

# Bridge Management

## 3

**Inspection,  
maintenance,  
assessment  
and repair**



**Edited by J.E. Harding, G.A.R. Parke and M.J. Ryall**



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## BRIDGE MANAGEMENT 3

This volume contains the papers presented at the Third International Conference on Bridge Management, held at the University of Surrey, Guildford, UK, on 14–17 April 1996.

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Assessment and Repair

EDITED BY

J.E. Harding, G.E.R. Parke and M.J. Ryall

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# Preface

Many national governments and local authorities throughout the developed world are now spending more of their road transportation budget on maintaining and repairing their existing bridges than on building new ones. This is in response not only to a policy of lower public spending due to the world recession but also to the recognition that the road network must remain open and free-flowing to retain, if not improve, economic growth. Bridges, of course, are an integral part of the road system and the cost of collapsed, closed or damaged bridges can run into millions of pounds.

Most bridges built before the 1950s are carrying loads which far exceed those envisaged by the original designers, and the dramatic increase in volume and intensity of traffic on our roads since then is, in part, resulting in the progressive deterioration of our bridge stock. In the UK alone, joining the European Union and the opening of the Channel Tunnel link with France are beginning to result in an unprecedented flow of normal and heavy traffic, especially 40 ton trucks, on the already crowded highways.

The threat to some bridges is further compounded by the ingress of water due to a number of factors, such as detailing deficiencies, construction problems and the breakdown of waterproofing barriers. A lack of understanding of the implications of salting on our roads in the past has meant that sodium chloride solutions have, in many cases, impregnated bridges and caused damage to steel reinforcement and prestressing cables in concrete bridges, to bolts and other fixings in steel bridges, and to expansion joints and bearings. The last have also often suffered from damaging vibrations. A significant number of brick arch bridges, so popular during the industrial revolution, are suffering the effects of overload and are cracking, bulging and, in some cases, subsiding.

Bridges cannot last for ever but can be properly managed to secure their maximum useful life. Existing bridges need constant monitoring and proper rehabilitation. New bridges need to be designed and detailed to reduce maintenance to a minimum.

The First International Conference on Bridge Management in 1990 may be seen in part to have sounded the alarm regarding the state of our bridges. The second in 1993 demonstrated the serious nature of the problem, and the papers in this volume have shown the extraordinary amount of energy spent in research, development and implementation of practical measures not only to save our bridge stock but also to ensure that future bridges are designed and built on a more cost-effective basis. The Third International Conference has provided over 100 papers from 30 countries which are included in this volume. It is hoped that the book will provide a useful reference manual for all bridge engineers involved in bridge

management, to refine the bridge management process for the future and to encourage a rational attitude to the design and maintenance process.

The University of Surrey, which has hosted this series of conferences, has sustained an interest in bridge engineering for many years, and this is evidenced by the Msc course specialising in the area which has attracted quality students both from overseas and from the UK. Many of the latter have attended part time while being active in design-related consultancy. The Department of Civil Engineering has also been active in research related to bridges, notably in stress determination in prestressed concrete bridge structures, in the buckling of steel-plated bridge units, as well as in the use of fibre composite materials in new bridge types and in the strengthening of old ones.

Our sincere thanks go to our sponsors who have consistently encouraged and helped us through this series of conferences, and also to our technical committee for assisting in selection of the abstracts of the papers presented. It is, of course, the authors of the material included in this volume who play the key role in such an enterprise, and the support staff who provide the background administration to ensure the success of the event, but it is also important to thank the delegates for providing such a stimulating and challenging atmosphere for the exchange of ideas and information during the conference events.

*J.E. Harding  
G.E.R. Parke  
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*Guildford  
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# 1 BRIDGE MANAGEMENT OBJECTIVES AND METHODOLOGIES

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## **Abstract**

This paper examines the objectives of bridge management in terms of risks to bridges and concludes that an effective management system needs to consider all potential risk factors. Since the currently available computer-based systems are primarily aimed at prioritising maintenance work on the basis of faults and deterioration alone, it is considered necessary to develop the methods further to cover other risks such as those arising from inadequate original design requirements.

## **1 Introduction**

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

1. Material deterioration and development of structural faults;
2. Inadequate original specification of materials and methods, e.g. the use of ASR prone aggregate in concrete or colliery shale as structural backfill;
3. Increased traffic loading since original design;
4. Inadequate original design requirements, e.g the shear capacity rules in pre 1970 bridges, lack of adequate scour protection requirements etc.

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

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For this reason, it is not sufficient for the bridge authorities to repair or strengthen only those bridges which have shown deterioration. Other bridges which are at risk due to other factors also require attention. Indeed the current Highways Agency bridge rehabilitation programme covers, in addition to "steady state" maintenance which deals with general deterioration, bridges and other structures deemed to be at risk for a variety of reasons. Any effective management strategy therefore has to forecast the work necessary by taking account of the overall risks faced by the bridges rather than by considering deterioration alone. This is also important because a minor fault in an important part of the bridge may signify a greater risk than a more extended fault in a less critical area.

The purpose of this paper is to examine the overall objectives of bridge management in terms of such risk considerations, the tools necessary to carry out the tasks effectively, and to focus on the computerised decision making procedures known as bridge management systems (BMS's).

## 2 Objectives

Reliability, in the form of  $p_f$ , the probability of failure, or  $\beta$ , the reliability index, is an accepted means for describing the overall risk to the safety of a bridge [1]. To calculate the reliability parameters of a bridge, it is necessary to carry out a probabilistic analysis of the safety of the bridge. However, in an approximate sense, for particular types of bridges or failure conditions, the conventionally calculated load capacity in the form of the available  $\gamma_{f1}$  may be used to describe reliability.

Let us assume, for the purpose of a schematic representation, that the reliability index  $\beta$  for the whole population of the bridge stock is individually calculated and the number of bridges for each value of  $\beta$  is expressed in a distribution graph as shown in Fig. 1. The newer bridges are likely to be on the right of this distribution and some "at risk" bridges such as those with severe deterioration, or those regularly affected by flooding, HGV collision, or those with ASR, are likely to be located towards the left. Let us assume that the bridges to the left of  $\beta_{CR}$  are those that are either showing clear signs of distress, or are calculated to have a critically low factor of safety i.e. are liable to fail at any time.

It is reasonable to expect that, without any management action, the overall reliability distribution of the bridge stock will tend to move leftwards. Some bridges will of course deteriorate much more slowly than others. If the bridges with  $\beta$  less than  $\beta_{CR}$  are the only ones repaired or replaced at year 0, after a period of time, say at year  $x$ , the number of bridges to be repaired will be much greater, as shown in Fig. 1. After a number of similar periods the numbers of bridges to be strengthened could reach unmanageable proportions. The overall purpose of bridge management is, therefore, to prevent such a scenario from developing. For this reason, some form of intervention has to take place well before bridges reach the stage indicated by  $\beta < \beta_{CR}$ . In order to carry out such intervention in the form of strengthening or replacement, as large numbers will be involved, selected groups of bridges have to be assessed for their reliability. The purpose of the assessment is to identify those bridges that have deteriorated beyond a particular condition point, say  $\beta_e$ ,  $\beta_e$  being the target reliability for assessment. The assessment criteria corresponding to  $\beta_e$  will normally be less

onerous than the design criteria which will correspond to the design target reliability, say  $\beta_d$ .

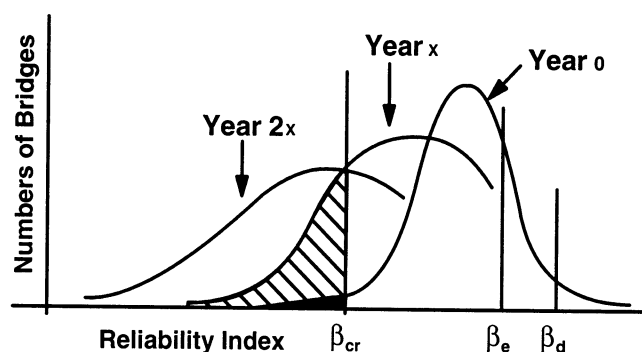


Fig. 1. Bridge safety deterioration.

### 3 Management tools

Based on the above, it seems that in order to put in place an effective management procedure a number of tools are required. These are as follows :

1. **Bridge stock reliability distribution.** It is essential to have an overall picture of the reliability of the bridge stock. It is also necessary to determine the relative rates of loss of reliability for different types of bridges. These need to be based on extensive collection of data relating to bridge condition and deterioration, and the evaluation of reliability for typical bridges, accompanied by the assessment of load capacity for the others. This exercise will have to be carried out at periodic intervals.
2. **Intervention criteria for selecting vulnerable bridges.** As described earlier, it is essential to identify the bridges which are at risk, or likely to reach a state of risk more quickly than others, so that preventative actions can be taken in good time. Criteria need to be developed to make such selections rationally by taking into account the overall needs of the whole bridge stock.
3. **Bridge specific assessment criteria and methodology.** The assessment of structural adequacy needs to be at the core of any effective comprehensive bridge management procedure. The assessment methodology needs to be rational and flexible i.e. it needs to cover bridge specific assessment loading and whole life performance related targets for assessing bridges of any given age. The required remedial work for inadequate bridges need to be determined on the basis of risk related options including non-destructive testing and monitoring. The present rules do not have any of these flexible options.
4. **Bridge Management System.** An essential function of management is to take on board the results of inspections and assessments and prioritise the necessary

remedial activities for the whole network. Similarly, once the funds are available, these need to be allocated in accordance with the strategic needs. At the scheme level also it is necessary to prioritise the work on the basis of cost-benefits. In order to carry out these functions, the bridge authority and its agents need to have a consistent decision making system which is able to interact with the output from all the other activities, such as assessment, and utilise the necessary economic and strategic information.

The Highways Agency has initiated an extensive R&D programme aimed at developing the items described above. A number of projects have already been commissioned, and others are being formulated. In this paper the proposals for item 4, a computer-based bridge management system, will be discussed in some detail.

#### **4 Bridge Management System (BMS)**

##### **4.1 Bridge maintenance activities**

The Highways agency is at present considering the options available for implementing a formal bridge management system (BMS) which could be used centrally as well as by its agents in order to make the current procedures more rational and consistent. In assessing the options, it has been necessary to examine the management activities involved, and the developments taking place in the related fields of bridge assessment and inspection methodologies, and how these advanced methods will be best utilised by a BMS.

The Highways Agency's expenditure on bridge management activities is mainly devoted to the 15 year Bridge Rehabilitation Programme. Although the programme formally started in 1988, the various components have grown out of a variety of needs perceived at different times. The programme was initiated mainly to deal with the current backlog of substandard bridges. However, some of the activities, by their very nature, will continue to be necessary beyond the current programme. The main components are steady state maintenance, assessment and strengthening and upgrading.

Certain procedures are common to all these components. For example, in addition to the bridges in the assessment and strengthening part of the programme, others such as the post-tensioned bridges and those in the support upgrading category also need to be assessed and, if necessary, strengthened. Similarly, inspections are a prerequisite for all these activities. Also, many remedial actions can have options, and the economic implications of these options need to be considered when prioritising the work.

##### **4.2 Component procedures**

The general procedures involved in carrying out the various maintenance and rehabilitation activities are as follows:

1. Bridges are selected for inspection, and the inspections are carried out.
2. Results of the inspections are examined and the following assessments are carried out:-

- a. Some routine maintenance activities, such as clearing drains, are carried out on a regular basis. Others may be initiated by an assessment of the severity of the reported faults and problems.
  - b. Needs are assessed for any hazard management measures which may become necessary, for instance when foundation scour is evidenced or when HGV strike is considered likely. Formal assessments are necessary for this, although such problems do not usually have any calculable effect on the load carrying capacity of a bridge.
  - c. Inspection results are also used to assess bridges with specific problems, such as the ASR affected bridges or the post-tensioned bridges, or the bridges in the assessment and strengthening programme. Such assessments usually provide an estimate of load carrying capacity.
3. All the three types of assessments referred to above, result in options in terms of remedial actions, and these options need to be costed as far as possible.
  4. Finally, decisions have to be taken, based on economic and other considerations, at two different levels. Centrally, annual bids and allocations have to be made by optimising all the options for the network. At scheme level, i.e. for a particular bridge, options have to be considered for determining the best course of action.
  5. To carry out all the above procedures, a background database is required which ideally contains all the necessary information such as bridge data, route data, records of past inspections etc.

#### **4.3 Currently Available Bridge Management Systems**

Formal procedures are currently in place in the Highways Agency for most of the above activities, e.g. the Bridge Inspection Manual, the Assessment Code and the associated standards, and the Assessment and Strengthening Programme implementation standards. Some others are currently being developed and most are regularly reviewed. One very important component, a central computer-based "decision" process is at present missing. This part of the management procedure is usually referred to as the Bridge Management System (BMS).

A number of BMS's are now being developed in various countries, the most advanced of these being the PONTIS system developed in the USA. The following is a brief description of the main features of this system, which is broadly typical of the rest.

PONTIS comprises of three main modules- Recommendations, Optimisation and Improvement, which have the following features:

1. Each bridge is divided into a series of elements, the system being capable of considering 160 different types of element. A typical bridge would consist of no more than 10 element types from the possible 160. For each element type the system requires a definition from 9 different condition "states".

In order to reduce the computational complexity of the problem when considering all possible bridges, PONTIS takes an entirely statistical approach. It considers the element as part of a family of elements divorced from the individual bridges when considering the network wide bridge maintenance needs.
2. This approach allows the Recommendations module to estimate the probability that a particular element type will deteriorate to a worse condition during the

evaluation period (usually two years). The cost implications of this in terms of necessary maintenance treatments are established and the costs and benefits of carrying out maintenance work now, rather than later, are determined.

3. The Optimisation module takes the element by element projections and sums them into work needed on each bridge of the network, providing costs and benefits for each maintenance project. The benefits estimated are those which result from deferral of maintenance, road user delay costs being ignored.
4. The Improvements module is designed to consider the ability of the bridge to carry traffic and provides costs and benefits associated with either strengthening or widening, if these are deemed necessary. Improvements are then considered with the maintenance schemes defined by the other modules and ranked on a cost benefit basis.

The Improvements module appears to be only partly developed as some of the functions described in the manuals do not, in fact, work.

5. PONTIS has been designed only for bridges, and other highway structures cannot be considered.

#### **4.4 Limitations**

The above procedure used by PONTIS is similar to that adopted generally for such systems. On the basis of the available information, it seems that this procedure suffers from the following limitations:

1. It covers only the routine maintenance and deterioration part of the HA's bridge management activities. Although the Improvements model aims in future to include assessment methods, it is doubtful if this can be satisfactorily achieved, given the continuing nature of the developments in this area.
2. The influence of the defects on the reliability of the bridge is ignored; hence the assessment of load carrying capacity is not involved. The bridge elements are considered as being totally independent of the bridge.
3. It is assumed that inspections will identify all serious defects. This is not always possible since some serious defects may be hidden below the road surfacing and waterproofing, or otherwise not accessible for inspection.
4. It is assumed that repairs can bring a bridge back to the as new condition. This is not true in most cases since, any major deterioration such as corrosion or cracking, will transfer the load to other sound parts of the bridge, thereby over-stressing those parts. No amount of repairs can bring the permanent stresses back to the repaired parts fully. Therefore, whatever maintenance activities are carried out, bridges in general will have a gradual reduction of reliability throughout their life.
5. It is assumed that regarding repairs, there are only two available options- to carry out or not to carry out the repairs. This is increasingly found not to be the case. Now-a-days commercial products of a range of durability are available; also the advent of techniques such as plate-bonding means that the extent of strengthening (ie the amount of the bonded plates) can also be chosen.
6. When considering maintenance options, the cost of traffic delays becomes a major factor. PONTIS does not include traffic delay costs at present.

7. PONTIS is intended to be a self contained system. Hence it incorporates the methods and information necessary for the prediction of element deterioration. Such methods are constantly being improved as a result of R & D and there is the danger that any BMS with such methods already incorporated may need frequent updating.

#### **4.5 Present Highways Agency BMS Related Developments**

If a "decision" system has to cover the whole spectrum of the bridge maintenance activities, it seems that the procedure used by the currently developed BMS's will not be adequate.

Advanced reliability based bridge assessment methodologies are now being developed by the Highways Agency and others to determine risk-related options for remedial actions, and it is likely that these will be available in a few year's time. To make full use of such tools, therefore, it is necessary that a corresponding "decision" system, incorporating a database and a financial optimisation process, is developed as soon as possible. Such a system will connect up to, rather than incorporate, the various inspection and assessment functions listed earlier. Ideally, it should have adaptable pre and post-processors so that continuing development of the other component procedures, such as improvements in assessment methods, will not affect this core system.

The Highways Agency has recently commissioned a framework study for developing such a system. In the meantime, it is also considering a trial use of PONTIS to see if the system can be modified to meet its overall needs.

## **5 Conclusions**

In conclusion it can be said that the primary aim of bridge management is to maintain the reliability of the bridge stock at an optimum level by forecasting future needs sufficiently in advance, and by deploying the best maintenance strategy possible with the available resources. Although bridge deterioration is an important factor in formulating such a policy, risk and reliability considerations should govern.

## **6 Acknowledgements**

This paper is being presented with the kind permission of the Chief Executive of the Highways Agency, Department of Transport.

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## 2 RECONSTRUCTION AND REPAIR OF STEEL HIGHWAY BRIDGES DAMAGED BY THE GREAT HANSHIN EARTHQUAKE

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### Abstract

On January 17th in 1995, the Hanshin district located in the west of Japan was hit by a strong earthquake (Great Hanshin Earthquake) with a magnitude of 7.2, and many elevated highway bridges and piers on the Kobe No. 3 Line of the Hanshin Expressway Public Corporation (HEPC) collapsed or sustained serious damage. A technical committee and three subcommittees were established for the reconstruction and repair of the collapsed and damaged structures soon after the earthquake. The activities, especially of the subcommittee for steel structures chaired by E. Watanabe, are introduced in this paper.

Keywords: Bearing, bridge pier, earthquake disaster, reconstruction, repair, seismic design, steel girder, strengthening

### 1 Introduction

The number of collapsed and damaged bridges, bridge piers and bearing systems on Kobe No. 3 Line of HEPC is summarised in Table 1. The seriousness of the damage is decided according to references [1][2], and the number of damaged piers and girders is referred to in reference [2]. The number of bridge girders indicates the number of spans, and the number of bearing systems means the number of support lines on the piers. The heaviest damage of bearings is classified as a rank of A. The collapse of bridge girders are mainly caused by the collapse or excessive deformation of piers. The number and ranks described in Table 1 may be changed in accordance with the progress of detailed investigation and examination. As far as the foundations of piers are concerned, serious damage has not been found.

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Typical damage of steel piers and steel girders is listed in Table 2. Many patterns of damage have been observed.

Table 1. Number of damaged piers and girders on Kobe No. 3 Line of HEPC.

Ranks of damage	Steel piers	Concrete piers	Steel & concrete bridge girders	Bearing systems
As(Collapse)	3	65	29	--
A (Serious)	8	84	64	220
B (Moderate)	12	107	243	195
C (Light)	112	246	215	206
D (None)	28	510	753	348
Total	163	1, 012	1, 304	969

Table 2. Typical damage to steel structures due to earthquake.

Structure	Typical damage
Steel piers	<ol style="list-style-type: none"> <li>1)Collapse due to buckling</li> <li>2)Local buckling of stiffened plates of column</li> <li>3)Overall buckling of stiffened plates of column</li> <li>4)Crack along welded portions</li> <li>5)Yield and/or excessive deformation of anchor bolts at column bases</li> <li>6)Slipping of fasteners(H.S.G.B)</li> </ol>
Steel girders	<ol style="list-style-type: none"> <li>1)Buckling of box girders due to collapse of substructures</li> <li>2)Yield and deformation of flange plates, web plates and cross beams, especially at ends of girders</li> <li>3)Damages of steel bearing systems</li> <li>4)Damages of devices for preventing bridges from fall</li> <li>5)Slipping of fasteners(H.S.G.B)</li> <li>6)Failure of expansion joints</li> </ol>

## 2 Emergency repair

The bridges and bridge piers which had collapsed were promptly dismantled. The dangerous bridges and piers which were seriously damaged were supported by temporary bents as shown in Fig. 1, to prevent them from falling down and collapsing due to aftershocks.

The dangerous piers were also repaired temporarily as an emergency measure. Typical repair methods performed prior to permanent improvement are illustrated in Figs. 2 and 3. The improvement means a strengthening and increase of ductility. The damaged portions of a concrete pier are repaired by setting steel cover plates around the

pier and filling concrete between the pier and these steel plates as shown in Fig. 2. The buckled plates of a steel pier are repaired by adding stiffeners welded on their outer surfaces as illustrated in Fig. 3. These emergency repairs were performed within about one month of the occurrence of the earthquake.

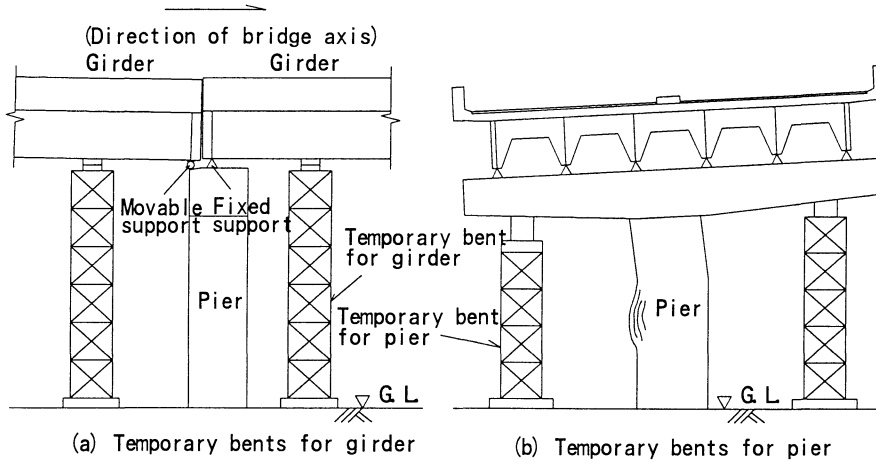


Fig. 1. Temporary supports for damaged girders and piers.

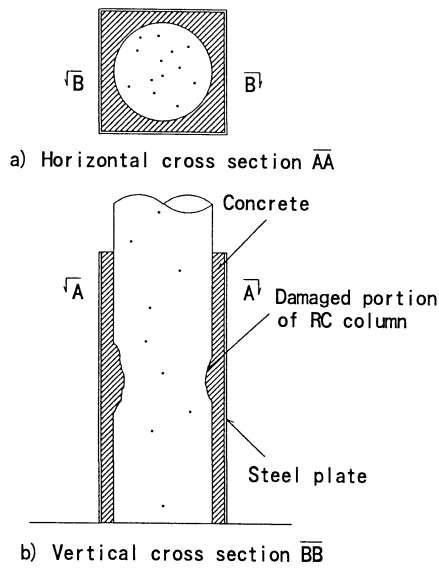


Fig. 2. Emergency repair for damaged concrete pier.

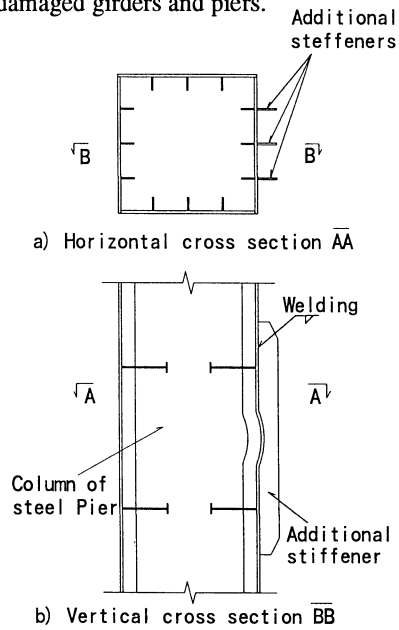


Fig. 3. Emergency repair for buckled steel pier.

The seismic coefficient adopted in the design of the emergency repair is 0.1, which is approximately equal to half of the usual design intensity of 0.2 - 0.3.

### 3 Classification of reconstruction and repair

A technical committee has been established in HEPC to investigate policies for the permanent improvement and reconstruction of the damaged structures on Kobe No. 3 line on the basis of reference [3]. It consists of three subcommittees in charge of steel structures, concrete structures and foundation-pile structures. The main activities of the subcommittees are the investigation of reconstruction and repair methods, their design criteria especially for seismic design, etc. Through the various investigations and discussion in the committee and subcommittees, the fundamental measures to be applied to the piers and girders of the Kobe Line have been determined as listed in Table 3.

Table 3. Classification and measures for collapsed and damaged piers and girders.

Ranks of damage	Measures
As(Collapse)	Reconstruction
A (Serious)	Reconstruction, or repair and improvement
B (moderate)	Repair and improvement
C (Light)	Repair and improvement
D (None)	Improvement

There is a need to check the restoring lateral strength, against strong earthquakes like the Great Hanshin Earthquake, in the seismic design of the structures such as steel and concrete piers, girders, bearings, devices for preventing bridges from falling down, etc. A dynamic response analysis considering elasto-plastic phenomena is required in seismic design. The earthquake waves to be used in this analysis are the ground motions actually recorded in the Great Hanshin Earthquake, and the measured maximum horizontal acceleration is 818 gals.

### 4 Repair and improvement methods

The permanent repair methods for buckled steel piers are shown in Fig. 4. The slightly buckled plates of steel piers are reformed by heat treatment or a pressing procedure as shown in Fig. 4 (a), and the cracks in welded members are gouged and then re-welded. The parts with serious damage are replaced with new members as shown in Fig. 4(b). The selection of these methods depends on the degree of the damage.

After repair, an improvement is carried out to increase the restoring lateral resistance against strong earthquakes. A typical method for improving an existing steel pier is shown in Fig. 5 (a). Concrete is placed in a steel column up to the required height in order to increase the buckling strength and ductility of the pier. The alternative method

now under examination is shown in Fig. 5 (b), in which longitudinal stiffeners or corner angles are added to the stiffened plates of a steel pier. The new anchor bolts should be added at the base of the pier, if the strength of existing anchor bolts is insufficient to resist strong earthquakes.

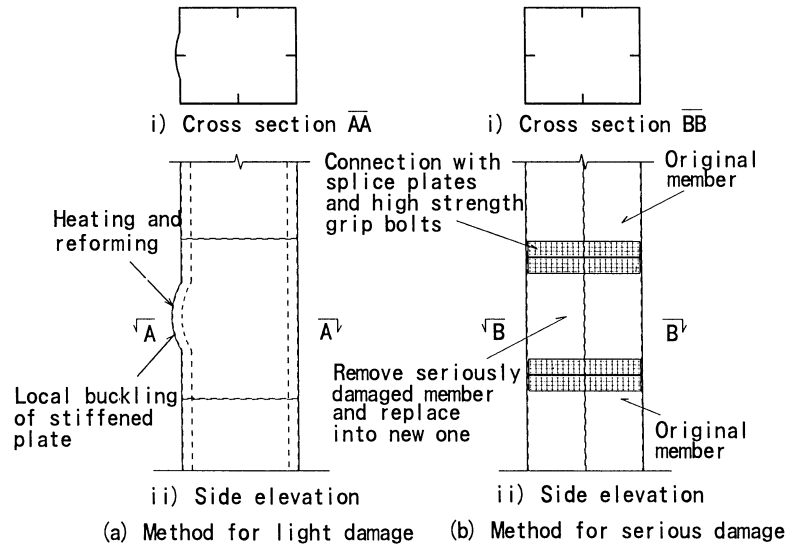


Fig. 4. Repair methods for existing steel piers.

The damaged portions of steel girders are also repaired by methods similar to those adopted for steel piers.

## 5 Reconstruction methods

The following fundamental policies are adopted for the reconstruction of collapsed steel bridges.

1. Stiffened deck plates, not reinforced concrete slabs, are adopted, if possible.
2. Rubber base-isolation systems, not steel bearings, are adopted, if possible.
3. Continuous girders, not simply supported girders, are adopted, if possible.

The purpose of the first policy is to reduce the seismic inertia force due to the weight of the superstructure, and the intention of the second is to avoid and reduce the seismic force. The third is adopted to prevent the bridge from falling down, even if the bearing systems accidentally collapse.

A hybrid pier consisting of a reinforced concrete column and a steel box beam, as shown in Fig. 6, is used in the reconstruction of collapsed concrete piers for the purpose of shortening the construction period and decreasing the seismic force through use of a light steel cross beam. The detail of the connection of the reinforced concrete column and steel beam is as described below:

1. The vertical reinforcing bars for the reinforced concrete column reconstructed by using the existing footing are inserted into the steel beam.

2. The appropriate shear connectors are arranged on the inner surfaces of the flange and web plates of the steel beam.
3. Concrete with plasticiser and expansive additive is placed into the connection.  
The strength and ductility of the hybrid pier has been assured through experiment.

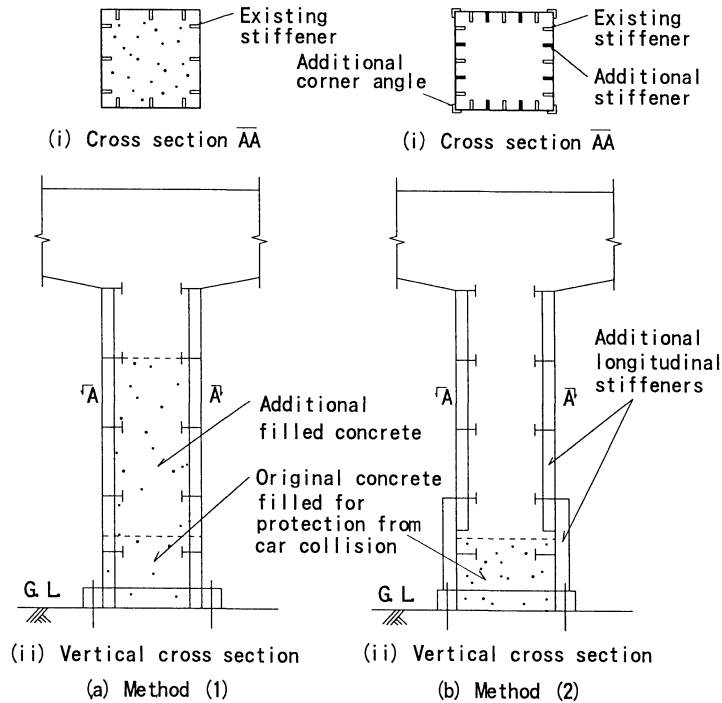


Fig. 5. Typical alternative methods for improvement of existing pier.

## 6 Other methods for improvement

End cross beams with knee-type bracing are replaced with more rigid cross beams with full webs to ensure sufficient lateral strength of the superstructure against the unexpectedly large seismic force, though knee-type cross beams have been adopted as a standard structure in HEPC (see Fig. 7). This alteration is adopted not only in the bridges to be reconstructed but also other existing bridges.

A semi-continuous connection system is adopted in existing simply supported bridges. In this connection system, only the web plates of adjacent bridges are connected with splice plates as shown in Fig. 8. The purpose of this connection system is to prevent the bridges from collapsing, and to decrease vibration and noise due to vehicles.

The devices connecting adjacent bridge girders to prevent them from falling down are replaced and designed in such a way that they can resist about 1.7 - 2.0 times larger seismic force than those used in current design.

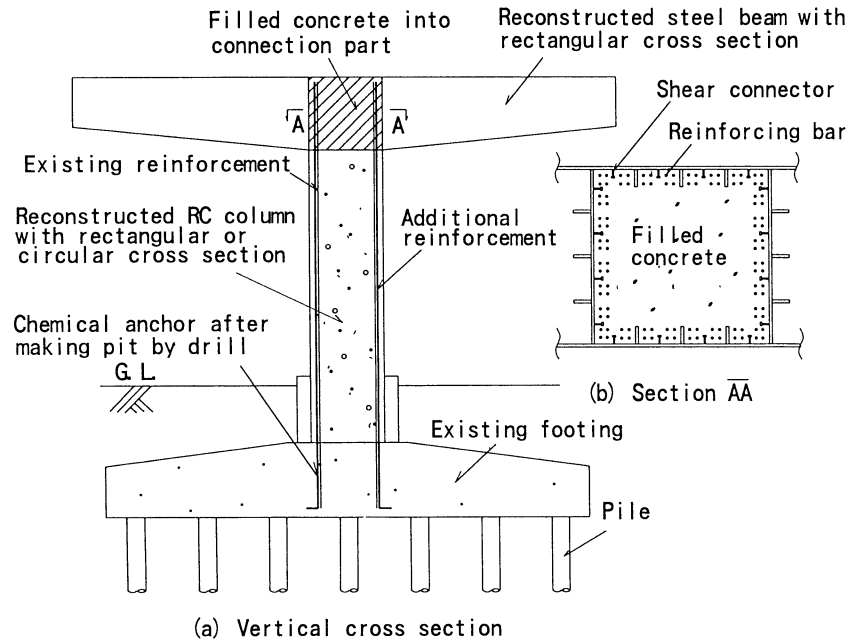


Fig. 6. Hybrid pier with RC column and steel cross beam.

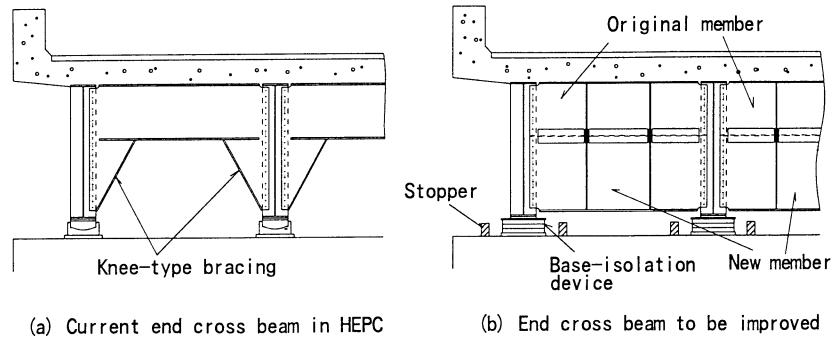


Fig. 7. Improvement of end cross beam.

Stoppers, for preventing bridges from falling, are newly installed on the pier in the case of special bridges such as curved bridges with a small radius of curvature, skew bridges, cantilever bridges, bridges with high bearing-height, etc. (see Figs. 7 and 8).

High strength friction grip bolts which slipped in the earthquake are replaced with new ones, if serious damage is found in sample inspections.

Existing steel bearings are strengthened so that they can resist about 1.5 - 2.0 times larger seismic force than those used in current design, or replaced with rubber base-isolators.

## 7 Documents for reconstruction and repair

The following draft documents have been prepared through the activities of the subcommittee for steel structures, for guiding seismic design and site work for reconstruction, repair and improvement of steel structures on the Kobe Line of HEPC.

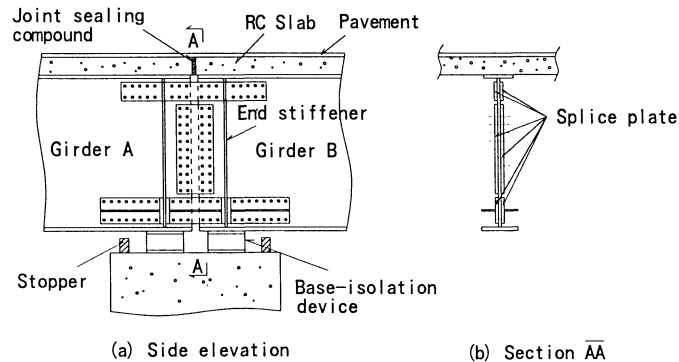


Fig. 8. Semi-continuous connection of simply supported girders.

1. Guideline for reconstruction and repair for damaged structures.
2. Repair and strengthening manual for damaged steel piers.
3. Repair and strengthening manual for damaged steel girder bridges.
4. Design manual for replacing reinforced concrete slabs with stiffened steel decks.
5. Design and construction manual for hybrid piers consisting of reinforced concrete columns and steel box cross beams.
6. Design and construction manual for temporary supports for reconstruction and repair work.
7. Design and construction manual for new bearing systems.
8. Design manual for devices for preventing steel bridges from falling down.

Most of the activities of the subcommittee for steel structures were performed rapidly and the permanent repair and improvement work started about three months after the earthquake. The site works will be finished around the end of 1996.

## 8 Conclusion

In this paper, the activities of the subcommittee for steel structures established in HEPC have been introduced. The reconstruction, repair and improvement methods using steel structures in relation to the collapsed and damaged steel and concrete structures have been briefly summarised.

## 9 Acknowledgement

Special thanks are due to the members of the subcommittee for steel structures for their valuable and devoted contribution to the activities.

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### 3 AN EXPERT SYSTEM USING CASE-BASED REASONING FOR SELECTING RETROFITTING METHODS OF FATIGUE DAMAGE ON STEEL BRIDGES

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#### **Abstract**

The maintenance and administration of existing civil engineering structures are becoming important subjects, and engineers with experience are required for them. Therefore, expert systems may be effectively used in this field.

A diagnostic knowledge-based expert system has been developed by the authors for the maintenance and administration of steel bridge. Repair and retrofit methods have been selected for fatigue damage of steel bridges. The systems involve a knowledge-base and an inference engine. Firstly, the knowledge included in the knowledge-base was obtained from 165 cases of fatigue damage. The production rules, causal relations, in the knowledge-base were assumed to have a certainty factor. Secondly, the inference engine has the function of machine learning. But, the expert systems using causal relations have a difficulty of acquiring the knowledge-base. A case-based reasoning mechanism is able to solve the problem of knowledge acquisition.

In the present paper, an expert system is built using case-based reasoning to select the repair and retrofit methods of fatigue damage. This system has a case-based database of 205 past cases, and 4 kinds of inference processes (retrieval, modification, restoration, and storing). The inference results obtained from this system which apply to actual cases are discussed.

Keywords: Expert system, fatigue damage, steel bridges

#### **1 Introduction**

Recently, maintenance and administration of existing civil engineering structures are becoming important subjects, and the maintenance and administration operations for such structures are complex and wide ranging. In the case of steel bridges maintenance and administration are becoming more and more important. At present, the selection of

repair and retrofit methods for the fatigue damage of steel bridges is dependent on the highly developed expertise of experts and empirical knowledge from many years. In fields like this, knowledge-based expert systems can be effective tools. Accordingly, the authors have been building a knowledge-based expert system to select repair and retrofit methods for fatigue damage of steel road bridges [1][2][3][4]. This knowledge-based expert system uses, as a method of expressing knowledge, a causal relation which is a kind of production rule.

The knowledge base expressed as a causal relation is built from past damage cases, and results from knowledge of external and internal causes of cracks, joint actions, cracking modes, and repair and retrofit methods. It is desirable, however, for a reasonable selection of repair and retrofit methods to add to the number of attributes to be considered. On the other hand, it is difficult to make a generalised rule, using knowledge which consists of many attributes. Thus, acquiring knowledge becomes a bottleneck.

As a method of solving the problem of knowledge acquisition, case-based reasoning [5][6][7] became of interest and has been studied to. Case-based reasoning is a technique to retrieve a case database from past cases similar to the given requirement, modifying and storing the similar cases, and solving the problems. Unlike traditional knowledge-based expert systems using rule-based reasoning, case-based reasoning can utilise damage cases from the past as they are. Therefore, case-based reasoning enables the acquisition of knowledge easily, just by collecting and arranging damage cases and evaluating wide-ranging problems based on similar cases.

In this study, the system, reported in the literature [1], for selecting repair and retrofit methods for fatigue damage of steel road bridges was rebuilt by using the technique of case-based reasoning so that it could cope with the bottleneck problem in acquiring knowledge, and the system was applied to damage cases from the past to evaluate the usefulness of the system.

## 2 Case database

Data from 205 cases were acquired in accordance with the attributes shown in Fig. 1, from cases of fatigue damage to steel road bridges in the past [8][9]. The cases studied are of fatigue cracks in the super-structures of steel road bridges.

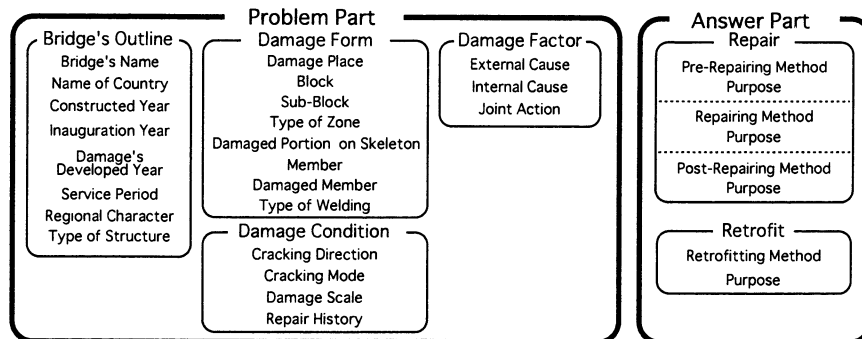


Fig. 1 . Attributes of case-based database.

Each of the case data consists of a problem part and a solution part. Attributes considered in the problem part are (i) those outline details of bridges such as year of construction and types/structures, (ii) those for damaged areas such as damaged spots and damaged members, (iii) those for damage condition such as crack directions and scales of damage, and (iv) those for causes of damage including external and internal causes of cracks. In the solution part, data such as repair and retrofit methods were classified into attributes for repair and those for retrofit to subdivide such data.

### 3 Case-based reasoning

#### 3.1 Concept

In case-based reasoning, a case database is first built up by accumulating collected cases systematically on the basis of their characteristics. Then, this database is retrieved for cases (including failure cases) in the past similar to a given subject, and the problems are solved by modifying the solutions. Case-based reasoning stands between "rule-based reasoning" where reasoning is conducted based on only rules and "model-based reasoning" where, a large-scale case database being used, reasoning is conducted based on only matching. Case-based reasoning is constituted as shown in Fig. 2, and functions of its constituents are as follows:

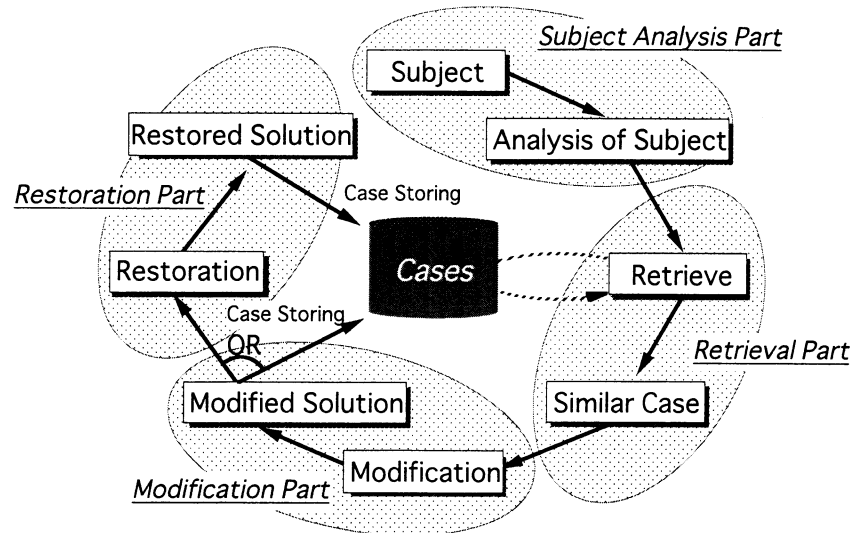


Fig. 2. Case-based reasoning.

1. *Subject Analysis Part*: Index information to retrieve for similar cases in the past is extracted from the subject.
2. *Retrieval Part*: The index information obtained in the Subject Analysis Part being used, the case database is retrieved for similar cases in the past. If a case extracted by search coincides with the subject case, the search result can, as it is, be used as a solution of the subject. Abstraction matching, partial matching, causal matching, and so forth are available as matching methods.

3. *Modification Part*: In a case where the case extracted by retrieval is similar to the subject, the difference between the subject and the similar case being taken into account, the solution is modified. There are two modification methods: one of directly modifying the solution of a similar case, and the other of modifying the process in which the solution of the similar case was deduced.
4. *Restoration Part*: In a case where a solution is modified in the Modification Part, there is a possibility that the modified solution is contradictory or wrong. In this Restoration Part, therefore, such modified solutions have to be verified and causes, if any, removed.
5. *Case Storing*: As solutions obtained in the Modification Part and the Restoration Part can be regarded as new cases, they are stored into the case database. Even solutions of failure are stored into the database since they are useful.

## 4 Construction of system

### 4.1 System configuration

The configuration of this system is as follows. The case-based reasoning part, which is the core of this system, consists of an inference engine and a case database. The man-machine interface could be made user-friendly, GUI being adopted. The inference engine has a case retrieval function and a case storing function. The case database partially materialised the function of the object-oriented database.

### 4.2 Processing procedure

Fig. 3 shows the processing procedure of this system. When this system has been booted up, case data are loaded into the interior data structure of the system. Then, when the problem part of a subject is input, the case database is retrieved, the abstraction and partial matching methods being used. Here, this system is built to perform two types of retrieval: one based on all the attributes and the other based on only the user-input, important attributes. Each type of retrieval produces more than one result. The user modifies the problem part of the subject with reference to the results obtained, and retrieval is conducted again. By repeating this process, the solution can be improved toward the optimum level.

Lastly, when no further modification of the problem part is needed, the user chooses the optimum solution from the results displayed and, as restoration, makes an evaluation of the propriety of the solution. If the solution is judged appropriate, the solution is stored into the case database as a successful case, and if inappropriate, as a failure case.

### 4.3 Retrieval

The retrieval part was arranged so that two types of retrieval could be conducted: one based on all attributes and the other based on only the attributes which a user judged important and input in a dialogue form. Each type of retrieval brings about plural solutions. The search procedure is as follows:

4.3.1 Abstraction matching of attribute values (Fig. 4 (1))

Firstly, abstraction matching is performed. In this matching, even when there is a difference between the value of a certain attribute of a subject's problem part and that of a case's, they are regarded as in agreement as long as the difference falls within a tolerance.

4.3.2 Agreement degrees of attributes (Fig. 4 (2))

As a single attribute allows description of plural attribute values, the values of a certain attribute of a subject's problem part are not necessarily all in agreement with those of a case's. In this case, if both the subject's problem part and the case's problem part have at least one attribute value, the number of values in agreement is divided by the number of values of the subject's problem part or that of the case's, whichever is larger. The result of the division is called the "degree of agreement" of attribute values.

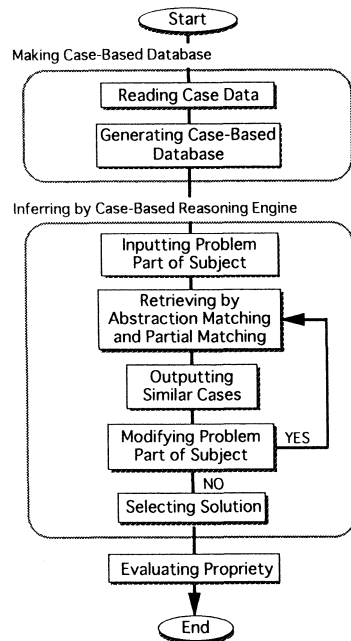


Fig. 3. Flowchart of system.

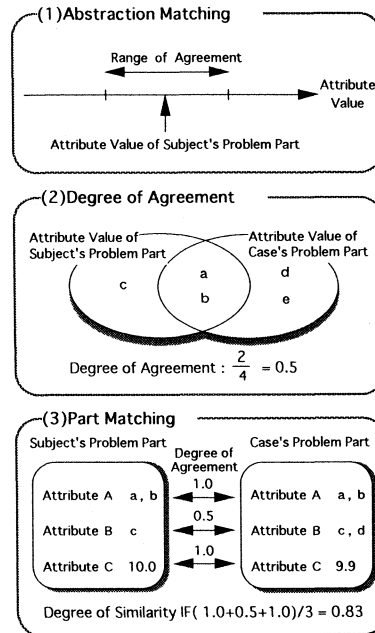


Fig. 4. Retrieval method.

4.3.3 Partial matching of cases (Fig. 4 (3))

In this matching method, the larger the number of attributes in agreement or the larger the number of important attributes in agreement, the higher the similarity is considered to be. In this study, in case of retrieval based on all attributes, similarity is evaluated by calculating the average of agreement degrees of all attributes.

4.4 Modification

Automatic modification of retrieval results (similar cases) was not considered in building this system. Consideration was given, however, so that the user could derive an optimum solution by modifying the problem part of a subject with reference to the

retrieval results and repeating the retrieval. This system provides users with necessary information, which helps them modify the problem parts of subjects efficiently.

#### **4.5 Restoration**

Users are supposed to evaluate the propriety of solutions obtained. If any solution proves to be a failure through its evaluation, the user may directly restore the solution or store the solution into the case database as a failure case, which, from the next retrieval onward, enables users to avoid the same failure.

### **5 Execution of system**

For damage cases of actual bridges being used, the propriety of this system is to be verified. When this system has been executed, the retrieval window appears from the main window. Attribute values of Table 1 are input into the retrieval window. In the case of attributes which the user judges important, the button is clicked, which produces a possible search on the basis of only such important attributes. Here, attributes for damage causes, i.e., external and internal causes and working force, were specified as important attributes. After inputting a subject, the execution button in the retrieval window is clicked and the retrieval starts. After the retrieval, the retrieval result are displayed on the basis of all the attributes shown and on the basis of the important attributes shown. The result of execution of this system is shown in Table 1.

#### **5.1 Mukoujima Bridge**

In case of the Mukoujima Bridge (in Japan), this system was run for fatigue cracks which had occurred in gusset plates at connection areas (hanger fitting areas) between hangers and main girder flanges. Mukoujima Bridge is a Langer bridge with an effective span of 118 m and a rise of 18 m. Within about three years of its completion, general vibration of the bridge and local vibration of hangers occurred, and fatigue cracks were found around rivet holes of gusset plates at upper and lower ends of hangers and in welds between gusset plates and main girder flanges in the central area of the bridge. This bridge was retrofitted by tying each hanger to the next with two wires for the purpose of improving the vibration behaviour.

Accordingly, as the retrieval conditions, "vibration due to wind", "stress concentration" and "joint action 2" were input into the items of the external cause, the internal cause, and the working force, respectively. Table 1 shows the reasoning results. Cases for which the "degree of similarity" is indicated are the results of reasoning on the basis of all the attributes, and cases for which the "degree of similarity" is indicated are the results of reasoning on the basis of only the damage causes. Shown in the shadowed sections are attribute values of perfect agreement or those by the abstraction matching.

Table 1. Inference results for Mukoujima Bridge.

Attribute	Bridge's Outline								
	Case	Bridge's Name	Name of Country	Constructed Year	Inauguration Year	Damage's Developed Year	Service Period	Regional Character	Type of Structure
	1	Unknown	Japan	1963	Unknown	1983	16~20	Unknown	Langer Bridge
	2	Unknown	Japan	1980	Unknown	1981	0~5	Unknown	Langer Bridge
	3	Umasita Bridge	Japan	Unknown	Unknown	Unknown	Unknown	Water's Edge	Langer Bridge
	4	Umasita Bridge	Japan	Unknown	Unknown	Unknown	Unknown	Water's Edge	Langer Bridge
	5	Umasita Bridge	Japan	Unknown	Unknown	Unknown	Unknown	Water's Edge	Langer Bridge
	9	Unknown	U.S.A.	Unknown	Unknown	Unknown	Unknown	Unknown	Unknown
Subject		Mukoujima Bridge	Japan	1968		1971	0~5		Langer Bridge

Attribute	Damage's Form								
	Case	Damage's Place	Block	Sub-Block	Type of Zone	Damaged Portion on Skeleton	Member	Damaged Member	Type of Welding
	1	Bridge's Middle Area	Others	Hanger Connection Plate	Weld Zone	Stiffening Girder, Additional Plate of Stiffening Girder	Upper Flange of Stiffening Girder, Gusset Plate	Upper Flange of Stiffening Girder	Fillet Welding
	2	Bridge's Middle Area	Main Girder	Hanger Termination	Weld Zone	Hanger, Hanger	Web Plate of Hanger, Flange of Hanger	Web Plate of Hanger	Fillet Welding
	3	Bridge's Middle Area	Others	Hanger Connection Plate	Weld Zone	Stiffening Girder, Additional Plate of Stiffening Girder	Upper Flange of Stiffening Girder, Gusset Plate	Upper Flange of Stiffening Girder	Fillet Welding
	4	Bridge's Middle Area	Others	Hanger Connection Plate	Weld Zone	Stiffening Girder, Additional Plate of Stiffening Girder	Upper Flange of Stiffening Girder, Gusset Plate	Upper Flange of Stiffening Girder	Fillet Welding
	5	On Terminal Supporting Spot	Others	Hanger Connection Plate	Weld Zone	Stiffening Girder, Additional Plate of Stiffening Girder	Upper Flange of Stiffening Girder, Gusset Plate	Upper Flange of Stiffening Girder	Fillet Welding
	9	Unknown	Main Girder	Cross Zone of Cross Beam and Main Girder	Weld Zone	Cross Beam, Main Girder	Upper Flange of Cross Beam, Web Plate of Main Girder	Web Plate of Main Girder	Fillet Welding
Subject		Bridge's Middle Area	Main Girder	Hanger Connection Plate	Weld Zone	Main Girder, Additional Plate of Main Girder	Upper Flange of Main Girder, Gusset Plate	Gusset Plate	Fillet Welding

Attribute	Damaged Condition				Damage Factor			
	Case	Cracking Direction	Cracking Mode	Damage Scale	Repair History	External Cause	Internal Cause	Joint Action
	1	Vertical to Welding	d	Middle	Nothing	Vibration due to Wind	Stress Concentration	Joint Action 2
	2	Parallel to Welding	c	Small	Nothing	Vibration due to Wind	Stress Concentration	Joint Action 2
	3	Parallel to Welding	d	Middle	Nothing	Vibration due to Wind	Stress Concentration	Joint Action 2
	4	Parallel to Welding	d	Middle	Nothing	Vibration due to Wind	Stress Concentration	Joint Action 2
	5	Parallel to Welding	d	Middle	Nothing	Vibration due to Wind	Stress Concentration	Joint Action 2
	9	Parallel to Welding	d	Small	Nothing	Welding Defect	Stress Concentration	Joint Action 2
Subject		Vertical to Welding	f		Nothing	Vibration due to Wind	Stress Concentration	Joint Action 2

Attribute	Repair						Retrofit		Degree of Similarity $\downarrow$	Degree of Similarity $\uparrow$	
	Case	Pre-Repairing Method	Purpose	Repairing Method	Purpose	Post-Repairing Method	Purpose	Retrofitting Method			Purpose
	1									0.68	1.00
	2	Gouging	Removal of Welding Bead	Stop Hole	Arrest of Cracks	Grinding	Smoothing Surface	Tied by Cable	Improvement of Vibration Behavior	0.63	1.00
	3							Splice Plate with H.T.Bolt	Improvement of Strength	0.62	1.00
	4							Tied by Log	Improvement of Vibration Behavior	0.62	1.00
	5							Splice Plate with H.T.Bolt	Improvement of Strength	-	1.00
	9			Stop Hole, Rewelding	Arrest of Cracks, Restoration of Original Condition	Grinding, Peening	Smoothing Surface, Reducing Out-of-plane Displacement			-	0.67
Subject								Tied by Cable	Improvement of Vibration Behavior		

As a result of reasoning based on all the attributes (degree of similarity), 4 cases (Nos. 1-4) were obtained, and as a result of reasoning based on only the attributes for damage causes (degree of similarity), 9 cases (Nos. 1-9) were obtained. In the case when all the attributes were taken into account, a counter measure, "tied by cable," was obtained from the reasoning to improve the vibration behaviour. In addition, another countermeasure for the same purpose, "tied by logs," was also obtained. In fact, Mukoujima Bridge underwent a retrofit by the method of "tied by cable" to improve its vibration behaviour. This proved that the reasoning results were appropriate.

## 6 Conclusion

In this study, a knowledge-based expert system for selecting repair and retrofit methods for fatigue cracks occurring in superstructures of steel road bridges has been built through the method of case-based reasoning. In the case of traditional systems based on rule-based reasoning, there is a bottleneck on knowledge acquisition due to the necessity of making rules of experts' knowledge. On the other hand, case-based reasoning has the merit that knowledge can easily be acquired just by collecting and arranging cases. Thus, this system was run based on repair and retrofit cases of actual bridges. The results proved the reasoning results of this system to be appropriate. On the other hand, systems using rule-based reasoning can not handle subjects which require knowledge not found in their rules, or in case of traditional database systems, no solution can be obtained unless retrieval conditions are met perfectly, whereas retrieval results from wide ranges can be obtained from case-based reasoning systems because they can retrieve for similar cases. Besides, search time was reduced as compared with Mikami, Tanaka, *et al* systems using causal network models [1][2][3][4].

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## 4 PREDICTION OF BRIDGE SERVICE LIFE USING TIME-DEPENDENT RELIABILITY ANALYSIS

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### Abstract

A bridge deterioration model is an essential component of a computerised bridge management system (BMS). Existing BMSs use Markov chain theory to model the deterioration process as a decay of condition ratings over time. An alternative approach, based on time-dependent reliability theory, is proposed. The new approach is in principle a generalisation of the Markov chain models. Rather than addressing the stochastic nature of condition rating the proposed approach seeks to model the random time using survival analysis.

Keywords: Bridge management system (BMS), Markov chain theory, time-dependent reliability, survival analysis, bridge deterioration models, bridge life prediction.

### 1 Introduction

In developing a computerised bridge management system (BMS) there is need to model the bridge deterioration process. The idea is that if we knew the rate and/or pattern of bridge deterioration, we would be able to predict the remaining lives as well as future performance of the bridges in the network. Both of these are important inputs to the decision models of a bridge management system.

The bridge deterioration process has often been modelled as the decay of bridge performance over time. Earlier models tend to be simple with linear and deterministic deterioration curves. More recent models used in the Indiana State's BMS [1] and FHWA-sponsored BMS software *Pontis* [2], are based on Markov chain theory. The deterioration process, recognized as a stochastic process, is represented by the transition probability matrix:

$$\mathbf{P} = \{p_{ij}\} \quad (1)$$

where  $p_{ij}$  is the probability of the bridge transiting from state  $i$  to state  $j$ , in one step. A stochastic process  $\{Z_n: n = 0, 1, \dots\}$  is defined as an indexed set of random variables,  $Z_n$ . It depicts the value of the system state  $Z_n$  at each time point,  $n$ . In the Markov-chain bridge deterioration model, the system states are indicated by some ratings of bridge condition.

Existing bridge deterioration models based on the Markov chain theory invariably assumes that a bridge can either remain in the current state or deteriorate to the next state in one transition. Also, the worst state  $M$  in a state space of  $\{1, 2, \dots, M\}$  is considered an absorbing state; which means that once the process enters the state it will never leave it. The stochastic nature of the deterioration process is thus described by the transition matrix of this format:

$$\mathbf{P} = \begin{bmatrix} p_1 & 1-p_1 & \cdots & 0 & 0 \\ 0 & p_2 & \cdots & 0 & 0 \\ \vdots & \vdots & \ddots & \vdots & \vdots \\ 0 & 0 & \cdots & p_{M-1} & 1-p_{M-1} \\ 0 & 0 & \cdots & 0 & 1 \end{bmatrix} \quad (2)$$

where  $p_i, i=1, 2, \dots, M-1$ , represents the probability of remaining in the  $i$ th state in the next transition. Notice that  $p_M$  is equated to 1 since  $M$  is an absorbing state.

The expected condition of the bridge at a future time  $n$  or conversely, the expected time to reach any specific future state, can be calculated by using the following relationship:

$$\pi^{(n)} = \pi^{(0)} \times \mathbf{P}^n \quad (3)$$

where  $\pi^{(n)}$  is the state probability vector at any time  $n$  and  $\pi^{(0)}$  is the initial state probability vector. The stochastic process is thus fully specified once the transition matrix  $\mathbf{P}$  as well as the initial states are known.

There is wide acceptance in the use of Markov chain theory in deterioration modelling but three issues need further investigation, namely:

1. Suitability of the Markov chain theory in bridge deterioration modelling.
2. Suitability of condition rating as the bridge performance measure.
3. Methods of estimating the transition probabilities,  $P_{ij}$ .

The authors briefly discuss these issues in Section 2 to provide the background for introducing an alternative approach to model bridge deterioration process. It is later shown that the proposed method, based on time-dependent reliability theory, amounts to the generalisation of the Markov chain model.

## 2 The state of the art

### 2.1 Markov chain theory

Markov chain theory is founded on two fundamental rules, namely: *memoryless* and *homogeneous*. The memoryless rule stipulates that the future states of the process depend only on the current states; while the homogeneous rule requires that the rates of transition from one state to another, remain constant throughout the time. Translated to the Markov-chain bridge deterioration model this is saying that the transition probabilities depend only on the current states and not on the ages of the bridges. Thus, a 5-year old bridge and an 80-year old one, should they be of the same ratings, are equally likely to stay in their current state in the next transition. *Pontis* [2] assumes this to be valid and uses only one transition matrix for the whole life span.

Indiana BMS develops separate transition matrices for each age group [1]. In this way, the stochastic nature of the deterioration process depends on both the current states as well as the ages of the bridges. One disadvantage of this effort however, is that by zoning there is a much smaller sample size within each group, which reduces the precision of the estimators.

### 2.2 Performance measure / state

Almost all existing bridge deterioration models use condition rating as the measure of bridge performance. Indiana uses the rating system introduced by the U. S. Federal Highway Administration (FHWA) [4]. Numeric ratings of 0 to 9 are used to indicate the physical conditions of bridge decks, superstructures and substructures, with 9 representing the "excellent condition" and 0 the "failed condition". *Pontis* [3] uses a different system of rating between 1 and 5. 1 is the 'best' and 5 is the 'worst' state.

It has been observed that condition rating is not adequate as a performance measure [2]. Condition rating does not reflect the structural integrity of a bridge, nor the improvement needs. Indeed, many major bridge management decisions, for example, posting have been based on the load rating. Besides, because condition rating is in the ordinal scale, we cannot compare two ratings by their difference or ratio. There is actually suggestion that bridge deterioration process modelling should include load rating [5].

### 2.3 Estimation of transition probabilities

Two methods of estimating the transition probabilities have been considered and discussed by Jiang *et al* [1] for the Indiana BMS. We will call them the 'Frequency' approach and the 'Regression' approach. In the Frequency approach, the transition probability  $p_{ij}$  is estimated by:

$$\hat{p}_{ij} = \frac{n_{ij}}{n_i} \quad i, j = 1, 2, \dots, M \quad (4)$$

where  $n_{ij}$  is the number of bridges originally in state  $i$  which have moved to state  $j$  in one step, and  $n_i$  is the total number of bridges in state  $i$  before the transition.  $p_{ij}$  has been shown to be a maximum likelihood estimator (MLE) [6]. From Eq.(4) it is clear

that this approach would require at least two sets of inspection data pertaining to two different points in time.

In the Regression approach, only one set of bridge data is needed. A regression function is first obtained by regressing condition ratings on ages. Transition probabilities are then estimated by "fitting" the regression function with the transition matrix. This involves seeking an optimal solution to minimize the difference between the expected condition rating, from the regression function, and that derived from the transition matrix.

The approach adopted by Pontis [2] for estimating the transition matrix, is in reality the 'Frequency' approach. However, instead of relying on bridge data, which is deemed to be scarce at the beginning of system implementation, the proportions of bridges to transit from one state to another are to be elicited from the bridge experts. As more data becomes available, after subsequent inspections in the following years, the probabilities are to be updated using the Bayesian method.

### 3 Reliability-based performance measure

#### 3.1 Time-dependent reliability

In previous sections we have seen the treatment of condition ratings as the response variable, subject to the influence of ages and other explanatory variables. Since the ratings are random variables, it makes sense to talk about the probability of these ratings reaching or exceeding a certain threshold value within the time interval  $[0, t]$ . We can indeed establish the relationship of this probability, or rather, its complement we called reliability, with time. This is the basis of time-dependent reliability theory.

We define the reliability function  $S(t)$  as the probability of survival of a system within the time  $[0, t]$ . In other words, it is the probability that the time to failure exceeds the time,  $t$ :

$$S(t) = P [T > t] \quad t \geq 0 \quad (5)$$

$T$  is a non-negative random variable representing the time to failure and is commonly known as the failure time or lifetime. 'Failure', in this context, refers to the event that the state hits a well-defined threshold value for the first time.

The reliability function explicitly expresses the reliability of a *new* bridge at any point in time. For an in-service bridge we would use an equivalent function known as *hazard function*,  $h(t)$ . The hazard function specifies the instantaneous rate of failure at time  $t$ , given that the individual survives up till time  $t$ . It can be proven that:

$$h(t) = f(t) / S(t) \quad (6)$$

where  $f(t)$  is the probability density function of  $T$ . Given the distribution of  $T$  in any of these forms, information about the remaining life and future bridge performance can be determined. As an example, the mean residual life can be calculated using:

$$m(t) = E [T - t | T > t] \quad (7)$$

The question now is how are we to estimate the lifetime distribution.

### 3.2 Estimation of the bridge reliability function

Estimation of the reliability function from lifetime data is the subject matter of *survival analysis* commonly used in industry for reliability testing of machines, and in biomedical fields for prediction of life expectancy. It is indeed a regression analysis of lifetime  $T$ , rather than the condition rating, on the explanatory variables, called 'covariates' in survival analysis. Also, it fits a distribution function rather than the expected value to the field data. What is unique in this statistical technique is the presence of censored observations. Censored observations are not complete. If we know that a bridge reaches a certain well defined threshold value at an age  $y$ , we have a complete observation;  $y$  is the lifetime value. However, if we found at the time of bridge inspection, that a bridge had not reached the limiting value, then we have a right-censored observation. This observation though incomplete is still useful for it tells us that the lifetime of the bridge goes beyond its present age. If instead, we found that a bridge had already surpassed the limiting value at the time of inspection, we then have a left-censored observation. We know that the lifetime of the bridge is less than or equal to the present age.

Based on this concept and using the 1991 NBI data from the state of Indiana, a parametric fitting with Weibull distribution was carried out using SAS procedure 'lifereg' [7]. This procedure handles doubly censored observations and includes checks on the significance of the estimated parameters. The threshold value for 'failure' was specified at the condition rating of '3'. This was to follow Jiang *et al* [1] so a comparison with their results could be made. Three covariates had been considered; material type, average daily traffic (ADT), and a categorical variable to indicate if the bridge has previously been rehabilitated. It was found that ADT, does not significantly affect the time to failure and was thus dropped from the analysis.

The Weibull parameters were computed from the results of the analysis. For illustration, parameters corresponding to threshold value of '3' are given in Table 1 and related distributions plotted in Fig. 1. The curves suggest that steel bridges tend to take a shorter time to reach condition rating '3' when compared with concrete bridges. Also, rehabilitation work undertaken on bridges does have a positive effect in extending the bridge lives.

Table 1. Fitted parameters of Weibull distribution (threshold value = '3').

	Conc. - rehab.	Conc. No rehab.	Steel - rehab	Steel No rehab.
$\alpha$ , Scale param.	1.43E-08	6.08E-09	4.08E-08	1.73E-08
$\gamma$ , Shape param.	3.6315	3.6315	3.6315	3.6315

A close examination of the reliability functions suggests that they may indeed be a plot of an infinite number of two-state Markov chains along the time axis. To investigate, we arbitrarily divide the time axis into various disjoint intervals of unit length. By considering the binary states of survival and failure we then obtain, for each time interval, a two-state Markov chain:

$$P_m = \begin{bmatrix} p_m & 1-p_m \\ 0 & 1 \end{bmatrix} \quad (8)$$

where  $1-p_m$  is the probability of failure in the interval  $[m-1, m]$  and  $p_m$  is the probability of survival beyond the interval. We know that for a two-state Markov chain with the format as in Eq. (8) the unconditional probability of survival at time  $n$  in the future is given by:

$$\prod_{m=1}^n P_m = \prod_{m=1}^n p_m \quad m = 1, 2, \dots \quad (9)$$

This is indeed the non-parametric estimator for  $P\{T>t\}$ ; viz., the reliability function.

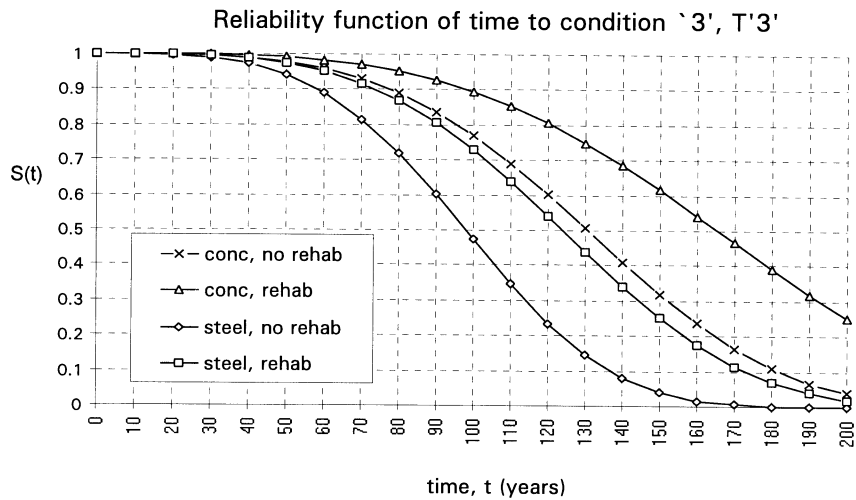


Fig. 1. Reliability functions of bridges fitted by Weibull distribution.

To further investigate the nature of the reliability function we proceeded to derive a homogeneous Markov chain from the survival model. By successively redefining the limiting value for 'failure', as condition ratings '3', '4', ..., '8'; and performing the survival analysis in each case, we obtained distributions of  $T_{\cdot 3}$ ,  $T_{\cdot 4}$ , ...,  $T_{\cdot 8}$ . These distributions together give a complete description of the deterioration process, as illustrated in Fig. 2.

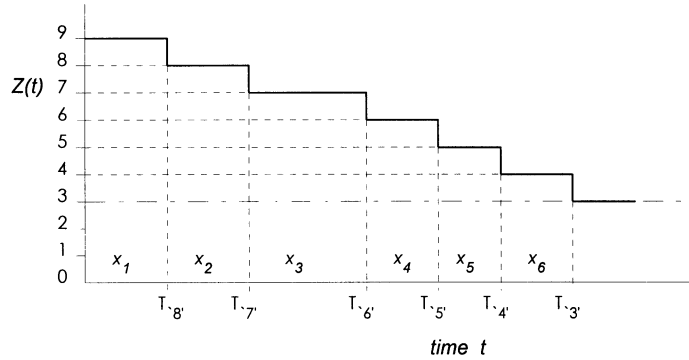


Fig. 2. A sample function of bridge deterioration process.

The random variable  $X_i$  is the duration that the process takes to stay in state  $i$  and is sometimes called the *sojourn time*. Notice that if  $X_1, X_2, \dots$ , have independent and identical exponential distributions we would have a continuous-time Markov process. In the present case, they are neither exponential nor identical.

We next define  $Q_{ij}(t)$  as the probability that after entering state  $i$  the process will next move to state  $j$  in an amount of time less than or equal to  $t$ . Put in another way, it is the probability that the random time for the process to move from  $i$  to  $j$  is less than or equal to  $t$ . In the special case where the bridge condition only deteriorates but never improves we have:

$$\begin{aligned} Q_{12}(t) &= P[X_1 \leq t] = P[T_{8,7} \leq t] \\ Q_{23}(t) &= P[X_2 \leq t] = P[T_{7,6} - T_{8,7} \leq t] \\ &\vdots \\ &\text{etc.} \end{aligned} \quad (10)$$

We know that  $T$  are Weibull distribution and Eq.(10) can be solved as a convolution:

$$P[T_1 - T_2 \leq t] = \int_0^\infty F_{T_2}(t + \tau) f_{T_1}(\tau) d\tau; \quad \text{for any } T_1 \text{ and } T_2. \quad (11)$$

We considered a transition period of 1 year and set  $t = 1$ . Transition probabilities were obtained by solving Eq.(11) numerically. Table 2, shows the transition matrix for concrete bridges which have not been rehabilitated previously.

#### 4 Discussions and conclusions

This paper has discussed existing methods for estimating transition probabilities of Markov-chain bridge deterioration model. An alternative approach using time-dependent reliability and survival analyses has been proposed and some preliminary results shown. The advantages of the proposed method include:

- Use only one set of data.
- Sojourn time is not restricted to geometric or exponential distribution.
- Use of probability-based measure, like reliability function facilitates computation of risk needed in life cycle cost analysis.

However, like the Markov chain models discussed in this paper, the proposed model could be used only for decisions regarding a network of bridges, rather than on individual bridges.

Table 2. Transition matrix for concrete bridges without previous rehabilitation.

	9	8	7	6	5	4	3
9	0.93	0.07	0	0	0	0	0
8	0	0.87	0.13	0	0	0	0
7	0	0	0.83	0.17	0	0	0
6	0	0	0	0.76	0.24	0	0
5	0	0	0	0	0.77	0.23	0
4	0	0	0	0	0	0.85	0.15
3	0	0	0	0	0	0	1

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## 5 RELIABILITY BASED REASSESSMENT OF AN EXISTING CONCRETE BRIDGE – A CASE STUDY

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### **Abstract**

A rational framework for reliability based reassessment is suggested and illustrated through the reassessment of an existing concrete motorway bridge subject to increased traffic loading. In this case where traditional approaches for reassessment indicated that strengthening of the bridge with an estimated cost of MECU 5 was necessary, the use of the suggested reassessment framework revealed that no strengthening of the bridge was necessary at all.

Keywords: Reassessment, Safety Formats, Reliability Analysis, Pile Strength Models.

### **1 Introduction**

As a part of the general evaluation of the bridges on the Danish motorway system a routine reassessment of the load carrying capacity and the residual life of the motorway bridge over the Gudenå river was initiated in 1994 by COWIconsult for the Danish Road Directorate.

The Gudenå bridge was originally opened in 1971 as the first part of the north-south oriented motorway of Jutland connecting, as a part of the Europe road 45 system (E45), Norway in the north with the African continent in the south.

The bridge structure is a reinforced concrete pile deck structure with a total length of 400 metres and a width of 26 metres. The pile deck super structure is composed of simply supported slab sections with spans of about 15 metres. The super structure is supported by a sub structure consisting of columns which themselves are supported on driven concrete piles. The bridge structure is illustrated in Fig. 1.

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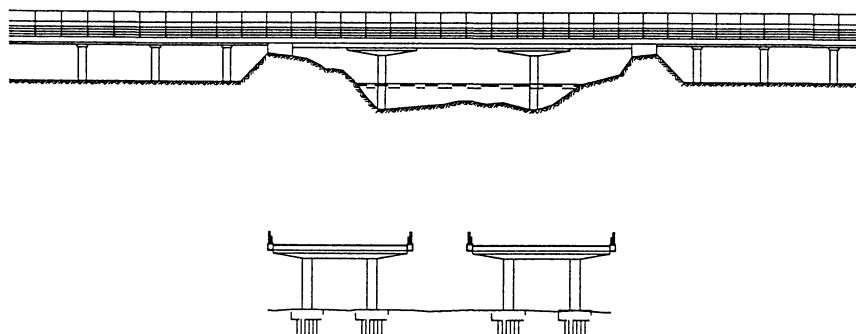


Fig. 1. Illustration of the Gudenå bridge structure.

A requirement for the load carrying capacity of the bridge, was that it should comply with bridge classification 100 (the class roughly corresponds to the maximum allowable load - in tons - from an extraordinary vehicle, applied simultaneously with an ordinary - 50 ton - vehicle) with reasonable maintenance costs for a residual service life exceeding 20 years.

The initial reassessment of the bridge was based on the same assumptions and the same structural data as used originally in the design of the bridge. The result of the initial reassessment indicated that the actual classification of the bridge was 70 for the super structure and 40 for the sub structure, i.e. a significantly lower carrying capacity than required. As a first indication of the costs associated with the necessary strengthening of the structure, in order to upgrade the overall bridge classification to class 100, an amount of MECU 5 was estimated.

Recognising that the actual condition of the bridge appeared to be excellent, and that the safety format underlying the original design basis takes into account uncertainties which may be reduced or even eliminated in a reassessment situation, it was decided to investigate the possibility of performing the reassessment of the bridge based on an alternative safety format. Thereby, it would be ensured that the knowledge concerning the actual condition of the bridge would be taken consistently into account, leading to a less conservative assessment of the bridge, which again may lead to a reduction of the upgrading costs.

## 2 Safety formats for structural reassessment

Two alternative approaches may be followed in establishing a safety format which takes into account actual available information regarding an existing structure.

One approach which is broadly operational but lacks flexibility is based on the load and resistance factor design (LRFD) safety format. The idea in this approach is that the load and resistance factors are modified in accordance with the level of information

available about the structure and the loading. In this way the safety factors may be reduced as more information about the actual loading and the actual structural state is collected. The LRFD safety format is sometimes referred to as a deterministic safety format as the uncertainties are taken into account through the safety factors.

Another approach which is more involved, but also more detailed and flexible, is based on a direct assessment of the reliability of the structure corresponding to a given (uncertain) loading and a given (uncertain) structural state. In this way the probability that the structure experiences collapse, excessive deterioration or excess of other limit states, may be directly assessed and compared with acceptance criteria.

Reliability methods are widely used in the design of LRFD based structural design and reassessment codes and should be seen as a further detailing in the treatment of information rather than an alternative approach to the deterministic safety formats.

When using reliability methods for reassessment of existing structures the knowledge about the state of the structure (material strength, loading, deterioration state etc.) is represented through probability density functions. The family of the individual density functions and the corresponding density function parameters may be established through frequentistic material (e.g. obtained through testing or inspection of the particular structure at hand) as well as through subjective information (based on experience or data from similar structures) or any combination of frequentistic and subjective information within the framework of Bayesian statistics.

Modern structural reliability methods have, during the last two decades, been developed to a state where they are readily applicable in practical applications. A comprehensive description of structural reliability methods and commercially available tools for reliability analysis is given in [1] from which core references to the relevant literature may also be found. In the following, no details will therefore be given on reliability methods but rather on their general application in the framework of reliability based structural reassessment, and in particular for the case of the Gudenå bridge reassessment.

### **3 General reassessment framework**

When a structure is designed, the knowledge about the structure 'as built' is associated with uncertainty regarding geometry, material properties, loading and environmental conditions.

A part of this uncertainty is due to inherent randomness which may be present e.g. material properties and loading characteristics, but a substantial part of the uncertainty arises from extrapolation of information. In this way, for example, the uncertainty associated with material properties in the design phase contains a significant contribution from the fact that the materials manufacturer may not be known and because the material batch characteristics may not be known.

The probabilistic models used in the design and in the assessment of a structure are hence merely reflecting the imperfect knowledge about the structure, and this knowledge may be updated as soon as the structure has been built.

An important task in the assessment of existing structures is therefore to perform a successive process of collecting and utilising information about the condition of the structure, the behaviour of the structure, and the loading on the structure.

Given that the requirements regarding the present and future use of a structure are specified, the reassessment process is a decision process of identifying the measures which will lead to the most economical fulfilment of these requirements.

Such measures may be to inspect and collect information regarding the geometry of the structure, the material properties, the deterioration of the structure, the static and dynamic behaviour of the structure, and the loading on the structure.

Measures may also be taken to repair or strengthen the structure or even to replace the structure.

The flow in a practical applicable reassessment procedure is shown in Fig. 2.

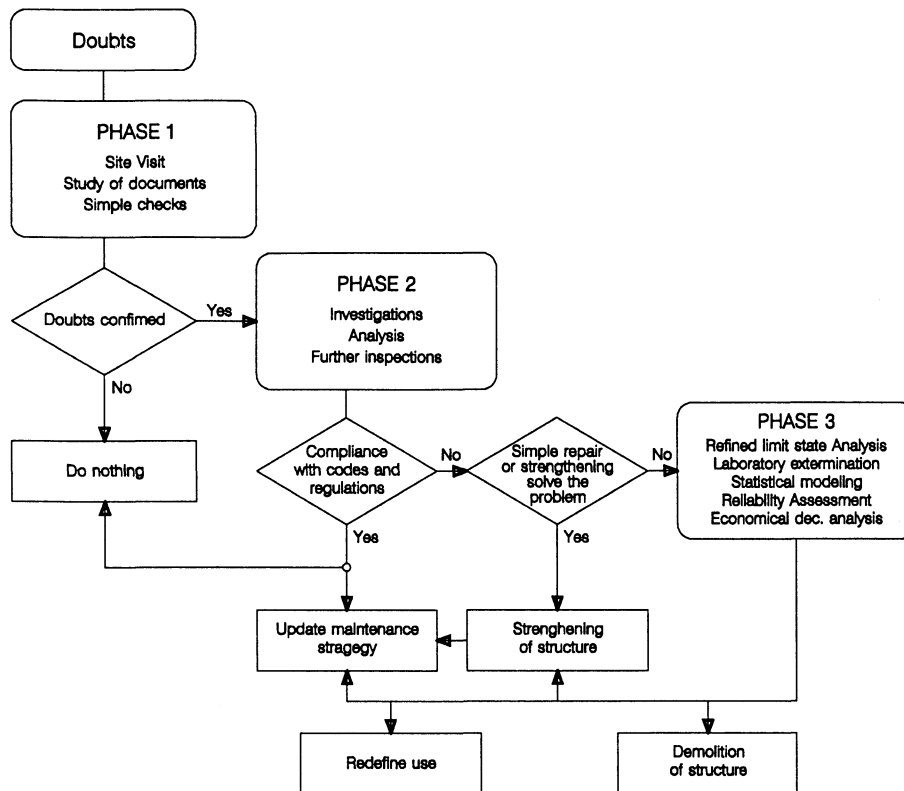


Fig. 2. Flow diagram for reassessment of existing structures.

An important aspect in the reassessment procedure is that the knowledge about the structure is established and refined in an adaptive manner according to actual need.

Load capacity and durability of the particular structure is initially reassessed based on simple structural analysis methods and readily accessible data. On this basis, the extent to which the structure fails to comply with the given requirements is evaluated.

Furthermore, the way in which a refinement of the knowledge about the bridge may best reveal the reason for not complying with the given requirements is identified. Such refinements may be based on detailing the structural analysis methods as well as on further collection of material data.

A successive assessment of an existing structure as described above may hence involve evaluations, which in terms of refinement and detailing, span from purely heuristic, experience based statements over application of deterministic safety formats to instrumentation, testing and probabilistic analysis.

#### **4 Reassessment of the superstructure**

With respect to the reassessment of the superstructure the successive scheme, as described in the previous section, was applied. Following the initial reassessment based on the same safety formats and knowledge about the structure as used in the original design, a LRFD safety format developed especially for reassessment of existing bridges was applied, see e.g. [2]. This safety format allows for modifying the safety factors according to the actual level of knowledge about the geometry of the structure, the strength of the materials and the precision of the structural analysis methods used.

The actual knowledge about the structure allowed significant reductions of the safety factors and following the updated reassessment, it was identified that only two failure modes were still critical for upgrading the bridge to class 100; namely, the positive moment capacity of the deck at mid-span and the punching shear strength of the deck over the supporting columns.

By refining the structural analysis methods to take into account the plastic capacity of the concrete deck, however, it was verified that the positive moment capacity is sufficient and hence only the punching shear strength remained a problem. By sensitivity analysis the governing parameter for the punching shear strength with the particular geometry and reinforcement at hand was identified as the concrete compression strength and therefore emphasis was focused on this.

The concrete compression strength assumed in the original project material was given as a cube compression strength of 20 MPa. When considering the overall state of the bridge, and recognising that the compression strength usually will increase as the concrete ages, this value appeared to be very conservatively assessed, and therefore it was decided to test the actual compression strength of the concrete in the superstructure. To this end 6 concrete cylinder specimens were drilled out of the bridge. The cylinders were compression tested and it was found that the characteristic (5% percentile with 95% confidence) compression strength of the concrete is 33 MPa, i.e. significantly higher than the value first assumed.

Based on the result of the concrete compression tests it was decided that the punching shear strength was sufficient and concluded that the load carrying capacity of the superstructure was sufficient for upgrading the bridge to class 100 without strengthening.

In addition to the reassessment of the load carrying capacity of the bridge the durability of the bridge was also assessed according to the increase in the loading.

Detailed finite element analysis of the bridge indicated that formal crack widths could

be expected in the range of 0.3 - 0.5 mm in the most critical areas of the bridge (lower side of the deck, mid span, and upper side of the deck, over the supports). By inspection of the bridge, however, it was found that there were no indications of cracking and by laboratory examination of concrete samples it was confirmed that carbonation was insignificant. By arguing that chloride penetration (due to de-icing salt) only will be a relevant damage process for the upper side of the deck, which is protected by a water-insulating membrane, it was concluded that the durability of the bridge was sufficient.

## 5 Reassessment of the sub-structure

As previously mentioned the initial reassessment of the sub-structure, based on original project design information and pile driving records, revealed that the carrying capacity of the sub-structure corresponds to class 40. As the determining parameter for the class of the sub-structure is the compression strength of the piles, attention was focused on a refined reassessment of these.

For driven piles, an increase in the load carrying capacity, as compared to the strength estimated from pile driving formulae, can normally be expected after the piles have been driven and the soil in the immediate vicinity of the piles has recovered. For the usual cylindrical constant diameter piles, this increase in the load carrying capacity will normally take place during the first year after the piles have been driven, but for the piles used for the present structure, which are piles with footings, see Fig. 3, the increase in carrying capacity may be expected to take place over a substantially longer period of time i.e. over several years. Furthermore, as piles with footings induce more severe disturbances in the soil around the piles during pile driving, larger increases in the carrying capacity can be expected for such piles. For this reason, the reassessment of the load carrying capacity of the piles concentrated on establishing and verifying models of the pile compression capacity incorporating the increase due to soil recovery. The present presentation focuses on the general methodology and approach in the case considered. For a comprehensive description of the pile load capacity modelling the reader is referred to [3].

First a model was established for the present load carrying capacity of the piles. It is assumed that the pile load capacity may be described by the sum of basically two contributions, namely a contribution from the pile surface (shaft and foot) and a contribution from the pile tip, see Fig. 3. As the piles are located in a two layer soil structure the pile load carrying capacity may be expressed by:

$$Q_p = Q_1 + Q_2 + Q_3 + Q_4 \quad (1)$$

where

$Q_1$  is the contribution from the pile shaft in cohesion soils.

$Q_2$  is the contribution from the pile shaft in friction soils.

$Q_3$  is the contribution from the surface of the pile foot.

$Q_4$  is the contribution from the pile tip.

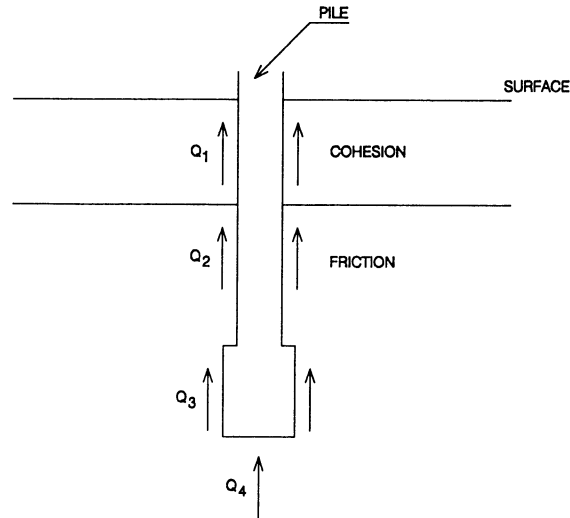


Fig. 3. Principal illustration of the pile and the soil structure.

It may be assumed that the pile tip carrying capacity at the time of the pile driving can be estimated from the pile driving journals and by application of the standard pile driving formula from the Danish codes (DDR). However, by comparison of pile load carrying capacities determined by the DDR and static pile capacity tests performed immediately after pile driving, see e.g. [4] a discrepancy is observed.

This discrepancy may appropriately be described by a systematic term (bias) and a random term (noise).

The relationship between the capacity of the piles estimated through the pile driving expressions  $Q_{DDR}$  and as obtained by compression tests  $Q_P$ , can therefore be given by:

$$Q_P = K Q_{DDR} + \Sigma \quad (2)$$

where the bias factor  $K$  and the noise term  $\Sigma$  are model parameters estimated by the maximum likelihood method.

One month after the piles were installed, four static pile compression tests were performed and the results of these tests can be used to estimate  $K$  and  $\Sigma$  for the present pile capacities. At the time of the pile compression tests it may be assumed that full friction is established on the shaft area on the pile feet. The relation between the pile load carrying capacity estimated by DDR and the static test results can hence be given by:

$$Q_P = Q_3 + K Q_{DDR} + \Sigma \quad (3)$$

In the period following the static pile capacity tests it is assumed that all possible pile shaft load capacity has been established and the present pile load carrying capacity may

therefore be written as:

$$Q_P = Q_1 + Q_2 + Q_3 + K Q_{DDR} + \Sigma \quad (4)$$

where:

$$\begin{aligned} Q_1 &= c_u A_{cf} \\ Q_2 &= S_u A_{fs} N_m \\ Q_3 &= S_u A_{ff} N_m \end{aligned}$$

where  $S_u$  is the effective stress,  $A_{cf}$  and  $A_{fs}$  are the surface areas of the pile shafts in the cohesion and the friction soil respectively and  $A_{ff}$  is the surface area of the pile foot shaft.  $N_m$  is a factor modelling the participating friction.

By modelling  $N_m$ ,  $c_u$ ,  $K$  and  $\Sigma$  as random variables and fitting the parameters of  $K$  and  $\Sigma$  by use of the Maximum Likelihood Method, see [5], the probability distribution function  $F_{Q_P}(x)$  for the piles may readily be determined through:

$$F_{Q_P}(q_P) = P(q_P > Q_P) \quad (5)$$

where the right hand side gives the probability that the uncertain pile capacity  $Q_P$  is lower than a certain value  $q_P$ .

The probability density function for one of the piles is illustrated in Fig. 4.

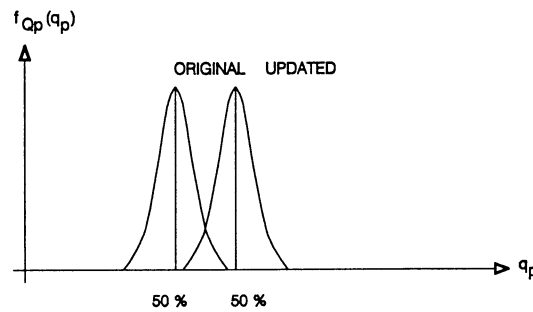


Fig. 4. Illustration of the probability density function of the compression strength for one of the analysed piles, before and after, additional pile compression strength tests.

Corresponding to normal practice the characteristic pile capacities to be used with the deterministic safety formats, in the classification of the bridge, shall be assessed as the 50% percentile value (indicated in Fig. 4) i.e. the mean value of the pile capacity. In this way there is no benefit gained by having an estimated pile capacity with a low coefficient of variation in comparison to an estimated pile capacity with a high coefficient of variation, which is why in general the mean value is a bad choice for a characteristic value.

In order to verify the probabilistic model for the pile compression strength it was

decided to perform on-site compression tests of three of the four piles tested at the time of construction. Furthermore the results of the compression tests were planned to be used in order to update the probabilistic model of the pile compression tests. The updated probability distribution function for the pile compression strength, i.e. the distribution function of the pile compression strength, conditional on the outcome of the experiments  $\mathbf{X}$ , may be determined by:

$$F_{Q_P}(q_P | \mathbf{x}) = P(q_P > Q_P | x_1, x_2, x_3) \quad (6)$$

where the right hand side gives the probability that the uncertain pile capacity  $Q_P$  is lower than a certain value  $q_P$ , conditional on the observed results from the pile compression tests  $x_1$ ,  $x_2$  and  $x_3$ .

As the increase in the pile compression strength was quite significant, it was for practical reasons not possible to reach the ultimate compression strength of all three piles in the tests. In order to gain the most information possible from the tests, where the pile compression strength was not reached, a model was established linking the pile cap, load deflection curve, to the ultimate compression strength. The principle is illustrated in Fig. 4 where it can also be seen that a random variable was introduced in order to take into account uncertainties in the proposed model.

In advance of each of the tests, the probabilistic model of the pile compression strength was used in order to predict what the result of the next experiment would be, by use of the equation and the model illustrated in Fig. 5. It is worth noticing that all the predicted mean values of the pile strengths were within 10% of the test results.

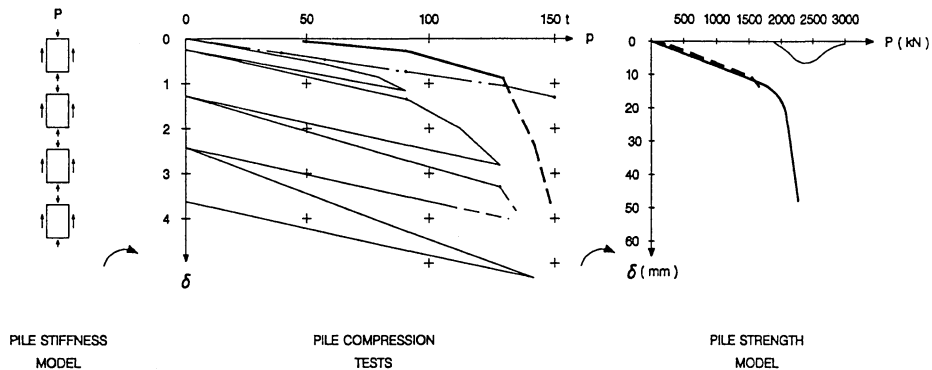


Fig. 5. Illustration of the modelling of the pile compression strength for the test piles which could not be loaded to their ultimate compression capacity.

Based on the updated probabilistic models for the pile compression strengths, updated probability distribution functions were established as illustrated in Fig. 4. In general the mean values of the pile compression strengths were increased significantly (10 -

20%) due to the test results.

Using the updated characteristic values (50% percentile) for the pile compression strength in the reassessment of the sub structure, it was found that only 10 of the piles did not meet the requirements for upgrading the bridge to class 100. For this reason it was decided not to use the deterministic safety format and to use reliability analysis directly instead.

For the assessment of the reliability of the individual piles, a probabilistic model was developed for the traffic loading on the bridge.

It should however be noted that reliability estimates at all times will be relative to the applied probabilistic model, which is usually influenced by a certain amount of subjectivity. Therefore it is important to be able to compare, using the same probabilistic model, the result of a reliability analysis of a structure, with an unknown reliability, with the result of a reliability analysis of a structure which is known to be safe.

As the traffic loading relevant for punching shear failure of the superstructure and for compression failure of the piles is the same, and the punching shear strength of the superstructure is governed by the concrete compression strength for which frequentistic material was available, the reliability with respect to punching shear failure of the superstructure was determined first. The result of this reliability analysis indicated that the failure probability was around  $10^{-5}$  per year, which is in the order of magnitude to be expected using the deterministic safety format for the type of structure considered. Using the same probabilistic model for the traffic load, and the updated probabilistic pile compression strength, it was found that the failure probability of the most critical piles is in the order of  $10^{-7}$  per year, clearly indicating that the pile compression strength is sufficient for upgrading the sub-structure to class 100 without strengthening.

## **6 Conclusions**

The use of deterministic safety formats, such as the LRFD safety format, for reassessment of existing structures, is often excessively conservative and may hence lead to unnecessary and costly repair, strengthening or rebuilding requirements. This is because most deterministic safety formats have been developed and calibrated for the new design of structures, where in general the prediction of the loading and the strength of the structure is associated with a significantly higher uncertainty as compared to the situation of an existing structure which may readily be inspected, instrumented and tested. Even when deterministic safety formats have been designed for reassessment purposes, the flexibility, with respect to including specific information about the structure, may not be sufficient. In these situations, reliability methods have proven to be extremely effective, as these, in conjunction with consistent probabilistic models of loading and structural strength, allow any information about the structure to be taken into account in the assessment of the reliability of the structure.

A rational framework for reliability based reassessment, applied in a case study concerning the reassessment of an existing concrete bridge, illustrates the great potential in applying an adaptive scheme for refining the level of knowledge about the structure, by detailing the structural analysis or by collection of data. The framework takes as its

basis the application of the readily available information and traditional safety formats for the reassessment, but can be used to identify in a successive manner how more information should be collected, and allows for the direct use of reliability methods.

## 7 Acknowledgements

The case study presented has been performed in a close and inspiring collaboration with A. M. Christensen from the Danish Road Directorate who is warmly acknowledged.

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## 6 REPAIR OF THE BASCULE PIER OF THE VILSUND BRIDGE

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### **Abstract**

In the early 1990s it became apparent that alkali-silica reactions in the structural concrete, ingress of sea water and continuing deterioration of the bascule pier of the Vilsund Bridge had reached a stage where a thorough repair of the structure was needed. The paper describes the repair project, focusing on the character of the problems, the repair design, and the construction works. In the final part of the paper comparisons are made with repairs undertaken on other alkali-silica damaged structures.

Keywords: Alkali-silica reactions, Bascule pier, Injection, Repair, Strengthening.

### **1 Introduction**

The bridge over Vilsund was built in the late 1930's carrying traffic from the island of Mors, to the north-western part of Jutland. The bridge has a total length of 381 m, and is formed by 6 spans, one of which is a 34 m bascule span.

The traffic capacity of the bridge is still considered to be sufficient and also the navigational conditions for passing ships are acceptable. The bridge has therefore, from a traffic service point of view, performed well.

However, shortly after the bridge was put in service, extensive cracking was observed in the bascule pier, and sea water was leaking through the reinforced concrete walls.

## 2 Description of the structure and previous repair efforts

The bascule pier, shown in Fig. 1, is a reinforced concrete caisson structure founded on concrete piles. The pier is divided into sections by internal walls and decks of reinforced concrete. The lower sections of the pier have been filled with mass concrete. Reinforced concrete plinths support the bascule and built-in reinforced concrete columns support the adjoining bridge span and the toothed rim pinions for opening and lowering the bascule.

In the mid-1950s extensive cracking of the concrete above sea level was repaired. The cracks were assumed to be caused by alkali-silica reactions (ASR).

In the late 1970's an approximately 0.4 m thick barrier of reinforced underwater-concrete (Colcrete) was cast around the outer face of the pier below sea level. The barrier was expected to stop excessive ingress of sea water into the pier and deterioration of both structural and mass concrete. The repair proved to have no observable effect on the degradation of the pier and this is ascribed primarily to defects in the construction of the barrier.

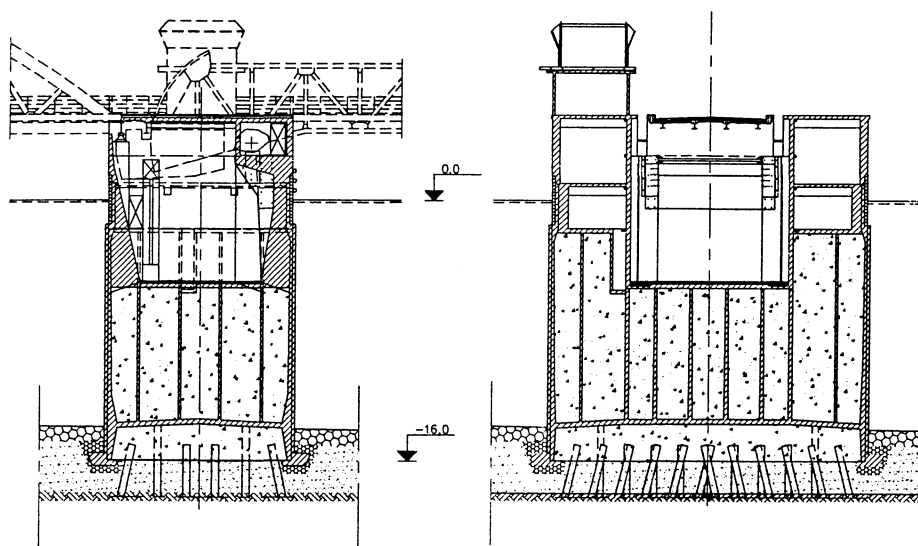


Fig. 1. The bascule pier.

## 3 Condition survey

In 1993-94 a detailed condition survey revealed that the degradation of the concrete in the pier had reached a state where an overall repair of the structure was needed in order to ensure the structural safety of the pier in general, and of some critical structural components in particular.

The survey confirmed that expansions due to ASR in the past, had led to extensive cracking of the structural concrete both on the outer walls and the inner parts of the

pier. Typically cracks are 1-3 mm wide. However, based on experience from other structures and laboratory tests on samples of concrete from the pier, it was concluded that future expansions due to ASR will be negligible. The laboratory tests included both fluorescent microscopy of thin sections and residue of reactivity analyses.

The extensive cracking of the concrete has, however, allowed excessive ingress of sea water into the pier (approximately 40 l being discharged from the pier every hour) and caused severe leaching and degradation of the concrete. Furthermore, large amounts of chlorides and sulphates from the sea water have been deposited in the concrete, which has accelerated the degradation of the structure even more. It was assessed that the compression strength of the concrete on average has been reduced approximately 30%, however, in some areas of the pier the concrete has totally disintegrated.

From the observations summarised above and evaluations of the load carrying capacity of the pier and its structural components, it became apparent that a thorough repair of the pier could not be postponed any longer.

#### 4 The repair strategy

Due to the urgency of some of the repairs it was decided to divide the repair of the pier into 2 phases.

The repairs in phase 1 were primarily located around the plinths supporting the bascule bearings, Fig. 2, where structural failure was considered imminent due to cracking, deteriorating concrete and corrosion of the embedded reinforcement, combined with the high loads on the plinths from the bascule.

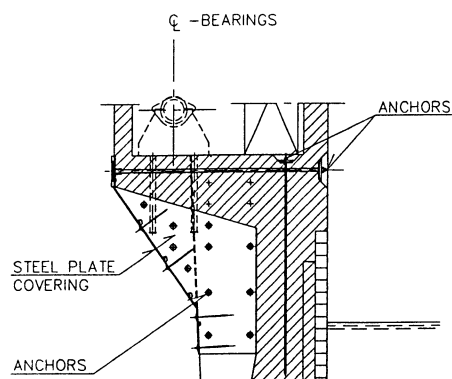


Fig. 2. Repair of the plinth for the bascule bearing.

In general, however, phase 1 included repairs of the structural components where the degradation of the concrete was considered to have caused the most severe weakening of the structure, giving rise to doubts about structural safety.

The repair of the plinths comprised of attaching a steel sheet covering around the plinths, combined with post-tensioned anchors, Fig. 2. Cracks in the plinths had previously been injected with epoxy-resin and damaged concrete removed.

Phase 2 includes an overall repair and strengthening of the pier in order to hinder future degradation and bring the overall structural safety of the pier in accordance with current Danish codes and load regulations.

The repair and strengthening comprise, in outline, removal of the existing underwater Colcrete barrier, underwater casting of a reinforced concrete plate, under the pier, and casting of an external reinforced concrete wall around the pier, founded on new piles and connected to the existing structure by means of post-tensioned transverse anchors. The outer face of the new concrete wall is provided with a steel plate covering which will also be the form for the wall during casting.

Internal repairs of the pier comprise of conventional concrete repair and extensive injection of cracks, honeycombs and cavities with grout and epoxy-resin. An outline of the repair is shown in Fig. 3.

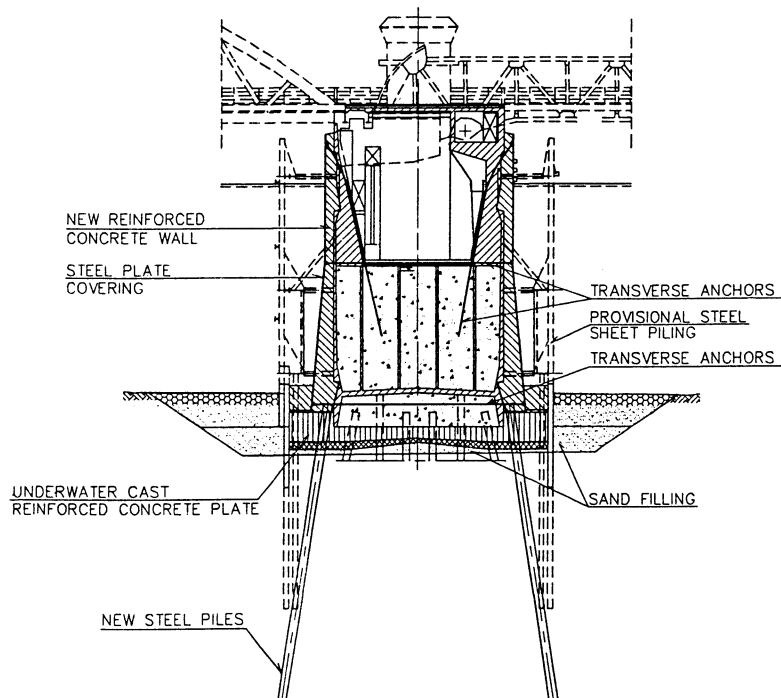


Fig. 3. Repair and strengthening of the bascule pier.

## 5 Construction works

After the first injection efforts on the plinths undertaken in phase 1, it became obvious that the cracking of the structural concrete was even more extensive and wide-spread

than expected from the condition survey. Epoxy-resin came out of cracks at random and far from the injection holes. A new injection tactic had therefore to be adopted. It was decided to try to limit the injection area by using fast-setting epoxy-resin to inject the concrete around the circumference of the plinths. However, a vast amount of epoxy-resin was used to inject the cracks in the plinths, with no account for the whereabouts of the major part of the material. After the injection work was completed, holes for post-tensioned anchors were drilled, and the steel sheet covering and post-tensioned anchors installed. Holes for anchors, 2-3 m deep, had to be drilled very precisely, not damaging the existing reinforcement and the bolts for the bascule bearings. Construction works in phase 2 commenced early in 1995. The external wall is to be constructed in a dry building pit behind a temporary steel sheet piling around the pier. Traditional working methods will then be applied for the construction of the wall, transverse anchors, displacement locks, steel plate covering etc. However, before the dry building pit can be established, part of the existing Colcrete barrier around the pier has to be demolished, new steel piles have to be driven; the sand filling beneath the pier must be removed and a new reinforced concrete plate has to be cast underwater, under the pier. These underwater works and the provisional steel sheet piling around the pier are expected to be completed in 1995.

At the end of July 1995, the new piles have been completed, removal of the sand filling beneath the pier started and the provisional steel sheet piling is under construction. Furthermore, repair and strengthening works are being carried out inside the pier.

## **6 Comparison with other projects and concluding remarks**

An almost identical repair was carried out in 1989-1991 on the bascule pier of the Oddesund Bridge. This bridge is located less than 40 km from the Vilsund Bridge and was, like the Vilsund Bridge, built in the late 1930's. The problems in the bascule pier at Oddesund were very similar to those now seen in the bascule pier at Vilsund, however, the pier at Oddesund also suffered settlement due to a slanting pile foundation.

In both structures the main reason for the degradation of the structural concrete is expansions due to ASR caused by the use of reactive aggregates in the concrete. For both structures, it has been concluded that the expansions due to ASR have diminished and have almost ceased.

Expansion due to ASR is known to be dependent on the amount of alkali and reactive-silica in the concrete. When one of the reactants is depleted, the expansion stops. In addition to these two components, water and calcium hydroxide are also limiting factors. Due to the extensive ingress of sea water over a period of 40-50 years in both piers, large amounts of alkali have been added to the concrete, suggesting that the expansion has stopped because the amount of reactive silica remaining in the concrete is very limited. It has therefore been concluded that the disappearance of expansion due to ASR is permanent.

This conclusion has of course been decisive in the choice of the repair strategy.

Observations on the bascule pier of the Oddeund Bridge after the repair was completed, have confirmed this basic assumption concerning ASR.

The Vilsund Bridge and the Oddeund Bridge are just two of many alkali-silica damaged structures in Denmark. Experience from these two bridges confirm that the only viable repair strategy when dealing with alkali-silica damaged structures seems to be a "wait and see" strategy, and knowledge of the residual reactivity is therefore crucial for planning the repair. Especially for marine structures that are constantly subjected to the ingress of sea water, and hence alkali, there seems to be no repair alternative but patience or provisional efforts until the reactive silica in the concrete has been depleted.

## 7 INVESTIGATION, REPAIR AND STRENGTHENING OF A VIADUCT'S STEEL PIERS

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France

### **Abstract**

In the late seventies a temporary steel viaduct was built beside a damaged bridge, carrying a heavily trafficked road crossing a railway line, and which was to be rebuilt. A few years later damage was found in some piers which were repaired and reinforced. Further inspection revealed several cracks in the same piers which were repaired and reinforced. Inspections again revealed several cracks in the same piers and showed that the strengthening had not been carried out sufficiently well and a second repair was undertaken. This needed to be done to a defined time scale in order to be completed by the target date for the opening of the new bridge.

### **1 Introduction**

In the early seventies severe damage was found in a reinforced concrete bridge built in 1941 over the PARIS-LYON and LYON-GENEVE railway lines, which carried 45,000 vehicles per day.

In 1976 a general inspection noted a considerable increase in damage in the deck and led to doubt about the behaviour of the bridge under such heavy traffic. Further investigation and assessment provided evidence of the impossibility of strengthening the bridge which was made of poor concrete fabricated in bad conditions due to the war. Therefore the decision was made to close the road to traffic as soon as possible, and to design and build a new bridge at the same location.

Two temporary steel viaducts were erected in 1977 alongside the existing bridge, one on each side, in order to reduce the access difficulties to the neighbouring Rhône crossing.

## 2 Description of the temporary steel viaducts

Each viaduct consisted of eight spans ranging from 15.40 m to 34.20 m supported by seven steel Y cantilever piers erected on the platforms between the railway tracks.

Each one way deck was made of prefabricated units ranging in length from 18.80 m to 30.80 m for a single width of 3.50 m, which were joined end to end with pins and side to side with bolts, so as to form a two lane 7.015 m wide carriageway. The units were entirely shop manufactured, including the anti-skid surfacing, in order to minimise the completion process on site. The north viaduct, as well as carrying a 1.30 m wide sidewalk, had to carry a 0.50 x 0.60 m container for pipes and feeders.

The piers, which were the same type for both viaducts, consisted of a 5.76 m long V shaped plate girder and a 1.03 m long box girder trunk, entirely shop welded and fastened on site with 30 high strength bolts (Fig. 1).

## 3 First damage and strengthening

### 3.1 Damage

In May 1982 a routine inspection of the north viaduct noted that 8 bolts on pier 6 and 14 bolts on pier 7, were missing at the south end of the connection fastening the cantilever girder to the trunk, with a slight opening between the two tie-plates on pier 7. The north lane of the carriageway was immediately closed to traffic in order to limit the transverse effect of live loads until the broken bolts were replaced - which was done at the beginning of June. At the end of July an overall bolt check was carried out on the whole viaduct, which revealed that a few bolts were missing again on piers 6 and 7.

As another closure of the viaduct would have had a dramatic effect on the local economy and traffic pattern, an investigation was undertaken to assess the structural strength of the Y piers and consider the options for their maintenance. Site visits highlighted:

1. an important volume of traffic on the north lane of the carriageway, practically comprising of only heavy trucks, the wheels of which rolled very close to the kerb;
2. a permanent loading of the last three spans due to traffic lights on the west abutment, with trucks waiting end to end;
3. a wide range of vibration and dynamic effects induced by braking and starting actions;

and this explained why only piers 6 and 7 were causing concern.

Further calculations taking into account this loading case showed a permanent transverse imbalance increased by the presence of the sidewalk and the container for feeders and pipes which did not exist on the other viaduct. Therefore tensile stresses in the bolts at the south end of the connection could reach, or exceed, the permissible stress, depending on the actual impact or dynamic effect of live loads.

### 3.2 Strengthening

The reinforcement consisted of a short corbel which was fixed under the south cantilever of the transverse girder onto the trunk. It comprised:

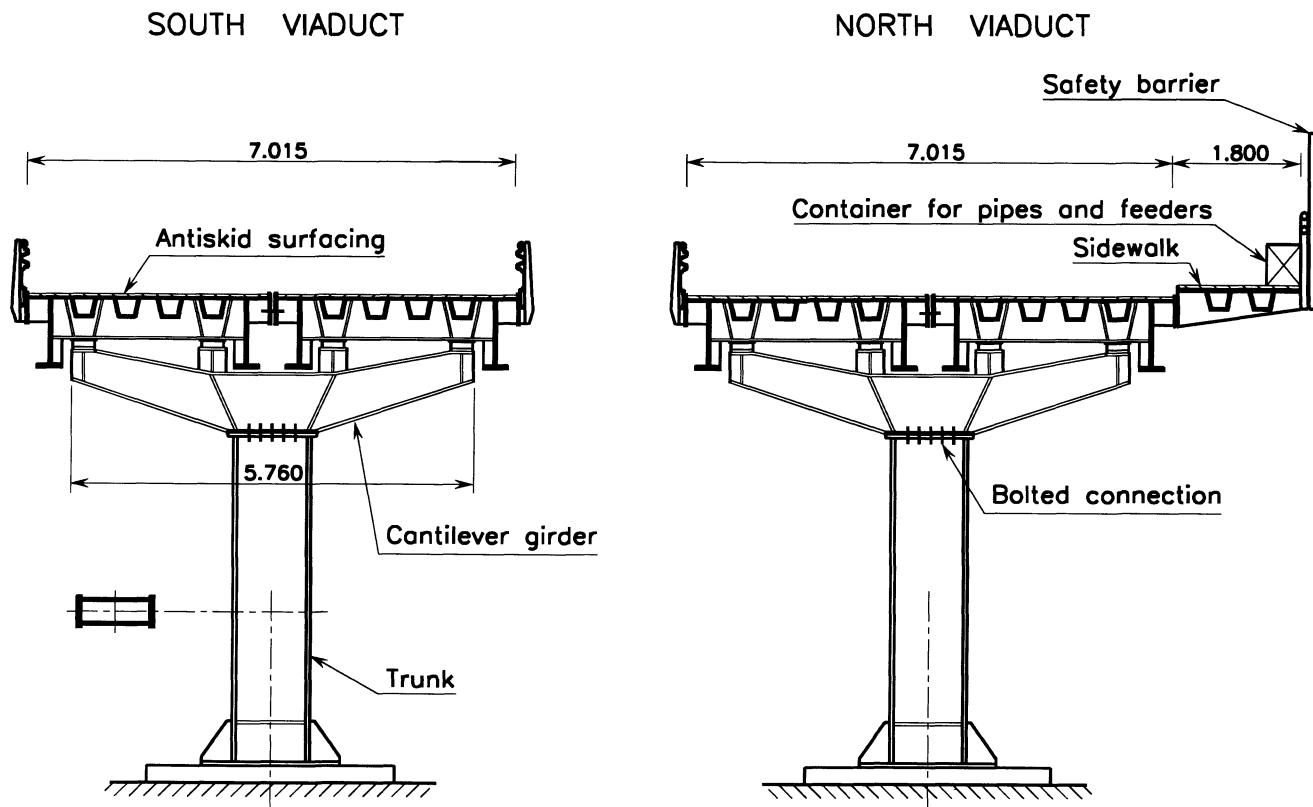


Fig. 1. Elevation of piers and cross section of deck.

- a web, varying in depth from 500 to 80 mm, with a rounded cut out at the lower top angle;
- a top flange, tied to the bottom flange of the cantilever girder with 4 x 5 high strength bolts;
- a bottom flange welded with the web to the trunk through a vertical plate; in order to keep the cantilever girder as a demountable piece of the pier (Fig. 2).

Only piers 6 and 7 were fitted with corbels in August 1982, and in the same time 3 bolts, which had been replaced in May, were found broken and had to be replaced again.

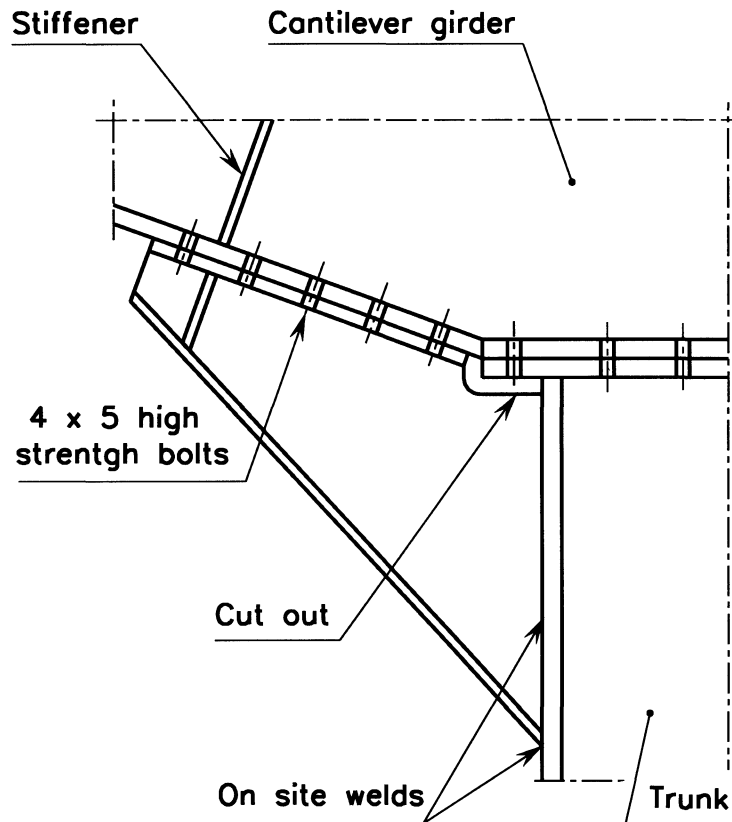


Fig. 2. Reinforcement as designed.

#### 4 Further damage and strengthening

##### 4.1 Damage

Every other month routine inspections did not note anything until February 1985 when 5 bolts were found broken on piers 6 and 7, still at the south end of the cantilever girder to trunk junction, and these were replaced, as well as 3 bolts fixed in 1982 which were getting loose.

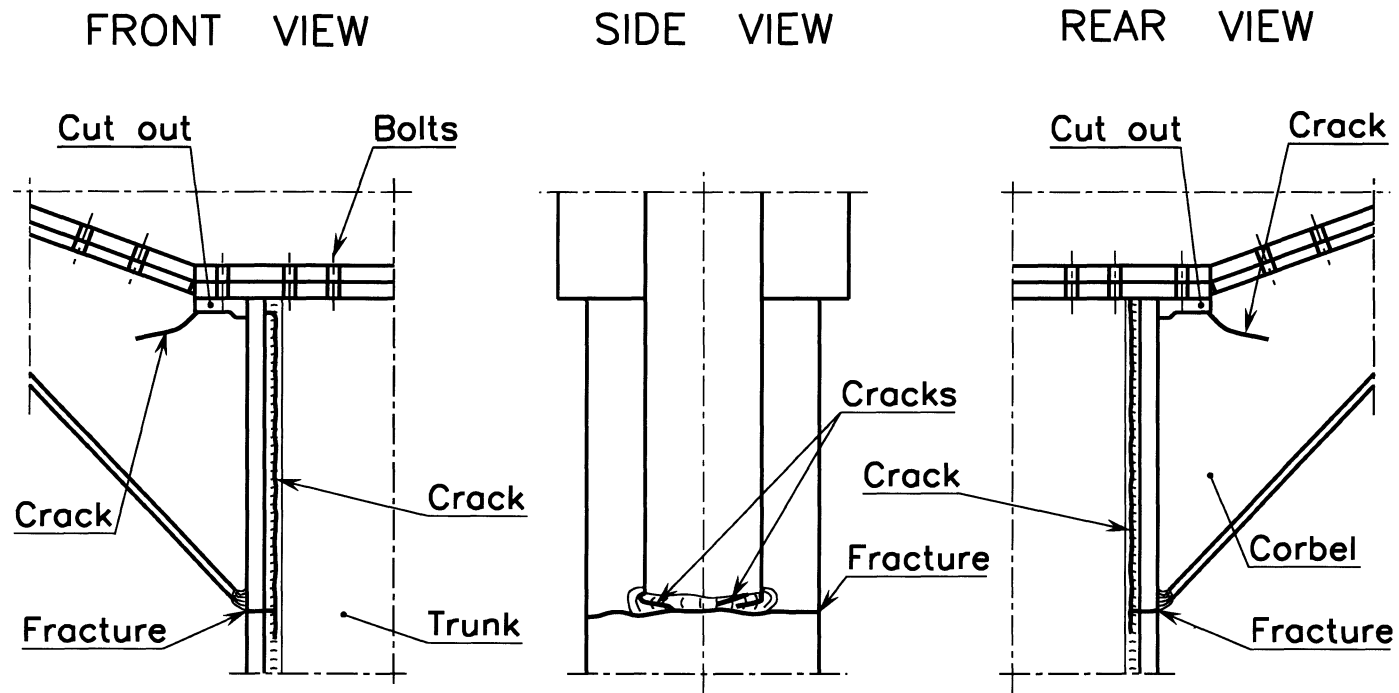


Fig. 3. Cracks and fracture on pier 6 (only damaged welds are represented).

A detailed inspection carried out in April 1986 reported that one bolt was missing on pier 5 and cracks had developed on piers 6 and 7.

On pier 6 cracks occurred in the reinforcement:

1. in the web, from the cut out at the lower top angle, with a length of 150 mm; in the weld joining the bottom flange to the trunk; and propagated into the flange of the latter, then into the fillet welds on both webs of the box girder. In actual fact the south flange of the trunk was completely broken (Fig. 3).

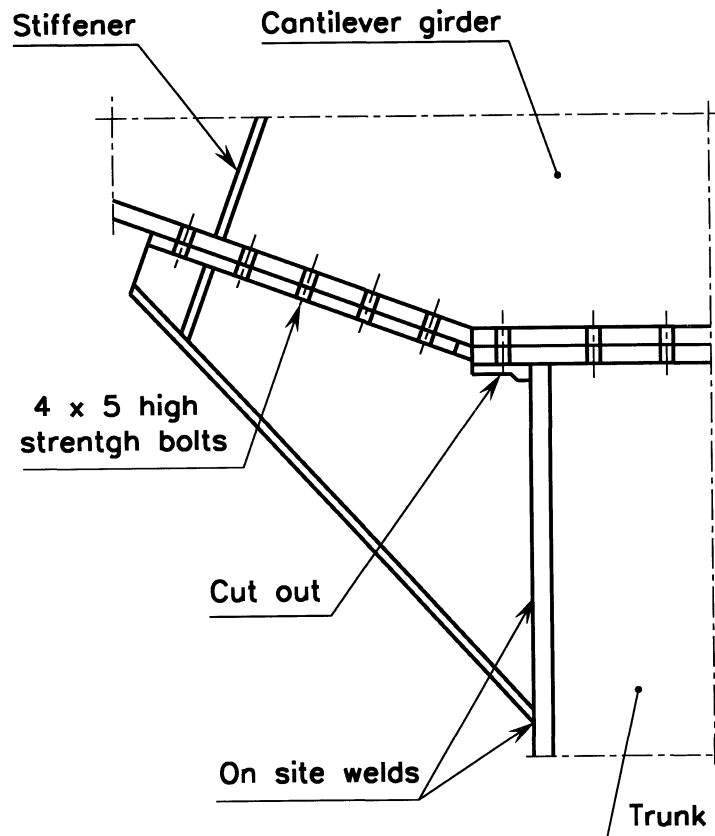


Fig.4. Reinforcement as achieved.

On pier 7 damage was restricted to cracks in the weld at the bottom of the reinforcement.

A thorough investigation brought out faulty details in the reinforcement which had not been provided as designed (Fig. 4). As a matter of fact:

1. the cut out in the web was not rounded, but comprised angles, with a notch from which the crack had developed (Fig. 5);
2. the vertical plate for attaching the corbel to the trunk was missing, for unknown reasons.

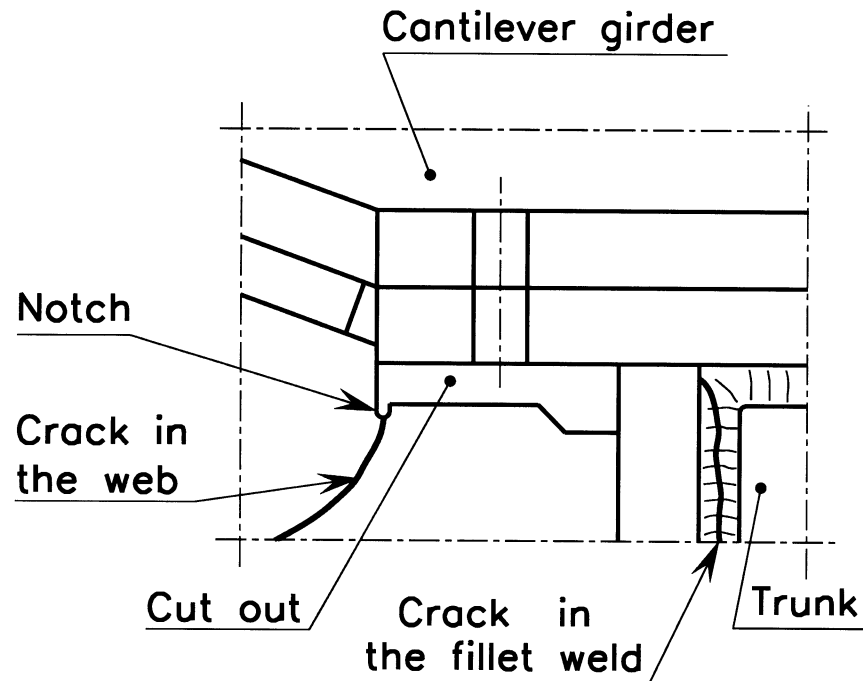


Fig. 5. Detail of the front face on pier 6.

As there was no connection between the top flange of the reinforcement and the top plate of the trunk, high stresses concentrated in the weld joining the corbel's bottom flange to the trunk, and fatigue effects induced the cracks and the fracture which could have led to the collapse of the pier.

#### 4.2 Strengthening

New reinforcement was immediately carried out which consisted in removing the corbels, repairing the cracks, and dressing the trunk over the whole height with two wide plates welded to the flanges and connected on both sides to the cantilever girder with high strength bolts (Fig. 6).

#### 5 Conclusion

As no further damage had occurred up to the opening of the new bridge in 1989, the second strengthening was considered satisfactory. This case history demonstrates the need to pay attention to details, even for temporary bridges.

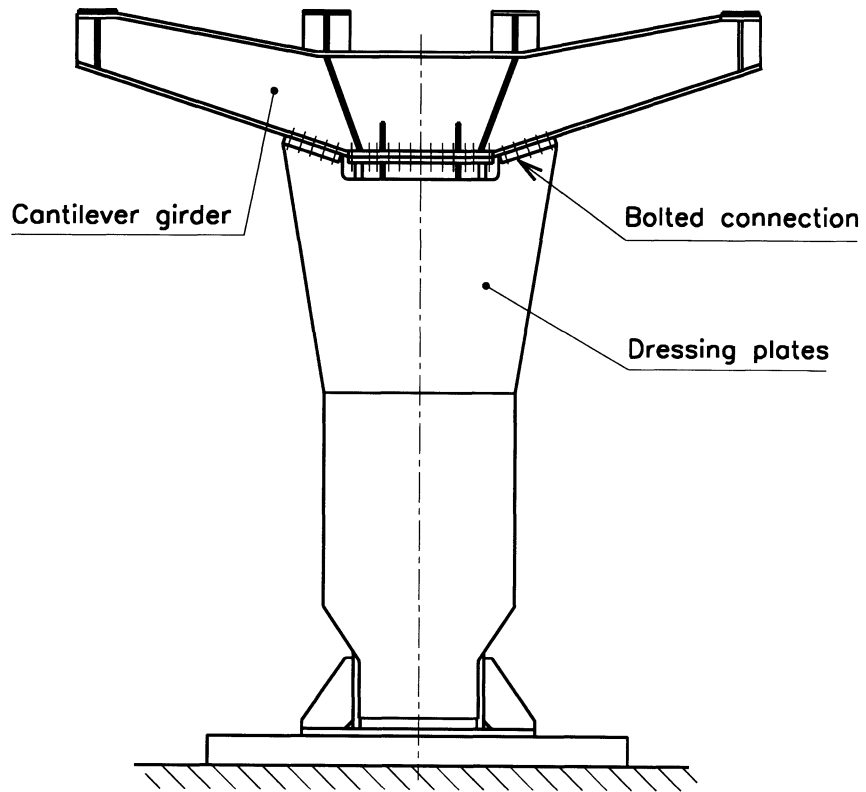


Fig. 6. Second strengthening on pier 6.

## 8 THE CHARMAIX VIADUCT (SAVOY FRANCE) PIER MOVEMENTS

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### Abstract

The paper describes repair work on a 346 metre long crossing of the Charmaix river to cope with movement of the ground support. Jacking is used to adjust the position of the bridge pier.

### 1 Introduction

The Frejus tunnel links France (Modane) to Italy (Bardonecchia) in the Alps. The access road to the tunnel on the French side, built on the edges of the mountains, traverses the Charmaix torrent over a bridge 346 metres in length comprising of 9 spans : 34.0 m + 7 x 40.0 m + 32.0 m. This is a prestressed beam viaduct with 4 beams below pavement, in curved plane and pier height up to 30 metres.

As the initial survey of the slopes did not indicate any foreseeable problems, the construction work was basically sited in an area of scree which proved unstable.

After completion of the viaduct (1978), the analysis of troubles centred on the girders - distortions under beam bearings, bottom areas of beams of successive spans in contact with concrete, despite an initial clearance of 50 mm allowed during construction, and ground instrumentation to confirm the movement of the two slopes of the Charmaix gorge.

The slopes displaced, on several ten metres thicknesses, at a quasi constant speed sliding by 2 cm per year, with an increased speed following the melting snow. Some pier displacements were approximately 20 cm in length, along the slope (Fig. 1).

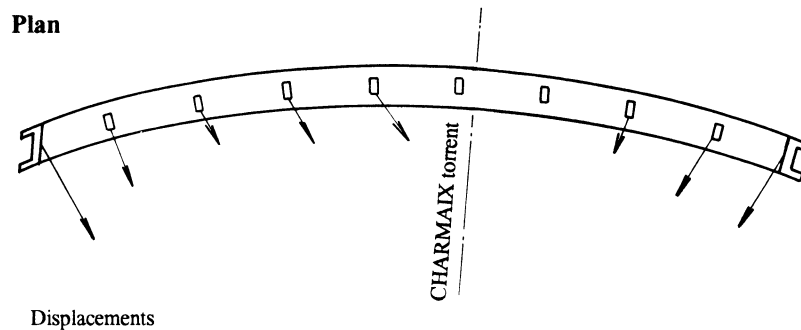


Fig. 1. Ground movements.

## 2 Location report of repair works

The owner of the bridge, the State Road Management, in collaboration with the D.D.E. of Savoy the works authority, TONELLO engineering office decided to follow a preliminary study of the repair work to free the piers from their moving foundations and to install an interface at ground level to permit, with every option for free operation, periodic replacement of the construction work to its initial location. The preliminary design included two possibilities; either to move the access road to the other side of the valley or to repair the bridge.

A short listed bid was produced giving the in principle total freedom of the bridge on the ground.

Taking into account the technical and financial proposals, the works were entrusted to VSL. They included work on each pier, following work on an experimental pier to prove the system, see Fig. 2.

The work comprised of the following steps:

- Installation of a jacking system able to displace the pier along axes x, y, z and in addition, also allow rotation around each axes.
- Enlargement of the foundation sole plate with the addition of guide walls.
- Formation of a prestressed reinforced base foot at the bottom of the pier.
- Jacking the pier under the base foot and against guide walls, vertically using 4 x 1000 t and horizontally by using 8 x 200 tonne jacks.
- Separating the pier from the sole plate by sawing.
- After positioning special sliding bearings and operating jacks, moving the pier on the sole plate to compensate for movement of the ground.

For the abutments, the work consisted of moving the girders on sliding supports installed for the movement operations.

Periodically, the company has had to move back the piers to hold the viaduct in position.

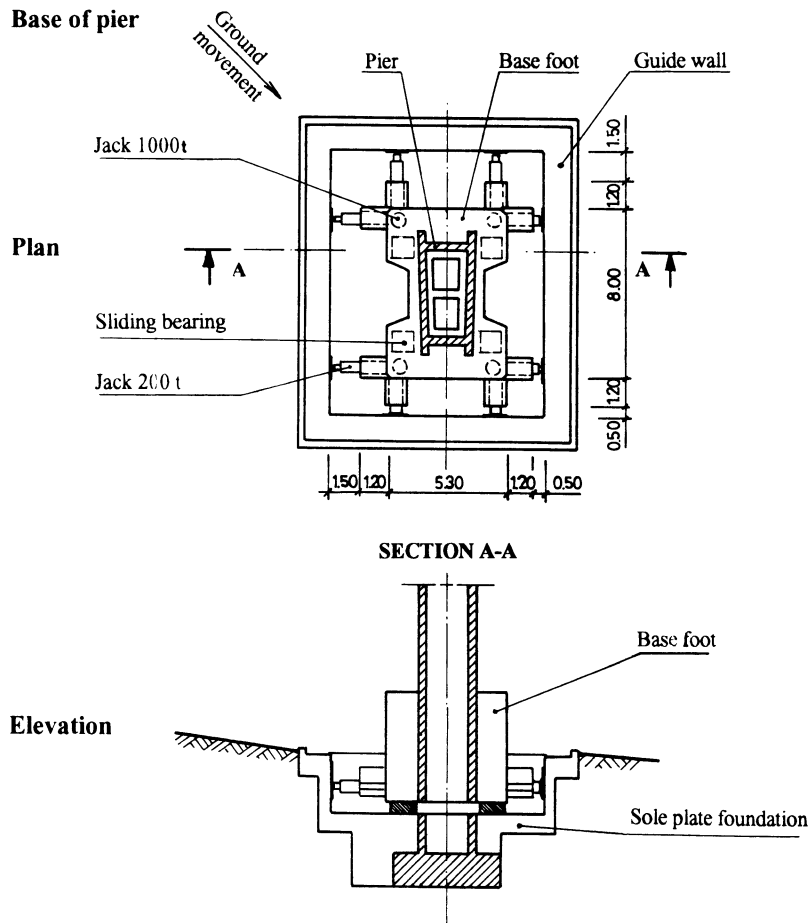


Fig. 2. Details of pier jacking arrangement.

### 3 Conclusion

As a model was made, movements, especially those of the girders, were confirmed by the structure which restored the efficient operation of the construction by movements at the foot of the piers. This single process saved a great deal of trouble if you imagine the alternative of transferring the road to the other side of the valley.

Although basically a very simple idea, the work demanded design, preparation and very careful execution to the smallest detail to prevent any improvisation during the manoeuvring operations.

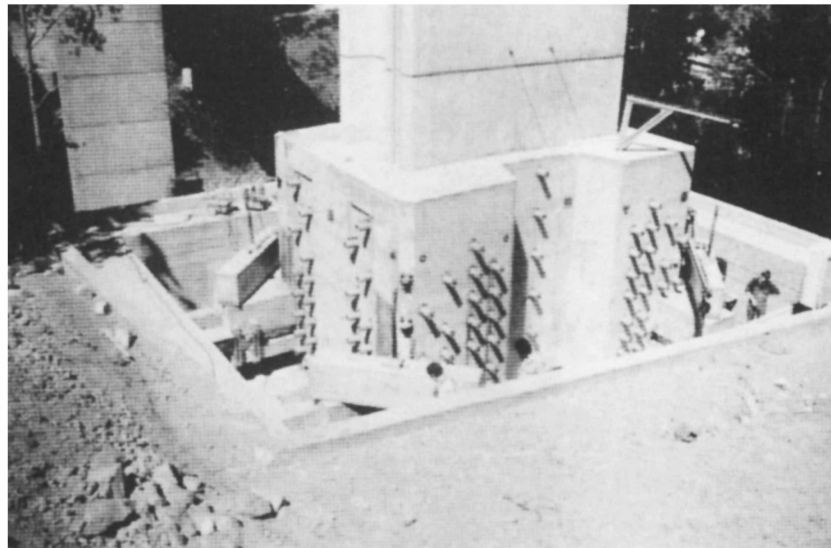
This experience shows that complex engineering solutions can be presented then carefully designed and finally put into practice by a specialised contractor.

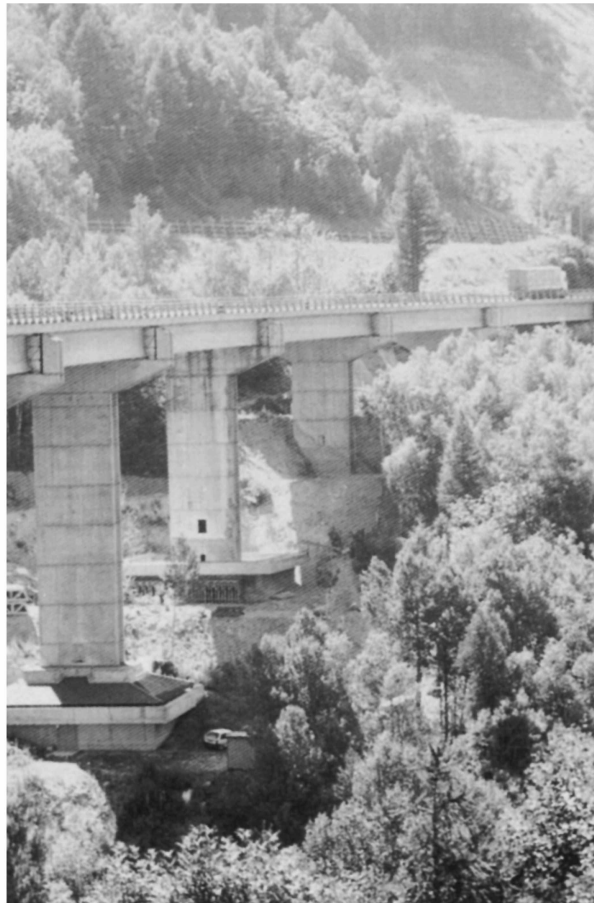
General views



Sole plate foundation

Base foot





## 9 INVESTIGATION AND REHABILITATION OF TSING YI SOUTH BRIDGE, HONG KONG

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### Abstract

Prompted by the presence of large areas of concrete delamination and corroding reinforcement on the piers, the Highways Department commissioned an investigation to assess the structural condition of, followed by a rehabilitation programme to, the 600 m long Tsing Yi South Bridge (Fig. 1). The investigation included the evaluation of the condition of existing concrete and reinforcement as well as the extent of ingress of aggressive agents, while the rehabilitation programme consisted of removal and replacing the defective parts of the bridge. Due to the very heavy traffic demand, the bridge was opened for vehicular traffic above and sea traffic below throughout the repair.

Keywords: Appraisal, carbonation, chloride, contract, corrosion, investigation, rehabilitation, spalling.



Fig. 1. Layout of the bridge.

## **1 Introduction**

Tsing Yi South Bridge was constructed in the 1970s to provide the only land access for vehicular traffic and to carry utility services between Tsing Yi Island and Kowloon. The bridge consists of five prestressed concrete box girders linked by shear hinges to form four main spans each 122 m long and two side spans 61 m long. The girders are supported on hollow rectangular reinforced concrete piers which are in turn supported by circular foundation piers sited upon caisson foundations. Pier 1 stands on Tsing Yi Island, Piers 2, 3 and 4 stand in water while Pier 5 is on an operating container terminal at Kwai Chung.

In early 1991, during a routine maintenance inspection, engineers of the Highways Department observed extensive spalling of concrete on the piers and rust staining on the arches. Ove Arup and Partners, Hong Kong Limited, were commissioned in August 1991 to investigate the extent of the degradation and the possible causes of the deterioration to appraise the structural adequacy of the bridge and subsequently, to prepare and administer the contract for the recommended repair.

The investigation took about a year, followed by establishment of the rehabilitation contract. The rehabilitation works was completed in October 1994.

## **2 Investigation**

### **2.1 Investigation plan**

In order to conduct a thorough but cost-effective investigation on the material condition, a phased programme of preliminary inspection, primary testing and typical testing was adopted. The investigation started with the preliminary inspection, which consisted of several site visits made by experienced engineers to visually assess the condition of the piers, the arches and the foundation piers. Sounding tests were also carried out on selected areas of the bridge structure. It was concluded from the preliminary inspection that the arches were in a relatively satisfactory condition, possibly because of the relatively higher grade concrete used than the piers, hence less investigation followed. On the other hand, the piers all appeared to be in poor condition, although they were exposed to varied intensity of wind and rain in their respective positions. Extensive testing was therefore planned for the piers. In order to optimise the quantity of tests, Pier 1 was selected for primary testing because of its lower access cost due to its land siting. Reduced amount of tests were carried out to the other piers which served to confirm test data obtained from Pier 1 and to reveal the variation in the condition of the individual piers. Provision was also made to allow indicative testings for the foundation piers to provide a comprehensive review on the existing state of the whole structure.

### **2.2 Investigative methods**

Several non-destructive site testing methods were adopted. They were supplemented by laboratory tests carried out on samples retrieved from site to give physical, mechanical and chemical data of the existing concrete. Site testing and sampling methods adopted for this project included:

1. Visual survey;
2. Sounding test;
3. Carbonation test;
4. Reinforcement cover measurement;
5. Half-cell electro-potential test;
6. Concrete powder extraction;
7. Coring for concrete samples;
8. Removal of cover concrete.
9. Laboratory tests carried out on the samples included:
  - Chloride content profile;
  - Sulphate content profile;
  - Compressive strength test;
  - Cement content and aggregate/cement ratio.

### 2.3 Investigation results

#### 2.3.1 Piers

1. *Visual condition:* The visual survey demonstrated that extensive spalling and cracking of concrete existed in large areas on random positions of the piers, with the greatest damage occurring on Pier 2 (Fig. 2). In all cases the damage consisted of delamination and spalling of the concrete from the horizontal link reinforcement which was fairly extensively corroded. In most cases the main reinforcement was not severely corroded. A thermo-graphic survey was carried out for Pier 1 to confirm the visual survey and sounding test results. On the internal surfaces of the piers the extent of spalling and delamination was very much reduced (about 10% average).

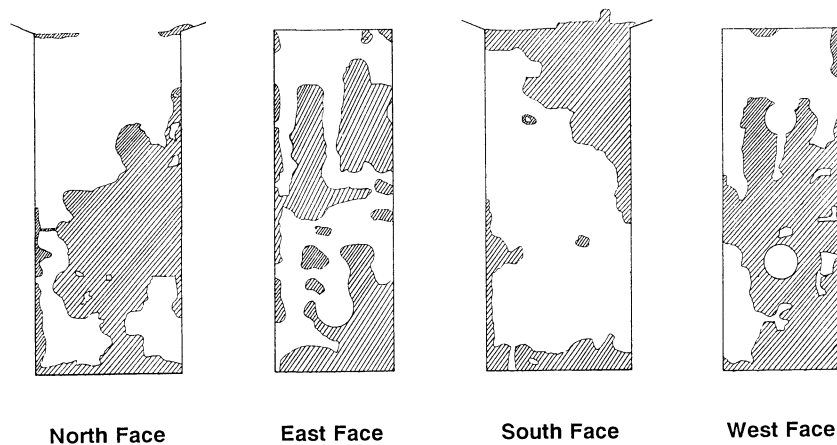


Fig. 2. Delaminated areas of Pier 2.

2. **Compressive strength and cement content:** The specified 28 day cube strength for the pier concrete was 3000psi (20MPa). Cores extracted revealed a mean concrete strength of 32MPa with a minimum value of 20MPa. The cement content at 18% was generally consistent with the strength of concrete in the core sample.
3. **Carbonation depth:** The depth of carbonation varied considerably throughout the pier structures giving mean values between 19 mm and 37 mm (Fig. 3). There was no direct correlation between the position of the sample and carbonation. However, there was a trend that the pier with the highest core strength and cement content also had the lowest mean carbonation depth.
4. **Chloride content:** This was determined by taking drillings at incremental depths of 5-50 mm, 50-100 mm and 100-150 mm into the concrete. On the external faces of the piers nearly every set of drillings showed decreasing chloride content with depth from the surface (Fig. 3). On the internal faces, this profile was not so marked and the chloride values were much lower.

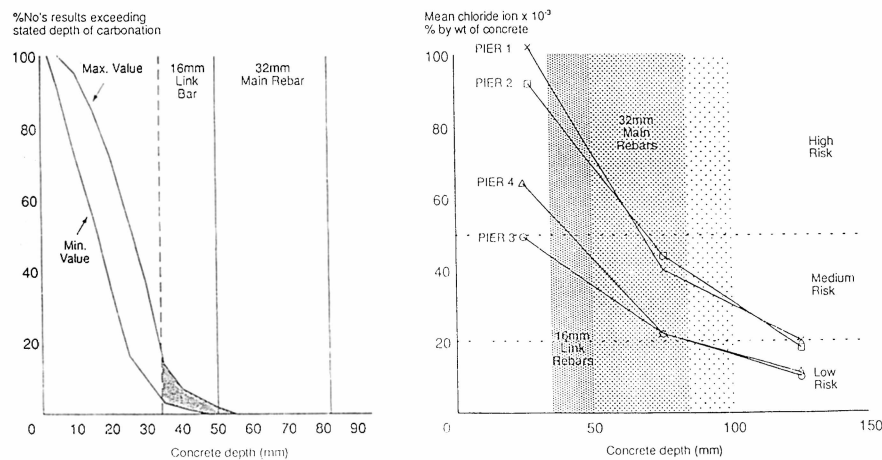


Fig. 3. Extent of carbonation and chloride ingress.

5. **Sulphate content:** The results indicated that sulphate had not penetrated into the concrete from the external environment.
6. **Steel bar condition:** By exposing the embedded steel bars in delaminated areas, the links of the piers were found to be highly corroded with over 50% of the steel area lost in the most severe case. However, it was found that most of the main bars suffered mild corrosion only with a minimal loss of cross-sectional area. For areas not delaminated, it was concluded that steel bars were not suffering from severe corrosion, as the readings in half cell electro-potential tests were generally less negative than expected for severely corroding steel.

### 2.3.2 Arches

There was no wide scale delamination and corrosion although localised reinforcement corrosion and concrete spalling was noted. The specified 28 day cube strength was 4,500 psi (31MPa). The mean core strength was 49MPa and the lowest value was 37MPa. The mean cement content was 20%. Both carbonation depth and chloride content were very low. It was also apparent that the reinforcement was in good condition with minimal loss of cross section.

### 2.3.3 Foundation piers

The compressive strength of the extracted concrete cores had a mean value of 48MPa. The chloride content was found to be high for the top 50 mm cover. However, concrete cover to the reinforcement was found to be about 100 mm and carbonation was found to be limited to within the top 10 mm.

## 2.4 Durability Assessment

The external surface of the piers were suffering from advanced deterioration of concrete cover and rusting of link reinforcement. Carbonation had reached the depth of the link steel which thus became de-passivated. Chlorides had penetrated to the depth of the link steel in concentrations sufficient to cause rapid corrosion. It appears that the durability of this outer layer was inadequate.

The concrete of the arches were exhibiting only minor chloride contamination probably because this concrete was of higher quality and thus more able to prevent penetration and absorption.

The concrete of the foundation piers was found to have a high compressive strength and the reinforcement was protected by sufficient cover. Although it was contaminated by high level of chloride at the surface, the contamination did not progress deep enough to reach the reinforcement.

## 3 Rehabilitation

### 3.1 The repair work

In order to give a long term future life and relatively little maintenance to the structure, the entire concrete surface to a depth of 100 mm was removed from all the pier external surfaces even in areas where the concrete appeared to be "sound". This exposed all the horizontal link steel and the outer layer of main reinforcement. Although there were areas of sound concrete, significant levels of chloride and corrosion of reinforcement might have already commenced and cause spalling later. Any corroded link steel was cut away and replaced. The exposed main reinforcement was thoroughly cleaned by water grit blasting and protected by a zinc-rich primer, followed by the recasting of the cover using a proprietary flowable grout concrete with mechanical material properties similar to the substrate and having excellent resistance to chloride ion penetration.

For the arches and internal surface of the piers, a sounding survey was performed to identify the delaminated areas. The concrete cover was then removed to a minimum depth of 15 mm beyond the reinforcing bars. After the bars were cleaned by water grit blasting, a bonding coat was applied to the bars immediately before the

replacement of concrete cover using a polymer modified mortar, followed by days of careful curing.

A surface protective coating was applied to the arches and the piers after the repairs to provide further protection against carbonation and chloride penetration.

### 3.2 Structural appraisal

To ensure that an adequate safety factor could be maintained throughout the repair works, an appraisal on the structural soundness of the bridge, in particular the piers, was performed by checking the record calculations against the as-built drawings. The only highway loading considered in the original design was HA type loadings to BS153: Part 3A. This was found to be comparable to the HA type loadings to BS5400: Part 2 for the same carriageway geometry.

The original design calculations show that the typical pier cross section had reached a stress level up to about 71% permissible stress in the concrete and 24% permissible stress in the mild steel reinforcement under critical load condition.

The removal of the 100 mm thick concrete cover would weaken the piers. As the bridge was required to safely support the traffic on top during repair and resist typhoon loading, a two stage repair was proposed. The surface of each pier was divided into the corner region and the central region. The corner region was the quarter width measured from the edge of the four vertical faces of each pier. The central region was the remaining portion at the centre of the four vertical faces. Stage one repair included the replacement of the outer layer of 100 mm thick concrete at the corner region. Stage two, which was the replacement of the outer layer of 100 mm thick concrete at the central region, would commence after the newly replaced concrete at the corner region had attained the specified strength.

To verify this two stage proposal, two pier breakout sections to resemble the piers were considered in the checking of stresses during repair (Fig. 4).

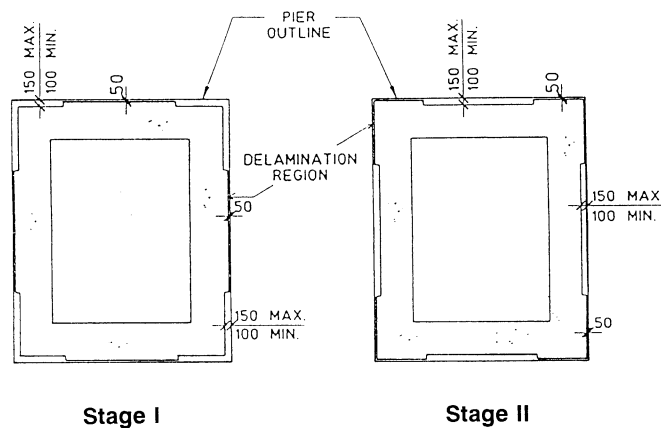


Fig. 4. Bridge pier repair stages.

The first breakout section was the pier section with the outer 150 mm thick concrete removed in the corner region to simulate the case when a minimum of 100

mm thick concrete was to be removed. A further 50 mm thick layer of concrete was discounted in the central region of the breakout section, as the 50 mm cover of the piers was found to be mostly ineffective structurally. A 4 mm reduction in the 32 mm diameter main reinforcement bars was also considered in the checking section to allow for the worst corroded condition. Similar checking procedure was carried out for the central breakout section. The calculations showed that during repair work, the working stresses in the piers would reach 92% permissible stress in concrete and 39% permissible stress in main steel reinforcements under critical load conditions.

At the ventilation openings, the pier cross section is reduced and the stresses are much higher than those in the typical section. Temporary strutting to these openings (Fig. 5) was therefore employed during the whole course of repair. Restriction on HB type vehicles using the bridge was imposed during repair, as they were not allowed for in the original design.

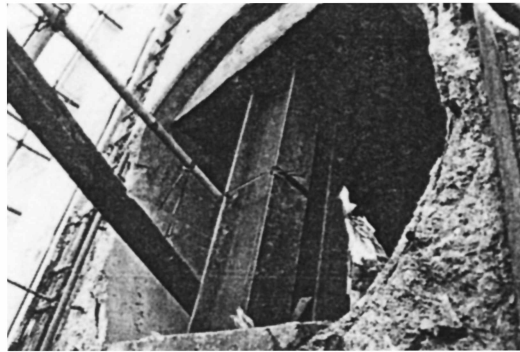


Fig. 5. Temporary strutting to ventilation openings.

### 3.3 The repair contract

The repair works described above were administered by a Bills of Quantities re-measurement contract, as the quantity of patch repairs needed verification on site. The contract was tendered by specialist contractors for concrete repairs in the Government's restricted list, since the repair called for specialised skills.

Quality of material and workmanship, which were the key to the success of the repair, were monitored by checklists of actions and acceptance criteria to be followed by the contractor. The actions included the preparation works, the repairs, and the tests to be performed. The checklists were signed by a qualified Quality Control Engineer when each action was performed satisfactorily.

In order to minimise damage to the bridge in the event of a severe accident which might severely disrupt the operation of the bridge, the piers were repaired in alternate spans. In this way, the weaker piers under repair were assisted by the adjacent stronger piers through shear transfer at the tip of the deck cantilevers. The sequence of repair based on this principle was:

1. *Phase I*: Repair for Pier 1, Pier 3 and Pier 5, with Pier 5 completed first to avoid disturbing the peak operation season of the container terminal.

2. *Phase II*: Repair the remaining Pier 2 and Pier 4 after the repair for the first three was completed.

Patch repair for the pier internal surfaces could be carried out immediately after repair concrete of the external face of the same pier had reached the specified strength. Patch repair for the arches could be carried out at any time before the final protective coating was applied.



Fig. 6. The bridge under repair.

#### 4 Conclusion

In Hong Kong many civil and building structures are constructed of reinforced concrete and exposed to hot and humid weather. In some cases the environment is particularly severe and structures may deteriorate faster. Although this bridge is a fine example of how a deteriorating structure can be assessed and restored through thorough yet cost-effective investigation and repair, it is always best to consider durability and maintenance at an early stage in the design process in order to provide adequate durability and ease of maintenance.

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# 10 REHABILITATION OF A HISTORIC BRIDGE OVER THE SAND RIVER NEAR VIRGINIA, SOUTH AFRICA

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## Abstract

During February and March 1988, the Free State province of South Africa was hit by devastating floods. The old Sand River arch bridge near Virginia in the Free State, which was built in 1926, was totally inundated during the floods and apart from the approaches which were washed away the bridge super-structure was severely damaged. Future planning of roads in the immediate vicinity of the bridge made provision for a new bridge over the Sand River, thus replacing the existing bridge. Closer investigation revealed that the bridge exhibited far greater inherent strength than previously believed. Through innovative design, a solution was found which obviated the construction of a new bridge and led to substantial cost savings to the client.

Keywords: Bridge, rehabilitation, river, South Africa, floods.

## 1 Introduction

During February and March 1988, the Free State province of the Republic of South Africa was hit by devastating floods. The provincial road network was disrupted to such an extent that Bloemfontein, the provincial capital, was cut off from the rest of the country for several days. Although several bridges were completely washed away, the approaches of 20 other bridges were also washed away. The biggest problem was the reconstruction of the bridge approaches to re-establish road links.

One of the bridges which was damaged during these floods was the historic Sand River Bridge No.96 near Virginia which was built in 1926. The importance of this bridge lies in the fact that it forms part of the link between the Gold Fields of the Free State and Lesotho which provides labour for the mines. Owing to a sharp increase in

traffic volumes, the route was to have been upgraded as a matter of course. The planned new road alignment also made provision for the construction of a new bridge which would have made the existing bridge obsolete.

The prohibitive cost of a new bridge resulted in a closer investigation into the feasibility of repairing and utilising the existing structure rather than building a new bridge at a greater cost.

## **2 Existing bridge**

### **2.1 General**

The existing structure was built in 1926 and consequently very little as-built information was available. No structural drawings could be located and the only drawing found was a location plan which did not even show the bridge position. The bridge is of an arch-type construction and due to its age is deemed to be historic.

### **2.2 Founding conditions**

A visual inspection of the bridge site revealed that the bridge piers were founded on sandstone rock. Sandstone was also found at various locations in the river bed in the vicinity of the bridge. Based on the visual assessment and the absence of any scour in the river bed, it could be assumed with reasonable certainty that the bridge was firmly founded on rock and further geotechnical investigations were deemed unnecessary. The estimated safe bearing pressure of the sandstone is 1000 kPa.

### **2.3 Structure**

The sub-structure consists of mass concrete piers, abutments and wingwalls. The total length of structure is approximately 90 metres, made up of nine arch spans of reinforced concrete. Mass concrete walls constructed on the sides of the arches kept the rubble infill placed on the arches in place. The single roadway was provided by placing premix on the infill and constructing sidewalks on the walls. Steel handrailings were added. The total height of the structure above the river bed was 11 metres and the bridge width was 4.6 metres with an effective roadway width of 3 metres.

An interesting feature of the bridge was the fact that the two outside openings between the abutments and the piers were closed up by means of concrete walls. The reason for this is unknown. It is presumed that the abutments needed structural support but the closure of the openings would definitely have had a negative influence on the hydraulic capacity of the structure.

## **3 Floods of 1988**

### **3.1 Extent of floods**

The flood in the Sand River was not investigated by the Department of Water Affairs as it was not deemed to be as severe as the flooding in the rest of the province. Hence it is difficult to give accurate figures. However some calculations were made and are summarised below: