addition documents received from other BRIME partners, technical information from conference proceedings, scientific magazines and the proceedings of Bridge Congresses over the last few years were analysed.

Summaries were prepared of the most relevant papers, publications and documents, which gave a brief but precise synopsis of the contents of the documents.

The length of the summaries varied accordingly to the content of the documents, ie, longer summaries were prepared for those works that dealt directly with the subject of selecting the best alternative from several options for the repair/replacement of a bridge.

The information collected can be divided into two groups:

- Publications that deal, in general terms, with bridge management systems ie their origin, need, fundamentals, characteristics, use, research and future developments.

- Documents that describe specific BMSs developed in some countries or analysing particular modules of these BMS that are mainly focused on issues that are relevant to this work such as the criteria for repair or replacement, maintenance strategies or the evaluation of user costs.

7.2.2 Review of existing decision system for bridge repair/replacement

The questionnaire on bridge management that was described in Chapter 2 contained a question concerning the criteria used to determine whether a structure should be repaired or replaced. The replies showed that most countries do not use a specific tool for such decisions. The exceptions are *Denmark, Finland* and the *USA*. *Denmark* uses a prioritisation program and *Finland* a repair index.

In the *USA,* a variety of bridge management systems are used and the methods developed for decision making are summarised below:

- PONTIS. The optimum policies are developed at a network level based on the minimum expected life-cycle costs over an infinite planning horizon.

- BRIDGIT. The bridge level actions are developed by minimising the expected life-cycle costs over a 20 year period. The optimum sequence of actions and the optimum time to take the action are considered. Actions may be triggered by the need for upgrading, rehabilitating deteriorated structures, replacement, etc. Benefits are determined as cost savings to the user.

- State Specific Systems. Five states have developed their own BMS: Alabama, Indiana, New York, North Carolina and Pennsylvania. The New York BMS has a specific tool for decision making but it has not been implemented by engineers using the system.

In *France*, decisions are made using engineering judgement with several levels of control both technical and financial. Likewise, engineering judgement is used in *Germany* and *Spain*. *Great Britain* uses cost benefit analysis, future needs and engineering judgement.

Norway

In *Norway*, proposals for repairing damage are based on a description of the damage and the condition assessment, and are prepared using a coding system describing the type of works and processes involved in the repair. Cost estimates are prepared for the proposed action, and an indication is given as to the year in which these activities should be undertaken so as to ensure that the specified standard is maintained.

When the cost of repairs recommended following a major inspection or special inspection exceeds 20% of the bridge replacement value, alternative strategies should be investigated.

At least two different strategies should be investigated depending on what is available. In addition to maintenance costs, road user costs and any costs to society, if affected by the various strategies, are also taken into account.

The following strategies may be considered:

- Temporary action: Minor repairs that enable major works or bridge replacement to be postponed.

- Major action: Extensive repair work over a short period that significantly extends the remaining service life of the bridge.

- New element/bridge: No repair work undertaken; however, the existing element/bridge is replaced at the end of its service life.

For each strategy different technical solutions may be considered.

When maintenance costs exceed 50% of the replacement value, the third strategy must be considered.

The net present value of the selected strategies is estimated and this forms the basis for selecting the optimal strategy. Factors that normally do not enter into cost estimate are also included before the final decision is made. Such factors may include: age of the bridge, remaining service life, carrying capacity, bridge width/road curvature, vertical clearance, traffic safety, future usage, aesthetics, historic value, etc.

Germany

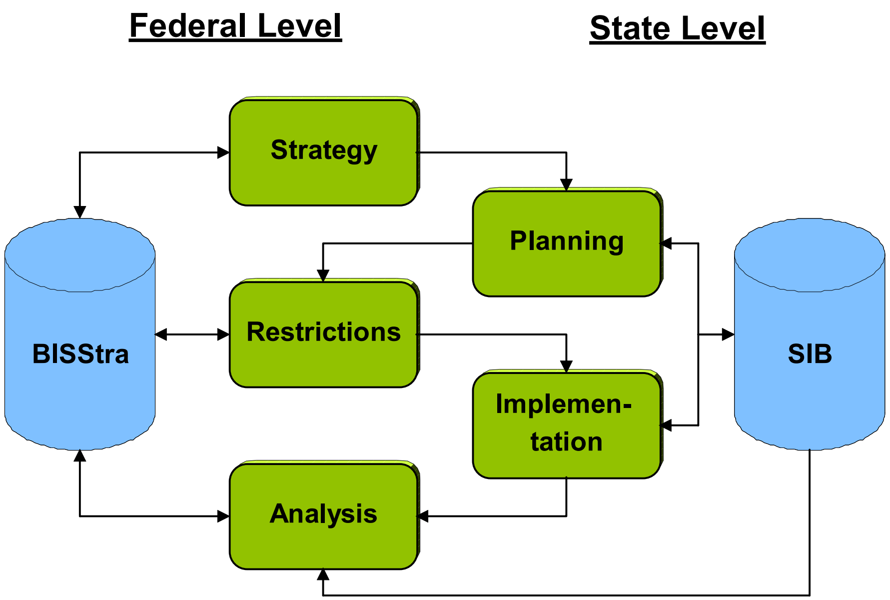
The *German* Federal Ministry of Transport is currently developing a comprehensive management system for structural maintenance (Figure 7.1). The planned management system is to provide the Federal Ministry with an overview of the current condition of structures at the network level, estimate future funding requirements and develop strategies for achieving long-term objectives and carrying out routine maintenance. In addition, it will provide the state bridge authorities with the programmes of work required to obtain improvements at the project level that maintain structures in an acceptable condition and meet network level strategies, long-term objectives and budgetary restrictions.

The first task at the state level is to develop and compare the maintenance options for each structure for the planning period ie the planning process. This results in a prioritisation of maintenance measures at the project level and a budget estimate, ie a list of all the projects scheduled for the planning period at the state network level. The resulting program is then fine tuned at the federal network level in the controlling process. This is undertaken to ensure that the final programme meets specified network level objectives (Haardt, 1998).

The planning process consists of recording and evaluating the condition of each structure in accordance with the German inspection rules DIN 1076 and RI-EBW-PRÜF. (These are guidelines produced by the Federal Ministry of Transport, Building and Housing for standardised recording, condition assessment and investigation of the results of inspections according to DIN 1076.) Structures in a critical condition are given priority for maintenance. The remainder are subjected to the maintenance planning process and the results from the inspections are used to determine the requirements for maintenance. This may require an additional evaluation of the structure if the results of regular inspections are not detailed enough for maintenance planning. This could be a structural assessment or a more detailed inspection. The ideal times for maintenance are determined on the basis of deterministic deterioration models for the individual bridge elements. Suitable maintenance options are identified and the resulting changes in condition that would be achieved are predicted. Alternative actions are then ranked using cost/benefit analyses to determine the preferred solution at the project level. A network-wide comparison of cost/benefit-ratios is used to provide an urgency rating for the preferred option for the planning period. This is used to determine the financial requirements at the network level and results in the first draft of the maintenance program. Budgetary restrictions make it necessary to optimise this maintenance program at the network level. In some circumstances, this optimisation will change the proposed measures at the project level if the budget is not sufficient to carry out all the proposed maintenance measures. In future a computer system will be used by the administration for maintenance planning. It will use cost/benefit-analysis at the project level and optimise the maintenance program for a limited budget.

The controlling process is carried out at the federal level and uses information from the Federal Ministry of Transport database (BISStra) and the results from the state level planning process to develop the final programme. As part of the controlling process, expenditure forecasts are prepared, analysed and updated, the draft maintenance programs are analysed and rated, and annual expenditure on maintenance that has previously been undertaken is reviewed. This information is used to determine the available budget, amend the proposed measures and update the technical rules (DIN 1076, ZTV-ING, RI-EBW-PRÜF).

The maintenance programmes are implemented by the state administrations. They plan and produce the required documentation for carrying out the works. Factors that are taken into consideration include the ability of the agencies to resource the work, and the possible combining of maintenance options, for example treatment of a number of bridges along a length of road. This is currently done manually but it is planned to develop a computer programme that will take these restrictions into account. The final programme is announced, funds allocated and the work implemented and documented. The results are submitted to the following year’s planning and controlling processes. This planning module can be extended to include project preparation, administration of measures and documentation.



Controlling Process (Federal Level):

- Analysis of maintenance practice
- Fixing of aims and restrictions
- Database "Bundesinformationssystem Straße (BISStra)"

Planning Process (State Level):

- Object related maintenance planning
- Yearly maintenance programmes
- Medium term demand

Implementation Process (State Level):

- Implementation of measures
- Documentation
- Database "Straßeninformationsbank (SIB)"

Figure: 7.1 Maintenance Management, Federal and State Level, Germany

Whilst the database and inspection procedures have been developed, the following sub-modules will be developed in the future:

- catalogues of maintenance options and costs, and deterioration models
- evaluation and selection of maintenance options
- determination of the draft programme and financial requirements.

A phased plan for the completion of the system has been prepared and it is planned that it will be fully implemented at both Federal and State levels by 2005 (Haardt, 1998).

7.2.2.1 Other management systems used abroad

Some countries, that did not take part in the questionnaire, have developed or are developing their own systems.

Japan

The first version of the Japanese management system was completed in 1995. This system determines the most effective maintenance plan under a given set of financial conditions.

The management system consists of two main modules, the "condition module" and the "planning module". The "condition module" is used to determine the condition of the entire bridge based on the condition of its main components. The "planning module" is used to optimise planning. It generates a maintenance programme that gives the bridges to be repaired and the urgency of the repairs under the given financial conditions. The system contains cost tables that allow a comparative evaluation of various repair alternatives. So far, it has been used for planning on an annual basis.

Poland

Since 1989, Poland has developed a management system for maintenance planning, which includes decision-making procedures at various organisational levels ie local, national etc. The basic function is for planning maintenance over a 1-year period, taking into account data from the bridge inventory, construction details and bridge condition.

To optimise the allocation of available resources, linear programming is used, taking into consideration the replacement value of all the bridges in a region, the condition of the bridges and additional statistical data such as the number of bridges and their surface area. These results are used to determine the annual maintenance costs on the basis of cost tables.

As part of the optimisation process, the available resources are distributed among the individual bridges. A number of parameters are taken into account including:

- costs carried by the organisation responsible for running/managing the network and users
- comparison of service-life costs with the cost of new construction
- technical criteria (ie simplicity of the repair works)
- durability (ie high deterioration rate in an aggressive environmental)
- influence of traffic (ie high volume of traffic or absence of alternative routes)
- urgency of the repair, restrictions etc.

Sweden

The Swedish bridge management system contains inter-disciplinary strategies for planning and control measures, as well as operative planning and implementation of measures.

Two models are available for operative planning and procurement. The first model is used for routine maintenance, ie preventive maintenance and minor measures. The second model, called SAFEBRO, is used for major maintenance.

A database that contains the technical solutions available and their costs support the planning procedure.

Switzerland

Switzerland is developing the KUBA-MS system as a prototype BMS. The system is intended to fulfil the following objectives:

- to determine the ideal maintenance policies, in economic terms, with and without budgetary constraints
- to determine the consequences of deviating from this strategy
- to take account of the costs incurred by operating companies and users
- to determine the optimum measures for any planning period
- to determine both the short-term and medium-term financial requirements
- to indicate the affect of different budgetary restrictions on the average structural conditions.

7.2.3 Review of commercial bridge management systems

A detailed study was made of two commercial bridge management systems: PONTIS and DANBRO. Both of the systems have a modular structure. Each module and each component of the programs was studied on the basis of the information provided in the technical manuals. The study focused on the methodology used for deciding whether to repair or replace structures and to evaluate user costs.

PONTIS is a Bridge Management System which is currently used in the USA (more general information on PONTIS is given in Chapter 2). The optimum policies are developed on a network level and are based on the minimum expected life-cycle cost over an infinite planning horizon. This study analysed the objectives of this system and its organisation. Each module was studied, in particular the Maintenance, Repair and Rehabilitation Optimisation Model (MR&R) and the User Cost Model. The objective of the MR&R Optimisation Model is to find for each element of each bridge in each environment, the policy which minimises the long-term maintenance funding requirements while maintaining an acceptable risk of failure. The User Cost Model provides inputs to the Improvement Optimisation Model that compares the savings in user costs due to replacement or improvement with the cost of the investment.

DANBRO is the BMS that is currently used in Denmark. It can be used for maintenance work at several levels. Deliverable D7 *Decision on repair/replacement* gives a description of each level and analyses the objectives and organisation of the system, its components and its modular structure, with emphasis on the optimisation of repair and rehabilitation works, and maintenance works. DANBRO provides a selection of possible rehabilitation strategies, gives an economic evaluation of each one and, using an optimisation module, selects the alternative with the lowest global cost. The system also contains a Maintenance Module that can automatically print out the work orders at set intervals for the local bridge engineer. This is done after decisions have been made on which bridge components are to be maintained, the maintenance works that are to be carried out on these components, the starting date of a maintenance job and the time interval between repetitions.

DANBRO is a simpler system than PONTIS and has only one repair optimisation module compared with the two optimisation modules in PONTIS: the MR&R Optimisation Model and the Improvement Optimisation Module. MR&R activities retard or repair the effects of deterioration but they do not directly change the level of service of the bridge, while improvement activities usually change it. Finally the Integrated Project Programming Model in PONTIS combines the results of MR&R and Improvement Optimisation models.

PONTIS computes user costs as the sum of three components: accident costs, vehicle operating costs and travel time costs. Each component is evaluated using a different equation. DANBRO uses only one formula for this calculation which is based on: traffic counts, distribution of vehicle types, length of detours, lower speeds on detours or through the working area, traffic delays; unit cost per km and unit cost per hour for each vehicle type.

7.2.4 Theoretical models for repair or replacement

Two theoretical models have been selected from the models found in the literature review: the model proposed by D. M. Frangopol (University of Colorado) and that proposed by F.A.Branco and J. Brito (University of Lisbon).

7.2.4.1 Frangopol

The method developed by Frangopol for determining the optimum inspection and repair program for new and existing bridges is based on minimising the expected life-cycle costs while maintaining an acceptable level of reliability. The method determines the optimum inspection technique and repair program: type of inspection, number of lifetime inspections, number of lifetime repairs, and the timing of the inspections and repairs.

This method incorporates:

- the effectiveness of inspection techniques and their different detection capabilities
- an event tree which covers all repair possibilities
- the effects of ageing, deterioration, and subsequent repair on structural reliability
- the time value of money.

The expected total life-cycle cost C_{ET} includes the initial cost C_T and the costs of preventive maintenance C_{PM}, inspection C_{INS}, repair C_{REP} and failure C_F. Accordingly, C_{ET} can be expressed as

$$C_{ET} = C_T + C_{PM} + C_{INS} + C_{REP} + C_F \qquad [7.1]$$

The objective is, as described above, to develop a strategy that minimises C_{ET} while keeping the lifetime reliability of the structure above a minimum allowable value. To implement an optimum lifetime strategy, the following problem must be solved:

Minimise C_{ET} subject to $P_{f,life} \leq P^*_{f,life}$ [7.2]

where $P^*_{f,life}$ = maximum acceptable lifetime failure probability (also called lifetime target failure probability).

An event tree is used to investigate all possible repair events associated with the inspections. For each case, the structural cross sectional dimensions, corrosion rate v, number of inspections, loads, allowable reliability level, and median detectability of the inspection method $\eta_{0.5}$ are given. The expected total costs associated with different inspection/repair strategies are then obtained. The optimum strategy is based on the likelihood of following the various paths on the event tree.

This approach has a number of limitations:

1) One of the assumptions used to compute the optimum lifetime solution is that if damage is found then a repair action will follow (the repair cannot be delayed). When performing special inspections, the ability to detect damage is dependent on the quality of the inspection technique being used. No repair will be made unless the damage is detected.

2) The damage models are oriented towards structural problems. The European road network exhibits a different environment. The medium age of structures is lower than that of structures in the USA and faults and occurrences of damage on European bridges are less of a structural nature and more related more to traffic safety and durability.

3) The objective of this method is to determine the optimum inspection/repair program for a bridge during its service life and not to choose the most appropriate repair option for a deteriorated bridge at a certain time. Therefore this method requires a large amount of initial data much of which is unknown and has to be estimated; thus leading to greater uncertainty in the final result.

7.2.4.2 Branco and Brito

The decision making criteria developed by Branco and Brito are part of a global management system which includes a periodic inspection strategy and the selection of repair works obtained using a knowledge-based interactive system. The repair decision module is based on a cost/value economic analysis that compares repair costs and their subsequent benefits for the expected remaining service life of the structure, for each repair alternative.

The methodology used quantifies the global costs of building, using and replacing each bridge and predicts the benefits during their life cycle. To perform this analysis, a global cost function C was developed:

$$C = C_0 + C_I + C_M + C_R + C_F - B \qquad [7.3]$$

where C_0 are the initial costs, C_I the inspection costs, C_M the maintenance costs, C_R the repair costs, C_F the failure costs and B the benefits. The failure costs include structural failure costs and functional failure costs: traffic delays, detours, etc.

Every repair decision is made according to a cost effectiveness index (CEI) for each option that indicates how well the proposed repair compares to the do nothing option. The greater the coefficient for a particular option, the better investment. In the calculation of CEI, the repair costs C_R, the failure costs C_F and the benefits B are considered. For each option the CEI is quantified by:

$$CEI = \frac{(C_R + C_F - B)_{Repair}}{(C_R + C_F - B)_{No\ action}} \qquad [7.4]$$

The CEI coefficient may be used at different levels of action, namely:

- Level 1. To compare different solutions for the repair of the same defect.
- Level 2. To prioritise the repair of different defects on a bridge. The maximum CEI for each defect is used for comparison between different defects.
- Level 3. To prioritise the repair of different bridges on a network. The accumulated maximum CEI's of each group of repairs on each bridge are compared for different bridges.

7.3 DECISION SYSTEM FOR REPAIR/REPLACEMENT

7.3.1 Theoretical models for repair or replacement

Based on the review of methods for deciding on the most appropriate action for a deteriorated bridge, a procedure for helping the engineer to choose the best repair option was developed and is described below. It takes account of safety, durability, functionality and economy, and is based on a global cost analysis that considers all the costs involved in designing, constructing, inspecting, maintaining, repairing, strengthening and demolishing a bridge, as well as the road user costs over the service life of the bridge. To perform this analysis, a global cost function C was developed as follows:

$$C = C_C + C_I + C_M + C_R + C_F + C_U + C_O - V_S \qquad [7.5]$$

where C_C are the construction costs, C_I the inspection costs, C_M the maintenance costs, C_R the repair costs, C_F the failure costs, C_U the road user costs, C_O other costs and V_S the salvage value of the bridge.

The objective is to develop a strategy that minimises C while keeping the lifetime reliability of the structure above a minimum allowable value. To implement an optimum lifetime strategy, the following problem must be solved:

$$\text{Minimise C subject to } P_{f,life} \leq P^*_{f,life} \qquad [7.6]$$

where $P^*_{f,life}$ is the maximum acceptable lifetime failure probability (also called lifetime target failure probability).

The repair options considered using this method restore the initial service level (design) of the bridge, but exclude methods that up-grade the structure eg increase its width or load carrying capacity. However, this method could be used to compare options for upgrading a bridge assuming the options being compared give the same level of improved service.

7.3.2 Methodology

The method considers alternative options for the repair or replacement of a deteriorated bridge or a bridge which is functionally inadequate.

The global cost of each alternative is evaluated and the selection of the most suitable repair/replacement option is based on a comparison of the costs. The method allows the choice from a number of different options that depend on numerous factors that can be of a very different nature (ie loss of lives, average daily traffic flow etc).

In this method the factors are considered independent or, at least, semi-independent, although that is not always the case (ie the traffic volume may be affected by repair work on the bridge as drivers may take an alternative route to avoid being delayed by the repair works).

The possible options take into account the use of different types of repair and the different times when each of the repairs can be implemented during the service life of the bridge. Replacement of the structure is considered as another alternative.

This method is structured in the following phases:

i) identification of the factors

ii) evaluation of the factors

iii) comparison of alternatives and selection of option.

Any cost incurred during the analysis period must be included in the evaluation of the global cost for each option. Its value must be discounted to time T_0, common to all options, which is usually the time when the study is made. This is calculated as follows:

$$C_{i,T_0} = C_i \frac{1}{(1+r)^{T_i}} \qquad [7.7]$$

where r is the net discount rate of money, and C_i is the cost incurred during the year T_i .

In this way all other costs incurred during the analysis period will be discounted to time T_0, giving a total cost as follows:

$$C = \sum_{i=1}^{n} C_i \frac{1}{(1+r)^{T_i - T_0}} \qquad [7.8]$$

This cost is then used to compare the various options.

Regarding the updating procedure, all the costs incurred during a given year are considered as being incurred at the end of that year.

7.3.3 Identification of the factors

The identification of the factors to be taken into account in comparing the repair/replacement alternatives is of great importance, since those aspects not considered will be excluded from the rest of the study.

In fact, the identification phase implies a certain pre-evaluation in which, in a global and approximate way, rough values for the factors are considered; this makes possible to discard effects that will have an insignificant effect on the cost..

On the other hand, the identification of the factors establishes the degree of detail of the study, a general study with a few factors highly aggregated (ie which include many different aspects) or a detailed study with many factors highly disaggregated.

The degree of detail that establishes the identification of factors, conditions their evaluation and the comparison of the alternatives later on. It is difficult to evaluate the highly aggregated factors since each of them comprises many variables of a different nature that are hard to analyse as a whole. On the other hand, the highly disaggregated factors are easier to evaluate, although the selection process is more complicated and more entry data are required.

A list that contains the factors that are the most relevant for selecting the best alternative for repair or replacement a typical bridge is given below. This gives an indication of the factors that should be considered but it should be adapted for each specific bridge. On some occasions it may be necessary add or remove factors or to sub-divide them to provide more detail.

C_I	**Inspection costs**
C_M	**Maintenance costs**
C_R	**Repair costs**
C_{RA}	Structural assessment costs
C_{RR}	structural repair costs
C_F	**failure costs**
C_U	**road user costs**
C_{UD}	traffic delayed costs
C_{UR}	traffic re-routed costs
C_{URT}	time costs
C_{URO}	Vehicle operating costs
C_{URA}	Accident costs
V_S	**salvage value**
C_O	**other costs**

7.3.4 Evaluation of the factors

The value of most factors tends to have an objective base and it is usually to make a quantitative evaluation. However it is sometimes difficulties to estimate their value for several reasons: lack of data, accuracy, etc. For example, if a repair option requires the lane width to be reduced by 15%, it would be difficult to estimate the increase in accident rates.

The value of some factors is more subjective and depends on, among other things, social and economic factors, which makes it difficult to quantify their value. Some examples are the value of lives lost in an accident, the destruction of structures that have a cultural or historical value and the social impact caused by the closure of a bridge.

In cases where a specific factor gives rise to a benefit, it must be included as a negative cost when evaluating the cost of this alternative. For example, if one of the options results in a reduction in journey times.

In any case, when a study of alternatives is being carried out, only those factors whose value gives rises to differences between some of them will be considered. Those factors whose value is the same for all alternatives will be disregarded, since they will not affect the comparison of the alternatives. For example, the cost of the construction of the original bridge may not considered in decision making because it is the same for all the alternatives.

The evaluation of all options must be done for the same analysis period, even if they have different service lives. There are two methods that can be used to take account of differences in service life:

- to assume that shorter service life alternatives will be replaced as many times as necessary to equal the longest expected service life

- to reduce the analysis period to that of the option with the shortest expected service life and to attribute a salvage value to the remaining options

Guidelines for estimating the value of each factor are given in the following sections. Where possible the estimates should be taken from actual data.

7.3.4.1 Inspection costs

Inspection costs are those incurred during the regular inspections that are carried out as part of the management of bridge structures. They do not include inspections that are carried out as part of an assessment of load carrying capacity undertaken when some form of structural deficiency is suspected. Also, they do not include the benefits obtained in terms of an increase in the bridge safety reliability as a result of an inspection. Inspection costs can be divided into *labour costs* and *equipment costs.*

Labour costs include all the fees of the personnel that perform the inspection and of those who feed the data into the computer database. *Equipment costs* include depreciation of any capital equipment used, expendable items and the time spent transporting equipment from one bridge to the next.

There are several methods for calculating bridge inspection costs:

- automatic computation based on the dimensions of the bridge, its location, with standard rates for inspectors and equipment, and a schedule of inspections

- use of regression techniques with data from previous years for similar bridges

- an annual cost.

7.3.4.2 Maintenance costs

Maintenance costs are those involved in preserving a bridge at its design level of service and excludes major structural work They are often uniformly distributed over the life of a bridge, and include only the small repairs that are recommended following periodic inspections.

Maintenance work is proportional to the size and the age of the bridge. As structures age and maintenance costs increase it may become more economic to replace a bridge rather than continue spending on maintenance. Because of the increasing maintenance cost with time, the estimate of these costs is time dependent.

The are several options for estimating these costs:

- an automatic computation in which the yearly maintenance costs of the bridge are a percentage of its construction costs (this can vary with its age)
- regression techniques using data from previous years for the same or similar bridges
- an automatic computation based on the total current maintenance costs for the bridge stock and on the dimensions of the bridge.

The simplest method of predicting annual maintenance costs is to take a fixed percentage of the cost of construction, typical values that have been suggested vary from 1.0% to 2.0%.

7.3.4.3 Repair costs

Repair costs are those for main structural work and include the costs of any structural assessments associated with the repair. For the cost analysis, it is considered that there is no other structural repair work on the bridge.

If replacement of the bridge is one of the alternatives being considered, then the cost of replacement is included as a Repair Cost.

Bridge repair costs can be divided into:

$$C_R = C_{RA} + C_{RR} \qquad [7.9]$$

where C_{RA} are *the structural assessment costs* and include the fees of the personnel carrying out the inspection, depreciation costs of the equipment used, expendable items and the fees involved in the preliminary structural design of the repair options that were considered.

and C_{RR} are the *structural repair costs* which include labour, materials, equipment, administration and quality control involved in the application of the repair.

If replacement is an option then C_{RA} would include all the costs derived from the project for the new bridge and the demolition project of the existing bridge. C_{RR} would include construction, supervision and administration costs of both the construction of the new bridge and demolition of the existing bridge.

For a global economic analysis, repair costs can be estimated using data from other repairs on the same type or similar bridges, taking into account the severity and location of the defects, their accessibility, the area of deck to be repaired and the repair method.

7.3.4.4 Failure costs

Failure costs C_F include all the costs resulting from any failure that causes a bridge to be closed to traffic, this may range from serious damage to actual collapse. The costs associated with structural failure can be obtained from the probability of failure P_f and the cost of collapse C_{FF}. Even though structural failures rarely occur under normal circumstances, these costs should still be included in an economic analysis and they are effectively the insurance costs.

$$C_F = P_f \, C_{FF} \qquad [7.10]$$

In the economic analysis, the estimate of the probability of failure considers, in a simplified way, a linear variation in time during the service life of the bridge. A probability failure path based on degradation mechanisms and the associated reliability index could also be used. Such an approach has the disadvantage that it involves the need to consider in the mathematical modelling a large number of parameters which affect the partial factors for design and assessment and, hence, the acceptable reliability level. Among other parameters, the following would have to be included: size and importance of the structure, degrees of redundancy and ductility, design life, type and modes of failure, frequency of inspection and maintenance, and scope and data acquired from in-situ inspections. The complexity of such an analysis and the difficulty in obtaining reliable data currently limit its use to very important and onerous projects.

The cost of collapse can be divided into the costs of bridge replacement, loss of lives, equipment, and architectural, cultural and historical value.

Bridge replacement costs include the extra expense involved in replacing a bridge that still has some years of remaining service life. This is done by comparing the cost of replacing a bridge that has failed with the cost of replacing it at the end of its service life. The replacement costs are essentially those of constructing a new bridge and traffic disruption costs during the period of the works.

Costs arising from loss of lives and equipment comprise: the value of the lives and injuries to anyone as a result of the failure (or what society is prepared to pay to save them), the value of their vehicles and the disruption to services. The latter include any electricity, water or gas supplies crossing the bridge that were interrupted as a result of the failure. These costs can be estimated from current traffic values and normal insurance values for vehicles and people.

The architectural, cultural and historical costs are a way of over-valuing bridges that are especially important from these points of view.

Failure costs can be omitted when comparing a number of options, as they will be similar for each one. If the probability of failure or the cost of collapse for one option is significantly larger than for the remaining options then it is necessary to include failure costs in the comparison.

7.3.4.5 Road user costs

Road user costs C_U correspond to the costs attributed to the reduction in the level of service provided as a result of the works being undertaken on the bridge. This may be increased journey times as a result of congestion at the bridge or detours made either as a result of closure of the structure or to avoid the congestion. When doing the analysis it is assumed that other bridges on the same road have no direct effects on these costs. They can be divided into:

$$C_U = C_{UD} + C_{UR} \qquad [7.11]$$

where C_{UD} are the costs due to delayed traffic and C_{UR} are the costs due to traffic detours.

In order to evaluate road user costs, it is necessary to predict future traffic growth. This can be done in terms of the annual volume of traffic, using a regression analysis or other statistical techniques. The daily distribution of traffic flow over the bridge must also be considered to take account of peaks in the traffic flow eg during rush hour periods. This takes account of volume of traffic and the number of heavy vehicles and is based on measurements or on typical distributions.

The costs due to traffic delays C_{UD} are those caused by the slowing down of traffic crossing the bridge, especially during rush hours. They are estimated from consideration of the average delay time and hourly value of time for the average user.

$$C_{UD} = ADT_L \cdot C_{H,L} \cdot t_L + ADT_H \cdot C_{H,H} \cdot t_H \qquad [7.12]$$

where:

ADT_L: average daily light traffic flow

ADT_H: average daily heavy traffic flow

t_L: additional waiting time, in hours, for light vehicles

t_H: additional waiting time, in hours, for heavy vehicles

$C_{H,L}$: unit cost per hour for light vehicles

$C_{H,H}$: unit cost per hour for heavy vehicles.

The costs due to traffic detours, C_{UR} are those that arise when traffic is re-routed from one bridge, because of congestion at the bridge or because of it has insufficient structural capacity. They are estimated from consideration of the costs associated with additional travel time C_{URT}, additional vehicle running expenditure C_{URO}, and the increase in the traffic accident rate C_{URA}.

$$C_{UR} = C_{URT} + C_{URO} + C_{URA} \qquad [7.13]$$

The costs associated with additional travel time C_{URT} due to traffic detours can be calculated from the following formula:

$$C_{URT} = ADT_L \cdot C_{H,L} \cdot t_L + ADT_H \cdot C_{H,H} \cdot t_H \qquad [7.14]$$

where:

C_{URT}: costs due to additional travel time due to traffic detours.

ADT_L: average daily light traffic flow.

ADT_H: average daily heavy traffic flow.

t_L: additional travel time, in hours, for light vehicles.

t_H: additional travel time, in hours, for heavy vehicles.

$C_{H,L}$: unit cost per hour for light vehicles.

$C_{H,H}$: unit cost per hour for heavy vehicles.

The costs associated with additional vehicle running expenditure C_{URO} can be calculated from the following formula:

$$C_{URO} = ADT_L \cdot C_{km,L} \cdot d_L + ADT_H \cdot C_{km,H} \cdot d_H \qquad [7.15]$$

where:

C_{URO}: costs due to additional vehicle running expenditure due to traffic detours.

ADT_L: average daily light traffic flow.

ADT_H: average daily heavy traffic flow.

d_L: additional length of detour in km for light vehicles.

d_H: additional length of detour in km for heavy vehicles.

$C_{km,L}$: unit cost per km for light vehicles.

$C_{km,H}$: unit cost per km for heavy vehicles.

The additional accident costs C_{URA} may be calculated from:

$$C_{URA} = t \cdot ADT \cdot \sum_i r_i \cdot c_i \qquad [7.16]$$

where

ADT: average daily traffic flow.

t: time when the increment of the accident rate occurs.

r_i : increment of the accident rate for type *i* accidents.

c_i : the cost of type *i* accident.

i: type of accident. These are classified in three groups: fatal accidents, injuries caused by accidents and damage to materials.

Programmes have been developed in several countries for evaluating road user costs under different circumstances. For example, in the United Kingdom a computer programme called QUADRO (QUeues And Delays at ROadworks) provides a method for assessing the cost imposed on road users while road works are being carried out. These include, road user delays (value of time), vehicle operating costs and accident costs.

7.3.4.6 Salvage value

The salvage value of a bridge is its value at the end of the analysis period. An estimate of the salvage value must be made when the analysis period is shorter than the service life of the structure. It can be estimated by assuming that its value is zero at the end of its service life and it is equal to the cost of construction when the bridge is put into service. The value at some intermediate point may then be interpolated from these two extremes.

7.3.4.7 Other costs

Other costs (Co), cover other aspects of a different nature that can give rise to additional costs for some alternatives and whose influence can be important in some cases. Some examples are given below:

- restrictions in the use of the structure eg reductions in vertical clearance, reductions in lane widths, lane closures, removal of hard shoulder from service,..
- influence of the proposed option on other users eg pedestrians and cyclists
- absence of alternative routes for light and/or heavy traffic that require special measures, eg, construction of a temporary bridge
- for bridges used by public transport eg buses, coaches and school transport, the absence of alternative transport over the same route eg rail
- influence of the repair works on other modes of transport (railway, high speed, etc) that may cause traffic disruption on them, limitations on the repair works ie working hours, night time working hours, etc.
- economic affect on local businesses eg disruption to traffic crossing a bridge may affect shops and local industries in the vicinity of the structure
- environmental impact of the works on the local community eg noise, dust and contaminants
- loss or reduction of historic, patrimonial, aesthetic, religious and traditional values of the bridge at all levels ie national, regional and local
- additional expenses incurred during the works ie staff, boards, beacons and other signalling
- convenience of a given alternative from the point of view of the use of available equipment, stocked materials, similar actions in nearby places, etc.

7.3.5 Comparison of the alternatives

As stated above, the selection of alternatives is based on minimising the total cost over the analysis period.

In the model described above, the repair index (RI) is used to determine the relative costs of each option, this is usually done using the do nothing option as the reference; the smaller the coefficient for a particular option, the better investment. In the calculation of RI, the inspection costs C_I, the maintenance costs C_M, the repair costs C_R, the failure costs C_F, the road user costs C_U, other costs C_O and the salvage value V_S are considered. For each option the RI may be quantified by:

$$RI = \frac{(C_I + C_M + C_R + C_F + C_U + C_O - V_S)_{\text{Repair or replacement}}}{(C_I + C_M + C_R + C_F + C_U + C_O - V_S)_{\text{No action or reference alternative}}} \quad [7.17]$$

The economic analysis considers a certain number of parameters whose accuracy cannot always be guaranteed: values of discount rates, inspection costs, maintenance costs, probability of structural collapse, evolution of traffic, etc. It is therefore useful to know the sensitivity of final results to each parameter in order to try to estimate more carefully those that have the most influence.

The RI coefficient may be used at different levels of actions, but its principal goal is to compare and select the best alternative for the repair or replacement for a bridge.

On the other hand, the method allows the global cost of each alternative to be calculated and the alternatives to be ranked in terms of cost. It also can be used to evaluate the differences in cost if any action is deferred and for this to be included as one of the options.

This method can provide useful information and enable comparison of different actions on a range of bridges on the network, on the basis of a consistent set of criteria.

7.3.6 Example

An example of the application of the economic analysis has been developed in Deliverable D7 *Decision on repair/replacement*. A brief description of the example is given below.

The example is based on a comparison of six options for the repair or replacement of a deteriorated bridge. The general characteristics of the bridge are:

- construction and opened to traffic: 1950
- assumed service life: 100 years
- year of the analysis: 2000
- at present (2000) the bridge has problems with its structural carrying capacity, and heavy traffic is rerouted
- discount rate: 1.5%

The following six alternatives are considered:

Alternative 1

The bridge will be replaced in 2016. Between 2001 and 2015 the heavy traffic will be rerouted. In 2016, year of the construction of the new bridge, all the traffic will be rerouted. The assumed service life of the new bridge is 100 years.

Alternative 2

The bridge will be repaired (*Repair 1*) in 2001. During the three month period of the repair the heavy traffic and half of the light traffic will be rerouted.

Alternative 3

The bridge will be repaired (*Repair 2*) in 2001 and again in 2026 ie the repair will have a shorter life than Repair 1. During the two repairs (two months for each) the heavy traffic and half of the light traffic will be rerouted.

Alternative 4

Is the same as the alternative 2 (*Repair 1*) but delayed five years ie the bridge will be repaired in 2006.

Alternative 5

The bridge will be repaired (*Repair 3*) in 2001. The repair cost of this alternative is lesser than the repair cost of the alternative 2 (*Repair 1*) but the repair works last six months. During the repair the heavy traffic and half of the light traffic will be rerouted.

Alternative 6

The bridge will be replaced in 2001. In 2001, year of the construction of the new bridge, all the traffic will be rerouted. The assumed service life of the new bridge is 100 years.

Figure 7.2 shows the different costs of each alternative: inspection costs, maintenance costs, repair costs, etc. Figure 7.3 presents the comparison of the alternatives for each cost.

Table 7.1 presents the main results of the economic analysis. The main conclusion from this analysis is that the best option is the second followed by the fifth. As can be seen the RI of the first option is very high which shows that this option is inadvisable.

Table 7.1: Results of economic analysis

	RI	Ranking
Alternative 1	4.692	6
Alternative 2	1.000	1
Alternative 3	1.063	3
Alternative 4	2.347	5
Alternative 5	1.001	2
Alternative 6	1.793	4

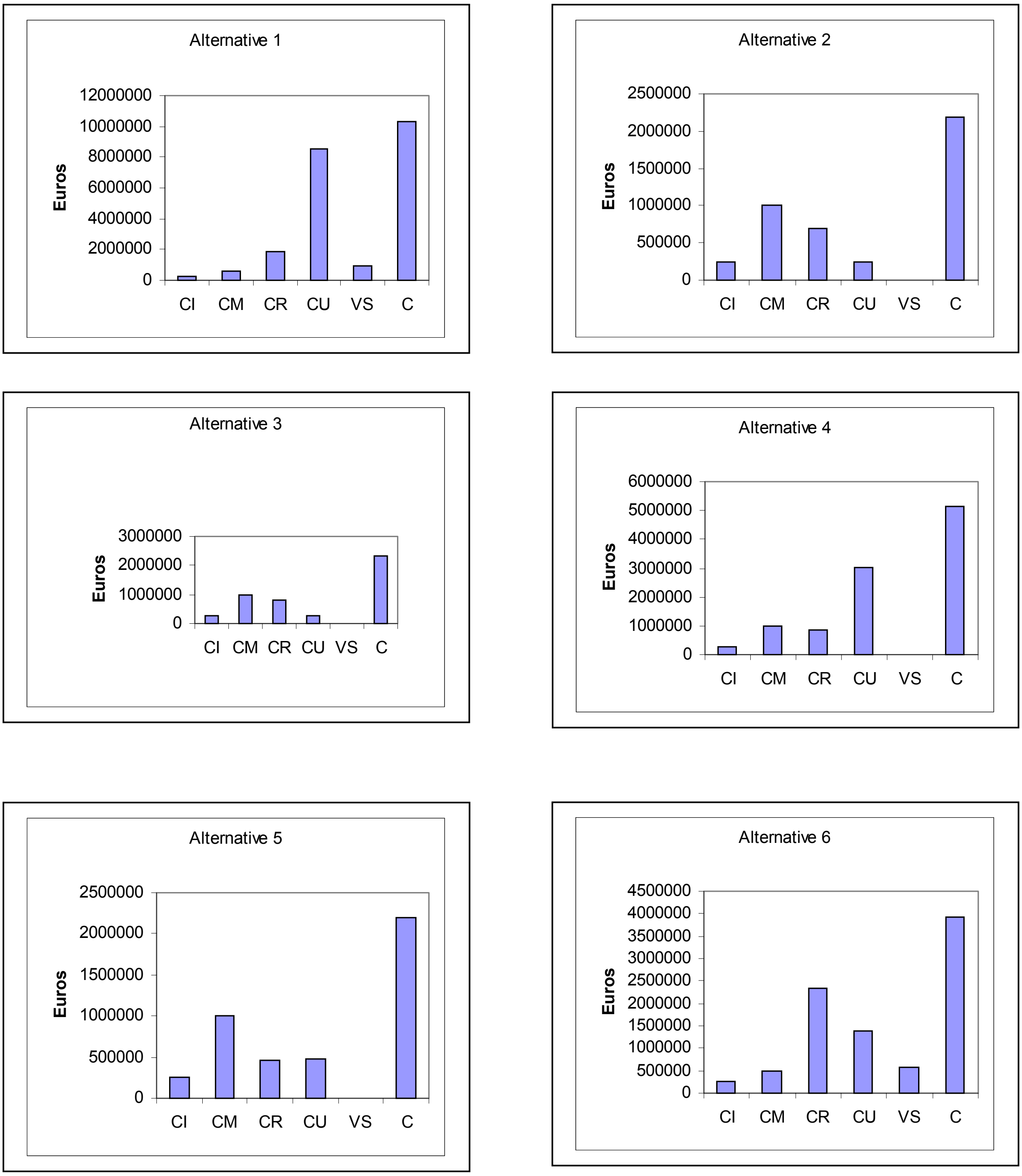


Figure 7.2: Cost of each alternative

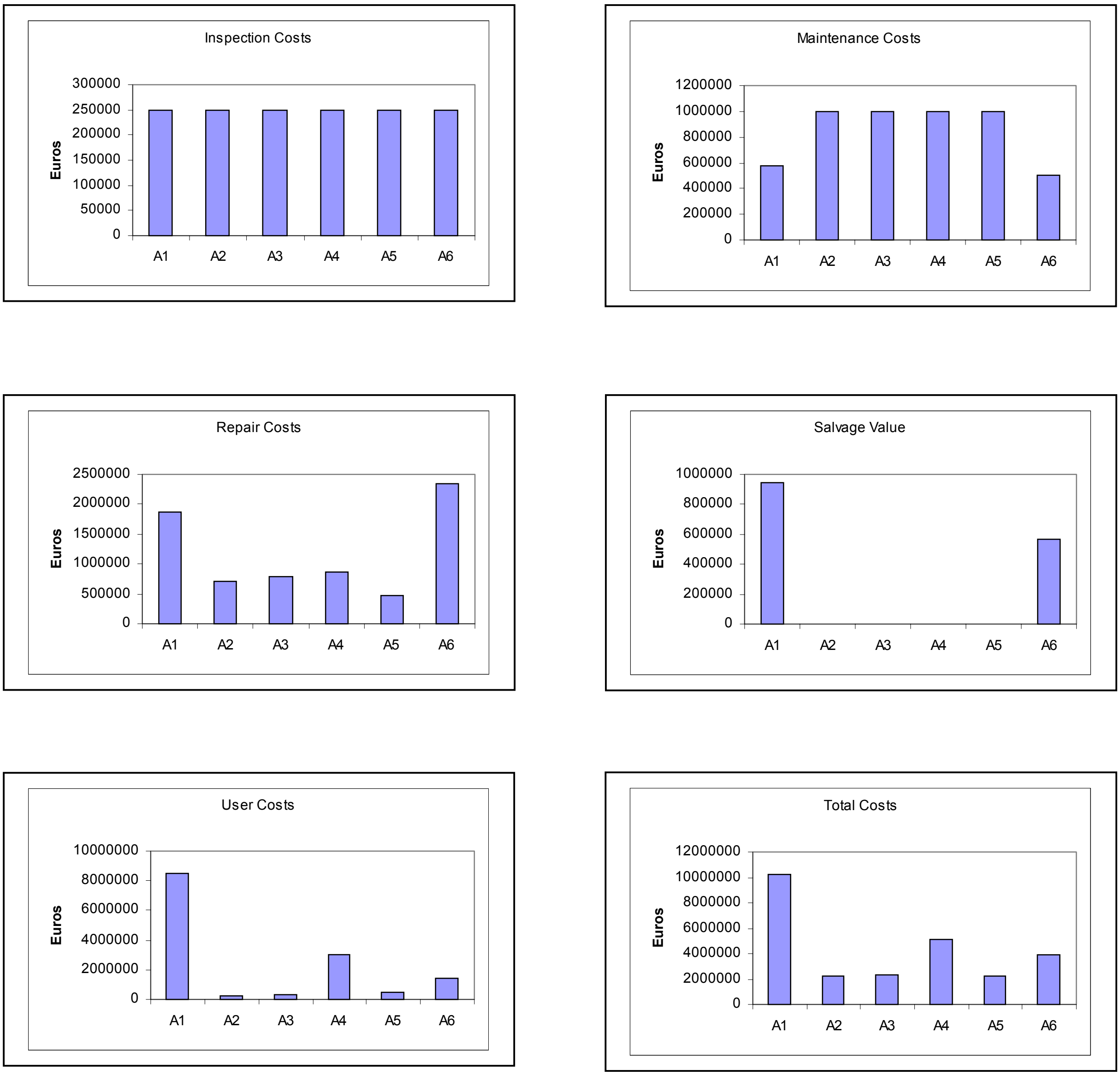


Figure 7.3: Comparison of each alternative

7.4 CONCLUSIONS

This chapter has described a method for making a decision between possible repair options for a deteriorated bridge. The method is based on a global cost analysis that considers all the costs involved in designing, constructing, inspecting, maintaining, repairing, strengthening and demolishing a bridge, as well as the associated road user costs, during the service life of the bridge.

The objective is to develop a strategy that minimises the global cost while keeping the lifetime reliability of the structure above a minimum allowable value.

The method examines the various options for the repair or replacement of a deteriorated bridge with inadequate load carrying capacity or functional problems. The global cost of each alternative is evaluated using a set of different factors and the selection of the most suitable repair/replacement alternative is based on a comparison of these costs. The method allows a choice among alternatives depending on numerous factors that can be of a very different nature.

The possible alternatives must take into account the use of different options, including replacement of the structure and the different times during the service life of a bridge when they can be implemented.

The work described in this chapter has concentrated on repair options that would restore the initial (design) service level of the bridge. It has not been considered upgrading the structure ie by widening, strengthening etc. Nevertheless, the method can be used to compare different options for upgrading a structure to a given level of service. If the level of service is different for some of the alternatives it will be necessary to complete the method taking into account the benefits that accrue from each of the alternatives for the repair or replacement of the bridge.

The results of this study will be used to assist the development of a framework for a bridge management system. It should also stimulate further improvements of existing decision procedures and the development of new ones.

7.5 BIBLIOGRAPHY

Ahlskog J J (1990). *Bridge Management-the answer to the challenge*. Bridge Evaluation, Repair and Rehabilitation, Andrzej Nowak. Kluwer Acadamic Publishers 1990.

Andersen N H (1990). *DANBRO – A Bridge Management System for many levels*. Bridge Evaluation, Repair and Rehabilitation, Andrzej Nowak. Kluwer Acadamic Publishers 1990.

Asociación Técnica Española de Pretensado (1995). *Reparación y refuerzo de estructuras de hormigón. Guía FIP de buena práctica.* Edita Colegio de Ingenieros de Caminos, Canales y Puertos, Madrid.

Asociación Técnica Española de Pretensado. *Recomendaciones para la conservación de puentes pretensados H. P. 7-92*. Edita Colegio de Ingenieros de Caminos, Canales y Puertos, Madrid.

Binet C (1996). *Condition and repair cost estimate of the french road bridge assets*. Recent Advances in Bridge Engineering, J. R. Casas, F. W. Klaiber and A. R. Marí (Eds.). CIMNE Barcelona 1996.

Branco F A and Brito J (1995). *Decision criteria for concrete bridge repair*. Structural Engineering International Number 2.

Branco F A and.Brito J (1996). *Bridge management from design to maintenance*. J. R. Casas, F. W. Klaiber and A. R. Marí (Eds.). CIMNE Barcelona.

Brito J, and Branco F A (1992). *A Decision System for Bridge Management.* 14Th Congress New Delhi IABSE. March..

Brito J, Branco F A and Ibañez M (1994). *A Knowledge-Based System for Concrete Bridge Inspection*. Concrete International, ACI, Detroit, MI, Feb.

Clausen P (1992). *Introduction to Bridge Management Systems*. 14 Th Congress New Delhi 1992 IABSE.

Collier C A and Glagola C R (1998). *Engineering economic and cost analysis*. Third edition. Addison Wesley Longman, Inc.

COWI (1999). *Bridge Management System,* January.

Das P C (1996). *Bridge Management Methodologies*. Recent Advances in Bridge Engineering, J. R. Casas, F. W. Klaiber and A. R. Marí (Eds.). CIMNE Barcelona 1996.

Das P C (1998). *New Developments in Bridge Management Methodology*. Structural Engineering International, Volume 8, Number 4, November 1998.

EC BRITE/EURAM. (1993). *Assessment of Performance and Optimal Strategies for Inspection and Maintenance of Concrete Structures Using Reliability Based Expert Systems*. Report P3091. Aalborg, Denmark.

Estes A C and Frangopol D M (1998), *Optimum reliability-based inspection/repair strategy based on minimum expected cost*. Structural Safety and Reliability. N. Shiraishi, M. Shinozuka, and Y. K. Wen, eds., Balkema, Rotterdam.

Frangopol D M and Estes A C (1997). *Lifetime Bridge Maintenance Strategies Based on System Reliability*. Estructural Engineering International Number 3.

Frangopol D M and G. Hearn (1996). *Managing the life-cycle safety of deteriorating bridges*. Recent Advances in Bridge Engineering, J. R. Casas, F. W. Klaiber and A. R. Marí (Eds.). CIMNE Barcelona 1996.

Frangopol D M and.Estes A C (1998). *Optimum design of bridge inspection/repair programs based on reliability and cost*. Symposium on "The Management of Highway Structures" organised by the Institution of Civil Engineers and The Highways Agency in London. May.

Frangopol D M Lin K Y and Estes A C (1997). *Life-cycle cost design of deteriorating structures.* Journal of Structural Engineering. October.

Golabi K, and P D Thompson (1990). *A Network Optimisation System for Maintenance and Improvement of California's bridges*. Bridge Evaluation, Repair and Rehabilitation, Andrzej Nowak. Kluwer Acadamic Publishers 1990.

Haardt P (1998). *Concept of a Management System for the Maintenance of Bridges and Engineering Structures*. Federal Highway Research Institute. Bergisch Gladbach.

Hawk H and Small E P (1998). *The BRIDGIT Bridge Management System*. Structural Engineering International, Volume 8, Number 4, November.

Huvstig A (1998). *Bridge Maintenance Training: experience and future trends*. Operation and Maintenance of Large Infrastructure Projects, Vincentsen and Jensen (eds). 1998 Balkema, Rotterdam.

Jones A E K, and Cusens A R (1997). *Whole-life costing*. Structural Engineering, Volume 75, Number 7, April.

Lauridsen J, Bjerrum J, Andersen N H and Lassen B (1998). *Creating a Bridge Management System*. Structural Engineering International, Volume 8, Number 3, August 1998.

Middleton C R (1997). *Concrete bridge assessment: an alternative approach*. Structural Engineering, Volume 75, Nos 23 and 24, December.

Naumann J (1998), *Development of a Bridge Management System in Germany*. Operation and Maintenance of Large Infrastructure Projects, Vincentsen and Jensen (eds). 1998 Balkema, Rotterdam.

Papers based on presentations made at The International Symposium (1996) "The Safety of Bridges" in London July 1996. *Safety of Bridges*. Edited by P. C. Das. London 1997.

PONTIS Technical Manual.

PONTIS User's Manual.

Proceedings of the International Symposium on Advances in operation and maintenance of large infrastructure projects (1998). Copenhagen, Denmark 10-13 May 1998. *Operation and maintenance of large infrastructure projects*. Edited by L. Vincentsen and J. S. Jensen. A.A. Balkema, Rotterdam.

Proceedings of the NATO Advanced Research Workshop on Bridge Evaluation, Repair and Rehabilitation (1990). Baltimore, Maryland, U.S.A. April 30 – May 2. *Bridge evaluation, repair and rehabilitation*. Edited by A. S. Nowak. NATO ASI Series. Vol. 187.

Proceedings of the US-Canada-Europe Workshop on Bridge Engineering (1997). Dubendorf and Zurich, 11 – 15 July 1997. *Recent advances in bridge engineering. Advanced rehabilitation, durable materials, nondestructive evaluation and management*. U. Meier and R. Betti (Eds.).

Proceedings of the US-Europe Worshop on Bridge Engineering (1996). Barcelona, Spain 15-17 July. *Recent advances in bridge engineering. Evaluation, management and repair.* J. R. Casas, F. W. Klaiber and A. R. Marí (Eds.). CIMNE Barcelona.

Road Directorate, Denmark Ministry of Transport (1995). *Bridge Maintenance on the Danish main road network*.

Road Directorate, Denmark Ministry of Transport, *Bridge Management System.*

Sinha K C, Bowman M D, Jiang Y, Murthy S, Saito M and A. Tee (1990). *Emerging Methodologies for Bridge Management Systems*. Bridge Evaluation, Repair and Rehabilitation, Andrzej Nowak. Kluwer Acadamic Publishers 1990.

Söderqvist M K and Veijola M (1998). *The Finnish Bridge Management System*. Structural Engineering International, Volume 8, Number 4, November.

The Department of the Environment, Transport and the Regions (1998), *"Getting the best roads for our money" An introduction to QUADRO*. March.

Thoft-Christensen P (1996). *Bridge Management Systems. present and future*. Recent Advances in Bridge Engineering, J. R. Casas, F. W. Klaiber and A. R. Marí (Eds.). CIMNE Barcelona 1996.

Thompson P D, Small E P, Johnson M and A R. Marshall (1998). *The PONTIS Bridge Management System*. Structural Engineering International, Volume 8, Number 4, November 1998.

Yanev B S (1998), *Bridge Management for New York City*. Structural Engineering International, Volume 8, Number 3, August 1998.

Yañez M A and A J Alonso (1996). *The actual state of the Bridge Management System in the state national highway network of Spain*. Recent Advances in Bridge Engineering, J. R. Casas, F. W. Klaiber and A. R. Marí (Eds.). CIMNE Barcelona 1996.

CHAPTER 8

PROGRAMMING MAINTENANCE

8.1 INTRODUCTION

The previous chapter described the development of a methodology for determining the most appropriate course of action for a particular structure. In practice, the funds available for maintenance are rarely sufficient to repair all deteriorated or damaged structures. Authorities for bridge maintenance must therefore decide which structures should be repaired in a given year. The most appropriate maintenance policy may not necessarily be to repair those structures that are in the worst condition. It should optimise the planned maintenance work to give the maximum long-term benefit. This means balancing the costs of deferring work on some structures against the benefits of bringing forward the repair of others. For example, if a structure is likely to deteriorate quickly it might be beneficial to repair it ahead of those in an apparently worse condition. In other cases it might be economically beneficial to repair structures along a given stretch of road. The aim must be to produce a maintenance strategy that gives best value in the long-term whilst ensuring that all structures remain safe and give an acceptable level of service.

This chapter reviews methods for prioritising bridges in terms of their need for repair, rehabilitation or strengthening. It examines current methods for prioritisation and optimisation of bridge maintenance at both project and network level, and identifies the objectives and constraints, which play a key role in decision making when planning maintenance strategies. Simple procedures are developed for selecting bridge structures for inclusion in the maintenance programme and for ranking bridges with respect to the impact of their location in the road network. The costs, which should be taken into account when examining different maintenance strategies, are identified. In addition, an alternative method for project level optimisation to that presented in Chapter 7 is also described.

8.2 OPTIMISATION OF BRIDGE MAINTENANCE

In order to allocate funds for bridge maintenance either in the current budget year or in the longer term, the following basic information is needed:

- the bridge structures that are in the worst condition and their priority ranking (this is a number based on the urgency of the repair work -see below)
- the condition of the bridge stock as a whole
- the bridge structures that have a reduced load carrying capacity
- the bridges that have a load restriction
- the bridges that need major repair work, preventative maintenance or regular maintenance work
- the importance of the location of each bridge in the network both for the local community and for the region as a whole

- the importance of the route on which the bridge is located in the road network
- the consequences for the community and for the regional economy if the usage of the bridge has to be restricted eg load restriction, closing lanes or closure of the bridge itself.

Ideally the funds allocated to bridge maintenance should take account of all these factors to ensure that the optimal maintenance programme is planned for the life of each structure ie the costs over the life of the structure or structural element should be minimised. This can be effectively achieved by breaking the optimisation process down into three levels.

8.2.1 First level - condition rating prioritisation

The first level of the optimisation procedure is to prioritise those bridges that need repair or maintenance work. This requires knowledge of the condition and deterioration rate of the bridge structure and its elements so that the most appropriate time to carry out the maintenance or repair work can be determined. For the level 1 optimisation, this is done using the condition assessment of the bridge structures obtained using the methods described in Chapter 3. They are obtained by carrying out periodic inspections of bridge structures. Different methods for condition assessment of bridge structures have been developed in different countries and some of these methods have been described in Deliverable D2.

An example of how this is done for a stock of bridges is shown graphically in Figure 8.1. The graph presents bridges that have been sorted according to their condition rating, the higher the value the worse is their condition. In reality some structures with a lower condition rating could have a more urgent case for repair than a structure with a higher condition rating. This mainly occurs when there is an imminent risk to public safety and urgent action is needed.

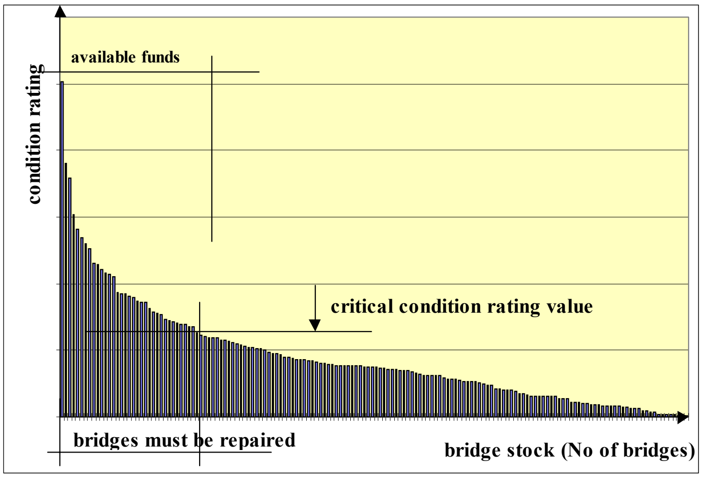


Figure 8.1: First prioritisation based on the condition rating

The results obtained from sorting bridges in this way depend on the method used for condition assessment. Another method for prioritising bridges is based on their deterioration rates and an example is given in Deliverable D3. Deterioration rates can be assessed by plotting condition against age. The slope of the curve at different ages gives an estimate of deterioration rate. The prediction of future condition can be estimated by an extrapolation of the condition-age data. The best estimate can be obtained if data on condition assessment are available from the time of construction. In most cases such data are available only for newer structures. For structures where past data are not known, especially for the period when deterioration has occurred, such predictions of future condition are more unreliable. By comparing the predicted future condition of bridge structures over a specified period of time, it is possible to prioritise bridges based on their deterioration rates. For example, consider two structures of different ages, one currently in a reasonable condition but quickly deteriorating (ie a steep slope of the condition-age curve such as a relatively new structure that is deteriorating quickly). The second, in poor condition but deteriorating slowly (ie the condition-age curve has a lower slope than for the first structure such as an older structure in which deterioration has developed slowly over a longer period of time). If some time after the last inspection, the predicted condition of the first structure becomes worse than that of the second structure, then the first bridge would have a higher priority for maintenance than the second, all other things being equal. The final decision is still based on the expert judgement, if there are too few available data on past condition assessments.

An example of estimating the deterioration rate of a bridge structure is presented in Figure 8.2. It shows an estimate of the results of inspections of highway structures in Slovenia over a ten-year period, where the oldest structures were about 18 years old at their first inspection. The analysis was made using the CAE hybrid neural network method. The ordinate is normalised to the number of structures. Curve C5 (Category 5) shows the structures in a very good condition and curve shows C1 structures in a poor condition. The condition is estimated for the whole structure. Curves C1 and C2 are explained by the fact that during inspections some structures are always found in a poor and very bad condition and these results have an impact on the overall findings.

If there are sufficient funds available and there are no other constraints then the first level optimisation can be used to identify the bridges that are to be programmed for maintenance by selecting those structure with a condition rating greater or equal to a pre-selected value. As in practice this is not the case because there are always constraints on what can be done, the most common being lack of funding, it is necessary to develop an optimised maintenance programme which gives best value within the constraints that have been imposed.

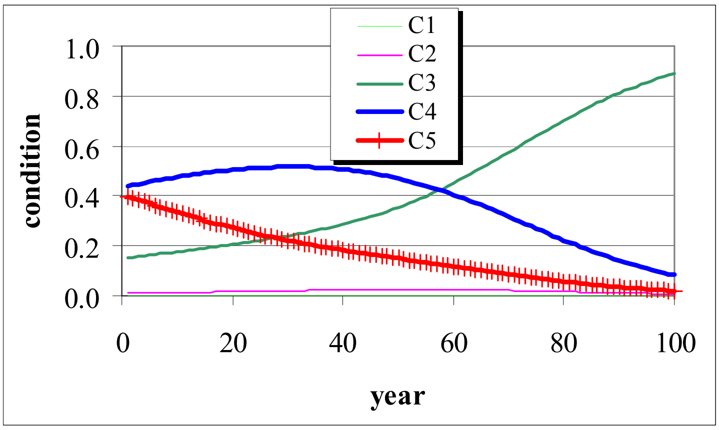


Figure 8.2: Estimated deterioration rates based on the results of 10 years inspection; Maximum age of structures at first inspection was 18 years

8.2.2 Second - condition rating prioritisation

The second level of optimisation is to rank those bridges that have been identified as in need of repair in order of priority. The priority ranking needs to take account of the assessed safety of the structure which is expressed by the safety index β, the estimated remaining service life of the structure, and the importance of the structure in the road network, as well as the condition assessment. The safety index (β) is a quantitative measure of acceptable performance level. It usually lies between 2.00 and 3.75 and depends on the inspection level, element and system behaviour and is described in more detail in Deliverable D12. The result of this analysis is a priority ranking of candidate bridges. This can be done on the whole bridge stock, but is usually performed on a smaller number of the candidate bridges, which have a similar condition rating, but their position and impact on the road network are different. This is described in more detail in Section 8.3.

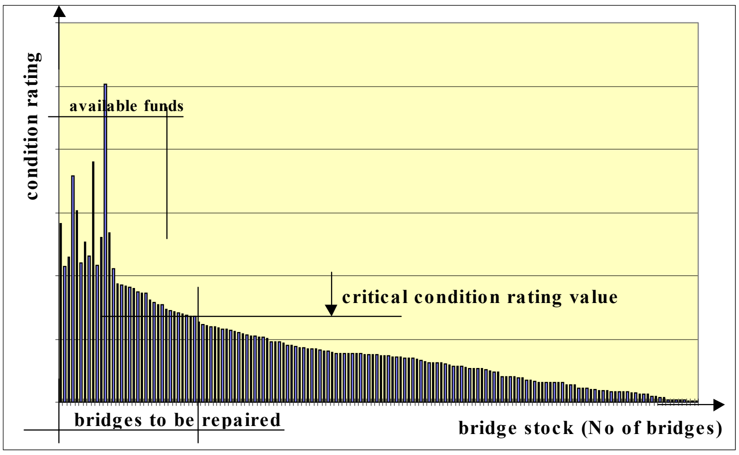


Figure 8.3: Second prioritisation based on the constraints of the locations of bridge structures in the network

The impact of the importance and functionality of the bridge within the network is determined on the basis of a variety of factors relating to its value to the community. These include: the road classification, traffic volume, bridge location, the heritage value of the bridge, weight restriction, vertical clearance, width of the bridge and the length of detours in the event of a bridge closure. In seismic regions, seismic resistance is also taken into account and a higher priority is given to structures with deteriorated substructures.

A simple model was developed for evaluating the impact factor and then ranking the bridges. This was done using the CAE hybrid neural network method described by Peruš and Žnidarič [1998]. The impact factor is a single value which, as indicated above, takes into account the importance and functionality of the bridge structure. The method for determining the impact factor and a simple case study of bridge ranking are described in Deliverable D12. A brief description of the model is given below in Section 8.3.

Figure 8.3 presents graphically one of the possible solutions for ranking bridge structures, taking into account the constraints of the position of the structures in the network. It shows that after the second prioritisation is carried out, bridges with a lower condition rating can be ranked higher than bridges with a higher condition rating. For example consider two equal bridges, one with a lower condition rating situated on a road with a very high traffic volume in an urban area and the other with a higher condition rating situated on the road with a low traffic volume in a rural area. Due to additional constraints such as traffic volume, location on the network, the bridge with the lower condition rating will be given higher priority than the bridge with the higher condition rating. A few other possible solutions are presented graphically in Deliverable D12.

The final step is to determine the optimised maintenance programme within the constraints that have been imposed – usually the funds available. There is still however a need for engineering judgement in reaching the final decision in order to take account of issues such as political factors or the need for urgent intervention in order to ensure public safety.

8.2.3 Third level - maintenance optimisation

The third level of the maintenance optimisation process is to prioritise bridges by considering different maintenance strategies and taking into account the costs of each strategy. The optimisation can be made for each bridge in the bridge stock - project level optimisation - and for a selected number of bridges taken from the priority ranking analysis or for the whole bridge stock - network level optimisation. The final prioritisation of the maintenance options for the network level may differ from that for the project level because of the different constraints between the project level and network level optimisation.

As it is often not possible to carry out maintenance in the current year on all structures that need repair, because of restrictions on funding, different long-term maintenance strategies may be needed for some bridges. There is no strict definition of the "long-term planning". Theoretically the long-term planning should be made for the life of the structure, but there are too many uncertainties concerning the future. For example, deterioration rates both of the structure's components and of the structure itself, traffic volumes, load level, durability of repairs and economic growth. For this reason, a shorter period is advisable and this may be the period a government is in office or a longer period if the economy of the country is stable an inflation is low.

8.2.3.1 Project level optimisation

To determine the optimal maintenance strategy for a structure, ie to minimise the costs required to keep a bridge structure at an acceptable safety and serviceability level, requires information from the structural inventory, the results of previous inspections and its previous maintenance history. When planning maintenance strategies, the practicality of the proposed repair work must be taken into consideration as well as the duration of the maintenance works.

The length of time that is taken into consideration when planning the maintenance work can be limited:

- to the estimated remaining service life
- to an extended estimated remaining service life or
- to a shorter period than the expected remaining service life.

The following data is required from the bridge inventory:

- location of the bridge
- nature of the obstacle crossed (eg river, sea, valley, railway, other roads)
- accessibility of the whole structure and its elements ie requirements for scaffolding, moving platform, abseiling etc.
- type of structure and its main dimensions.

The following data is required from the inspection reports:

- the elements of the bridge structure which are damaged or contain defects
- the location of any defects
- type, extent and severity of any defects. If possible a more exact quantitative description of the extent of defects should be assessed on site during the inspection, otherwise it must be made later in the office based on plans of mapped defects, which are made during the in-depth inspection. Photographs can also be used if they are taken in such a way that the assessment can be made satisfactorily.

This information can be used to determine suitable maintenance options and their associated costs. For standard repair techniques the costs can be obtained from previous work whereas the costs of techniques that have not been applied previously may need further investigation.

An economic evaluation is then made of each of the proposed maintenance options by taking account of the total costs ie both the direct engineering costs in carrying out the selected repair option and the associated indirect costs. Indirect costs include administration and traffic delay costs (ie user delay costs, vehicle operating costs, costs associated with any weight restrictions or width restriction, the costs of any detours and accident costs). In assessing the traffic delay costs over a period of years the rate of traffic growth must be taken into account especially for heavily trafficked roads. As maintenance strategies for each bridge consider costs over the long-term, costs incurred in the future must be discounted to present day values so that a direct comparison can be made of the different maintenance options [Vassie, 1998]. As traffic delay costs often significantly exceed direct costs, it is recommended that they are recorded separately from the engineering costs.

There are several short and long-term maintenance strategies that can be applied to a deteriorated or substandard bridge structure and each has different consequences:

- Do nothing. In this case the consequences of further deterioration of the bridge structure or its elements must be taken into consideration as well as the additional costs associated with future repairs.

- Regular maintenance work. Regular maintenance work is usually carried out at prescribed time intervals. It only applies to certain types of maintenance work, which must be done at regular time intervals during the lifetime of the structure. The scope of the regular maintenance work and time intervals are usually known since construction (or design if nothing changed during the construction) and depend mainly on the type of the structure, location of the structure and equipment built into the structure. Some typical examples of regular maintenance work include: cleaning expansion joints, drainage systems both on the structure and on embankment slopes, the surface of the carriage-way and the ground around the bridge, and renewal of anti-corrosion protection on steel railings, safety barriers and lighting columns. If inspection reveals that the condition of an element that has been subjected to regular maintenance work is satisfactory, further maintenance can be deferred.

- Preventative maintenance work. This type of maintenance work is carried out to prevent or to postpone the initiation of deterioration of the structure. In most cases preventative maintenance work is connected with corrosion of reinforcement or structural steelwork and aims to prevent or slow down corrosion, if it has already started. The decision to carry out preventative maintenance work is usually taken if the quality of the structure or some part of it does not reach the design requirements for durability. For example, the concrete cover depth is less than that specified in design or due to visible signs on the structure which indicate that deterioration will take place if no action is carried out. Visible signs include wetting of the concrete surface due to defects in the drainage system or inadequate design. Other defects include carbonation of the concrete surface, ingress of chlorides and cracked or deteriorated paintwork on structural steelwork. Preventative maintenance includes repairing defects in the drainage system or replacement if it is inadequate, application of surface coatings, painting of structural steelwork, renewal of waterproofing membranes and pavements and sealing of cracks in the pavement surface. For corrosion prevention, the first application of surface coatings may depend on the depth of the carbonation front or chloride ions with respect to the reinforcement, but this practice varies in different countries, eg in the UK silane is applied immediately after construction.

- Repair work. This is undertaken to repair damage caused by deterioration or other effects such as vehicle impacts and to slow down the rate of future deterioration. The aim is to reinstate to an acceptable level the functionality of a structure or its components. The work may sometimes be applied simply to reduce the rate of deterioration or degradation, without significantly enhancing the current level of functionality. The repair work is not necessarily intended to restore the structure or its element to their original level of functionality and/or durability. The time of application of the repair depends on the severity, extent and cause of deterioration as well as the chosen repair technique. Usually some boundary conditions which have an impact on the quality of the repair work must be fulfilled, for example if the chosen repair technique is dependent on environmental and weather conditions, this must be taken into consideration when planning the repair.

- Strengthening/rehabilitation. This strategy is carried out with the intention of restoring the structure back to its original condition or increasing its load carrying capacity, functionality and/or durability. The chosen strategies depend on the type of structure, the material used in its construction and the present level of the load carrying capacity, functionality and durability.

- Replacement. This strategy is taken into account when the costs for repair or strengthening exceed the replacement costs of the structure. This level depends on several criteria, which are more thoroughly presented in Deliverable D7. If replacement is deferred, additional measures must be taken into account to provide an acceptable level of service and safety (eg load restrictions, lane closures, temporary propping and frequent inspection intervals). Such measures may substantially increase indirect costs.

There are several techniques that can be used to evaluate the optimal maintenance strategy for individual bridge structure. One approach is to compare the amount of money saved by deferring maintenance work from the current year with the long-term costs due to the additional deterioration. A suitable measure for determining the optimal strategy is "cost/benefit" analysis [Vassie, 1997 and Mechanical Engineer's Handbook, 1998]

$$\boldsymbol{CR}_{j,t_0-t_i} = \frac{\boldsymbol{C}_{j,t_0} + \boldsymbol{CA}_{j,t_0-t_i}}{\boldsymbol{CD}_{j,t_0-t_i}} \qquad \text{(Eq. 8.2.1)}$$

where:

CR_{j,t_0-t_i} - ranking cost/benefit ratio for the j-th maintenance option during the period t_0-t_i.

$C_{j,t_0}+CA_{j,t_0-t_i}$ - resultant increase in life time costs and additional costs due to additional deterioration for the j-th maintenance option during the period t_0-t_i.

CDj,t_0-t_I - money saved in the current period by deferring maintenance work for the j-th maintenance option during the period t_0-t_i.

The higher the ratio the higher the priority for maintenance work because less money is saved by not doing the maintenance work in the current year compared with the long term costs. When comparing ratios for different repair strategies the best is the one that gives the best value of money.

It must be stressed that in some cases the optimal maintenance strategy may not be selected. This can occur on very busy routes where, due to the much higher indirect costs connected with the optimal maintenance strategy, more frequent periodic maintenance work with less disruption and lower associated indirect costs may be selected.

Project level optimisation is described in more detail in Chapter 7.

8.2.3.2 Network level optimisation

A similar approach as was presented in the previous paragraph (8.2.3.1) for the project level optimisation can be adopted for the network optimisation. For the network optimisation comparison of the amount of money saved by deferring maintenance work from the current year with the long-term costs due to the additional deterioration is carried out for the whole bridge stock. A suitable measure for determining the optimal strategy is "cost/benefit" analysis [Vassie, 1997 and Mechanical Engineer's Handbook, 1998]

$$\boldsymbol{CR^k}_{j,t_0-t_i} = \frac{\boldsymbol{C^k}_{j,t_0}+\boldsymbol{CA^k}_{j,t_0-t_i}}{\boldsymbol{CD^k}_{j,t_0-t_i}} \qquad \text{(Eq. 8.2.2)}$$

where:

CR^k_{j,t_0-t_i} - ranking cost/benefit ratio for the k-th bridge, it's j-th maintenance option during the period t_0-t_i.

$C^k_{j,t_0}+CA^k_{j,t_0-t_i}$ - resultant increase in life time costs and additional costs due to additional deterioration for the k-th bridge, its j-th maintenance option during the period t_0-t_i.

$CD^k j,t_0-t_I$ - money saved in current period by deferring maintenance work for the k-th bridge, it's j-th maintenance option and during the period t_0-t_i.

The higher the ratio the higher the priority for maintenance work because less money is saved by not doing the maintenance work in the current year compared with the long term costs.

To carry out the network level optimisation, the following data must be available:

- The period of time, for which the optimisation is to be carried out although this may change during the period over which costs are optimised.
- The number of bridges on for which the optimisation is carried out over the chosen period of time. Several maintenance options should be considered for each bridge. For each option it is necessary to estimate the life of the repair. These results are obtained from the project level optimisation.
- Planned construction work of new roads as they may affect the indirect costs, e.g. the new road may reduce the cost of detours.
- Other works that are planned on the road network.
- The expected level of available funds within each year over the period for which costs are being optimised.

- Any constraints that may affect the decision on which technical solution should be adopted or the time when the maintenance should be carried out. For example, consider a post-tensioned beam and slab bridge, with a heavily damaged beam containing broken and corroded tendons and a reduced load carrying capacity, crossing a railway and road. The first option is the removal and replacement of the damaged girder and the second option is to repair the girder in-situ by removal of deteriorated concrete and replacement of the broken or corroded tendons with external prestressing. The decision on which option to choose is based on the constraints imposed by the removal (demolition in-situ or extract the damaged girder from the superstructure) and replacement of the girder (connection with the rest of the superstructure) and the impact this has on user costs (closure of the bridge for the period of removal and replacement). In some instances, the technical solution may be affected by the weather for example good weather conditions may be required if the selected option involves applying coatings or waterproofing membranes.

Network level optimisation is an iterative process and in practice there are also some unforeseen factors that may cause the maintaining authority to deviate from the optimum maintenance program. Such factors are:

- natural disasters eg floods or earthquakes
- co-ordination of maintenance work on a group of bridges on the same road
- co-ordination of maintenance work on bridges with the pavement maintenance on the road
- political decisions
- available funds for the maintenance of the whole bridge stock.

Coordination of maintenance costs on a group of bridges or on bridges with pavement maintenance can reduce the traffic delay costs, but may increase the direct costs as a result of delaying the maintenance work.
Prioritisation of the maintenance programme based on political decisions is usually not optimal and cannot usually be justified technically and economically. If political decisions are taken into account in developing the maintenance programme for some bridges, direct and indirect costs may increase due to planned maintenance work being deferred on other bridges. It is recommended that the effect of political decisions on the maintenance programme, if not professionally and economically justifiable, is not taken into account.

If there are groups of bridges for which a large expenditure would be required to bring them up to the required standard, for example structures in seismic regions, a special programme with its own funding for upgrading bridge structures should be initiated. Failure to do this would result in a large expenditure on strengthening and upgrading these structures which may impose a severe constraint on the funds available for other structures.

The optimal maintenance programme may need to be revised to take account of these additional factors. It is unlikely that the revised programme would be the same as the optimal programme for individual structures. The modified programme would specify which structures would be included in the maintenance programme in the short term, which structures will be taken into account for advanced optimal maintenance programme within a specified number of years and for which structures the optimal maintenance programme will be delayed by specified number of years. The time periods involved vary in different countries.

8.3 PRIORITY RANKING

Prioritisation of bridge structures based on their condition rating gives an indication of the relative condition of each structure. However bridges with the highest priority (ie in poorest condition) may be located on less important roads than bridges with a lower priority. Therefore additional parameters need to be taken into account to rank bridges in a more realistic way. A method for doing this is described in detail by Peruš and Žnidarič [1998] and is summarised below.

As stated in Section 8.2.2, the four factors that affect the priority ranking (R_A) of a bridge structure are: safety index (β), condition rating (R_C), impact factor (I_F) and the remaining service life of the structure (S_L).

Safety index β takes into account the resistance and the load effects of a structure. Usually its value is in the range 2 to 3.75 and sometimes higher, with the safety of the structure increasing with increasing safety index.

Condition rating R_C is a measure of the condition of a structure and is derived from the results of inspections and any tests that have been carried out and is described in more detail in Chapter 3.

The impact factor reflects the economic and political effects of undertaking maintenance on a structure. Its value ranges from 0 to 1. As an example, the value of the impact factor may depend on the location of the structure. It would be higher if the maintenance was to be carried on a structure on a highway with a high volume of traffic in an urban and/or industrial area than on a structure on a rural road that carries a low volume of traffic.

The remaining service life S_L is evaluated in years and because of the lack of satisfactory deterioration models, which would take into account the parameters that influence deterioration; the estimate of remaining service life is based on engineering judgement.

The problem faced by the maintenance engineers is how to develop a model for priority ranking (R_A) that takes account of the four factors listed above.

The factors that affect the priority ranking can be regarded as input values and the resultant priority ranking the output value. To determine unknown output variables from known input variables, a database is needed that contains reliable data that covers the full range of the input variables likely to be encountered. The database should include both measured and assessed values of the output and the corresponding input variables. This can be represented by a sample vector, which can be used to describe one particular set of variables. The input and output variables correspond to the components of this vector. For example, if a bridge with R_C = 12.1, S_L = 35 years, β = 3.5 and I_F = 0.67 has a ranking value R_A = 7.3, then the sample vector is defined as *{12.1, 35, 3.5, 0.67, 7.3}*. The database for modelling priority ranking consists of a finite set of sample vectors.

There are a number of methods that can be used to solve this problem. In the CAE method, each of the output variables corresponding to the vector under consideration (ie a vector with known input variables and unknown output variables) can be estimated from the formula:

$$r_k = \sum_{n=1}^{N} C_n \cdot r_{nk} \qquad \text{(Eq. 8.3.1).}$$

Where

$$C_n = \frac{c_n}{\sum_{j=1}^{N} c_j} \qquad \text{(Eq. 8.3.2)}$$

and

$$c_n = \exp\left[\frac{-\sum_{i=1}^{L}(p_i - p_{ni})^2}{2w^2}\right] \qquad \text{(Eq. 8.3.3).}$$

Where r_k is the k-th output variable (e.g. R_A), r_{nk} is the same output variable corresponding to the n-th vector in the database, N is the number of vectors in the database, p_{ni} is the i-th input variable of the n-th vector in the data base (e.g. R_C, S_L, …), p_i is the i-th input variable corresponding to the vector under consideration, and L is the number of input variables.

Eq. 8.3.1 suggests that the estimate of an output variable is computed as a combination of all output variables in the database. Their weights depend on the similarity between the input variables p_i of the vector under consideration, and the corresponding input variables p_{ni} pertinent to the sample vectors stored in the database. C_k is a measure of similarity. Consequently, the unknown output variable is determined in such a way that the predicted vector composed of given (input) and estimated data (unknown output) is the most consistent with the sample vectors in the database.

The parameter w is the width of the Gaussian function, which will be called the smoothness parameter. It determines how fast the influence of data in the sample space decreases with increasing distance from the point whose co-ordinates are determined by the components (input variables) of the vector under consideration. The larger the value of w, the more slowly this influence decreases. Large w values exhibit an averaging effect. Ideally, in the case of uniformly and densely distributed data, w should correspond to a typical distance between data points. In this case the CAE method yields a smooth interpolation of the functional relationship between the input and output variables.

In some applications, a non-constant value of w gives better results than a constant value. When using non-constant w values, Eq. 8.3.1 can still be used, but proper, locally estimated values of w_i should be taken into account. The formula for c_n (Eq. 8.3.3) can be rewritten as:

$$c_n = \exp\left[-\sum_{i=1}^{L}\frac{(p_i - p_{ni})^2}{2w_i^2}\right] \qquad \text{(Eq. 8.3.4),}$$

where different values of w_i correspond to different input variables.

It should be noted that the mathematical derivation of Eqs.8.3.1 - 8.3.3 given by Grabec and Sachse [1987] is based on the assumption of a normal distribution of the input data. The extension of the applicability of these equations to non-constant w values (Eq.8.3.4) is, however, based on physical considerations and described by Fajfar and Peruš [1997]. Whereas a constant w corresponds to a sphere in an L-dimensional space (L is the number of input variables), a non-constant w value corresponds to a multi-axial ellipsoid in the same space.

The choice of an appropriate value of w depends on the distribution of data, on its accuracy and on the sensitivity of the output variables to changes in the input variables. Some engineering judgement, based on a knowledge of the investigated phenomenon, and trial and error, is needed to determine appropriate value(s) for w.

8.4 MODELLING THE PRIORITY RANKING

Priority ranking is, as described above, a function of four main parameters:

$$R_A = R_A(R_C, S_L, \beta, I_F)$$

As it is very difficult to express the relationship between these variables in an explicit form, the alternative solution is, as described above, a non-parametric description by CAE, where Eq. 8.3.4 is used. According to the above notation, the expressions can be written as:

$$c_n = \exp\left[-\frac{(R_C - R_{Cn})^2 + (S_L - S_{Ln})^2 + (\beta - \beta_n)^2 + (I_F - I_{Fn})^2}{2w^2}\right]$$

and finally:

$$R_A = \sum_{n=1}^{N} R_{An} \cdot \frac{c_n}{\sum_{j=1}^{N} c_j}$$

To use the above terms normalised sample vectors are required. Usually, the normalisation (for interpolation only) is done as a linear transformation from original sample space to abstract sample space, where individual components range from 0 to 1.

8.4.1 Impact factor I_F

The Impact Factor I_F is in essence composed of two factors, the importance factor (IMF) and functionality factor (FF). The importance factor takes into account the position of the bridge on the network and importance of the network itself. It is defined by four functions that describe:

- road class
- volume of traffic
- location of the bridge
- historical value.

The functionality factor is also defined using four functions, which describe the impact of different functional deficiencies of the bridge. These four functions describe:

- vertical clearance
- width of the bridge
- possible detours
- weight restriction.

Importance and functionality factors are determined using the CAE hybrid neural network. The impact factor (I_F) is simply calculated using the following normalised expression

I_F = (IMF+FF)/8.

If the impact of the earthquake vulnerability of a bridge structure is taken into account, then impact factor is simply multiplied by a factor IE,

I_F = IE×(IMF+FF)/8.

where IE is defined as:

IE = 1+G,

where G is the designed ground acceleration expressed as a fraction of the acceleration due to gravity. A more detailed description of the methods for determining the importance and functionality functions is described in Deliverable D12. It is recognised that this simple expression for impact factor needs further development and analysis, although a case study of priority ranking presented in Deliverable D12 gives satisfactory results.

8.5 PREDICTION OF FUTURE CONDITION

One of the consequences of deferring maintenance work as a result of the constraints discussed above is that the bridge will continue to deteriorate. Therefore, a model for predicting the future deterioration rate is needed in order to ensure that it does not deteriorate below an acceptable level and that, if required, suitable measures can be imposed to ensure the safe passage of vehicles. Some methods for doing this are described in Chapter 6.

Deterioration may be the consequence of a single mechanism or there may be several processes involved. To use this data a periodical assessment of the bridge structure as a whole and/or its element must be made and put into a properly structured database. Using this method it is important to quantify different deterioration processes in terms of severity and extent. As an example the condition of a structure or its elements can be graded as 5 - very good condition, 4 - good condition, 3 - satisfactory, 2 - poor, 1- very bad. With curves of condition grades versus time some predictions of future deterioration can be obtained. There are also several methods for doing this. One approach is to analyse either the whole bridge stock (figure 8.2), or each structure, or each structural components, or all structural components of a bridge stock. An estimate for the whole bridge stock may be in some cases misleading for estimating the deterioration of structural components. Figures 8.4 and 8.5 show the deterioration of a column over a period of three years, although the global assessment for the whole structure is satisfactory. Therefore in the future two analyses of deterioration rates should be made, one for global estimation of deterioration of structure and another for structural components.

Another approach is to use the transition probabilities and the current condition state in a Markov Chain analysis. In this case the prediction is based on the current condition state of the particular element under consideration and on how quickly other elements in the stock of similar type, age and condition have deteriorated in the past. The method is more thoroughly described by Vassie [1997].

Figure 8.4: Deterioration of a column 1997

Figure 8.5: Deterioration of a column 2000

8.6 CONCLUSIONS

The work in this chapter has shown that further research is needed to develop optimal bridge maintenance strategies. New mathematical tools, such as neural networks and genetic algorithms, are being developed for providing optimal maintenance strategies as well as the classical mathematical formulations. As developing maintenance strategies requires information on the past, present and future condition of a structure as well as on the previous maintenance works, databases have to be continually updated with additional data. These data are necessary to improve current models for optimising maintenance strategies as well as for developing new ones. They have to be organised in such a way that they can be easily manipulated for different purposes and used in the analysis with different methods. A systematic collection of all direct and indirect costs is needed from the past (if available) and the present and future planned repair work. Therefore, it is essential for the future planning of maintenance work that repairs are carefully inspected during the bridge inspections so that their effectiveness can be evaluated with time. Inspecting repairs depends on the repair technique used. In some cases, especially in very aggressive environments, monitoring new structures or repair work is recommended to obtain additional data for planning future maintenance strategies. These costs should also be taken into account when analysing the costs of planned maintenance strategies. Finally, the general framework of the optimisation can be developed, but it may require modification for use in different countries, because each will have different requirements.

8.7 REFERENCES

Fajfar P and I Peruš (1997). *A non-parametric approach to attenuation relations.* Journal of Earthquake Engineering, Vol. 1, No. 2, pp. 319-340.

Grabec I and W. Sachse (1997). *Synergetics of Measurement.* Prediction and Control. Springer-Verlag, Heidelberg.

Mechanical Engineers' Handbook (1998). *Investment Analysis.* Myer Kutz Editor, John Wiley & Sons.

Peruš I and J Žnidarič (1998) *Deterioration Categorisation and Priority Ranking by a Neural Network-like approach.* Strategies for Testing and Assessment of Concrete Structures, Guidance Report, CEB Bulletin 243, May.

Vassie P R (1997). *Managing Bridge Maintenance.* Proceedings International Conference on Structural Faults and Repair, Edinburgh.

Vassie P R (1998). *Whole life costing of maintenance options.* Papers, The Management of Highway Structures, 22-23 June.

CHAPTER 9

FRAMEWORK FOR A BRIDGE MANAGEMENT SYSTEM

9.1 INTRODUCTION

This chapter brings together the work described in the previous chapters to develop a framework for a bridge management system.

A bridge management system has as its heart a relational database for storing all the information required to carry out the management functions; this is called the bridge inventory. Much of this data already exists albeit in paper records. However, it should be appreciated that a considerable effort is required to compile and verify this existing data, collect missing data and thereafter to enter data modifications promptly to ensure that the database provides a reliable and up to date record. The main functions of the BMS are best catered for in separate modules that are attached to each other via the inventory. These modules consist of mathematical models, algorithms and data processing tools. The modules needed to manage bridge maintenance efficiently and effectively have been discussed in previous chapters and are:

- condition appraisal (Chapter 3)
- assessment of load carrying capacity (Chapter 4)
- rate of deterioration (Chapter 6)
- structural assessment of deteriorated structures (Chapter 5)
- deciding maintenance strategies and methods (Chapter 7)
- prioritising maintenance work (Chapter 8)

The modules also correspond with the main activities associated with managing bridges:

- various types of inspection
- testing
- assessment of bridges in different conditions
- preventative maintenance
- repair work
- strengthening
- replacement

Management systems sometimes exist for road pavements, lighting columns, retaining walls and embankments and street furniture such as sign gantries and sign posts. In theory all these elements of road infrastructure could be combined to form an all embracing infrastructure management system which would have some potential benefits. At the present time, however, it is considered that significant operational difficulties would arise because the operations at the site level are not yet sufficiently well integrated. The approach taken here is to produce the framework of a management system for bridges while taking account of the long term objective of combining this with management systems for other types of infrastructure. The first step to full integration would be to link the inventories for different types of infrastructure through their location on the road network using global positioning systems.

The application of these modules generates a maintenance programme that indicates:

- the maintenance work needed each year on each bridge in the stock and
- the recommended maintenance method and its estimated cost

that are necessary to keep all the bridges in the stock open to the full range of normal traffic at a minimum overall lifetime cost subject to any constraints that may be imposed from time to time such as a maximum annual budget for maintenance work. Where it is necessary to impose constraints the BMS should evaluate the consequences of imposition in terms of reduced life, increased lifetime cost and increased disruption to bridge users. The following sections discuss the background to each module and summarise the work described in previous chapters before describing the BMS framework and interconnections between the modules. Particular attention is given to the data requirements of each module and to applications at both network and project levels.

9.2 GENERAL APPLICATIONS OF THE BRIDGE INVENTORY

Typically bridge inventories contain a few hundred data fields which cover aspects such as:

- bridge identifiers - name / number
- bridge location - map reference, road name, route number, obstacle crossed
- bridge elements and components
- bridge dimensions
- bridge materials
- forms of construction
- year built and required life
- traffic data
- structural assessment history
- inspection history
- test history
- maintenance history
- bridge owner, maintenance agent, region, services

Queries and associated reports relating to this data can be carried out using normal database operations. An almost unlimited variety of queries can be posed using logical operators and criteria on selected data fields. A few examples of queries follow which will illustrate the possibilities:

Example 1: List the name, bridge number and age of all bridges in region A where the principal bridge inspection is overdue by more than 1 year.

Example 2: List the name, bridge number and location of all bridges using deck waterproofing membrane type B on roads subject to winter maintenance with rock salt de-icer.

Example 3: List the name, bridge number, location, inspection history and maintenance history of all bridges in region C that are classified as ancient monuments.

Example 4: List the name, bridge number and location of any bridge on the route M1 that has a weight restriction imposed.

The first part of each example indicates the information from the inventory that is required and the second part specifies the criteria that should be applied to ensure that the data reported only relates to bridges satisfying the criteria.

The first example is one that would be used regularly to check that the principal inspections of bridges are not being overlooked by mistake, an event that is quite possible on a few bridges when the stock contains, typically several thousand bridges.

Example 2 may arise if problems have been identified with a particular type of deck waterproofing membrane and it is required to identify the other bridges with this type of membrane especially if de-icers are used since these may penetrate into the deck and cause latent corrosion of the reinforcement.

The third example may arise if the heritage authority requests information on the condition and maintenance of bridges classified as ancient monuments.

The fourth example may arise during investigations into the effect of a load restriction on one bridge on traffic movements on its route.

These queries usually only take a few minutes to compose and can be saved for future use if necessary. The reports can be viewed on screen, printed or saved to file for electronic transmission.

This function of a BMS is very flexible and has numerous applications associated with day to day management activities.

9.3 CONDITION APPRAISAL

Bridges are usually designed and constructed to achieve a life of about 100 years hence it is important to monitor their condition periodically throughout their life in order to ensure that:

- they remain fit for purpose
- the level of deterioration is consistent with achieving the design life
- there are no obvious defects that affect the safety of the public.

These checks are the purpose of bridge inspection and the results can be used to provide information on the condition of a bridge. The term condition is quite general and means different things to different people. Guidelines for condition assessment based on a review of methods used in Europe and the United States are described in Chapter 3. In general it is based on the results of superficial, general and major bridge inspections. A fourth type of inspection, an in-depth inspection, is sometimes carried out on bridges that have to be repaired and comprises extensive measurements on site and investigations in the laboratory.

The review found that there are two concepts of condition assessment of the whole structure. The first is based on a cumulative condition rating obtained from a weighted sum of the condition states of each element of the bridge. The second is based on a condition rating class where the condition of the bridge is considered to be equivalent to the condition state of the element in the poorest condition. The first concept enables bridges to be ranked in terms of condition.

More advanced methods of condition assessment are also reviewed in Chapter 3. These include Artificial Intelligence methods such as Neural Networks, Fuzzy Logic and Genetic Algorithms and an example is given of the use of Neural Network model to categorise condition state of bridges suffering from reinforcement corrosion. It was found that the prediction of future condition remains a challenge, even when these advanced methods are employed. Further research is therefore needed on deterioration models and on the development of a database in order to be able to predict the future condition of bridges.

In order to use condition to monitor the deterioration of a bridge throughout its life it is necessary to make the definition more restrictive and precise and in particular it should be quantified. The work described in Chapter 3 identified two main approaches for quantifying condition:

(a) To make visual observations and simple tests to subjectively assess the condition on an arbitrary scale ranging, for example, from 1 (good condition) to 5 (very poor condition).

(b) To measure physical/ chemical parameters such as concrete strength, thickness of steel section, concrete resistivity and chloride content using more sophisticated tests.

Both approaches have significant disadvantages. The chief disadvantage of physical/ chemical measurements is that each measurement technique only takes account of one mode of deterioration and each element of the bridge may experience different deterioration mechanisms at different stages of their life.

The main causes of deterioration of construction materials and components are corrosion, freeze-thaw effects, alkali silica reaction and sulphate attack (Chapter 5). Each cause will require different tests to establish its presence, find the extent of the deterioration, determine its rate of development and assess the consequences. Consideration of corrosion of reinforcing steel in concrete demonstrates the difficulties. The following sequence of tests would be needed to monitor the condition throughout the life of the bridge:

a) determine the cause of corrosion (chloride or carbonation) using sampling methods
b) having diagnosed the cause of corrosion determine the extent of the problem over the surface of the structure by more extensive sampling to ensure statistical significance; for example measure the area and location of de-bonded concrete (spalled, cracked, delaminated) in regions of general corrosion
c) establish the consequences of chloride contamination or carbonation by measuring the depth of cover, depth of carbonation, chloride depth profile, the threshold chloride concentration for corrosion, and the time since corrosion initiation or the time to corrosion initiation
d) find where the reinforcement is already corroding by half cell potential measurements
e) establish the type of corrosion (localised or general) that is taking place by measuring the electrical potential gradient
f) measure the corrosion current density to estimate the rate of corrosion
g) expose the reinforcement to measure the remaining cross section of the steel bars in regions of localised corrosion

If these measurements were repeated periodically throughout the life of a bridge suffering from reinforcement corrosion the condition would be thoroughly monitored although it would still be difficult to express the condition in a concise quantitative form. The condition would be best represented by a multi-dimensional vector of the individual test results. In order to achieve satisfactory coverage of the bridge it would be necessary to carry out the measurement set at several locations. The high variance of results for several of the measurement techniques means that uncertainties would exist about how well the sample of measurement locations represents the condition of an element or the bridge. Real variations in the value of these measurements at different locations in the bridge lead to further confusion about the meaning of condition. Where measurements differ at different locations, suggesting variations in condition, it is debatable how the condition should be represented. Possibilities are:

(i) to take the average, medium or mode of the measurements at different points on an element
(ii) to take the worst case to represent the condition of the element

The first case is appropriate if an overall assessment of the condition of the element is required or if the variance is small. More often, however, the earliest occurrence of defects at some point on an element is required and in this situation the worst case approach is more appropriate. This draws attention to a fundamental point - why do we need to know the condition? It would evidently be interesting to know the average condition of a bridge especially for network management purposes. However average values are misleading at the project level because it is possible for part of an element to have some measurements indicating the presence of defects where the average of the measurements indicates no defects. This is especially likely due to the high variance of some types of measurement. The element may therefore require maintenance work even though the average condition appears to indicate no defects.

Even for network level management the worst case approach is more likely to give a better representation of condition since it reflects the need for maintenance more reliably. The above discussion about the interpretation of the condition of a bridge suggests that the need for different levels of maintenance may provide a simple and relevant measure of condition.

The cost of physical/ chemical testing as a general method for determining condition could easily exceed the cost of maintenance and hence this approach is only likely to be used in exceptional circumstances. Traffic management could be required in order to carry out these tests on some bridge elements and the associated traffic disruption would further count against the physical/ chemical test approach to assessing condition. Returning to the discussion relating to the question ‘why do we want to know the condition of a bridge or its elements?’ there are a few additional answers:

- To provide the opportunity to carry out simpler, cheaper and less disruptive maintenance procedures before further deterioration necessitates more complex, expensive and disruptive work.
- To provide a global view of the condition of the stock of bridges.
- To provide feedback to designers and builders about durability of construction materials and components so that work on improvements can be effectively targeted.

These potential benefits of assessing bridge condition must be compared with the costs of undertaking the testing work. It appears that in most circumstances the benefit: cost ratio will not be high enough to support the testing approach. Testing would however be necessary prior to maintenance work to establish the best method and the extent of the work required to achieve a durable repair. The amount of testing needed prior to maintenance would be much reduced if the testing approach had been adopted for assessing condition and this would clearly count in favour of this approach. The results from testing are also often required in order to carry out the assessment of load carrying capacity of deteriorated structures.

An important advantage of the testing approach is that it increases knowledge of deterioration mechanisms that will provide feedback to designers and managers to help them improve durability and lower deterioration rates.

Another way of limiting the amount of the testing work needed which is described in Chapter 3 is to use neural networks to try and derive a relationship between the test results and visual observations made by an inspector. After the neural network has been trained by supplying data on both visual observation and test results, it should be possible to just carry out the visual observation and to have this data improved by the neural network relationship. This should provide a more reliable condition assessment than that achieved with visual observations alone.

The main disadvantages of the subjective assessment of condition based on visual observations are:

- the subjectivity of the assessment can make the results vulnerable to bias
- visual observations cannot detect latent defects or the early stages of deterioration.

The first disadvantage can be largely overcome by developing a set of definitions for each condition state that are clearly discrete in the sense that there are distinct differences between the definitions for adjacent condition states. Discreteness limits the number of states that can be used to four or five in most cases. The effectiveness of a set of condition state definitions can be tested by arranging for a number of bridge inspectors to independently assess the condition state of a group of bridges in a statistically designed trial. A considerable amount of thought and iteration may be required to establish a satisfactory set of definitions and a number of sets may be needed to embrace different construction materials such as steel and concrete, and different forms of deterioration such as corrosion of steel and sulphate attack of concrete. The small number of states in a condition state system means that each state is associated with a maintenance strategy such as do nothing, preventative maintenance, minor repair work, major repair work, strengthening or replacement. This link between the condition state and maintenance strategy supplies a unifying theme for the BMS.

Condition state systems based on visual observations usually take account of both the severity and extent of deterioration. The severity of a defect is, however, usually of more significance than the extent in terms of maintenance needs. The extent of deterioration has more significance than severity in terms of the quantity and cost of maintenance work. Therefore in terms of the condition assessment the severity of deterioration is more significant whereas in terms of optimising maintenance costs the extent of deterioration is more significant. The limitation of condition assessment to visual observations of the severity and extent of deterioration usually means that is difficult to establish more than about three discrete condition states and this is barely adequate.

The second disadvantage relating to the limitations of visual observations for assessing condition is more important. Some defects that occur on bridges provide no visual indications and are classed as latent defects. Some latent defects ultimately produce secondary effects with observable indications when the primary latent defect becomes severe, but this usually occurs too late to prevent the necessity for major strengthening and refurbishment. Most defects only become visible when they have developed significantly. This means that more complex, costly, disruptive and extensive maintenance is needed than would have been the case if the deterioration had been detected sooner. In these circumstances the preventative maintenance strategy becomes, in effect, a disallowed option although systematic investigations are recommended to confirm the presence of latent defects. Preventative maintenance is often applied initially as part of the construction process but it generally has a limited life, which is short compared with the design life, and needs to be reapplied regularly if the protection is to be maintained. If the early stages of deterioration and breakdown of the protection provided by preventative maintenance are not detected, due to the limitations of visual inspection, then the time window for the effective reapplication will be missed, with the consequences described above.

The main advantage of the visual observation approach to assessing the condition of a bridge is operational. It can be carried as part of a bridge inspection without the requirement for additional access and traffic management and hence with little additional cost or disruption to traffic. The other main advantages are its simplicity and links with maintenance strategies.

The disadvantages associated with the two approaches to assessing condition discussed above suggest an approach comprising the best features of both. An approach based on the assessment of condition state by bridge inspectors can be recommended, but with the incorporation of sufficient non-destructive testing to enable latent defects to be detected and diagnosed in most circumstances. This approach will also permit more discrete condition states to be defined. It will not however evaluate the extent of deterioration of the area requiring maintenance. Further tests would be required if repair work becomes necessary although the preferred maintenance philosophy is to maintain the effectiveness of preventative measures applied during bridge construction so that the concrete remains undamaged. Preventative maintenance is generally applied to entire elements so there is no need for tests to determine the area requiring maintenance. An example of a condition state system for concrete bridges vulnerable to reinforcement corrosion is provided in Table 9.1.

The assessment of condition is usually carried out for each element of a bridge. This gives rise to questions about if and how the condition assessments should be combined to give an overall condition for the bridge. For project level management of a particular bridge it is probably best not to combine the condition assessments for each element since these relate most closely to the maintenance requirements. For network level management where the overall condition of a stock of bridges may be wanted, some type of aggregation of condition states must take place. Possible methods of aggregation are:

- the mean value the of condition states for all the elements of a bridge
- the median value the of condition states for all the elements of a bridge
- the mode value the of condition states for all the elements of a bridge
- a frequency distribution of condition states of the different elements comprising the bridge
- a weighted mean value
- worst case value.

Table 9.1: Condition state system for concrete bridges vulnerable to reinforcement corrosion

Condition State	Non Destructive Tests Used	Criteria	Maintenance Strategy
1.	Cover depth Carbonation depth Chloride depth profile Half Cell potential	No visible defects Cover depth > 30 mm Carbonation depth < 10 mm Chloride penetration depth < 10 mm Half Cell potentials in passive zone	DO NOTHING
2.	Cover depth Carbonation depth Chloride depth profile Half Cell potential	No visible defects Ratio of cover depth : carbonation depth < 1.5 Ratio of cover depth : chloride penetration depth < 1.5 Half Cell potentials in passive zone	Preventative Maintenance to retard carbonation and chloride ingress
3.	Half Cell potential	No visible defects Half Cell potentials in the active zone	Preventative maintenance to reduce the corrosion rate
4.	Half Cell potential Corrosion Current	Visible indications of corrosion Half Cell potentials in active zone Potential gradient low Corrosion current moderate or high	Repair concrete + Preventative maintenance to reduce the corrosion rate
5.	Half Cell potential Resistivity Chloride depth profile	Half Cell potentials in the active zone Potential gradient high Resistivity low Chloride penetration depth > cover depth	Repair damaged concrete + Preventative maintenance to reduce the corrosion rate and prevent the development of incipient anodes
6.	Remaining cross section of reinforcement by invasive examination Area of de-bonded concrete by observations and delamination soundings.	Remaining cross section < 90% Area de-bonded > 10%	Carry out an assessment of load carrying capacity and strengthen if necessary

The mean value does not always represent the required maintenance work reliably for reasons discussed previously and furthermore the mean value will be non integer which is inconsistent with the discrete nature of the condition state scale. The median or mode provide a better measure of the central value since they keep an integer value. All central value measures are quite poor in representing the amount of maintenance work required, which is the primary purpose for assessing condition. A frequency distribution of the number of elements in each condition state corresponds closely with the amount of maintenance required and also provides measures of the central value and dispersion of the distribution. It is also straight-forward to combine frequency distributions from a group of bridges to obtain an overall distribution for the group. The only disadvantage is that the condition of a group of bridges is not represented by a single numeric value. The frequency distribution does however allow the number of elements requiring a particular type of maintenance to be enumerated and this is probably more useful. Weighted mean values are sometimes calculated where the condition of each element is weighted according to its perceived importance. In a similar way mean values of condition state for the bridges in a group can be weighted according to their relative importance. The importance of an element or a bridge is certainly a significant consideration in bridge management because it is associated with the consequences of an element or a bridge failing or needing major maintenance which would cause varying degrees of disruption to users of the bridge. The importance of a bridge, however, has nothing to do with its condition and it is recommended that the two concepts are not combined. The importance of a bridge should be taken into account in the evaluation of the costs and disruption associated with maintenance work and in the generation of optimised and prioritised maintenance programmes. Similar arguments apply to the combination of condition and rate of deterioration. The condition assessment should be used simply to identify maintenance needs. The worst case value would indicate the most complex form of maintenance needed by each bridge but would not provide a central measure of condition.

The assessment of condition is primarily associated with the inspection module of the BMS. There are four levels of inspection that are generally adopted – superficial, general, principal and special. Superficial inspections take place annually and consist of a brief visual examination to elicit any serious defects, but no condition assessment is made. This type of inspection is often combined with the annual visit for basic routine maintenance to carry out activities such as cleaning drains and controlling the growth of vegetation. The results of superficial inspections are not usually recorded in the BMS. General inspections are carried out about every 2 years and consist of visual observations made without special access arrangements. An assessment of condition is made of those elements that can be observed, but some elements will be obstructed from view and hence cannot be inspected. A condition assessment will not be possible in these elements. Principal inspections are carried out about every six years and involve detailed visual observations supplemented by some non-destructive testing and sampling. Provision is made to enable the inspector to gain close access to all parts of the bridge and a condition assessment is made for each element of the bridge. Special inspections are carried out as required and not at a regular frequency. They are used to establish the cause and extent of the deterioration and are usually carried out prior to repair work so that it can be correctly specified. Special inspections involve the extensive application of non-destructive testing and material sampling. Condition assessments made during general and principal inspections are normally stored in the BMS. The results of special inspections are not always stored in the BMS. It is however recommended that test results are stored in the BMS since this will help when assessing the rate of deterioration.

The main purpose of bridge inspections can be summarised as:

- to decide if a more detailed inspection is needed
- to assess maintenance needs and strategy
- to assess the safety of users and to decide if a structural assessment is needed
- to reduce the risk of unexpected failure
- to comply with regulations
- to assess the condition of a bridge element.

Information about condition is stored in the inventory database and can be combined with other data in the inventory. For example the frequency distributions of the condition of different elements of a bridge can be aggregated to include only bridges

- in a given region or
- in a given age range or
- on a particular route or
- in a particular type of environment or
- within a given range of span length.

Alternatively the condition of elements that satisfy various limitations can be aggregated. Examples include:

- bridge decks with a particular type of waterproofing membrane
- bridge decks with a particular type of expansion joint
- bridge piers on roads treated with de-icing salt.

The above examples of criteria defining the selection of bridges or elements from the entire stock are very simple and it is possible to combine simple criteria to form a complex criterion using logical operators such as AND, OR, and NOT.

The discussion of condition assessment has been detailed because the information is of crucial importance and is used for all the other modules of the BMS, ie

- assessment of load carrying capacity
- rate of deterioration
- optimisation of maintenance costs
- deciding the maintenance strategy
- prioritising maintenance work.

9.4 ASSESSMENT OF LOAD CARRYING CAPACITY

The previous section discussed the assessment of condition which is one factor that decides whether or not maintenance is necessary. Maintenance needs, based on condition assessment are usually decided by considering the lifetime economics of the bridge. In other words maintenance is carried out if it leads to a reduction in whole life cost. Maintenance work can also be sanctioned for aesthetic, political, social or environmental reasons, but these are too unpredictable to be included in the BMS at present and hence must be left to the judgement of local engineers. Another factor that plays an important role in deciding on maintenance needs is the load carrying capacity and whether it is sufficient to sustain the applied loads.

The maintenance needs arising from an inadequate load carrying capacity are essential in nature. In other words if the load carrying capacity is inadequate, load restrictions must be imposed until the bridge is strengthened to maintain safety. The only exception to this rule occurs when there is compelling evidence that any failure would be gradual such that inspection and monitoring would permit loads to be reduced prior to an anticipated failure. Thus maintenance required because of inadequate load carrying capacity is more important than that needed because of poor condition and normally has a higher priority as a result. There is, however, a strong interaction between condition and load carrying capacity since deterioration of condition almost invariably reduces the load carrying capacity. In the last section it was seen that when the condition becomes sufficiently poor the recommendation was to carry out a structural assessment to check the capacity. The situation regarding the link between condition and load carrying capacity is less straight forward than it appears. The significant parameter is actually the difference between the actual capacity of a bridge at a given time and the required capacity based on the possible loads carried at that time; the condition only affects the actual capacity. Some bridges have considerable reserves of strength and can undergo substantial deterioration before their capacity becomes substandard. In these cases the need for maintenance is more likely to depend on the condition rather than the load carrying capacity. For example spalling concrete may become a hazard for users or the poor aesthetics of a deteriorated bridge may lead to a loss of public confidence before the capacity becomes inadequate. In other cases the difference between the actual and required load carrying capacity may be quite small and relatively small amounts of deterioration could make the bridge substandard. There are less reserves of strength in some parts of a bridge than in others and it is important to know the location of these structurally critical areas, because more attention should be given to condition assessment in these locations (Chapter 4).

The above discussion explains the necessity for structural assessments to establish a measure of load carrying capacity and the location of structurally critical areas on a bridge. Recommendations for methods of assessment of load carrying capacity are described in Chapter 4. These methods are based on a review of current assessment procedures used in the countries participating in BRIME, including details of the characteristics of existing structures, the standards used in design and assessment, and the experimental assessment methods. The aim is to show how suitable and realistic assumptions for material, and structural properties and traffic loads can be obtained and implemented in a structural assessment.

To assess adequately the resistance properties of structural elements, data and models of loads and material strength need to be gathered. With regard to loading, the work described in Chapter 4 covers traffic loads, and increases in traffic loading are taken into account by the application of extreme traffic situations and the definition of a sufficient safety level. With regard to material strength, summaries of time-independent statistical properties are referenced for reinforced and prestressed concrete, steel, masonry and timber structures.

Bridge assessment in the partner countries generally is based on classical structural calculations in which the load effects are determined by structural analysis. The rules used are provided mainly by design standards with additional rules relating to testing methods, including load testing. Bridge assessment is usually based on either a deterministic or a semi-probabilistic approach; partial safety factors are used in the semi-probabilistic approach. These methods are sometimes considered to be conservative. A new approach taking into account the uncertainties of variables is emerging and reliability calculations are beginning to be introduced. The target reliability index is becoming the governing factor for assessment.

Chapter 4 recommends an assessment methodology based on five assessment levels going from a method using simple analysis and codified requirements (level 1) to a sophisticated assessment using a full probabilistic reliability analysis (level 5).

The review of current practice for assessing load carrying capacity was the starting point for the modelling of deteriorated structures that is described in Chapter 5.

The assessment of bridge strength is also an important input for the cost evaluation of various maintenance strategies and the decision making process (Chapter 7) and for the priority ranking (Chapter 8). A knowledge of bridge strength is essential for the routing of exceptional load vehicles and for the safe management of traffic.

A pass/fail outcome to a structural assessment is only barely sufficient because its use is limited to a particular point in time and it provides only a crude assessment of the age at which a bridge may become substandard. An estimate of the date for the next assessment of capacity cannot therefore be made.

The evaluation of the variation of load carrying capacity with time remains a challenge and requires further investigation. Methods for quantifying the structural effects of material deterioration so that they can be incorporated into the assessment of the load carrying capacity of bridges are described in Chapter 5. The methodology followed is divided into three stages.

i) identification and diagnosis of the common forms of deterioration present in the European bridge stock. From this survey, it is apparent that corrosion of steel due to carbonation and chloride contamination is the most common problem, and that ASR, sulphate attack and freeze-thaw action also occur at a significant frequency.

ii) evaluation of the existing methods for incorporating deterioration in assessment e.g. reduced cross-sectional area, modified stress-strain relationship, modified bond properties.

iii) investigation of straight-forward methods of taking account of deterioration in the determination of structural strength of components. This is carried out for deterioration caused by corrosion, ASR and freeze-thaw action.

Models to predict deterioration are often based on experiments using laboratory specimens, and need to be calibrated by comparison with site measurements and non-destructive tests. Only those site measurements that can be carried out reasonably quickly and minimise the disruption to site operations can be realistically used for predicting deterioration. Deterioration processes do not develop at a linear rate due to specific conditions on site and this complicates the methods used for making predictions.

For bridge management purposes structural assessments are required in order to:

- determine the reserves of strength of different parts of a bridge at different ages and conditions

- estimate the date for the next structural assessment
- estimate the date at which any part of a bridge will become substandard.

These estimates will be derived from a knowledge of the rate of deterioration and the reserves of strength measured at the last structural assessment. The reserves of strength will also depend on changes in loading although these are not predictable. Changes in loading will therefore generate a need for an assessment when they take place. At present the algorithms linking strength and rate of deterioration are very approximate with the result that the estimated dates above will be conservative. An improved understanding of the strength of deteriorated bridges and the factors affecting the rate of deterioration should lead to improved algorithms and estimates. This should permit bridges to be strengthened or replaced before they become substandard thereby avoiding loading restrictions and the disruption that usually results. Improved algorithms should also enable a reasonable estimate of remaining life up to assessment failure to be obtained. The aggregation of residual lives for the bridges in a stock would enable the BMS to determine the number and location of bridges requiring strengthening or replacement each year and to adjust the level of preventative maintenance to reduce the rate of deterioration if necessary.

The major factor holding up the calculation of reasonable estimates of remaining life is establishing how deterioration influences the strength of a structure (Chapter 5). Deterioration can result from environmental influences and from faults associated with design and construction. Usually deterioration results from a combination of different problems and this makes it a difficult process to model. For example reinforcement corrosion in bearing shelves and cross-heads usually results from the failure of an expansion joint which leaks, allowing saline water to fall onto the concrete element which has insufficient falls and drainage. The salt water then ponds allowing chloride ions to rapidly penetrate the concrete causing corrosion, especially if construction practices resulted in the formation of cracks in the concrete surface. Many physical processes are involved and it is easy to see the modelling difficulties. In a similar way the effect of a known level of deterioration on the strength of an element is difficult to estimate because it also depends on:

- the location of deterioration
- the number or area of defects
- the severity of defects.

and condition only really accounts for severity. The real problem is the non-uniformity of deterioration. For example if a reinforcing bar was uniformly corroded over its entire surface, a reasonable estimate of its strength could be obtained from the cross section of steel remaining. In practice, however, reinforcement corrosion is never uniform and is often in the form of pits which represent an extreme non-uniform situation. For steel corrosion the effect on strength may not be limited to reduced dimensions, but may also involve the ductility which is known to be reduced by corrosion and especially pitting corrosion. The situation is further complicated because the strength depends on a number of different load effects namely: flexure, shear, bond, bearing and deflection. To assess the strength of deteriorated concrete necessitates a knowledge of the tensile strength, bearing strength and elastic modulus as well as the compressive strength. The composite action between steel and concrete in reinforced concrete depends on the bond between these two materials and the mechanism by which corrosion affects the bond is not well understood. In undamaged concrete the bond depends on the bar type and compressive strength of the concrete but it is not known how levels of corrosion insufficient to fracture the concrete affect the bond. When corrosion causes the concrete to fracture leading to spalls, cracks and delamination it is clear that the bond is significantly reduced, but the extent is not known. These uncertainties and shortcomings to knowledge mean that any attempt to relate deterioration and strength is based largely on the engineering judgement of experts and is inevitably likely to be very conservative. The current practice of measuring the compressive strength of concrete and the tensile strength of steel to take account of deterioration in structural assessments is necessary but not sufficient. More extensive testing is, however, not justified until there is a better understanding of how the non uniform variation of physical properties of steel and concrete affect the strength of these materials; this can only be achieved by fundamental research.

At the present time, the only effective ways in which a BMS can use information from structural assessments is to

- determine the reserves of strength assuming no deterioration
- locate the critical structural areas on a bridge
- assess the condition particularly in the critical areas
- use engineering judgement to take account of strength and deterioration, and decide whether strengthening is needed.

Another problem is to evaluate the effect of strength deficiencies in one element on the strength of the whole bridge. The stresses in one element are often redistributed into other elements so that the strength of the bridge is greater than would be expected from the strength of the individual elements. The combination of all these uncertainties means that in order to maintain the risk of failure at an acceptable level the assessment of the effect of deterioration on strength will have to be conservative. Bridges with low reserves of strength will probably need to be strengthened if they suffer any significant deterioration in critical areas.

The prediction of the load carrying capacity in the future and the relationship between strength and condition remain some way from being achieved.

9.5 RATE OF DETERIORATION

It is important to know the rate of deterioration of bridge elements because it allows future maintenance to be planned. This enables the bridge manager to assess the best time to carry out maintenance work. There are significant costs involved in carrying out maintenance work too soon or too late. The cost of maintenance is sensitive to the time when it is carried out for two reasons:

(i) in the calculation of whole life cost the cost of maintenance work shows a reduction by a factor of $(1.06)^{-n}$, where the discount rate is 6% and n is the age of the bridge when maintenance is carried out, to give the net present value (NPV).

(ii) maintenance costs change disproportionately for each unit increase in condition state as the complexity of maintenance operations increase; the level of disruption to users resulting from the maintenance work often mirrors the increased cost.

It appears that the best time to carry out maintenance is just before a transition occurs between one condition state and the next poorer condition state. This is because a large step increase in cost occurs at the time of transition whereas costs increase only slowly during the interval spent within a particular condition state. The current knowledge of how condition varies with time is not sufficient to estimate this ideal time for maintenance more than approximately, but it seems like a worthwhile venture nevertheless. It can be seen that the consequences of carrying out maintenance too late are more serious than doing the work too soon so it is best to err toward early maintenance. There have, however, been cases where the assessment of condition has been incorrect and maintenance work appropriate for a higher condition state has been carried out unnecessarily and wastefully. This emphasises that it is essential to carry out the condition assessment properly and demonstrates the serious waste of money that can occur if maintenance work is carried out much too soon.

The situation regarding disruption to users as a result of maintenance work is also influenced to some extent by whether the work is carried out too soon or too late. When account is taken of the growth of traffic each year there may be less disruption if maintenance is carried out too soon, but if early maintenance has the result of requiring an extra maintenance treatment during the life of the bridge then the whole life disruption will probably be increased. Carrying out maintenance too late will usually result in more disruption because the more complex maintenance treatment required will generally result in more extensive traffic management over a longer time period. Traffic management and delay costs increase when maintenance work is deferred as shown in Figure 9.1 because further deterioration increases the duration of repairs and traffic growth leads to higher traffic flows.

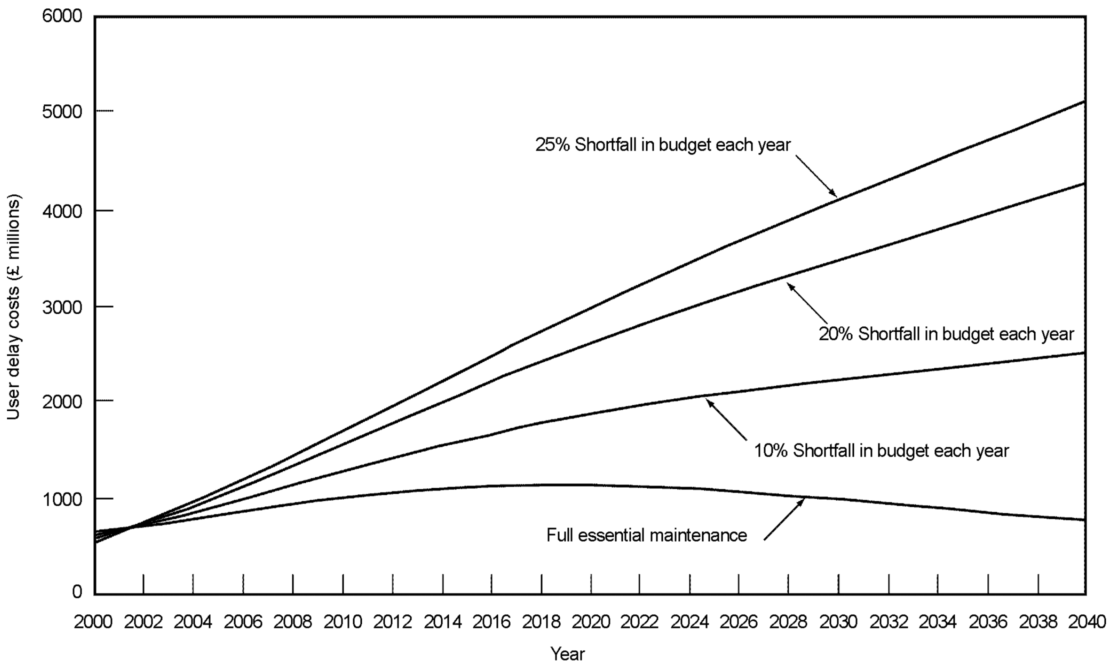


Figure 9.1: Traffic delay costs due to underfunding

A knowledge of the rate of deterioration also enables the bridge manager to

- estimate the residual life or time till the poorest condition state is reached
- decide on a suitable maintenance strategy for a bridge at different ages
- prioritise maintenance (rapidly deteriorating bridges have a higher priority because there is more chance maintaining too late)
- calculate a budget for bridge maintenance from information about how many bridges need maintenance each year.

One of the most important applications of deterioration rates is to determine the cost and disruption consequences of deferring maintenance work and less frequently of advancing maintenance work. It is often necessary for maintenance to be carried out at a non optimal time for operational reasons such as:

- budget limitations that mean some work has to be deferred
- to complete maintenance work on a bridge and avoid a return for a long period, the work on some bridges may need to be advanced or delayed
- to allow bridges on the same road to be maintained simultaneously to limit traffic disruption.

Deterioration is a natural process that should be expected to occur since it is unrealistic to expect a bridge or any other structure to remain serviceable for ever. The bridge manager's objective is to control the rate of deterioration so that the required serviceable life of the bridge is achieved. This objective can be satisfied by an appropriate design using durable materials or by applying maintenance at appropriate ages during the life of the bridge. In practice a combination of these two approaches is adopted in most cases.

The condition of a stock of bridges usually decreases as the average age of the stock increases. When the number of new bridges built during a period of time significantly exceeds the number demolished the average condition of the bridge stock tends to improve. If the average condition of a bridge stock is shown to be deteriorating too quickly it will be necessary to undertake a special programme of maintenance and replacements to retard the rate of deterioration and improve the average condition of the stock. The average condition and its rate of change give only an approximate measure of the rate of deterioration and its consequences. A better approach is to use the area enclosed by a graph of average condition versus age for the bridge stock as a measure of stock condition and the rate of change of this area as the rate of deterioration (Figure 9.2). This approach also indicates the rate of deterioration for bridges in particular age ranges and can therefore help in targeting maintenance work. Another useful procedure is to measure the rate of deterioration of groups of bridges that were in particular condition states a set period ago, say five years. This will indicate if the rate of deterioration is unusually high for groups of bridges in a particular condition state. The age and condition of bridges are the two factors that most influence the rate of deterioration of the bridge stock.

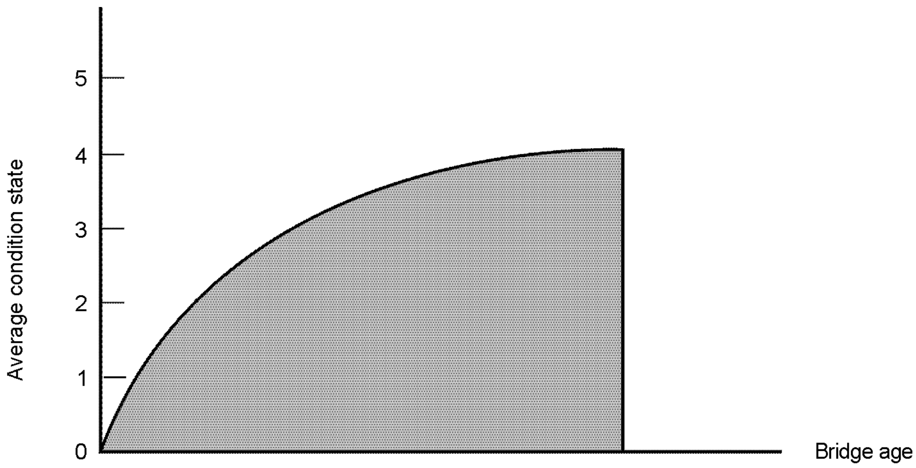


Figure 9.2: Condition of bridge stock represented by area under graph of average condition state vs. bridge age

The level of maintenance is the other factor that influences the rate of deterioration of the bridge stock although the effect it has on the rate of deterioration of particular bridges is even more marked. The condition of a bridge usually decreases until some maintenance work is carried out. Maintenance can have two effects:

- slowing the rate of deterioration
- improving condition.

Preventative maintenance has the first effect whereas repair work or rehabilitation should have both effects. A graph of condition state versus age for a bridge therefore consists of a number of discontinuous sections often leading to a saw tooth shape. The discontinuities occur at the ages when maintenance is carried out and result in a change of numeric value of gradient for preventative maintenance and a change in both sign and value of the gradient for repair/rehabilitation work (Figure 5.1). It should be noted that normally repair/rehabilitation work produces only a partial improvement in condition so that the condition when initially built is not recovered. Strengthening work on the other hand can raise the load carrying capacity to a value greater than that when built. In terms of its effect on the load carrying capacity of a bridge deterioration is most significant when it takes place in structurally critical parts of the bridge. However, it is unlikely, given the current state of knowledge, that it will be possible to estimate the rate of reduction in load carrying capacity from the rate of deterioration in the near future. The rate of deterioration may nevertheless be used to indicate when it is necessary to carry out a structural assessment.

To use information about the rate of deterioration to predict the condition state at a future age a procedure must be found to take account of the effect of maintenance work. One procedure is based on two factors:

- the immediate improvement in condition resulting from the maintenance work
- the change in gradient of the condition – time graph following the maintenance work.

Types of bridge or element with a high rate of deterioration can be identified and provide an indication as to whether the cause is poor design/materials or insufficient maintenance. This feedback can then be used to eliminate problems in the future.

Deterioration of bridges has a number of possible effects:

- reduces the strength
- makes it unsafe for users, for example due to falling masonry
- reduces the life
- impairs the appearance

It is the extent of these effects at different ages which largely determines the type of maintenance required. Although deterioration reduces strength this may not be significant; it will depend on the location of deterioration and the reserves of strength. A defect may or may not affect the safety of users; for example spalling concrete from the soffit of a bridge over a small river may have little effect on the safety of users whereas a similar defect in a bridge spanning a busy road or railway could have a serious effect on users. Deterioration generally results in a reduction in life although the magnitude will vary and will not always be significant. The appearance of a bridge is often impaired by deterioration and this can sometimes lead to a loss of public confidence in the structure.

It is difficult to generalise about the rate at which bridge elements deteriorate because different bridges and even different parts of bridges are exposed to different macro- and micro-climates. Even bridge elements of nominally similar construction and materials can have variations in concrete mix, cover depth and latent defects which can significantly influence the type of pathology and the deterioration rate. The two main approaches to determining rate of deterioration, physical and stochastic modelling are described in Chapter 6.

The approach described in detail in Chapter 6 is physical modelling applied to a particular deterioration process namely the ingress of chloride ions into concrete bridge elements. This has a limited goal in terms of the bridge management system as it deals with only a single deterioration mechanism; it is limited to the initiation phase of the corrosion process although it also deals with monitoring corrosion within a bridge and it proposes a durability surveillance system. It does however illustrate the difficulties in attempting to predict the rate of future deterioration.

The chloride ingress model has at present serious limitations as far as BMS is concerned. The model can predict when there will be a risk of corrosion initiation, but it cannot predict the corrosion rate of reinforcement. It can be useful for forecasting possible maintenance actions, but not for assessing deterioration and structural capacity.

Further research is therefore necessary to improve knowledge of chloride ion penetration models through the constitution of a database collecting measurements on site and in the laboratory, and most importantly to develop models for the propagation phase of corrosion.

9.6 DECIDING MAINTENANCE REQUIREMENTS

The primary decisions associated with the maintenance requirements for a bridge are:

- the maintenance strategy

- the maintenance method
- the extent of maintenance
- the age when maintenance is carried out.

The maintenance strategy for the network is often a policy decision. It is particularly important to select the appropriate strategy to minimise costs and maximise the effectiveness of maintenance. The options for maintenance strategy include:

(a) do nothing until a bridge becomes unsafe or substandard, when some form of strengthening or traffic restriction will be needed

(b) do nothing until the condition deteriorates to a benchmark value, when repair work will be needed to improve the condition

(c) carry out regular preventative maintenance to reduce the rate of deterioration thereby avoiding or delaying the need for repair work, strengthening or traffic restrictions.

Replacement of bridges when they become unsafe or substandard is an alternative to strengthening and the decision between the two strategies is usually based on economics.

The main advantages of strategy (a) are:

- no maintenance is necessary until a bridge becomes unsafe or substandard thereby deferring expenditure and traffic disruption to later in the life of the bridge
- the avoidance of maintenance costs and traffic disruption on bridges that do not become unsafe or substandard during their required life.

Both of these advantages will help to reduce the whole life cost.

The main disadvantages of strategy (a) are:

- the cost of strengthening work and the associated traffic disruption are high leading to increases in the whole life cost
- it is possible that large numbers of bridges may need strengthening at certain times reflecting the non uniform rate of bridge construction in the past. Industry has difficulty reacting to markedly non uniform maintenance requirements leading to delays in carrying out the work and a significant number of bridges with traffic restrictions, resulting in serious disruption to the movement of vehicles.
- defects may arise due to deterioration which although not affecting safety may increase the rate of deterioration and seriously detract from the appearance of the bridge. This can result in the need for strengthening at a lower age and a loss in public confidence about the safety of the bridge.

This strategy may be suitable for a very small number of bridges on which the optimal decision is to replace them and to let them last as long as possible.

The main advantages of strategy (b) are:

- no maintenance is needed until a bridge reaches the benchmark condition value thereby deferring expenditure and traffic disruption to later in the life of the bridge

- to retard the rate of deterioration thereby reducing the chance that strengthening work will be needed
- the avoidance of maintenance work and its associated costs and traffic disruption on some bridges where the rate of deterioration is low with the result that the benchmark condition value is not reached during the lifetime of the bridge.

These three advantages should help to reduce lifetime costs and traffic disruption.

The main disadvantages of strategy (b) are:

- The cost of repair work and the associated traffic disruption can be substantial.
- In years when large numbers of bridges need repair work the industry may not be able to react quickly enough leading to delays in carrying out the work. The increased rate of deterioration on these bridges will advance the time when strengthening is needed.
- If the benchmark condition triggering repair work is set at too poor a condition, the rate of deterioration can be increased and the visual appearance can be significantly affected before the benchmark value is reached.

The main advantages of strategy (c) are:

- Preventative maintenance is cheap in comparison with repairs and strengthening and can usually be carried out with little disruption to traffic.
- The rate of deterioration is retarded substantially, especially if the preventative maintenance is applied from new, and will delay or avoid the need for repairs and strengthening.

These two advantages will tend to reduce lifetime costs and traffic disruption.

The main disadvantages of strategy (c) are:

- More frequent maintenance is required; typically preventative maintenance requires re-application about every 20 years although further development work could result in longer intervals between preventative maintenance events.
- Preventative maintenance by its nature is applied to all the bridges in the stock before it is known whether or not it is necessary. Some bridges may deteriorate very slowly or have substantial reserves of strength and hence may not need repair or strengthening even without preventative maintenance. For these bridges preventative maintenance work would be wasteful, but our current state of knowledge is not sufficient to be able to identify them. It is therefore necessary to apply preventative maintenance to all bridges in the stock, although there may be scope for limiting the application to structurally critical zones and to areas vulnerable to deterioration such as areas exposed to salt spray or under leaking expansion joints.

These disadvantages will tend to increase lifetime costs.

The choice of strategy depends on many factors but the preventative maintenance strategy is usually preferred when:

- it can be applied from new
- a maintenance life of more than 20 years can be achieved
- the majority of the bridge stock is likely to require more than one session of repair work or strengthening work resulting from deterioration during its life.

In practice, in the past, the maintenance strategy has not been considered until defects were observed at which point preventative maintenance is no longer an appropriate maintenance strategy. The condition state of a bridge generally determines which maintenance strategies are possible.

The choice of maintenance method will be considered in some detail later in this chapter although it is pertinent to say here that the number of maintenance options from which a decision has to be make are substantially reduced when the maintenance strategy is pre-determined.

The extent of maintenance work should be decided on the basis of achieving a durable result with a life of at least 40 years. Partial repair work can only be regarded as a short-term measure that is rarely justified on economic grounds. The correct extent of maintenance work is normally decided on the basis of a thorough survey and tests carried out on the bridge.

The time at which maintenance is carried out can have a significant bearing on the efficiency and effectiveness of the strategy. Two possible approaches to deciding the best time to carry out maintenance are:

a) to consider the best maintenance option at a particular time, for example when funds become available (Chapter 7)

b) to consider the best maintenance option at regular intervals, say 5 years, in the future.

Approach (a) develops decision criteria that help to choose the best maintenance option for a given bridge at a particular time and is described in Chapter 7. It is based on a global cost analysis that includes safety, durability, functionality and socio-economic factors, and considers all the costs involved in construction, inspection, maintenance, repair, failure, road usage, and replacement. The strategy consists of minimising the global cost while keeping the lifetime reliability of the bridge above a minimum allowable value.

Three difficulties are encountered with respect to the application of this methodology. The first concerns the constitution of a database containing costs, especially indirect costs and failure costs. The second is related to the necessity of predicting the future behaviour of bridges and the probability of failure for the various alternatives. It is clear that, as indicated in previous sections, additional research work is needed for predicting the future deterioration of both structural elements and non-structural components, when the option involves deferring maintenance work for a significant period of time. The third is the difficulty of determining the best time to carry out maintenance: it may, for example, be better to delay the proposed maintenance work if the rate of deterioration is sufficiently low to avoid a transition in condition state. In some cases, it could be preferable to permit deterioration to continue for some time, incurring a transition in condition state, and then to carry out more extensive maintenance. Early maintenance is not necessarily the best and to determine the optimum time the following factors need to be considered:

- the current condition
- the rate of deterioration
- the future life required
- the maintenance cost
- the discount rate for calculating whole life cost
- the type of road and the traffic management needed for the maintenance work
- the current traffic flow rate and rate of traffic growth.

The second approach (b) does take account of these factors and hence determines the optimum time for maintenance in order to minimise lifetime costs and traffic disruption. The disadvantage of this approach is the complexity of the algorithms needed, although with the power of modern computers this should not be overemphasised.

An optimisation process involves the minimisation or maximisation of an objective function. It may also involve a number of constraints. For optimising bridge maintenance a typical objective function is the whole life cost, which requires minimisation. The whole life cost should include engineering, traffic management and traffic delay costs because on busy roads the latter can be a major contribution to the overall cost. Possible optimisation constraints are:

(i) a benchmark value for probability of failure
(ii) a benchmark value for condition
(iii) no constraint.

The first constraint implies that bridges will be maintained in a safe condition throughout their life. The second constraint implies that the condition of the bridge will not be allowed to deteriorate beyond the benchmark value. The 'no constraint' option is the least restrictive. It does not imply that unsafe bridges can continue in service because this is not permitted. If a bridge became substandard the maintenance options would be between strengthening/replacement or traffic restriction, the decision being based on the relative increases in whole life costs associated with the two options. On a busy road the cost of strengthening/replacement is almost certain to be less than the costs associated with continuous traffic restrictions. On lightly trafficked roads the cost of traffic restrictions may be less than strengthening/replacement, although site specific factors are likely to play a significant part in assessing costs. The 'no constraint' option would result in traffic restrictions being imposed on some bridges, although the consequences of the restrictions would be small.

The first constraint would by comparison result in no long term traffic restrictions being imposed on any bridge. In practical terms it is usually difficult to use the first constraint because the probability of failure is not known unless detailed structural assessments are made at regular intervals. Furthermore it would be necessary to know how the probability of failure changes with increasing age. This will depend on the rate of deterioration in structurally critical areas. However such relationships are not properly understood and it is unrealistic to expect the probability of failure to be predicted with any degree of accuracy in the short term, except for some deterioration mechanisms such as fatigue of steel. Condition values are however often available from inspections allowing the second constraint to be adopted. This constraint provides some assurance that the condition of individual bridges and the bridge stock will not deteriorate too far. This assurance is achieved at some cost compared with the no constraint option because the imposition of constraints will lead to a higher value of the minimum whole life cost.

Maintenance work clearly has to take account of both the load carrying capacity or probability of failure and the condition. In view of the current limitations of knowledge a pragmatic approach would be:

(a) to predict how the condition will change with time in the structurally critical zones

(b) to recommend a structural assessment when the condition deteriorates to a state consistent with a reduction of strength

(c) to either strengthen/replace or impose traffic restrictions if the assessment criterion is failed

(d) to consider carrying out repairs if these will reduce the whole life cost and carry out a further structural assessment at a suitable interval if the assessment criterion is passed.

Thus the need for strengthening or replacement is linked indirectly to the condition via a load assessment.

In most cases on major roads and sometimes on relatively minor roads the cost of bridge maintenance is dominated by the contribution of traffic management costs and traffic delay costs. Major savings can be made by maintaining traffic movements during maintenance work and by minimising the duration of restrictions even if this means providing temporary support to the bridge. There will of course be some parts of bridges on busy roads that can be maintained without implications for the traffic.

Decisions about maintenance options pertain to the circumstances associated with particular bridges such as the condition and extent of maintenance needed and hence are especially associated with project level bridge management. Decisions about the maintenance strategy can be policy led and are therefore more closely associated with network level bridge management. Network level management algorithms can be developed to predict the number of bridges requiring different degrees of maintenance each year but these algorithms cannot identify the particular bridges needing maintenance. This information can only be obtained from project level algorithms using bridge specific information. Network level information can be obtained by the aggregation of project level information and this can be employed to test the effectiveness of network level algorithms.

Ideally decisions about maintenance methods should involve aspects of safety, durability, functionality, economy, environment and sociology. Environmental and social factors are difficult to represent in monetary terms and have therefore not been considered by existing bridge management systems.

The followings costs should be evaluated when calculating the whole life cost of a bridge: design, construction, inspection, assessment, testing, preventative maintenance, repair, strengthening, replacement, demolition, traffic management, traffic delay and salvage value.

The ideal situation is for the spend on each of preventative maintenance, repairs and strengthening/replacement to be constant. This can only be achieved if there is sufficient money spent on preventative maintenance and repairs to control the deterioration rate at a reasonable level. If deterioration occurs too quickly the numbers of bridges requiring strengthening/replacement will increase to consume the entire maintenance budget resulting in further increases in deterioration rate and the number of substandard bridges (Figures 9.3a and 9.3b).

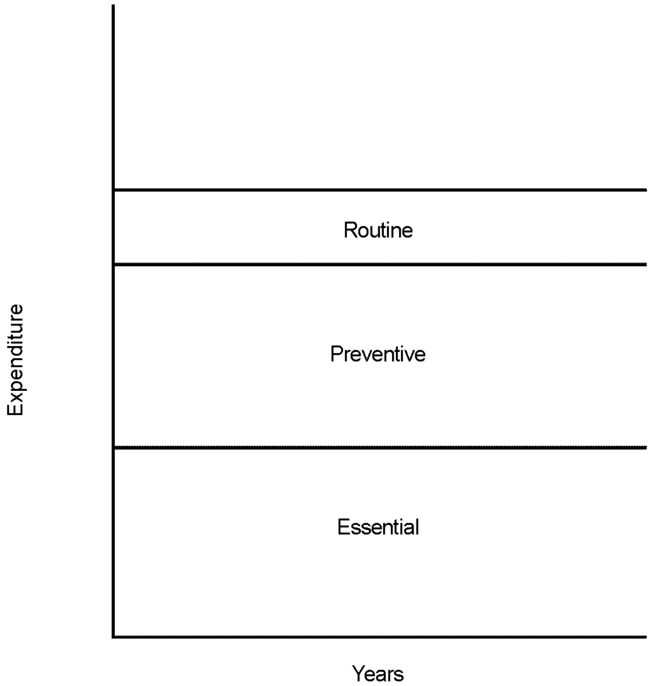


Figure 9.3a: Ideal bridge maintenance programme

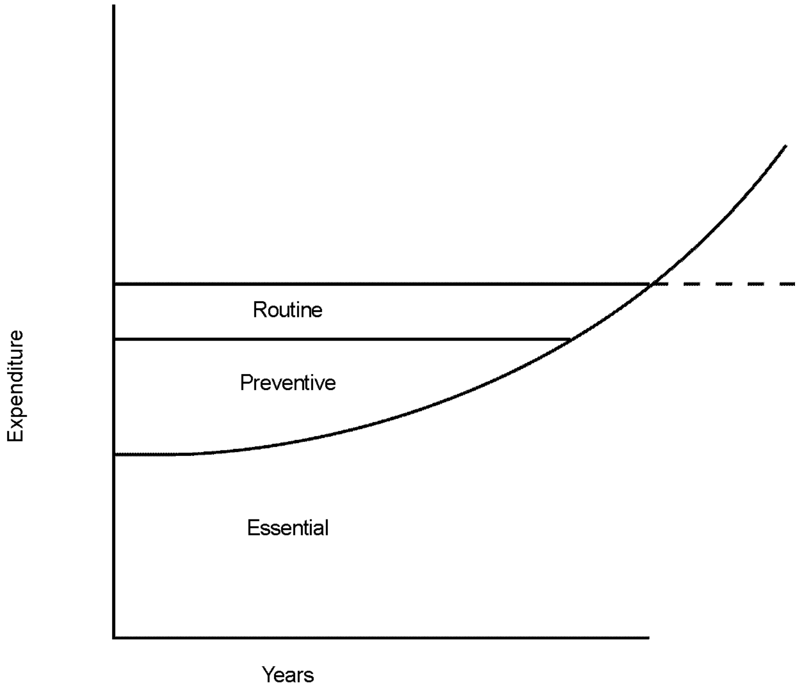


Figure 9.3b: Effect of long term underfunding

The optimisation process outputs an optimal maintenance programme that specifies what, if any, maintenance is needed on each element of every bridge in the stock each year in the future for the period of optimisation. The period of optimisation is not critical and can range from as little as 10 years to the bridge design life. All predictions are approximations and the further into the future that predictions are made the greater the errors. This is not however a major problem since maintenance planning is normally limited to about 10 years in the future where errors should be small. Regular updating of the data and regular re-application of the optimisation algorithm should ensure the reliability of predictions at least up to 10 years ahead. Optimisation algorithms can only take account of predictable processes such as natural deterioration. They cannot respond to events such as accidental damage, vandalism, natural disasters and political factors. Engineering judgement will still be necessary to deal with maintenance work arising from these events. Optimisation is primarily a project level management tool and while aggregation of the results for individual bridges provides a maintenance programme for the network that is optimal in some sense at the network level it cannot take account of the following actions that could reduce traffic management and delay costs:

- maintaining a group of adjacent bridges on a route in a single contract
- combining pavement and bridge maintenance
- combining several maintenance jobs on a bridge so that they can be done at the same time, involving the deferral of some work and the advancement of other work.

A broad outline of the main steps involved in bridge maintenance management is given in Figure 4. It involves two main algorithms:

(a) to optimise maintenance costs taking account of the rate of deterioration
(b) to calculate the rate of deterioration and predict future condition

Possible algorithms for (b) will be discussed in section 9.8 of this Deliverable. A possible algorithm for (a) is discussed here. The first step is to decide the maintenance strategy because this will substantially reduce the number of possible maintenance options. The approach is best explained by considering the tree diagram shown in Figure 9.5. Starting at year 0, which can correspond to any bridge age and condition, there will be a number of maintenance options (2 are shown in Figure 9.5) one of which is always 'do nothing'. Each maintenance option will have an associated cost so each branch of the tree will have a cost. The nodes at either end of a branch will have condition values representing the condition state before and after the maintenance work represented by the branch. These condition states are obtained using algorithm (b). Thus every node will have a condition state and a number of possible maintenance options represented by branches emanating to the right (increasing time). The optimisation process calculates the cost of each pathway through the tree and finds the pathway representing the lowest total cost. The cost of each pathway is simply the sum of the costs of each branch in the pathway discounting according to the time associated with each branch. In practice the number of pathways is very high (a three option tree over 20 years would have 3^{20} pathways) and although the calculations are simple a large amount of computer time would be needed. Dynamic programming can be used to eliminate redundant pathways in order to reduce the number that need to be costed and thereby to reduce the computer time to a reasonable value.

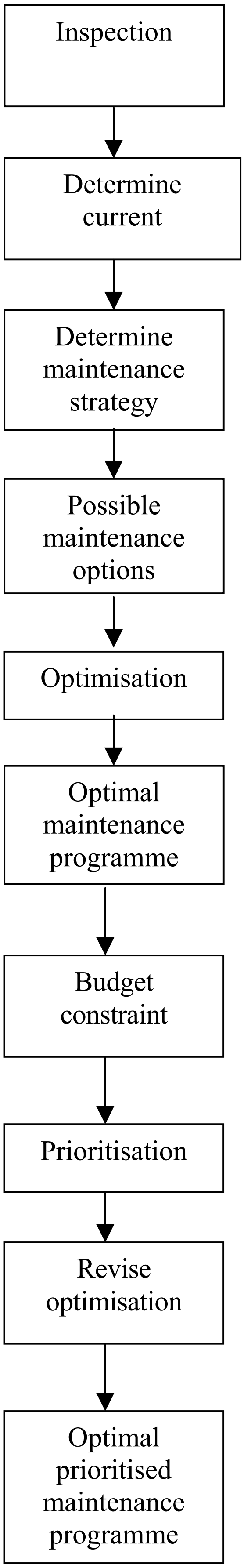


Figure 9.4: Main steps involved in bridge maintenance management

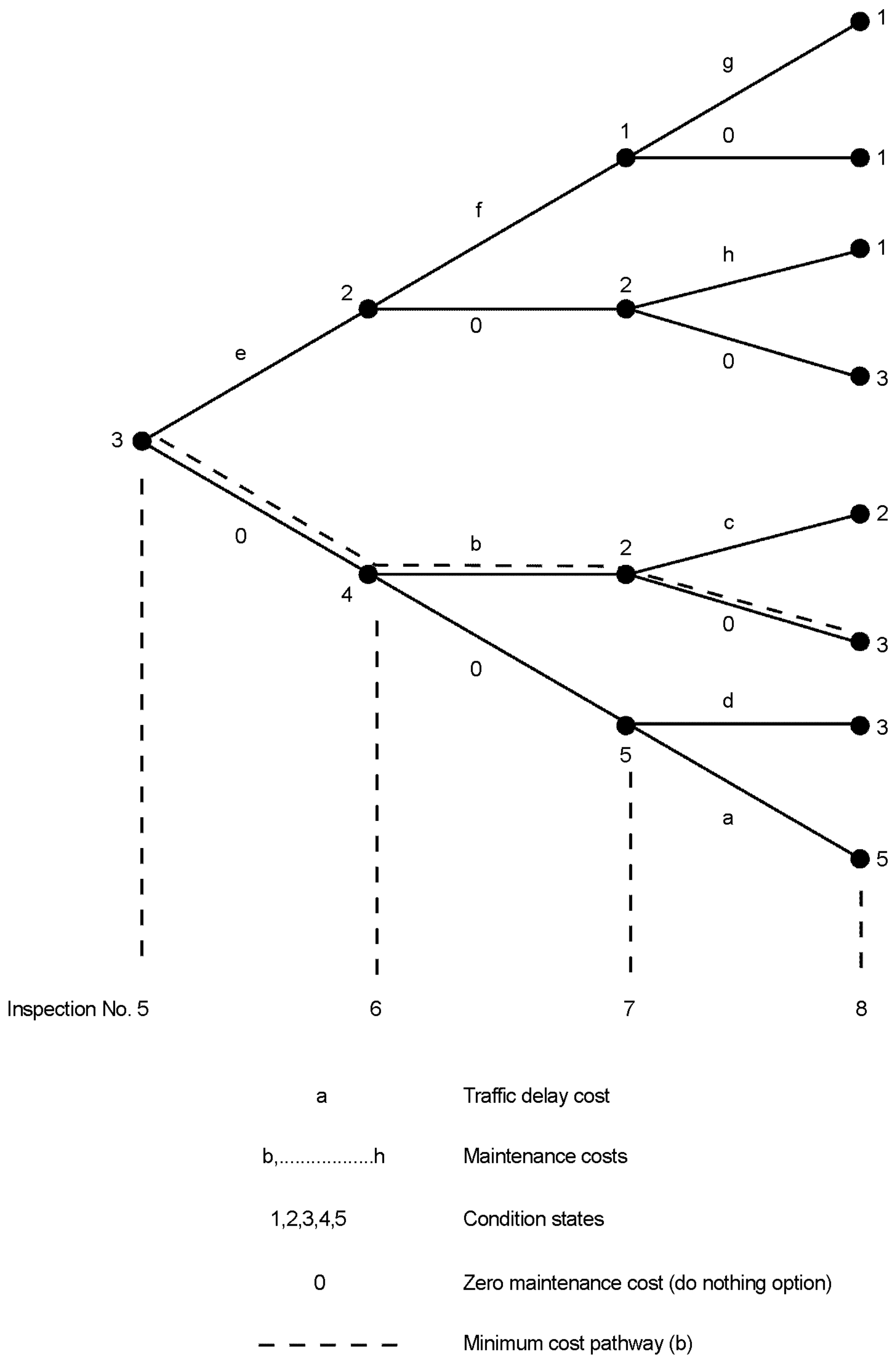


Figure 9.5: Optimisation tree diagram for two maintenance options per mode

9.7 PRIORITISING MAINTENANCE WORK

There are many factors that can lead to a need for prioritisation such as:

- policy decision to prioritise a certain type of maintenance
- policy decision to prioritise maintenance on a particular route

- policy decision to provide insufficient funding to carry out the optimal maintenance strategy.

The latter is by far the most common reason for prioritisation since the demand for public expenditure always seems to exceed the supply of revenue. The limitation of the maintenance budget is in effect a constraint on the optimisation process and it can be dealt with in this way although it complicates the algorithm and requires more computer time. In practice simpler methods have been adopted. One simple approach has the objective of minimising the number of bridges that have outstanding optimal maintenance work. This would involve ranking bridges in order of increasing maintenance costs so the ones with lower costs have higher priority and the bridges with high maintenance costs would be deferred for consideration next year. The consequence of this approach is that bridges with high maintenance costs may never be maintained. Clearly this approach is unsatisfactory even though the objective on which it is based is reasonable. A prioritised maintenance programme is by its nature sub-optimal and will result in increased life-time costs and traffic disruption. A better approach to prioritisation is to minimise these consequences. The steps involved in such an approach are as follows:

(i) In a given year form a subset of the bridge stock containing only those bridges requiring maintenance according to the optimal maintenance programme.

(ii) Assume for each bridge in the subset that the optimal maintenance work is deferred and call the resulting cost saving for a bridge the benefit.

(iii) Produce a new optimal maintenance programme for each bridge based on the assumption in (ii).

(iv) Calculate the increases in lifetime cost and traffic disruption for each bridge resulting from applying the assumption in (ii) and call this the cost of prioritisation for the bridge.

(v) Calculate the cost:benefit ratio for each bridge and rank the bridges in the subset in order of increasing value of this ratio.

(vi) The bridge with the lowest cost and highest benefit will have the smallest ratio value and thus the lowest priority for maintenance; this bridge will be selected for maintenance in the given year and removed from the subset.

(vii) Repeat step (vi) until the maintenance budget is consumed.

(viii) The bridges remaining in the subset when the budget is consumed will have their maintenance work deferred and will be considered for maintenance when it next becomes optimal.

In practice the above prioritisation process would only apply to bridges requiring non-essential maintenance. All bridges requiring strengthening or replacement would have the highest priority and this work would be carried out before the non-essential work was prioritised.

This prioritisation procedure is useful because it is objectively based. It does not consider subjective factors such as the environment, sociology, sustainability, aesthetics or historical value. These should be considered qualitatively by local engineers using engineering judgement to decide if the ranking of bridges should be modified.

Another approach to prioritisation is to consider all the factors affecting the priority and to combine them in some way to produce a priority index which can be used to rank the bridges (Chapter 8). This is a much simpler approach but suffers from subjectivity and hence possible bias concerning the values of some parameters and in the formula combining them. Subjective factors include the importance of the road, historical value and aesthetics. Objective factors include condition state, cost of maintenance, life of maintenance, life required and safety index.

The prioritisation process can have a profound influence on the maintenance programme and necessitates considerable care in order to minimise lifetime costs and disruption to traffic.

This is described in detail in Chapter 8 and the proposed methodology is divided into three levels as shown in Figure 9.6.

The first level of prioritisation is a ranking of bridges based on their condition rating that is based on the classification of condition into a number of classes. The purpose is to select the most damaged bridges having a condition rating above a given critical value. If enough funds are available, the bridges with a condition rating above the given critical value are repaired. In most cases however, the budget is limited, and it is not possible to maintain all of those bridges so a second level of prioritisation is necessary.

The second level of prioritisation is a priority ranking function R_A which takes into account the condition of the bridge (R_C), the safety index of the bridge (ß), the remaining service life (S_L) and the impact of the bridge on the road network (I_F). The impact factor takes into account the importance and functionality of the bridge, and is a function of road classification, traffic, location and historical value. The estimation of the remaining service life is based on engineering judgement. A fifth parameter, the seismic resistance of the structure, may be added in regions of seismic activity.

The model for ranking is therefore usually based on a function of the four parameters:

$$R_A = f\ (R_C\ ,\ ß\ ,\ S_L\ ,\ I_F\ )$$

A database containing sufficient empirical data is needed on a finite sample of ranked bridges in order to initiate the process. The CAE method is then used to predict the output variable R_A for a given bridge from the known input variables (R_C , ß, S_L , I_F) by taking into account the known relations between input and output variables of the sample of bridges. The CAE method is an optimisation method which can incorporate a knowledge-learning process or a neural network approach and requires a database containing sufficient, empirical data on a finite sample of the ranked bridges.

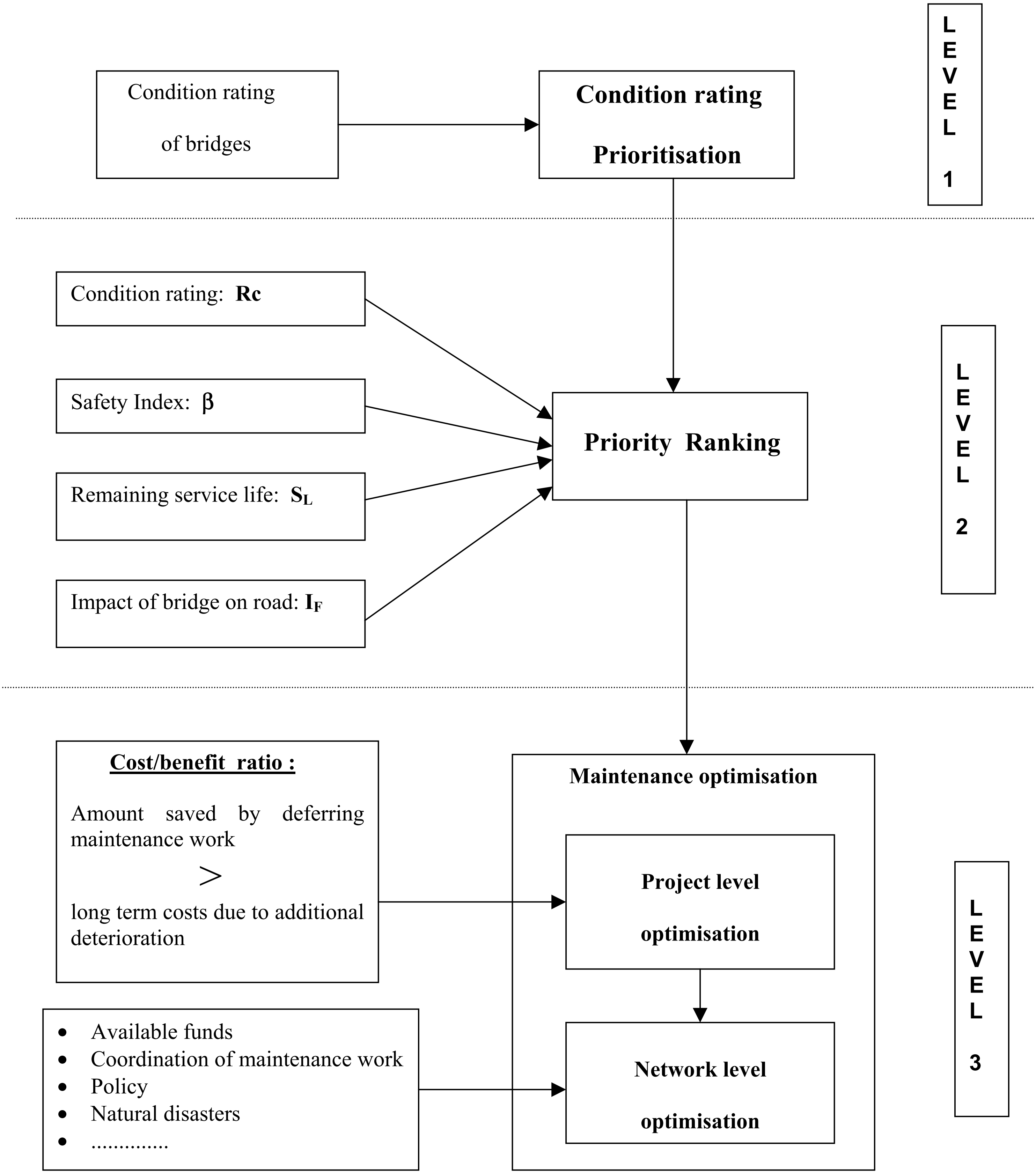


Figure 9.6: Scheme for prioritisation of bridge maintenance.

At the end of the process, a priority ranking of all bridges is obtained, but engineering judgement is still needed because all the constraints may not be considered (eg political decisions, urgent intervention for ensuring traffic safety, etc).

The third level of prioritisation is based on a maintenance optimisation for different selected maintenance strategies, taking into account the costs for each selected strategy. The optimisation is made for a particular bridge (project level optimisation) and then, for the whole stock (network level optimisation). The stock of bridges studied may be limited to the selected number of bridges resulting from the second level of prioritisation.

Project level optimisation involves an economic evaluation of each maintenance option and should take into account the total cost including both direct costs of repair and indirect costs (administrative costs, user delay costs, width restriction costs, etc.). The optimal maintenance strategy for a bridge, over a period $t - t_o$, is the one for which the amount of money saved by deferring maintenance work in current year is comparatively high compared with the long term costs due to the additional deterioration occurring during the period. This method is also called cost/benefit analysis, and the cost/benefit ratio, R, is expressed for a given bridge, for a chosen period $t - t_o$, and for a given maintenance option as:

$$R = \frac{\text{lifetime cost} + \text{cost due to additional deterioration}}{\text{money saved by deferring maintenance work}}$$

The lower the ratio, the lower is the priority for maintenance.

Network level optimisation uses the different values of the cost/benefit ratio obtained for all bridges to produce a ranking of bridges at the network level. This network level optimisation is an iterative process, which includes such factors as:

- available funds for maintenance of the whole bridge stock
- co-ordination of maintenance work for groups of bridges
- co-ordination of maintenance work on bridges with the maintenance work on the road
- political decisions
- natural disasters such as floods, earthquakes.

There are some common features between this approach and that described in Chapter 7 and summarised in Section 9.6. The methodology described in Chapter 7 is a global cost analysis for a given bridge, and it can be considered to be an alternative to the cost/benefit ratio method described above for the project level optimisation (third level). The two first levels described above are used to select a sub-set of the bridge stock on which maintenance prioritisation is to be carried out, to avoid having to apply the cost/benefit ratio to all bridges. Whichever method is chosen a network level optimisation is then required.

Limitations of knowledge on the subject make it difficult to give a preference between the global cost analysis or the cost/benefit ratio method in order to choose the best maintenance programme for a given bridge. An examination of the two methods shows that the first seems easier to apply and has the advantage of considering the whole life of the bridge. The second is intended to be used over a certain period of time and introduces the difficulty of needing to know the cost due to the additional deterioration, which requires the evolution of the deterioration process to be known with enough accuracy.

9.8 PREDICTION OF FUTURE DETERIORATION

It is clear from the preceding sections that a method for predicting future deterioration is a fundamental requirement for a BMS. A simple method is to conduct an extrapolation of the condition of a structure or its elements from the past data. This should be done on a homogeneous family of elements (like in the Pontis BMS), or on a homogeneous family of bridges (like in the French BMS). The French system considers the distribution of bridges in different condition classes as a function of their age over a period of seventy years and may be used to predict their future evolution. Many hypotheses should be made however, among them:

- changes in design and construction techniques have only a small influence on the durability
- continuity in the benefits of maintenance
- the absence of new causes of deterioration.

This method could be improved by switching from a deterministic analysis of the bridge transition among condition classes to a probabilistic analysis by introducing a Markov chain process and transition probabilities like in the Pontis BMS.

The probabilistic approach uses the condition state assessments made during bridge inspections. This information already exists and is cheap to collate. Measuring condition using the condition state assessments made by bridge inspectors is normally based on a set of discrete states which represent different stages in the deterioration process. There is a close association between the condition state and the appropriate type of maintenance which simplifies the interpretation of the condition measure. Different materials have different deterioration processes hence it is necessary to set up a condition state scale for all the common deterioration processes and materials of construction. A condition state scale consists of a number of states each of which is given a numeric value and a definition describing the stage of the deterioration process. The number of states is normally between 3 and 10. If the assessment is based entirely on visual observations, the number of states normally lies at the lower end of this range, whereas more states can be used if non-destructive tests and material sampling are used. When too few states are used the deterioration process is not adequately described, but if too many states are used it becomes difficult to differentiate between them, resulting in different inspectors making different condition assessments on the same element. Deterioration sometimes results in the formation of latent defects and in these cases it is recommended that the condition state should be based on visual observations, non-destructive testing and sampling. An example of a typical condition state scale for corrosion of reinforced concrete is shown in Table 9.1.

The general procedure is as follows:

(i) Sub-divide the bridge stock into sets of bridge elements with characteristics that indicate that they should deteriorate by similar mechanisms. Factors that are most likely to influence the deterioration process are the construction material, geographic location and when and how the element was previously maintained. As more information is obtained about deterioration processes the sub-division can be refined with the proviso that the number of bridges in each set is statistically significant.

(ii) the average condition state as a function of age usually fits quite well to a polynomial equation such as:

$$C(t) = a + bt + ct^2 + dt^3$$

where C(t) is the average condition state of the stock at time t years and a, b, c, d are polynomial coefficients

(iii) The beginning of the Markov Chain is represented by the tree diagram shown in Figure 9.7. The numbers at the nodes represent the condition state and the numbers associated with the branches of the tree such as p_{xy} represent the transition probability of going from state x to state y between consecutive inspections. Note the simplying assumptions:

$$y \geq x \quad \text{and} \quad y = x \quad \text{or} \quad y = x + 1$$

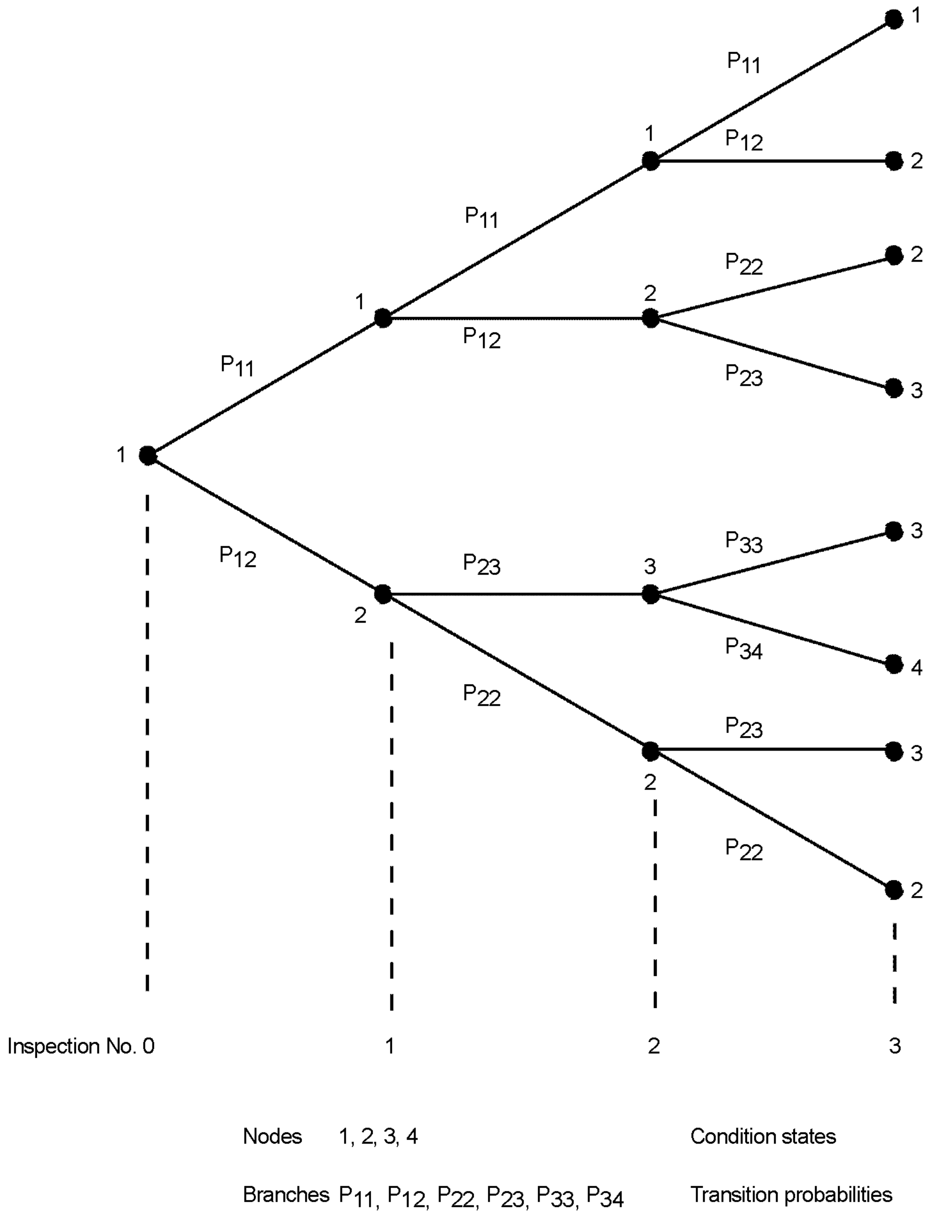


Figure 9.7: Markov chain diagram

This diagram can be used to determine the probability of being in a given state at a given time and to determine the average condition state at a given time.

(a) probability of being in state 2 at the second inspection is given by $p_{11}\, p_{12} + p_{12}\, p_{22}$

and

(b) the average condition state at the second inspection is given by

$$C_m(t,w) = p_{11}^2 + 2(p_{11}\, p_{12} + p_{12}\, p_{22}) + 3\, p_{12}\, p_{23}$$

where C_m (t,w) is the average condition state at time t determined by the Markov Chain. The set of transition probabilities is denoted by w.

Thus in order to find C_m (t,w) it is necessary to know the values for p_{11}, p_{22}, p_{33} and p_{44}.

9.9 A FRAMEWORK FOR A EUROPEAN BMS

A bridge management system that is able to answer the various objectives of the managers must be modular and must incorporate, at least, the following principal modules:

1. Inventory of the stock
2. Knowledge of bridge and element condition and its variation with age
3. Evaluation of the risks incurred by users (including assessment of load carrying capacity)
4. Management of operational restrictions and of the routing of exceptional convoys
5. Evaluation of the costs of the various maintenance strategies
6. Forecast the deterioration of condition and the costs of various maintenance strategies
7. Socio-economic importance of the bridge (evaluation of the indirect costs)
8. Optimisation under budgetary constraints
9. Establishment of maintenance priorities
10. Budgetary monitoring on a short and long-term basis

Figure 9.8 presents an architectural framework of a BMS including these principal modules with their main interactions. The framework takes into account the two levels of management (project level and network level) and is organised in order to show the contribution of each of the BRIME workpackages (WP 1 to WP 6).

9.9.1 Framework

The interrelationships between the BMS inputs, models and outputs are very complex as indicated by Figure 9.8. It would be difficult to produce a flow chart for the entire BMS using full names for inputs, outputs and models. There would be numerous intersections and the chart would quickly become very complicated to follow. Instead of a full flow chart the BMS has been broken down into various models and the inputs and outputs have been given for each model (Tables 9.2 and 9.3). The inter-connections between different models are explained in terms of outputs of models which act as derived inputs for another model (Table 9.4). The interconnections can also be seen in Figure 9.8 although the key for the codes for outputs, models and inputs will have to be used in order to interpret the framework. It is possible to traverse the framework starting at the output to find all the inputs required for the model. The models been divided into project and network level models.

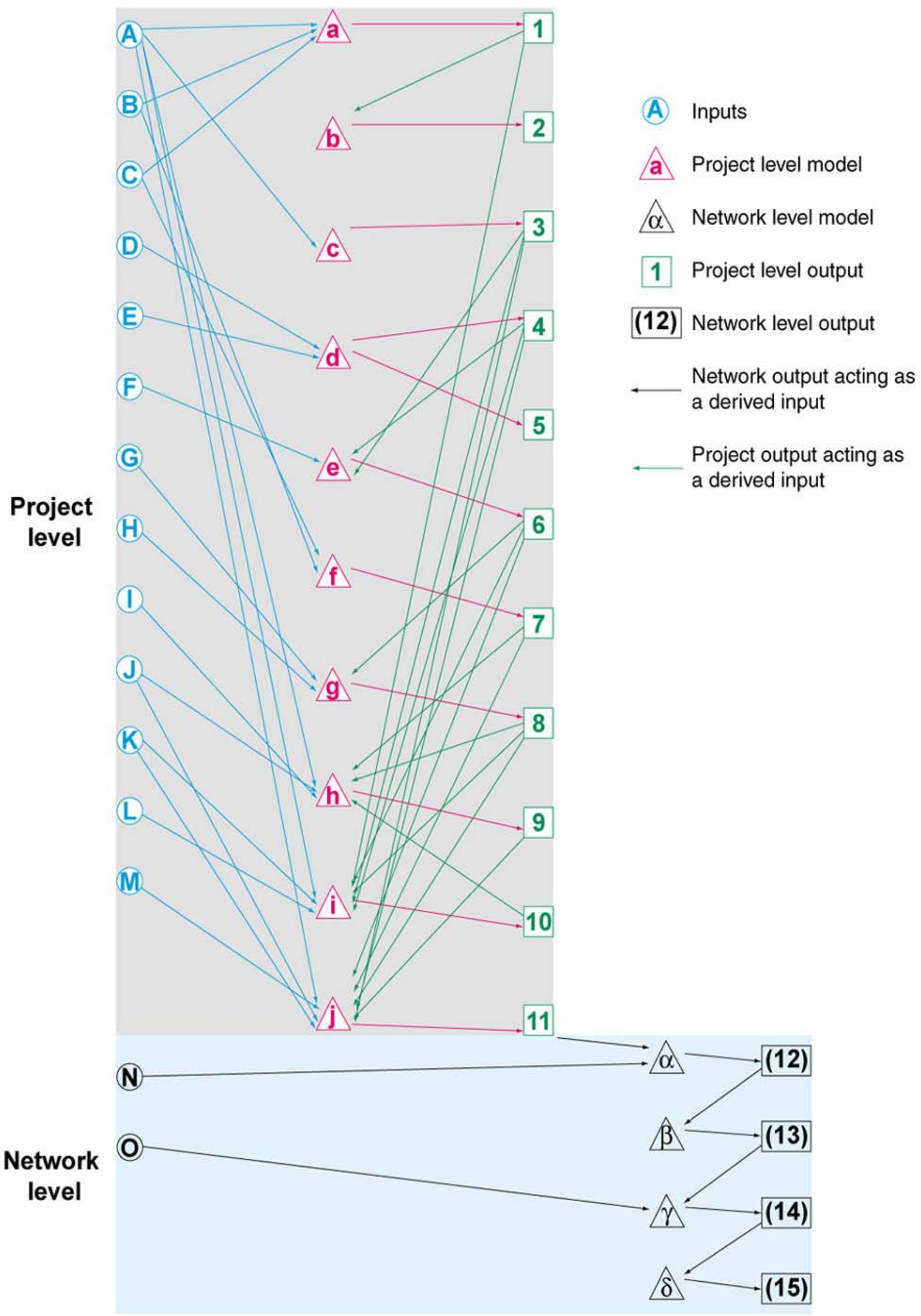


Figure 9.8: Interconnections between inputs, models and outputs for BMS

9.9.2 Models: (a to j) project level and (α to δ) network level

a. To use inspection observations, material testing and information in the inventory to derive a measure for the condition of each structural element and component of the bridge.

b. To combine the condition state values for all elements and components to provide a measure for the overall condition state of a bridge.

c. To use information in the inventory such as structural design calculations and as-built drawings to find the original load carrying capacity measured in terms of load, reduction factor or reliability index. The most structurally vulnerable parts of the bridge would also be identified.

d. The inspection and test histories for the particular bridge under investigation, and for all the other bridges in the stock with similar characteristics would be used to investigate the rate of deterioration and to predict how the condition of particular elements and the entire bridge would vary in the future. This variation with time could be represented as a condition state-time trajectory where a discrete condition state is associated with each year or some other agreed interval.

e. The purpose of this model is to predict the need for essential maintenance when the strength of the bridge becomes inadequate i.e. the bridge becomes sub-standard. This has always been difficult to achieve. The approach could be based on the following inputs:

- the latest load carrying capacity
- the structurally vulnerable parts of the bridge
- the condition state-time trajectory for the vulnerable parts
- information from the assessment history of the bridge.

Note that the first three inputs are outputs from other models (these have been called derived inputs) whereas the last input is an original input.

An estimate of the time when essential maintenance will become necessary can then be based on the original or latest assessment of capacity and the rate of deterioration near the vulnerable areas. If the capacity is only slightly greater than the minimum acceptable value, the bridge may become substandard in the future due to changes in loading even if the vulnerable areas are not deteriorating.

f. The cause of deterioration has a bearing on the maintenance methods that are effective in a given situation and may also influence the maintenance strategy. The extent of deterioration can also affect the choice of maintenance method and strategy; it will directly influence the cost of maintenance. The information from inspections and tests can be used to establish the cause and extent of deterioration.

g. Maintenance work and traffic restrictions to substandard bridges can result in disruption and delays to road users which have an economic cost. These costs can be calculated using traffic data such as traffic composition, vehicles per day and traffic growth rate, the duration of restriction and the type of vehicles needing to be re-routed. The latter can be obtained from assessment results. The impact of traffic disruption can influence the choice of maintenance method and strategy.

h. The optimal maintenance method selected for a bridge should, subject to various constraints, minimise the lifetime cost of the bridge. In other words if a different maintenance method was used the lifetime cost would be greater. The choice of maintenance method will depend on:

- information from the inventory such as access and repairability
- the maintenance strategy adopted
- the delay costs resulting from different types of maintenance
- the cause and extent of deterioration
- the required life, free from maintenance after the work is completed
- the costs and lives of different maintenance methods

i. The choice of maintenance strategy has an important bearing on the life of a bridge and the whole life cost. Maintenance strategies include replacement, strengthening, rehabilitation, repairs, preventative maintenance and do nothing. The primary distinctions are based on the condition and strength of the bridge. If the bridge is substandard due to insufficient load carrying capacity or falling masonry/concrete for example then essential maintenance is required. It is essential in the sense that if the maintenance is not done traffic restrictions must be imposed to make the bridge safe for users. It can be seen that the requirement for an essential maintenance strategy such as strengthening or replacement depends on safety rather than cost. If a bridge is not substandard, but has undergone considerable deterioration, rehabilitation or repairs are likely to be the chosen strategy. In this case the need for maintenance is based on reducing the lifetime cost by increasing the age of the bridge when essential maintenance eventually becomes necessary or by avoiding the need for essential maintenance altogether. Preventative maintenance is a possible strategy when deterioration has not yet occurred to a significant extent; it should reduce the rate of deterioration and lifetime costs. The choice of strategy can also depend on the element involved and other site specifics which would be recorded in the inventory. In particular the choice of maintenance strategy will depend on:

- the current condition state for the elements and bridge
- the load carrying capacity and critical parts
- information in the inventory
- required future life of bridge
- condition state-time trajectory for each element
- date when essential maintenance will become necessary
- delay costs associated with different strategies
- maintenance history and policy

j. An ultimate objective of a BMS is to establish an optimal maintenance programme for each bridge (project level) which will predict the timing and type of maintenance required to achieve both the safe operation of the bridge and a minimum lifetime cost. The optimisation will have to take account of the following factors:

- information in the inventory

- the choice of optimal maintenance method
- maintenance costs and lives
- delay costs
- the rate of deterioration (condition state-time trajectories)
- the life required of the bridge
- the extent of deterioration
- the date when essential maintenance becomes necessary
- the discount rate used in lifetime costing.

The first ten models are involved with analysing data and making decisions about the maintenance of particular bridges - project level bridge management. The next four models (α-δ) analyse and make decisions about a stock of bridges - network level bridge management. The bridges chosen to be within the stock can depend on many factors such as

- geographical region
- type of road –national roads, minor roads etc
- type of bridge – overbridge/underbridge

For various reasons it may not be possible or convenient to carry out the optimal maintenance work for each bridge. For example there may be insufficient funds or labour and efficiency can sometimes be improved for the network as a whole by deferring or bringing forward maintenance for particular bridges in order to co-ordinate the work and reduce traffic disruption. These considerations place constraints on the optimisation process of which the following are common examples:

- budget
- network efficiency
- policy

A constrained optimisation process produces an optimal maintenance programme subject to the constraints imposed. This is, of course, sub-optimal compared with the unconstrained optimisation. It has been found that the available funding for bridge maintenance is almost always insufficient to carry out all the work identified in the unconstrained optimisation. Thus the work needs to be prioritised in such a way as to minimise the whole life cost subject to limited funds being available each year. It is important that prioritisation should not affect safety hence bridges needing essential maintenance must be satisfactorily maintained or traffic restrictions imposed. The problem with prioritisation when it continues over many years is that the number of substandard bridges will progressively increase resulting in continuously decreasing funds for non essential maintenance, thereby creating a vicious circle.

The purpose of optimisation and prioritisation processes is to ensure that the money spent on bridge maintenance achieves the best value. In practice these bridge management techniques are usually first applied to a bridge stock that already has a significant number of substandard or deteriorated bridges. The manager may therefore not only want to ensure that his expenditure is achieving the best value, but also that policy targets for the condition of the stock and individual bridges are also being satisfied. Such policy target parameters may include the following:

- number of bridges with load restrictions of different degrees
- number of bridges with other traffic restrictions
- number of substandard bridges
- annual traffic delay costs due to restrictions and maintenance works
- number of bridges overdue an inspection
- number of replacements each year
- average condition of the stock
- number of bridges with condition state greater than X
- number of bridges containing one or more elements with a condition state greater than Y

The manager or owner will set targets based on these parameters so that he can monitor, each year, the safety, condition and disruption caused by the operation of the bridge stock.

These models monitor the implications of a given maintenance programme and budget and compare these with the policy parameter targets to find the degree of compliance. If the compliance is low it indicates that the budget is insufficient to achieve the targets and that it must be increased or the targets reduced. The final model estimates the budget needed in order to achieve a specific degree of compliance with the targets.

9.9.3 Basic inputs

A Inventory
B Inspection
C Test Data
D Inspection history
E Test history
F Assessment history
G Traffic data
H Duration of restriction
I Future maintenance free life (MFL) of repair
J Compendium of maintenance life/costs
K Future life required
L Maintenance history/policy
M Discount Rate
N Constraints
O Policy Parameter Targets

9.9.4 Calculations

a	Condition state of element	]	
b	Condition state for bridge	]	
c	Assessment of LCC assuming no deterioration	]	P
d	Rate of deterioration/prediction future condition	]	R
e	Predict future LCC	]	O
f	Cause/extent of deterioration	]	J
g	Traffic delays	]	E
h	Optimal maintenance method	]	C
i	Decide maintenance strategy	]	T
j	Optimal maintenance programme	]	

α	Prioritisation model	]	N
β	Implication model	]	E
δ	Comparison model	]	T
γ	Budget variation model	]	W
			O
			R
			K

9.9.5 Project outputs

1. Current condition state (elements)
2. Current condition state (bridge)
3. Original LCC and Critical areas
4. Condition state/time trajectory for each element
5. Condition state/time trajectory for bridge
6. Date for essential maintenance
7. Cause/extent of deterioration
8. Delay costs due to maintenance or restrictions
9. Optimum maintenance method
10. Best maintenance strategy
11. Optimal maintenance programme
12. Prioritised maintenance programme
13. Values of policy parameters
14. Degree of compliance with policy parameter targets
15. Budget needed to obtain say 90% compliance

Table 9.2: Project level BMS

Model	Input	Calculation	Output	Work-package
A	Inventory, Inspection, Test data	Condition state of element	Current condition state of element	1
B	*Current condition state of element*	Condition state for bridge	Current condition state for bridge	1
C	Inventory	Original LCC for bridge	Original LCC + critical areas	2
D	Inspection history Test history	Rate of deterioration Prediction of future condition	CS time trajectory for element CS time trajectory for bridge	4 4
E	Assessment history *CS time trajectory for elements/critical areas*	Predict future LCC	Date for essential maintenance	3
F	Inspection test data	Cause/extent of deterioration	Cause and extent of deterioration	5
G	Traffic data Duration restriction Load restriction	Delay model	Delay costs due to maintenance restrictions	5
H	Inventory *Delay costs* *Cause/extent of deterioration* Future MFL needed Maintenance strategy Maintenance costs / lives	Optimal maintenance method	Optimal maintenance method	5
I	Inventory Future life required *Current CS for elements* *CS trajectory for elements* *Date for essential maintenance* *Delay costs* Maintenance history/ policy *LCC and critical areas*	Decide maintenance strategy	Best maintenance strategy	5
J	Inventory Optimal maintenance method Maintenance cost/ lives Delay Costs Condition state time trajectory for elements Life required for bridge *Extent of deterioration* *Time for essential maintenance* Discount rate	WLC / optimisation	Optimal work programme	6

Italic = Project Level Output
LCC = Load carrying capacity
MFL = Maintenance Free Life
CS = Condition State

Table 9.3: Network level BMS

Model	Input	Calculation/ Assessment	Output	Work-package
α.	Optimal maintenance programme for each bridge **Constraints such as budget, political**	Prioritisation model	Prioritised maintenance programme giving dates and types of maintenance	6
β.	Prioritised maintenance programme	Implication model	Values of policy parameters each year	7
γ.	Values of policy parameters **Policy parameter targets**	Comparison model	Degree of compliance with policy parameter targets each year	7
δ.	Degree of compliance	Budget Variation model	Budget needed to satisfy say 90% of policy targets	7

For the input column of Table 9.3

Italics = Project level output
Bold Text = Network level inputs
Verdana Font = Network level outputs

NOTE When an output acts as an input for another model it is assumed that inputs for that output in the previous model are also inputs for the new model.

For example in model 4 the degree of compliance input was the output of model 3 and the inputs for model 3, the value of the policy parameters and the policy parameter targets, are therefore also inputs of model 4. Traversing backwards in this way can be used to find the primary inputs and modelal interconnections. For example the policy parameter targets are a primary input.

Table 9.4: Inputs associated with outputs (see Figure 9.8)

OUTPUT	DERIVED INPUTS	BASIC INPUTS
1		A, B, C
2	1	
3		A
4		D
5		E
6	3, 4	F
7		B, C
8	6	G, H
9	7, 8, 10	A, I, J
10	1, 3, 4, 6, 8	A, K, L
11	4, 3, 8, 9, 7, 6	A, J, K, M
(12)	11	N
(13)	(12)	
(14)	(13)	O
(15)	(14)	

All the inputs associated with a given output can easily be obtained from this Table. For example the inputs associated with output 6 are F, A and D where the A and D are derived inputs obtained from outputs 3 and 4. In Table 9.4, the derived inputs for a given output are themselves inputs.

9.10 CONCLUSIONS

This chapter has shown how results from the main bridge management activities such as inspections, assessments, testing, maintenance, prioritisation and replacement, described in Chapters 3 to 8 of this report, can be combined to produce a framework for a computerised bridge management system that will provide both project and network level information. The types of project level information generated include:

- measures of the condition of each structural element and component of a bridge and for the complete bridge
- the load carrying capacity of a bridge and its most structurally vulnerable parts
- the rate of deterioration of elements and components of a bridge enabling their future condition to be predicted
- predictions of when a bridge will become substandard in terms of the load carrying capacity
- identification of the maintenance requirements of a bridge
- guidance on effective maintenance strategies and methods
- programmes of maintenance work indicating the timing of specified maintenance methods needed in order to minimise the whole life cost of a bridge.

The types of network level information generated include:

- prioritised programmes of maintenance when the optimisation of the programme is constrained by factors such as a maintenance budget that is insufficient to enable all the work in the optimal programme to be carried out
- values of policy target parameters such as (a) the number of bridges with load restrictions at a given date, (b) the number of bridge replacements each year and (c) the average condition of bridges in the stock at a given date
- degree of compliance of measured policy target parameters with set benchmark values
- size of maintenance budget needed to achieve a specified degree of compliance.

Whilst the system was developed for the European Road network, it could also be applied to national and local road networks.

Ultimately it should be possible to combine management systems for pavements, earthworks, bridges (structures) and street furniture to achieve a route management system.

9.11 BIBLIOGRAPHY

Al-Subhi K et al (1989). *Optimising system level bridge maintenance rehabilitation and replacement decisions*. FHWA/ NC 89-002, Washington DC, 1989.

Bungey J H (1982). *Testing of concrete in structures*. Surrey University Press, Glasgow, 1982.

Darby J J, Brown P and P R Vassie (1996). *Bridge management systems : The need to retain flexibility and engineering judgement*. Bridge management 3, 212-218, E and FN Spon, London, 1996.

Das P (1996). *Bridge Management Objectives and Methodologies*. *Bridge Management 3.* 1-7, E and FN Spon, London, 1996.

Das P (1998). *Development of a comprehensive structures management methodology for the Highways Agency*. The management of Highway Structures Conference, Institution of Civil Engineers, London, 1998.

Department of Transport (1993). *Bridge Inspection Guide.* HMSO, London.

Department of Transport (1982). *Quadro 2 User Manual.* London.

Federal Highways Administration (1987). *Bridge management Systems.* FHWA DP-71-01, Washington DC.

Frangopol D M and Estes A L (1997). *Lifetime bridge maintenance strategies based on system reliability*. Structural Engineering International IABSE, 1997, 7 (3), 193-198.

ENPC (1994). *Gestion des ouvrages d'art.* Actes du colloque organisé par l'ENPC, Paris, 18-20 octobre, Presses de l'ENPC, pp 596.

Gusella V et al (1996). *Information System for the management of bridge owned by the province of Perugia*, Italy. Bridge Management 3, 592-602, E and FN Spon, London, 1996.

Harding J E et al (ed) (1990). *Bridge Management : Inspection, Assessment, Maintenance and Repair*, E and FN Spon, London, 1990.

Harding J E et al (ed) (1993). *Bridge Management 2*, London, 1993.

Harding J E et al (ed) (1996). *Bridge Management 3*, Thomas Telford, London, 1996.

Highways Agency (1994). *Inspection of Highway Structures. BA63/94. Design Manual for Roads and Bridges Volume 3*, HMSO, London.

Highways Agency (1997). *The Assessment of highway bridges and structures. BD 21/97 and BA 16/97*. Design Manual for, Roads and Bridges Volume 3 / Section 4, London, HMSO.

Hogg V and Middleton C P (1998). *Whole life performance profiles for highway structures*. The management of highway structures conference, Institution of Civil Engineers, London, 1998.

Institution of Civil Engineers (1998). *Supplementary load testing of bridges.* Thomas Telford, London. pp 59-63.

Institution of Civil Engineers (1998). *The Management of Highway Structures.* Conference Proceedings, Institution of Civil Engineers. London.

Jiang Y and Sinha K C (1989). *A dynamic optimisation model for bridge management systems*. Transportation Research Board annual meeting, 1989, Paper no. 88-0409, Washington DC.

Jiang Y et al (1989). *Bridge performance prediction model using the Markov Chain*. Transportation Research Record No. 1180, TRB, Washington, 1989.

Kreugler J et al (1986). *Cost effective bridge managemnet strategies*. FHWA/ RD-86/ 109, Washington DC, 1986.

Llanos J (1992). "*La maintenance des ponts, Approche économique.*" Presses de l'ENPC, pp176.

Ministère des Transports (1995). *Instruction Technique pour la Surveillance et l'Entretien des Ouvrages d'Art – 1ère partie : dispositions applicables à tous les ouvrages.* Direction des Routes et de la Circulation Routière, France, 19 octobre 1979 (révisée le 26 décembre 1995)

Odent N, Berthellemy J and Delfosse G (1999). *Impact d'une politique de gestion sur l'état d'un partrimoine d'ouvrages en béton armé.* Ouvrages d'Art No. 33, Service d'Etudes techniques des Routes et Autoroutes, décembre 1999.

OCDE (1992). *La gestion des ouvrages d'art.* Paris. pp132.

OECD. (1981). *Bridge Maintenance.* Paris.

Saito M and Sinha K (1990). *Timing for bridge replacement, rehabilitation and maintenance*. Transportation Research Board annual meeting, 1990, Paper no. 890336, Washington DC.

Service d'Etudes Techniques des Routes et Autoroutes (1996). *Image de la Qualité des Ouvrages d'Art (IQOA): Méthodologie, Procès-verbaux de visite, catalogues de désordres, et valise de formation.* France.

Shetty N K et al (1996). *A risk based framework for assessment and prioritisation of bridges*. Bridge management 3, 571-579, E and FN Spon, London, 1996.

Spackman M (1991). *Discount rates and rates of return in the public sector : Economic Issues*. Govt Econ service working paper No. 113, HM Treasury, 1991.

Sundararajan C R (1995). *Probilistic Structural Machanics Handbook*. Chapman and Hall, London, 1995.

US Department of Transportation, Federal Highways Administration (1988). *Recording and coding guide for the structure inventory and appraisal of the nations bridges*. FHWA-ED-89-044, Washington DC, 1988.

Vassie P R and Rubakantha S (1996). *A model for evaluating the whole life cost of concrete bridges*. Corrosion of reinforcement in concrete construction, 156-165, Society for Chemical Industry, 1996.

Winston W L (1991). *Operations Research.* PWS – Kent, Boston.

Y. Robichon C, Binet, B. Godart (1995). "*IQOA: Evaluation of bridge condition for a better maintenance policy.*" Symposium international de l'AIPC « Extended the lifespan of structures », San Francisco, 23-25 août.

CHAPTER 10

END USER, LINKS TO OTHER PROJECTS AND RECOMENDATIONS

The work undertaken in BRIME has provided a framework for a bridge management system that would enable the bridge stock on the European highway network to be managed efficiently and effectively. The implementation of this framework as a complete package would require a detailed inventory of the bridge stock to be established, and implies the development and application of reliable inspection, assessment and maintenance procedures applied consistently to the whole bridge stock. It is likely that this process would be take a number of years to complete. It would ensure the continued high performance of the network and, potentially, could save billions of Euros in construction, maintenance and traffic delay costs.

As most countries have already moved some way towards a national BMS, it is more likely that the proposed framework here would be implemented in a staged way. The modular structure of the framework means that some aspects could be implemented immediately and the BMS progressively refined as the inventory data is improved and the other aspects of the framework are phased in. It is likely that the BMS framework will be improved as the results of future research become available. In addition further improvements may be made as a result of experience gained from using the BMS.

10.1 END USERS

The end-users of the results of this project include international, national and local government highway organisations and agencies that are responsible for the safe operation of road networks. End-users also include consultants, contractors and construction companies that are involved in the day-to-day construction and management of bridges. At international and national levels, the findings from this study could influence matters of policy regarding safety and the administration and operation of highways, and highlight areas where action is required. It will also be of interest to those responsible for decision-making in the areas of transport policy, legislation, and research and development.

At a regional or local level, engineers charged with the upkeep of a section of highway infrastructure will benefit from the availability of information on methods of inspection, assessment and analysis, and from improved whole life cost models. Together these will improve the efficiency of operations, provide more reliable predictions of expenditure, and assist in the prioritising, planning and execution of inspection and maintenance works by enabling maintenance programmes to be optimised. Such information will also be of benefit to road operators and contractors concerned with maintenance works.

The findings will be of interest to all organisations responsible for the management of bridges at both network and local level, including national railway authorities, and owners of other infrastructure such as waterways, as well as highway authorities. Other organisations such as consultants employed to assess the load carrying capacity of bridges and test houses responsible for determining structural condition will also benefit from the outputs of Workpackages 1, 2 and 3.

10.2 LINKS TO OTHER PROJECTS

Bridge management is a diverse subject that covers many areas. These include:

- inspection and testing methods for determining the condition of bridge structures and their component parts
- degradation mechanisms for the different types of construction materials used in bridges
- structural analysis methods for determining the load carrying capacity of the different structural types
- effect of various forms of deterioration on bridge behaviour and strength
- materials science and the development of new materials for both new construction and the repair of existing structures and
- economic analyses to determine the most appropriate maintenance options.

In addition bridge management needs to take account of political initiatives such as sustainable development and to consider how bridges can be maintained in a sustainable manner.

This diversity means that projects undertaken in all these areas have some relevance to bridge management and, where appropriate, individual workpackages have taken advantage of work carried out in other projects. There are also numerous committees and Working Groups active in relevant areas. Some examples are given below:

- BRITE-EURAM: MILLENNIUM: *Monitoring of large civil engineering structures for improved maintenance*. The primary objective of this project was to develop and demonstrate an on-line strain measurement system with the capability of meeting the required specification for life prediction and maintenance control of large civil engineering structures surviving for the lifetime of the structure, ie for up to 100 years.

- BRITE-EURAM: SMART STRUCTURES: *Integrated monitoring systems for durability assessment of concrete structures* (http://www.gmic.dk/smart.htm). The objective of this project was to produce an integrated monitoring system so the inspection, maintenance and traffic delay costs can be reduced.

- COST 521: *Corrosion of steel in reinforced concrete structures*. The aim of this COST Action is to support the construction industry by technical and economic optimisation of the resources used to construct, monitor and maintain reinforced concrete structures.

These projects show that there is an increasing interest in determining the actual condition of bridges and this will have an enormous impact on bridge management when operative, ie when the true condition of a large proportion of the bridge stock is available.

Other projects that are relevant to BRIME are concerned with developing strategies for determining remaining service life of concrete structures, measurements of vehicle loads on structures, development of procedures for assessment of concrete structures and management of the road network. Theses include:

- BRITE-EURAM: 4062: *The residual service life of reinforced concrete structures*. The project demonstrated the viability of producing a practical User's Manual for the assessment of reinforced concrete structures.

- CONTECVET: EC Innovation Programme IN30902I: *A validated User's Manual for assessing the residual life of concrete structures*. The basic objective of this project was to scale up for industrial use, the innovative results from BRITE-EURAM 4062.

- COST 323: *Weighing-in-motion of road vehicles*. The aim was to develop and improve methods of measurement of dynamic loads applied to the road by commercial vehicles.

- DURANET: Targeted Research Action - Environmentally friendly construction technologies. Cluster 6: Improved performance of concrete in structures. The objective is to support the development and application of performance based durability design and assessment of concrete structures.

- FIB: Working party 5.3-1: *Assessment and residual service life evaluation of concrete structures*.

- RIMES: *Road information and management Euro-system*. The objective of this project was to provide comprehensive specifications to enable existing and future road information and management systems to contribute to European needs as well as serve the local requirements.

Whilst the examples given above illustrate how the results from other projects have been fed into BRIME, the results from BRIME are being used in other projects, in particular COST 345: *Assessment procedures for highway structures*. The objective of this project is to identify the procedures and documentation required to inspect and assess the condition of structures such as bridges, earth retaining walls, tunnels and culverts: it thus goes beyond BRIME to cover all types of highway structure. One of the areas that will be covered in detail in COST 345 that was not studied in depth in BRIME is the provision of information on the stock of highway structures. This is essentially the inventory data that is required as input to budgetary plans for maintenance works and operating cost models and also for establishing recommendations for maintenance options. COST 345 will also:

- define the requirements for future research work
- identify those structures not amenable to simple numerical analysis.

The results of the BRIME project will also assist the work of FIB Commission 5: *Structural service life*. The aim of Task Group 5.3: *Assessment, maintenance and rehabilitation*" is to develop a reliability based strategy, procedures and criteria for assessment, maintenance and rehabilitation of concrete structures to ensure cost optimal service life.

10.3 RECOMMENDATIONS FOR FURTHER WORK

The results from BRIME set out a framework for a bridge management system. However, further research is required before the framework can be fully implemented and detailed recommendations for future work are given at the end of the relevant chapters. Particular areas include:

- development of optimal inspection strategies
- modelling deterioration rates
- development of reliable methods for determining the load carrying capacity and remaining service life of deteriorated structures
- determining the influence of different types of cost on the results of life-cycle maintenance analyses and decision making processes
- investigating the durability and cost effectiveness of different repair techniques
- studying life-cycle maintenance options on large numbers of real and different types of structure
- parametric studies of different mathematical formulations (classical, neural networks, genetic algorithms) of life-cycle optimisation on a stock of structures.

Reliable methods of collecting appropriate data through routine inspections are required. At present, inspection methods focus on condition monitoring with a view to determining maintenance and repair options. It is often not possible to use the results directly in a strength assessment. There is a need to modify bridge inspection procedures so that the results can be used more directly in determining load carrying capacity.

Modelling deterioration rates is a key area as all structures deteriorate. The main cause of deterioration in bridge structures is corrosion of reinforcement, prestressing tendons and structural steelwork caused by chlorides. However, there are currently two fundamental weaknesses when trying to predict the time to corrosion initiation due to chloride ingress into concrete and both require further research. The first is the accuracy and limited amount of reliable input data (chloride profile, environmental load, material properties). In addition, data must be collected at different ages at the same location. The second is the accuracy of the threshold values for corrosion initiation for real structures.

More accurate modelling of deteriorated structures would enable the effects of deterioration to be quantified and the current and future load carrying capacity assessed. This would enable the optimum time for intervention to be determined. Further research work on modelling the remaining service life of structure and/or structural elements for individual and multiple deterioration mechanisms, at the project and network level, based on available data are needed.

Future developments are likely to include further application of the use of artificial intelligence methods in various aspects of bridge management and increased use of reliability techniques. Life cycle assessment is also likely to be used to minimise the environmental impacts of bridge management and to encourage a sustainable infrastructure system. Finally, the management of the highway network as a whole will mean that bridge management will become a part of a much larger asset management system that ensures that society gets maximum benefit from its investment in the highway infrastructure.

APPENDIX I

PROJECT PARTICIPANTS

Project Co-ordination Committee

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Framework for a bridge management system (Workpackage 7)

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BASt	Bundesanstalt für Straßenwesen
CEDEX	Centro de Estudios y Experimentacion de Obras Publicas
LCPC	Laboratoire Central des Ponts et Chaussées
NPRA	Norwegian Public Roads Adminstration
TRL	TRL Ltd
ZAG	Slovenian National Building and Civil Engineering Institute.

APPENDIX II

DELIVERABLES

Deliverable D1

Kaschner R, Cremona C and D W Cullington (1999). *Review of Current Procedures for Assessing Load Carrying Capacity,* BRIME PL97-2220.

Deliverable D2

Bevc L, Mahut B and K Grefstad (1999). *Review of Current Practice for Assessment of Structural Condition and Classification of Defects.* BRIME PL97-2220.

Deliverable D3

Bevc L, Peruš I, Mahut M and K Grefstad (2001). *Review of Existing Procedures for Optimisation.* BRIME PL97-2220.

Deliverable D4

Godart B and P R Vassie (1999). *Review of Existing BMS and Definition of Inputs for the Proposed BMS.* BRIME PL97-2220.

Deliverable D5

Cremona C, Kaschner R, Haardt P, Daly A F and D Cullington (1999). *Development of Models (Load and Strength)*, BRIME PL97-2220.

Deliverable D6

Cremona C, Godart B, Kaschner R, Haardt P and S Fjeldheim (1999). *Experimental Assessment Methods and Use of Reliability Techniques*, BRIME PL97-2220.

Deliverable D7

Astudillo R, Arrieta Torrealba J M, Velando Cabanas C and C Lozano Bruna (1999). *Decision on Repair/Replacement.* BRIME PL97-2220.

Deliverable D8

Blankvoll A, Fluge F, Larsen C K, Markey I, Raharinaivo A, Bevc L, Capuder M and I Peruš (2001). Bridge Management and Condition Monitoring. BRIME PL97-2220.

Deliverable D9

Bevc L, Peruš I, Capuder F, Mahut B and M Y Lau (2001). *Review Use of Neural Networks, Classify Defects and Guidelines for Condition Assessment,* BRIME PL97-2220.

Deliverable D10

Kaschner R, Cremona C and D W Cullington (2001). *Guidelines for Assessing Load Carrying Capacity,* BRIME PL97-2220.

Deliverable D11

Daly A. F (1999). *Modelling of Deterioration in Bridges*, BRIME PL97-2220.

Deliverable D12

Bevc L, Peruš I, Capuder F, Mahut B and K Grefstad (2001). *Procedures for Determining a Condition Rating and Guidelines for Prioritisation of Maintenance,* BRIME PL97-2220.

Deliverable D13

Godart B and P R Vassie (2001). *Bridge Management Systems: Extended review of Existing BMS and Outline Framework for a European System.* BRIME PL97-2220.

Deliverable D14

Woodward R J, Cullington D W, Daly A F, Vassie P R, Haardt P, Kashner R, Astudillo A, Velando C, Godart B, Cremona C, Mahut B, Raharinaivo A, Lau M Y, Markey I, Bevc L and I Peruš (2001) *Bridge Management in Europe – Final Report.* BRIME PL97-2220.