Optimisation of Preventative Maintenance Strategies for Reinforced Concrete Bridges

A Thesis submitted in partial fulfilment of the requirements for the Degree of Doctor of Philosophy

By

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September 2005

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Executive Summary

Bridge structures have vital role to play in the transport system. Unfortunately, like all engineering structures they are constantly deteriorating with time due to various causes such as metallic corrosion. A well-planned and implemented maintenance program can help to secure maximum service life at an acceptable cost. Preventative maintenance (PM) measures can often help to prevent, or postpone, the onset and progress of deterioration. However, there are currently significant uncertainties associated with the need for these maintenance actions and their degree of effectiveness. The aim of this project is to develop a methodology, which will set a basis for finding optimum PM strategies while incorporating the uncertainty of their effectiveness.

A review study was undertaken first to study the state of the art in the area of bridge maintenance management particularly in the UK and optimisation techniques. Based on the findings of this study a probabilistic procedure has been developed to incorporate the uncertainties and estimate the effectiveness of various PM measures by producing probability of failure ($p_f$) profiles related to the initiation of corrosion. To link the $p_f$ profiles of PM with their cost, and produce optimum PM strategies, an optimisation methodology using the principle of Genetic Algorithm (GA) has been developed. A spreadsheet - program was created using Excel XP and Visual Basic Programming 6 (VBA 6), to demonstrate the methodology and enable analyses to be performed efficiently. This provides a tool capable of identifying optimum PM strategies for Reinforced Concrete (RC) bridges, by delaying the onset of, or stopping, the corrosion process due to the ingress of chloride based de-icing salt. Sensitivities studies were also performed to identify significant trends and assess the relative importance of the various parameters involved.

The results are promising since optimum PM strategies can be produced even though these are dependent on the input data. In order to produce more realistic strategies and apply this methodology effectively, there is a need to enhance the quantity and quality of the available data. It is concluded that, these strategies can be used as an aid to gain better insight into the effectiveness of PM measures, and their use in planning the protection of bridge element from chloride ingress.
Dedication

To my beautiful family
Acknowledgement

I would like to express my sincere gratitude to my supervisor Dr Toula Onoufriou for her supervision and support throughout the research. Her infectious enthusiasm, depth of knowledge and interest for the work never ceased to amaze me and were a constant source of my own growth and understanding. I would also like to thank my co-supervisor Dr Mike Mulheron for sharing his encyclopedic knowledge on concrete technology and giving constructive comments and advice.

The funding provided by the Highways Agency (HA) and Transport Research Laboratory (TRL) is gratefully acknowledged. In particular, the enthusiasm and encouragement of Dr Victoria Hogg (HA), Mr Martin Potts (HA), Mr Awtar Jandu (HA) and Dr Richard Woodward (TRL) is much appreciated.

I want to thank all my colleagues, past and present, in the research rooms 12BB02 and 6AA02, who have been great fun and support from the first day. In no special order I would like to thank Julie Bregulla, Guanghui Li, Alexis Panagoulopoulos, Nick Photiou, Jo Sungil, Imran Rafiq, Bulent Iman, Lefteris Aggelopoulos, Ian Hackman, Nancy Katzantzi, and Renos Votsis. Special thanks to all my personal friends around the world. Gratitude is also due to Tony Thorne and Alan Smith for their help and patience with computing matters. Special thanks to the administration ladies, Bryony Turner, Penny Briggs and Jacky Halliday for their help and support.

Finally, I am truly grateful for having such a wonderful family. I am indebted to my amazing parents, Andrea and Giannoula, who have provided a positive environment filled with direction, encouragement, motivation, and love in which a rather shy kid could develop the confidence and determination to fulfil her dream. They never stopped believing in me and supporting me, both mentally and financially. I also thank my lovely sister Maria for always being there for me, encouraging me to go all the way and for her unconditional love. I thank my brother Niko, his wife Elena and their two marvellous kids Ioanna and Alexandra for bringing such a joy in my life.

I also wish to express my deepest gratitude to Renos, for his love, support, encouragement and patience thought the course of this work. Without him, this thesis would not have been completed.
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<thead>
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<th>Notation</th>
<th>Description</th>
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<tbody>
<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers</td>
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<tr>
<td>BM</td>
<td>Bridge Management</td>
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<tr>
<td>BMS</td>
<td>Bridge Management System</td>
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<td>BRIME</td>
<td>Bridge Management in Europe</td>
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<tr>
<td>CEB</td>
<td>Comite Euro-International Du Beton</td>
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<tr>
<td>CoV</td>
<td>Coefficient of Variation</td>
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<tr>
<td>CP</td>
<td>Cathodic Protection</td>
</tr>
<tr>
<td>CR</td>
<td>Concrete Replacement</td>
</tr>
<tr>
<td>DMRB</td>
<td>Design Manual for Roads and Bridges</td>
</tr>
<tr>
<td>ECE</td>
<td>Electrochemical Chloride Extraction</td>
</tr>
<tr>
<td>FHWA</td>
<td>Federal Highway Authority</td>
</tr>
<tr>
<td>FORM</td>
<td>First Order Reliability Method</td>
</tr>
<tr>
<td>GA</td>
<td>Genetic Algorithm</td>
</tr>
<tr>
<td>HA</td>
<td>Highways Agency</td>
</tr>
<tr>
<td>MCS</td>
<td>Monte Carlo Simulation</td>
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<tr>
<td>OECD</td>
<td>Organization for Economic Cooperation and Development</td>
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<td>PC</td>
<td>Portland Cement</td>
</tr>
<tr>
<td>PM</td>
<td>Preventative Maintenance</td>
</tr>
<tr>
<td>RC</td>
<td>Reinforced Concrete</td>
</tr>
<tr>
<td>SORM</td>
<td>Second Order Reliability Method</td>
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<tr>
<td>TRL</td>
<td>Transport Research Laboratory</td>
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<tr>
<td>WLC</td>
<td>Whole Life Costing</td>
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<tr>
<td>WS</td>
<td>Waterproofing System</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Reliability Index</td>
</tr>
<tr>
<td>CoV</td>
<td>Coefficient of Variation</td>
</tr>
<tr>
<td>sd</td>
<td>Standard deviation</td>
</tr>
<tr>
<td>ipop</td>
<td>Initial population</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>---------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$N_{\text{pop}}$</td>
<td>Number of chromosomes in the initial population</td>
</tr>
<tr>
<td>$N_{\text{pop}}$</td>
<td>Number of chromosomes in the population in every generation</td>
</tr>
<tr>
<td>$N_{\text{par}}$</td>
<td>Number of parameter</td>
</tr>
<tr>
<td>$N_{\text{gene}}$</td>
<td>Number of bits in the chromosome (binary encoding)</td>
</tr>
<tr>
<td>$N_{\text{good}}$</td>
<td>Number of chromosomes in the mating pool</td>
</tr>
<tr>
<td>Parent$_n$</td>
<td>A chromosome selected for reproduction</td>
</tr>
<tr>
<td>Offspring$_n$</td>
<td>Chromosome created from the mating two parent chromosomes,</td>
</tr>
<tr>
<td>X$_\text{rate}$</td>
<td>Crossover rate</td>
</tr>
<tr>
<td>M$_\text{rate}$</td>
<td>Mutation rate</td>
</tr>
<tr>
<td><strong>Terminology</strong></td>
<td></td>
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<td>--------------------------------------------------------------------------------------</td>
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<tr>
<td>Anodic reaction</td>
<td>A process involving loss of electrons.</td>
</tr>
<tr>
<td>Algorithm</td>
<td>A finite set of well-defined rules for the solution of a problem in a finite number of steps.</td>
</tr>
<tr>
<td>Absorption</td>
<td>The process whereby the concrete takes in a fluid to fill spaces within the material.</td>
</tr>
<tr>
<td>Adsorption</td>
<td>The process in which molecules adhere to the surface of the concrete.</td>
</tr>
<tr>
<td>Alkali-Silica Reaction</td>
<td>The reaction between the alkalies (sodium and potassium) in portland cement binder and certain siliceous rocks or minerals, such as opaline chert, strained quartz, and acsic volcanic glass, present in some aggregates; the products of the reaction may cause abnormal expansion and cracking of concrete in service.</td>
</tr>
<tr>
<td>Bridge</td>
<td>A bridge is a structure spanning and providing passage over a river, chasm, traffic intersection area, fjord, inlet or other physically obstacles and with a span length equal to or exceeding a certain distance.</td>
</tr>
<tr>
<td>Cathodic reaction</td>
<td>The process involving electrons gain.</td>
</tr>
<tr>
<td>Component</td>
<td>A non- monolithic part of a structure, such as a bearing or expansion joint.</td>
</tr>
<tr>
<td>Corrosion</td>
<td>The deterioration of materials by chemical interaction with the environment.</td>
</tr>
<tr>
<td>Chromosome</td>
<td>Any of the rod-like structures found in all living cells containing the chemical patterns which control what an animal or plant is like.</td>
</tr>
<tr>
<td>Deterministic method</td>
<td>Calculation method in which basic variables are treated as non-random.</td>
</tr>
<tr>
<td>DNA</td>
<td>The genome of an organism — its pool of hereditary material— is composed of an organic molecule called Deoxyribonucleic acid.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
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<tr>
<td>Element</td>
<td>A functional part of a highway structure, such as a beam or a deck.</td>
</tr>
<tr>
<td>Element (or component)</td>
<td>The analysis where the reliability of a single element (or component) in a structure is assessed.</td>
</tr>
<tr>
<td>Management Plan</td>
<td>A plan prepared by the Maintaining Authority which includes a schedule of all maintenance activities on a structure for the foreseeable future.</td>
</tr>
<tr>
<td>Mutation</td>
<td>Mutation is the way in which genes change and produce permanent differences.</td>
</tr>
<tr>
<td>Genes</td>
<td>Genes are lengths of DNA that carry information for a particular trait.</td>
</tr>
<tr>
<td>Genome</td>
<td>The entire complement of genetic material in a chromosome set.</td>
</tr>
<tr>
<td>Homogeneous</td>
<td>Material that have the same composition at every point (heterogeneous; antonym).</td>
</tr>
<tr>
<td>Isotropic</td>
<td>Materials having the same properties in all directions (whether axial, lateral, or any other direction) (anisotropic; antonym).</td>
</tr>
<tr>
<td>pH</td>
<td>The logarithm of the reciprocal of hydrogen-ion concentration in gram atoms per litre; provides a measure on a scale from 0 to 14 of the acidity or alkalinity of a solution (where 7 is neutral and &lt;7 is acidic and &gt;7 is basic).</td>
</tr>
<tr>
<td>Present Value Cost</td>
<td>A parameter that allows the discounted costs associated with maintenance proposals to be compared at a common point in time.</td>
</tr>
<tr>
<td>Probabilistic method</td>
<td>Calculation method in which the basic variables are treated as random.</td>
</tr>
<tr>
<td>Real value encoding system</td>
<td>Encode directly integer or real numbers.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>---------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Reliability</td>
<td>The probability that a devise will perform its purpose adequately for the period of time intended and under the operating conditions encountered. Covers safety, serviceability, and durability of the structure.</td>
</tr>
<tr>
<td>Single point crossover</td>
<td>One crossover point is selected; from the beginning of chromosome to the crossover point is copied from one parent; the rest is copied from the second parent.</td>
</tr>
<tr>
<td>Standard roulette rule</td>
<td>Parents are selected according to their fitness. The better the chromosomes are, the more chances they have to be selected.</td>
</tr>
<tr>
<td>Structural reliability</td>
<td>The ability of a structure to fulfil its design purpose for some specified time.</td>
</tr>
<tr>
<td>Substructure</td>
<td>Bridge elements below bearing level such as abutments, wing walls and piers.</td>
</tr>
<tr>
<td>Superstructure</td>
<td>Bridge elements above foundations such as deck, cables and beams.</td>
</tr>
</tbody>
</table>
Chapter 1

Introduction and layout of the thesis

1.1 Incentive for the study

It has been well said that "to the civil engineer the word 'impossible' is scarcely permitted. His professional duties call upon him to devise the means for surmounting obstacles of the most formidable kind. He has to work in the water, over the water, under the water; to cause streams to flow; ..... ; to construct chimneys that rival the loftiest spires and pyramids in height; to climb mountains with roads and railways; to sink wells to vast depths in search of water. By untiring patience, skill, energy, and invention, he produces in these several ways works which certainly rank among the marvels of human power" (Warne, circa 1877).

One of these wonders is the creation of bridge structures. However, like all engineering structures the bridges around the world are constantly deteriorating with time due to a range of causes such as metallic corrosion. The closure, or malfunction, of a bridge will represent a failure of serviceability and result in major financial problems, both locally and nationally. As deterioration accumulates and traffic requirements increase, such events can happen more frequently. Millions of pounds are spent every year to maintain the bridge stock, avoiding the realisation of such events. Although bridges cannot last forever, maintenance can help to secure their maximum service life. It has been suggested that preventative maintenance (PM) can both help to postpone essential rehabilitation work, and extend the service life of bridges in a cost effective manner. On the other hand, depending on the situation, it is sometimes more expensive to maintain deteriorated bridges than to build new ones.
Therefore, the reduction of maintenance cost is a challenge that must be met in future bridge maintenance planning.

In the UK there are about 155,000 bridges, which are maintained by the Highways Agency (HA), local authorities and private owners such as Railtrack, London underground and British Waterways (Mallett, 1986). Eighty percent (= 8,790 bridges) and thirty four percent (=43,860 bridges) of the bridge stock owned by HA and local authorities respectively, are built predominantly from concrete (Mallett, 1986; Department of Transport, 1987; Daly, 1999). Concrete is nearly always reinforced with steel to strengthen its naturally brittle behaviour. The most dominant cause of reinforced concrete (RC) highway bridge deterioration in the UK is corrosion of reinforcement due to chloride ion ingress, supplied by de-icing salts (Wallbank, 1989). In the UK alone, it is estimated that the annual cost of repairs of concrete structures due to steel corrosion exceeds £500 million (Hobbs, 1996). The corrosion of reinforcing steel in concrete structures is a global problem that has detrimental effect in many countries. For example in the USA the annual direct cost of corrosion for highway bridges is estimated to be $8.3 billion. However there are 583,000 bridges in the United States with some 235,000 bridges being manufactured from conventional reinforced concrete. Indirect costs to the user (e.g. traffic delays) were estimated to be tenfold that of direct costs (Koch et al., 2001).

Current maintenance strategies are often formulated on the basis of the condition of the bridge rather than its reliability (structural safety). Therefore uncertainties associated with the deterioration are not taken into account. With the use of probabilistic methods these uncertainties can be incorporated in the best possible way. Maintenance measures can be applied to ensure that the bridge reliability remains below an acceptable probability of failure, i.e. the probability that the structure will reach a specific limit state during its service life.

The adage “prevention is better than cure” is particularly true for bridges that, due to detrimental factors the safe and efficient operation of the bridge, maybe, at some point in the future, in danger if action is not taken now. This proactive management approach may not be essential now but it can be justified on economic grounds since it can enable the postponement of essential rehabilitation work.
The constantly growing demand for reliable bridges, with low maintenance cost has fuelled the need to investigate the use of PM measures for extending the service life and reducing the whole life cost of highway bridge structures.

While the effectiveness of different PM measures has been addressed before, mostly qualitatively, at the time of initiation of this project there was little information on, and assessment of, the quantitative degree of effectiveness of PM and optimum PM strategies against the common forms of deterioration cause encountered in real structures. As a step towards understanding the complicated behaviour of PM a probabilistic approach has been used to address, in a quantitative manner, its effectiveness while an optimisation method (genetic algorithm) is utilised for developing a tool for identifying optimum PM strategies.

1.2 Objective of the thesis

The objective of this research is to develop a procedure that will enable the incorporation of uncertainties associated with the need, and effectiveness, of PM measures related to the critical deterioration mechanism of RC bridges. Furthermore, it seeks to create a methodology, using a whole life optimisation process that will provide a tool for the development and comparison of time-dependent optimum PM strategies with more confidence. This research will:

- Identify the most critical deterioration mechanism of RC bridges.
- Examine the parameters involved in the deterioration mechanism.
- Assess the effectiveness of PM measures on the deterioration mechanism.
- Ascertain the appropriate limit state function for the application of PM measures.
- Develop a methodology that will incorporate the above for the development of optimum PM strategies by using genetic algorithm (GA) principles.
• Identify the data required for the GA methodology to become more beneficial. Improved data will provide the basis for selection of more realistic optimum strategies.

• Provide general guidelines on the use of such methodology for RC bridges.

### 1.3 Basic approaches in the study

Due to the large amount of uncertainties influencing the effectiveness of PM measures a probabilistic-based approach was considered to be the rational way to estimate the degree of effectiveness for various PM measures. In general the probability of failure (initiation of corrosion) is calculated as a function of time to illustrate the effectiveness of PM. For the probabilistic-based reliability analysis Monte Carlo Simulation method (see Appendix A) is used in this study due to its simplicity of application in solving such problems.

However, it is not enough to estimate the effectiveness of PM. To aid engineers to make decisions regarding the timing of the application of these measures it is also necessary to identify optimum PM strategies with minimum whole life cost. GA principles are utilised to develop the optimisation methodology for this study.

The probabilistic procedure and GA methodology developed is intended to be used for a specific bridge type and deterioration mechanism. However, the approach developed here is generic and can be developed further for use on different deterioration mechanisms, other types of bridges and bridge networks.

### 1.4 Outline of the thesis

This thesis contains seven chapters, five appendices and a list of references. Assumptions related to the formulation of the methodologies developed in this study are stated when they are encountered.

Following on from this introduction Chapter 2 presents a literature review. It discusses the bridge management and overview of the most commonly used bridge management systems around the world. Also this section describes the general study
for maintenance strategy on bridges, the probability-based analysis methods and optimisation techniques available for producing optimum strategies for bridges.

The scope of this PhD project is focused to RC bridges. Chapter 3 highlights the deterioration mechanisms of RC bridges and focuses on the corrosion of steel reinforcement due to chloride ion ingress into the concrete cover. The last section of the chapter presents recommendation for preventing the initiation of corrosion of RC bridges with the use of specific PM measures.

Chapter 4 presents, in detail, the procedure developed, using probabilistic techniques, to enable the prediction of the effectiveness of different PM actions. The chloride ion ingress model used permits the comparison of different PM interventions with respect to their effectiveness to reduce or stop the probability of initiation of corrosion. Uncertainties that influence the degree of PM action’s effectiveness are incorporated. The outcome is expressed as a probability of failure ($p_f$) with respect to the proposed limit state. A parametric study is performed to assess the sensitivity of the various model parameters, and their relative contribution, to the overall uncertainty of the degree of effectiveness of PM measures.

The key findings from Chapter 3 and 4 are then collated and incorporated in the development of an optimisation methodology. Chapter 5 presents the proposed optimisation methodology that is based on the principles of a genetic algorithm. This methodology provides a tool to identify optimum PM strategy for RC bridge elements. It is implemented with the use of an Excel program and illustrated through an example (Pilot) case. Furthermore a sensitivity study is carried out to examine and identify key parameters that affect the efficiency of the developed program.

Chapter 6 demonstrates the applicability of the developed GA methodology proposed in Chapter 4 and Chapter 3 through 24 case studies. The benefits from the potential application of the methodology are presented.

Chapter 7 concludes the findings, proposes some guidelines and learning points for producing optimum PM strategies as well as giving an outlook on future work.
Work presented in Chapter 4, 5 and 6 is novel. Chapter 3 contains elements of originality such as the categorisation of the effectiveness of different preventative maintenance applied to RC bridge elements.
Introduction and layout of the thesis

Chapter 1: Introduction

Chapter 2: Literature review
- Review of existing approaches to bridge management
- Effectiveness of maintenance measures and maintenance strategies
- Reliability analysis
- Whole life cost analysis
- Optimisation procedures

Chapter 3: Deterioration of RC bridges and related PM measures
- Basic theory of corrosion of reinforcement due to chloride ingress
- PM measures for corrosion due to de-icing salts
- Categorisation of the effectiveness of PM measures

Chapter 4: Development of probabilistic analysis procedure to illustrate effectiveness of PM
- Probabilistic analysis procedure for illustrating the effectiveness of PM
- Sensitivity analysis of the probabilistic variables of the model
- Probability of failure profiles based on the effectiveness of different PM measures

Chapter 5: Methodology for optimisation of whole life strategy for PM measures using GA
- Basic theory of genetic algorithm
- GA methodology for the optimisation of PM strategies

Chapter 6: Case studies examined
- Validation cases for the GA methodology
- Optimum PM strategies

Chapter 7: Conclusions

Figure 1.1: Framework of the thesis
Chapter 2

Literature review

2.1 Introduction to literature review

To prepare for this research work it has been necessary to survey the available literature in a number of areas. The review study aimed to identify, and assess, the current thinking in the generic area of the importance and effectiveness of preventative maintenance (PM) measures, as well as use sequence of application with the aid of optimisation methods. This chapter presents the findings to support the development of a methodology that will quantify the effectiveness of PM measures and apply them in an optimum approach.

Bridges of all types are associated with various forms of degradation. Since bridge structures are generally an important link within; and the most expensive element, of any highway network, highway authorities are obliged by law to maintain them. In the UK there are about 155,000 bridges, which are the responsibility of the Highways Agency (HA), local authorities and private owners such as Railtrack, London Underground and British Waterways (Mallett, 1986). The value of the bridges on the national network, owned by the HA is estimated at 23 billion Euros and typically comprise about 2% of its length and 30% of its value (Woodward et al., 2001). Millions of pounds are spent every year to maintain the bridges. Approximately 225 million Euros are spent for the 9,500 of HA’s bridge stock (Woodward et al., 2001). However, whilst the costs of maintenance are increasing constantly, funding is often inadequate (Cole, 2000). It has been suggested (Das, 1999; Frangopol et al., 2000) that PM can help to reduce the total cost of maintenance needed for the bridge useful life.
This literature review starts by giving a general introduction into the broad area of bridge management. With regard to bridge management the review focuses on bridge maintenance management and particularly to the importance of PM. Throughout this review it is clear that although some researchers (Vassie & Arya, 2000; Liu & Frangopol, 2004) have addressed the behaviour of PM very little information, mostly qualitative, exists on the effectiveness of PM especially in relation to specific deterioration mechanisms. Since, there are a lot of uncertainties (Das, 1999a, Das & Onoufriou, 2000a) concerning the applications and assessment of PM, a subsequent section of the literature survey presents the reliability method which has been employed in various aspects of bridge management. This information is directly relevant to this thesis and is essential in appreciating, and quantifying, the effects of PM measures. The final section reviews several optimisations methods currently used. Also particular applications of optimisation techniques for developing optimum maintenance/ repair/ inspection bridge strategies are briefly presented and discussed.

### 2.2 Bridge management

Bridge management (BM) philosophy was developed to look after a bridge or bridge stock from conception to the end of its service life (§ 2.7.1.3). It appears to have its roots as early as the Roman times. Not only had the Roman emperors “sent architectures, engineers, and skilled workmen to give supervision and guidance” during construction but “the builders of the bridges were held responsible for their stability for forty years, after which time they were repaid the deposit which had been required of them” (Shirley-Smith, 1964). This philosophy is been used as a means of managing better available funding while ensuring that bridges remain functional and safe throughout their life.

#### 2.2.1 Bridge management objectives

The term 'bridge management’ addresses all the activities necessary to ensure that a bridge structure remains fit and safe its entire life, on a daily basis management, such as inspections, testing, monitoring, and assessment of load capacity. From the results of these activities, it is possible to prioritise maintenance, repair, or replacements...
schemes. Bridge management key objectives can summarise below (Johnstone & Brodie, 1999; Das, 2000; Klatter & Van Noortwijk, 2003):

- Functional: To maintain the integrity and quality of the bridge stock, to provide sufficient repair and maintenance so as to ensure the bridge is in a serviceable condition until rehabilitation or replacement, to minimise disruption to users whilst undertaking management activities, to manage effectively operational programs and tuning them with other programs such as pavement management.

- Safety: To ensure public safety when the bridge is in use.

- Aesthetics (Svensson, 2000): To maintain an acceptable level of the appearance of the bridge.

- Sustainability: To program management activities (e.g. maintenance) in such a way that will meet the needs of the present generation but with future maintenance activities remaining manageable and without accumulated backlogs.

- Economic: To allocate, on an optimum basis, the available funding into the different necessary activities.

2.2.2 Bridge management levels

There are two primary bridge management levels:

- project level (management of individual bridge)

- network level (management of a stock of bridges).

Project level bridge management, deal with the information retrieved from the bridge management activities (Figure 2.1) for individual bridges, elements or components. The condition of the bridge is determined. The budget is based on the activities necessary to improve or maintain the condition of the particular bridge.
Network level bridge management, deals with the information retrieved from the bridge management activities (Figure 2.1) for the entire bridge stock or subsets of the entire stock (bridges in a given region). In this category the condition of the bridges is established based on the whole network. The required budget is determined based on the activities necessary for improving, or maintaining, the current condition of the network.

The activities involved by the two levels of bridge management have been defined by the following flow chart (Figure 2.1) (Das & Onoufriou, 2000a).

![Bridge Management Activities Flow Chart](image)

**Figure 2.1: Bridge management activities and relative accuracy required (Das & Onoufriou, 2000a)**

### 2.3 Bridge Management Systems

Until the end of the 20th century bridge authorities and governments failed to appreciate the benefits from forward planning at the design and construction phase of the need for regular maintenance and inspection during bridge service life. Therefore
bridge engineers of the 21st century have to deal with deteriorating bridges that either need repairing, strengthening, weight restrictions or even replacement. Unfortunately, poor 'health' conditions of bridges result into weakened bridges which in some cases caused fatal accidents. To prevent the reappearance of such accidents in the future, researchers around the world are trying to find a mechanism which can best ensure the implementation of the objectives of bridge management. This mechanism is called Bridge Management System (BMS) (Ryall, 2001).

According to the American Association of State Highways (AASHTO) BMS consists of “formal procedures and methods for gathering and analysing bridge data for the purpose of predicting bridge conditions, estimating network maintenance, repair and rehabilitation (MR & R) needs, determining optimal policies, and selecting projects and schedules within budget and policy constraints” (AASHTO, 1993).

The first generation of BMS was used to replace the rather inconvenient manual filling systems (e.g. card index systems; Ryall, 2001). Data from inventory and field inspections could be stored and process using for example statistics techniques for the type or age of the bridges inscribed. Such UK BMS are the National Structures Database (NATS; DoT, 1993), and BRIDGEMAN (§ 2.3.3). The second generation of BMSs have more advanced features in comparison with the first generation systems such as the ability to optimise maintenance actions (Flaig & Lark, 2000). The UK BMS HiSMIS (§ 2.3.3) can be categorised in this group.

As Blakelock (1993) state “the real power of any BMS lies in the ability to process information and produce output”.

2.3.1 The structure of BMSs

A number of BMSs are being developed in various countries. Even though there are differences between the developed BMS’s, due to the different needs that lead into their development, the basic components of all BMSs are essentially the same. The traditional components of a BMS are illustrated in Figure 2.2 (OECD, 1992).
Figure 2.2: Structure of model bridge management system (OECD, 1992)

Both AASHTO (1993) and an interim final rule which was distributed in the USA on December 1, 1993, suggest that a BMS should include four basic components (Czepiel, 1995):

- Data storage: Information regarding the condition rating of bridge elements from inventory or field inspections can be collected and stored. The condition rating can be achieved with many methods:
  - deficiency rating, e.g. Japan (Yokoyama et al, 1996)
  - bridge condition rating, e.g. New York: (Yanev, 1997)
  - maintenance priority number, e.g. UK Surrey Council: (Brooman & Steele, 2001)
  - bridge condition index, e.g. UK national: (Blakelock et al., 1999)
  - condition states, e.g. USA: (Hearn, 1998).

This information can be used as an input to the modelling modules to predict future conditions of the elements and be able to optimise maintenance actions under budget limitations.
- Cost and deterioration models: Cost models can calculate the cost of possible maintenance actions or rehabilitation or even user costs. Deterioration models can predict the state of bridge elements over future time. Based on the deterioration models necessary maintenance actions can be identified. They can be either deterministic or probabilistic in nature (James et al., 1991). Five well-known time-dependent deterioration models are briefly described below.

  o Marcov model: Most of the current deterioration models use the Markovian-chain process even though it has been identified to have various limitations (Frangopol & Das, 1999; Frangopol et al., 2001). Ng & Moses (1996) states that Marcov chain predicts, probabilistically, the condition of the elements based only on current conditions (memoryless) ignoring the age of bridges where the transition rates from one state to another remain constant (homogeneous).

  o Failure rate function: As reported in Van Noortwijk & Frangopol (2003) the failure rate function $r(t)$ is defined as :

$$r(t) = \frac{f(t)}{1-F(t)} = \frac{f(t)}{F(t)}$$

where $F(t)$ is the cumulative probability distribution and $f(t)$ is the probability density function of lifetime distribution. These models found application in electrical and mechanical engineering since in these fields failure means that the component is no longer functioning. However a bridge / bridge element, or component, has a range of ‘failure’ states and therefore a failure rate cannot be measured. Based on lifetime distributions the well-known models age replacement (replacement upon failure or upon reaching a predetermined age $k$, whichever occurs first) and block replacement (replacement upon failure and periodically at times $k, 2k, 3k, \ldots$) were developed (Barlow & Proschan , 1965).

  o Regression models: The regression models (NIST/SEMATECH, 2004) that best-fit data from actual deterioration process (e.g. Fick’s law of diffusion; Crank, 1975). Unfortunately there is not always
enough real world data to support the development of this type of models.

- Stochastic process: Example of stochastic process that can be used to model deterioration is gamma process. A gamma process has a gamma distribution with independent non-negative increments. Rijkswaterstaat’s model (Van Noortwijk & Frangopol, 2004) used this deterioration process.

- Time dependent reliability index: Since engineers are now more concerned than ever about the load carrying capacity (or structural reliability) of a bridge rather than the condition states of the bridge components/elements alone (Frangopol & Das, 1999) reliability deterioration was introduced. The reliability profile \( \beta(t) \) is defined as the variation of the reliability index with time (Thoft-Christensen, 1996; Estes & Frangopol, 1996). Frangopol & Das (1999) introduces different levels of reliability. The random variables affecting the lifetime distribution of the bridge are given in Frangopol et al., 2001. From the literature it is clear that reliability based models are constantly gaining ground. Reliability is further explained in Section 2.6.

- Optimisation models: These models use the results from the deterioration and the cost models and employ whole life costing analysis (§ 2.7), or an equivalent process, to identify the optimum strategy for bridge elements or for the whole bridge structure. Optimisation can be either by the top-down, or bottom-up approach (Figure 2.3; Small et al., 1999).
Budgets and standards are used to develop optimal policies which are then used to plan projects. Feedback is provided to refine the models. Budgets and standards may be modified to perform what-if analyses.

**Figure 2.3: Alternative BMS development philosophies (Small et al., 1999)**

- Updating functions: Actual data from field measurements can be used to update deterioration and cost models. A lack of reliable collections of such data is currently noticed. Bayesian estimation (Melchers, 1999) can be used for updating estimated probabilities of future condition values.

### 2.3.2 Engineering judgement

Common to all these systems is the temptation to believe that the input and output information are absolutely correct. That can not be true at least with the methods currently available. Furthermore no computer program is currently capable of dealing with the level of intuitive decisions that are required because of the lack of data, modelling or even understanding of various processes. BMS are not developed to manage bridges but rather to provide a tool to collect and process all the available information and to produce reports to aid the engineer in the decision-making process. Therefore, sound engineering judgement is still required to make a balanced final decision on what is both reasonable and applicable in a particular situation (Ryall, 2001).
2.3.3 Current BMSs around the world

Different counties have developed different BMS to meet their constraints and requirements. In the following section particular BMSs in the UK and USA are described where other international systems are briefly analysed in relation to this study.

2.3.3.1 UK bridge management systems

BMS development in the UK appears to have been accelerated by a 15 year program which started in 1987 by the Department of Transport (UK) (Baker, 1999) to restore to a good condition bridges and other structures. This program gave a new impetus for the development of new BMS that could provide better future management of the bridge stock.

Rendel Palmer and Tritton (RPT) developed a BMS called HiSMIS (Highway Structures Management Information System) in the late 1980’s. As Blakelock (1993) points out, since its formal release in 1990, HiSMIS was the most widely used BMS in the UK apart from the Department of Transport’s in-house system, NATS (DoT 1993). The layout of the HiSMIS system is shown in Figure 2.4.

![HiSMIS System Diagram](image)

Figure 2.4: HiSMIS system (Blakelock, 1993)
The database is formed from five distinct modules:

1. **History**: Contains information of the history of the structure (e.g. bridge, tunnel, retaining wall, etc) and any changes made to it. It can also store drawing schedules and other relevant background information.

2. **Inventory**: Contains all the fixed data about the structure and its restrictions.

3. **Inspection**: Records and process all type of inspections (general, principal, special, superficial; BD 63/94, 1994) for the condition of individual elements with a facility for prioritising actions.

4. **Maintenance/financial**: Records all maintenance, rehabilitation and replacement (MR&R) actions obtained from the inspection module, as planned and carried out along with their cost information.

5. **Programming/study**: A specifiable module where the user can add any additional information that seems necessary.

In addition there is a system administrator module to enable local administrators to adjust the system for meet local constraints and needs. The output is formed from enquiry and reporting generator. Figure 2.4 show schematically the output from ‘Enquiry and Reporting’.

Other BMS’s in the UK are:

- **SMIS (Structures Management Information System)**. This is a relative new bridge management system, developed by the HA, even though is seems that not all of software algorithms are yet completed. The major difference from other BMSs is that it determines maintenance needs based on structural adequacy, or safety, rather than solely on the condition state of the structures. That means that priority of maintenance, or repair, will be given to the structures that become unsafe even with minor deterioration rather to structures that have enough residual margin of safety even with extensive deterioration (Hayter & Allison, 1999).
- **BRIDGEMAN (BRIDGE MANagement system).** Developed by Oxfordshire County, Council. It is suggested (Darby & Vassie, 1996) that ‘Bridgeman’ is relatively inexpensive software that can be used flexibly to meet the needs of individual bridge managers and owners. The work priority is categorised in five levels (very low, low, medium, high, and urgent) and is based on cost benefit, safety or other reasons.

- **COSMOS (Computerised System for the Management Of Structures).** Written by Babtie Group. It is the interface between the user and commercial ORACLE database software (Steele et al., 2000). A distinctive feature of COSMOS is the computation of a Maintenance Priority Number (MPN) that produces a list, with elements, that require maintenance, according to this number (Brooman & Steele, 2001). Each element is assigned a MPV that is based on specific factors (condition factor, location factor and road factor). The MPV varies between a highest priority of 3.2 and a lowest of 142 (Brooman & Wootton, 2000; Brooman & Steele, 2001). In 1996 Surrey Council in the UK implemented COSMOS and further developed it with the addition of module subsystems (e.g. graffiti and pump management, site instructions), and full featured Intranet system. The first computer system implemented by Surrey was STREG (Structures REGister) developed in early eighties (Palmer & Cogswell, 1990). Then BRIDGIT (not to be confused with BRIDGIT USA) was developed by Surrey Council and Howard Humphreys but this ceased in 1993 due to its declining ability to store, and process, the increasing amount of data becoming available (Brooman & Wootton, 2000).

### 2.3.3.2 USA bridge management systems

In the USA the primary BMS in use today is Pontis. Pontis was developed in 1991 under an FHWA contract with Cambridge Systematics and Optima, Inc. (FHWA, 2004). Pontis is now licensed through the American Association of the State Highway and Transportation Officials (AASHTO) to over 40 states transportation departments and other agencies nationally and internationally even though National Highway System Designation Act [Public Law 104-59] officially repealed all the legislative requirements for BMS implementation (Small et al., 1999). Das (1996)
states that the Pontis system is the most advanced BMS developed. The Bridge Management Process Supported by Pontis can be seen in Figure 2.5.

![Bridge Management Process Supported by Pontis](Source: NHI Training Course 134056A, Pontis Bridge Management)

The bridge is divided into individual elements, or sections, made with the same material. Each element is assigned a condition state (from 1 to 5), which can be described as a quantitative measure of deterioration. Pontis is recognising the uncertainty in predicting deterioration rates therefore it views the bridge deterioration process as being probabilistic. The deterioration rates can automatically be updated after historical inspections are collected. To model the deterioration in Pontis a simple form of the Markov approach is used where in order to model the change of condition rating of each element, transition probabilities are applied. Furthermore, Pontis has the ability to estimate accident and user costs resulting from diversion and travel time delay costs, similar to QUADRO 4 (§ 2.7.2) in the UK. The delay costs are computed as the sum of three components: accident costs, vehicle operating costs and travel time costs where different equations are used to evaluate each component.

For the optimisation routine the Pontis system employs a top-down analytical approach by optimising over the network before determining individual bridge projects (Thompson, 1993; Czepiel, 1995).

One of the major advantages of the Pontis system is that it operates in a Windows environment which makes it easy to use. Unfortunately, there are a number of
limitations associated with the Pontis system such as the assumption that inspections will identify all serious defects. Further limitations of this system have been identified by Das (1996). Small et al., (1999) report that only a small number of the States that use Pontis actually use the system to generate results applicable for bridge management decision making. Most of the States use Pontis to support decision-making within the Bridge or Maintenance Department. However, it is expected that while systems progress and more information becomes available, States will employ BMS within their current practice.

An extended literature review of the USA BMSs can the found in the report prepared by Czepiel (1995).

**2.3.3.3 Other universal bridge management systems**

The following section summarises the main characteristics of different BMS around the world associated with this study focusing on maintenance optimisation and whole life costing. More details on various BMS can be found in Rodney et al., 2002; Woodward et al., 2001; Carter, 2002. Table 2.1 presents the system name and corresponding references of each examined country.
Table 2.1: International system names and corresponding references

<table>
<thead>
<tr>
<th>Country</th>
<th>System Name</th>
<th>References</th>
</tr>
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<tbody>
<tr>
<td>Canada</td>
<td>OBMS</td>
<td>Thompson et al.,(2000)</td>
</tr>
<tr>
<td>China</td>
<td>GZMBMS</td>
<td>Wang &amp; Nordengen, 2004</td>
</tr>
<tr>
<td>Denmark</td>
<td>DANBRO</td>
<td>Lauridsen &amp; Lassen, 1999</td>
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<td></td>
<td></td>
<td>Gharib, 2002</td>
</tr>
<tr>
<td>Finland</td>
<td>SIHA</td>
<td>Soederqvist &amp; Veijola, 1996 &amp; 2000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Soderqvist, 1998 &amp; 2000</td>
</tr>
<tr>
<td>Germany</td>
<td>SIB-Bauwerke</td>
<td>Krieger &amp; Haardt, 2000</td>
</tr>
<tr>
<td>Japan</td>
<td>J-BMS</td>
<td>Miyamoto et al., 2000</td>
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<tr>
<td></td>
<td></td>
<td>Miyamoto et al., 2002</td>
</tr>
<tr>
<td>Netherlands</td>
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<td>Bakker et al., 1999;</td>
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<tr>
<td></td>
<td></td>
<td>Klatter et al., 2002;</td>
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<td></td>
<td></td>
<td>Klatter and Van Noortwijk, 2003;</td>
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<tr>
<td></td>
<td></td>
<td>Van Noortwijk &amp; Frangopol, 2004</td>
</tr>
<tr>
<td>South African</td>
<td></td>
<td>Nordengen et al., 2000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CSIR, 2004</td>
</tr>
<tr>
<td>Switzerland</td>
<td>KUBA-MS</td>
<td>Roelfstra et al., 2004</td>
</tr>
</tbody>
</table>

- Inventory of existing bridge stock. All the countries have an inventory component into their computerised BMS to store information about the bridge in terms of its name, location, construction etc. Usually it includes drawings (design set and as-built set).

- Bridge condition. In general the condition is recorded after inspection (superficial, routine, general, detailed and special) and stored for individual elements and the whole bridge. In Germany only the condition of the whole bridge is recorded. The condition is mostly based on point rating scale.
• Prioritisation of maintenance work. Prioritisation module for generating an optimal (minimum cost) maintenance strategy subjected to constraints such as a minimum condition level in BMS.

  o Canada. For the optimisation of strategies a gradient search method (§ 2.8.1) is used which respects the overall budget constraints as highest priority and reports, but does not enforce, the degree to which performance targets are predicted to be met. However it also reports the level of funding required satisfying all performance targets or achieving the highest benefit while meeting as many of the funding constraints as possible.

  o China. The authority may identify the bridge condition and consider a priority for maintenance. A life-cycle cost model is used to decide the optimum budget of a maintenance plan for the authority.

  o Denmark. Based on the optimisation module bridges need repair are selected on the worst economic consequences to society if the work on them is postponed. Inputs in this module are the budgets for the next 5 year, the discount rate and the data from the economic evaluation of special inspections. The results may be altered due to factors that cannot be expressed in monetary terms (e.g. aesthetics or co-ordination with other works).

  o Finland. For prioritising individual bridges when composing the repair and rehabilitation program for the following years two indices are utilised. One is the repair index that describes the repair needs. The greater the index value the greater the need to repair. Maintenance activities and the given budget should reach a specific repair index level. Reconstruction and rehabilitation index identifies the bridges that have functional deficiencies or have researched the end of their functional and economical use.
- Germany. Prioritisation is performed on the basis of existing severity of damage, operational and traffic-related circumstances and available budgets all be based mainly on subjective estimates.

- Japan. A multi-stage optimal maintenance plan can be drawn up using a minimisation of the total cost (life-cycle cost) of maintenance measures and a maximisation of the quality of the bridge members restored by maintenance measures as objective functions. The inputs are indices for the durability and load-carrying capability of the bridge member under consideration and the maintenance budget upper limit value that is given by the user. The output proposes an optimal plan including the total cost and the degree of recovery of the durability and load-carrying capability indices. For the optimisation of the maintenance plan genetic algorithm (§ 2.8.1) is utilised.

- Netherlands. A condition-based analytical maintenance model, which is called LEM, and life-cycle cost analysis are used for the optimisation of maintenance measures.

- South Africa. The most suitable, optimised maintenance programme for a given financial year is determined, based on budget limitations and the relevancy of the identified defects as a result of bridge inspections.

- Switzerland. Actions required to maintain structural safety and serviceability are prioritised without entering an optimisation process. Optimisation on the project level consider 2 types of strategies (preservation and improvement) where are compared to find the economical optimal variants.

- Application of whole life costing (§ 2.7) analysis as budget planning (long term) module in their BMS. Whole life costing analysis is used in Canada, and Japan, Finland, Netherlands. Denmark use WLC analysis for strategies within the next 25 years only.


• Deterioration modelling prediction.
  
  o Canada. The Markov models are calibrated to reflect the history of the bridge stock, at a network level. However, the results of the global deterioration model can be altered with the use of the "adjustment factor" to reflect local characteristic at the project level.
  
  o China. To assess the durability of the identified bridge, a probabilistic method is used.
  
  o Finland. The system applies probabilistic deterioration models to find the condition distribution of network of bridges. Deterministic modelling of deterioration is used for the project level system.
  
  o Japan. The system uses multi-layered neural networks to predict the service life of each bridge member by calculating deterioration curves for durability and serviceability using regression analysis curves.
  
  o Netherlands. Only one component, one failure mode and only the uncertainty in the deterioration process is used in a condition-based analytical maintenance model, called LEM. The deterioration process is assumed to be a stochastic process (gamma process) with independent increments.
  
  o Switzerland. Markov chains are used in KUBA-MS to represent condition forecasts where the transition probabilities are determined using regression analysis from inspection results. The deterioration processes are assigned to materials and not to element types.

2.3.4 The Future for Bridge Management Systems

Most of the current BMS are based on the following assumptions (Frangopol & Das, 1999):

  • Maintenance needs are directly related to condition states (based on the deterioration extent; Vassie, 2000) of the structures.
• Conditions predictions are based on Markovian approach.

• The justification of any proposed work is that it will cost less if the work is done now than later.

However the decision for the maintenance, or replacement, of a bridge mainly depends on the load capacity (structural reliability) of the bridge rather solely on the perceived condition. Also as mentioned before, the Markovian approach has several limitations. Finally the cost of work should be based on the whole life of the bridge in the form of optimum program strategies for bridge specific or a network specific that may involve and other related programs work programme e.g. pavement maintenance.

New and improved techniques for the development and enhancement of BMS are currently carried out. It seems that the next generation of BMS will be based upon lifetime reliability and whole life costing.

Updating techniques (e.g. Conditional probability, Bayesian updating; Ang & Tang, 1975) can use inspection results to revise the reliability level of the bridge (Thoft-Christensen, 1999b). Maintenance actions will be applied to ensure that the condition of a bridge is above its target reliability level (Frangopol et al., 2000) rather than a specific condition state where reliability state refers to the structural adequacy. Optimum programs of actions will be based on whole life cost analysis and will incorporate the uncertainties and constraints involved in the process (Frangopol et al., 2000).

2.4 Bridge maintenance management in the UK

Focus is given here in the maintenance management and particularly in the UK’s, since a considerable research effort, mainly by the Highways Agency, was undertaken in this area to address the various issues associated with bridge management and develop appropriate methods and standards to ensure optimum use of the resources.
According to the OECD (1981) report, ‘Bridge maintenance’, the aim of bridge maintenance in the UK is “to maintain highway structures in a serviceable condition with particular regard to the safety of users, the preservation and integrity of the structure and the achievement of economy in the long term”. Accordingly it is not enough to apply maintenance to ensure the safety and the serviceability of the bridge but also to develop maintenance strategies to accomplish efficient utilisation of the financial resources.

The information from inspections, assessments, and tests can be used to decide the maintenance options required. An advice note entitled ‘Whole life assessment of highways bridges and structures’ is being drafted for publication by Highways Agency to determine options for future maintenance strategies and to provide input for maintenance works bids.

The overall procedure is shown schematically in Figure 2.6 (Das 1997, 1998 & 2000). It demonstrates that “the assessed current performance level is to be compared with the critical level (minimum acceptable level) appropriate for the element concerned to determine if the element is sub-standard now or is likely to be in the near future” (Das, 2000).

Figure 2.6: Whole life performance based assessment of bridges (Das, 1998 & 2000)
If the element is sub-standard now essential maintenance (§ 2.4.1) will be considered but if the element is not sub-standard now some preventative maintenance (§ 2.4.1) work may be justified due to reductions of future maintenance costs. Once the present performance is determined using the present state assessment (BD 21/01, 2001), the performance of the element is then projected into the future using a number of alternative maintenance strategies. Possible courses of action include (a) fully strengthen, (b) partially strengthen to maintain its current level of structural capacity, or (c) left to be replaced in the near future (Das, 2000). Then each action (planning costs, maintenance costs, traffic costs) necessary for these alternative strategies will then be submitted into the bidding process, described later.

2.4.1 Types of maintenance

The following maintenance types can be used to increase the life of a bridge structure by reducing the current rate of deterioration (BA 81/00, 2000).

1. Routine Maintenance

Routine maintenance “is the work of minor nature, which should be carried out at regular intervals to ensure the safety of the structural stock, keep the stock in good order and minimise deterioration” (BA 81/00, 2000). This involves the cleaning of drains and channels, removal of debris from bearing shelves, and graffiti/vegetation from the structure amongst others.

2. Preventative Maintenance (PM)

Preventative work is optional in that it is only recommended now but if it carried out will reduce essential work from arising prematurely at a future date. As such, “it is not essential now but may be justified on economic grounds” (BA 81/00, 2000).

Currently available PM options for reinforced concrete bridges are:

- waterproofing
- installation of positive drainage to avoid contamination
- repair of deck expansion joints
- make bridge decks continuous
- bearing replacement
- bridge enclosure
- scalers and surface coating
- concrete repairs
- coating reinforcing steel
- corrosion inhibitors
- cathodic protection
- electrochemical chloride extraction
- re-alkalisation
- anti-carbonation coatings.

However, the application of PM measures is not simple because of the uncertainties surrounding their effectiveness. Research is needed to identify their effect in the deterioration process, i.e. to decelerate, or prevent, chloride ion ingress by blocking the pores of the concrete, or changing its characteristic (diffusion coefficient). More discussion on these issues is given in Chapter 3 and 4.

3. Essential Maintenance

Essential maintenance “is required to maintain the safety standards of the structure e.g. structures, which are assessed to be inadequate for the 40 tonne loading, or other safety reasons (structural deficiencies in components / elements) should be repair” (BA 81/00, 2000). Furthermore, it “is required because the bridge’s condition is deteriorating to an unacceptable level, which may cause public alarm or
"make the structural behaviour unpredictable such as spalled concrete" (BA 81/00, 2000). Available essential options are:

- **Strengthening**: The structural enhancement of existing weak members in such a way as to restore or increase their ultimate in bending, shear or direct tension and compression.

- **Replacement**: Removal of major structural members and/or the addition of new ones.

### 2.4.2 Maintenance strategies

A maintenance strategy is a long-term plan that contains action plans (i.e. routine preventative and essential maintenance actions) for achieving a desired future state for a specific bridge structure or a network.

The Highways Agency proposed a strategic plan in 1997 to determine bridge maintenance needs for the future. Each year funds will be allocated for essential, preventative and routine work. In cases where essential work can not be performed interim measures such as weight restriction and temporary propping may need to be employed (Wallbank et al., 1999).

In an ideal situation, the expenditure would be shown as in Figure 2.7 (a) (Wallbank et al., 1999). However, if insufficient funding was provided each year, the amount of essential work required for structures to remain in service would start to increase with time as shown in Figure 2.7 (b) (Wallbank et al., 1999).

![Figure 2.7: Bridge maintenance programs](image)
Development of the strategic plan requires (Wallbank et al., 1999 and Das, 2000):

- The estimation of typical maintenance costs. Four maintaining agents were consulted by the Highways Agency and Maunsell consultancy, to estimate the costs of various maintenance operations and their associated traffic delay costs. Then an average cost for each activity is derived from the obtained data.

- The estimation of probability distribution for the maintenance intervals and application of the results to the range of bridge types and ages. A number of maintaining agents were consulted for their opinion, based on records and engineering judgment, to estimate for different types of bridge, the likely times that essential maintenance is applied i.e. assessing the rehabilitation rate. Multiplying the rate of rehabilitation with the number of bridges built in different years, the number of bridges of all types of bridge requires essential maintenance in any year could be obtained.

![Figure 2.8: Essential maintenance cycle times for reinforced concrete bridges (Wallbank et al., 1999)](image)

A similar procedure was adopted to predict future percentage of bridges requiring preventative maintenance in any year (Figure 2.9). PM measure is
assumed to be required at cycle times between 10 and 25 years with mode of 20 years. It was thought that where preventative maintenance is appropriate, a uniform 5.7% of the stock will be treated each year.

![Figure 2.9: Preventative maintenance cycles (Wallbank et al., 1999)](image)

For each preventative maintenance scenario, the number of bridges of each type constructed in a particular year is multiplied with the predicted rate of rehabilitation with and without preventative maintenance. This will provide the number of bridges to be maintained (with PM) or rehabilitated (without PM) in any particular year in the future.

- Backlog of work. In addition to the maintenance presented above, backlog work cost should also be estimated. This work arise from strengthening due to the assessments of 40 tonne vehicles, replacement of sub-standard parapets and strengthening of piers vulnerable to impact.

- Road user delay costs. These costs often exceed direct cost. By using QUADRO 4 (§ 2.7.2) an estimation of the road user delay costs can be calculated since they are based on a series of very broad simplifications and are sensitive to the assumptions made. Nevertheless this estimation can give an indication of the order of road user delay costs when applied to a bridge stock.
The overall strategic plan was the summations of four optimum strategies, which is shown in Figure 2.10.

![Highway Agency's predicted future expenditure on bridge maintenance](image)

**Figure 2.10: Highway Agency's predicted future expenditure on bridge maintenance (Das, 2000)**

However this strategic plan provides an overall guidance on the extent of future bridge maintenance expenditure. The onus remains on annual maintenance bids and prioritisation to actually carry out the programme (Das, 2000). The procedure (Haneef & Chaplin, 1999) can be summarised as follows (Das, 2000):

- Step 1. Prioritise by work type (committed, essential, etc).
- Step 2. Prioritise by category (pier strengthening, etc).
- Step 3. Select maintenance options with lowest Present Value (PV) (§ 2.7).
- Step 4. If the resulting cost profile for any work type is less than that in the strategic plan, select next higher PV option.

To achieve more effective use of the limited maintenance budget it is necessary to develop optimum strategies that will provide recommendations regarding the time application of various maintenance options while obtaining minimum whole life
costing (§ 2.7). Optimisation techniques (§ 2.8) can be applied to obtain such strategies.

2.5 Pro-active bridge maintenance management

Typical current management strategy is based on reactive process, i.e. a course of action is taken after deterioration is observed. For example, when a bridge assessment is carried out in the UK, actions will be taken if the assessment shows the bridge to be inadequate (BD 21/01, 2001). This is not the most effective use of available resources.

To achieve proactive BM approach, PM measures can be used to maintain a bridge in a serviceable condition while postponing the deterioration process. The difficulty of developing PM strategies lay on the assessment of the effectiveness of these measures.

2.5.1 Effectiveness of PM measures

There are broadly three methods to estimate the effectiveness of PM under a deterioration mechanism and are similar to those estimating the rehabilitation rate (Das, 1999):

- To estimate it based on expert opinions of experience bridge engineers. This is the simplest method.

- To collect available data that shows the degree of effectiveness (e.g. chloride concentration during the service life of silane) carried out by the maintaining authorities in the past.

- To develop effectiveness models by using reliability-based studies of whole life performance under different maintenance regimes.

The first method was used by a number of researchers studying the effect of PM measures to prevent, or stop, a specific deterioration mechanism (such as corrosion of steel reinforcement) but mostly they produced only qualitative results (§ 2.5.1.2). Table 2.2 and 2.3 were produced by Vassie & Arya (2000) to be used as guidance.
when choosing a suitable maintenance strategy. These tables just state the particular PM measures that can be used to protect reinforced steel bars against the chloride ion and carbonation mechanisms, and some general, but relevant, information for these measures. However they fail to give enough information of the degree of effectiveness of PM to delay/inhibit the corrosion process. Furthermore qualitative approaches do not take into account the what-if analysis options than quantitative methods can achieve.

Table 2.2: Characterisation of methods for preventing reinforcement corrosion (Vassie & Arya, 2000)

<table>
<thead>
<tr>
<th></th>
<th>Silane</th>
<th>Paint</th>
<th>WPM</th>
<th>Inhibitors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Protects against</td>
<td>Chloride</td>
<td>Chloride &amp; Carbonation</td>
<td>Chloride &amp; carbonation</td>
<td>Chloride &amp; carbonation</td>
</tr>
<tr>
<td>Effectiveness for intermittent wetting</td>
<td>Good</td>
<td>Very Good</td>
<td>Very Good</td>
<td>Very Good</td>
</tr>
<tr>
<td>Effectiveness for Ponding</td>
<td>Poor</td>
<td>Good</td>
<td>Very Good</td>
<td>Good</td>
</tr>
<tr>
<td>Aesthetics</td>
<td>Neutral</td>
<td>Improved</td>
<td>reduced</td>
<td>Neutral</td>
</tr>
<tr>
<td>Ease of initial application</td>
<td>Easy</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Easy</td>
</tr>
<tr>
<td>Ease of replacement</td>
<td>Easy</td>
<td>Moderate</td>
<td>Difficult</td>
<td>Not needed</td>
</tr>
<tr>
<td>Frequency of replacement</td>
<td>15'</td>
<td>10-15 years</td>
<td>20-25 years</td>
<td>___</td>
</tr>
<tr>
<td>Comparative cost of prevention</td>
<td>Low</td>
<td>Low</td>
<td>Moderate</td>
<td>High</td>
</tr>
</tbody>
</table>

* Silane has a relatively short track record; there is evidence that is fully effective for 10 years and its life is expected to be much longer.
Table 2.3: Characterisation of methods for stopping reinforcement corrosion (Vassie & Arya, 2000)

<table>
<thead>
<tr>
<th>Pre repair testing</th>
<th>Concrete repairs</th>
<th>Impressed Current CP</th>
<th>Desalination</th>
<th>Migrating inhibitor</th>
<th>Sacraficial CP</th>
<th>Realalkalisation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extensive</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Limited</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

| Propping during repair | Usually | Unlikely | Unlikely | Unlikely | Unlikely | Unlikely | Carbonation |

| Effective for corrosion caused by CO₂/Cl-/both | Both | Both | Chloride | Both | Both | Carbonation |

| Effects of repairs on users | High | Low | Low | Low | Low | Low |

| Post repair monitoring/ maintenance | Monitoring | Monitoring and maintenance | Monitoring | Monitoring | Monitoring | Monitoring |

| Preventative maintenance needed | Yes | No | Yes | Yes | No | Yes |

| Comparative Cost of repair | Very High | High | Moderate | Low | Low | Moderate |

The second method can be both a qualitative and quantitative method but unfortunately there is lack of available data. The third method is a quantitative approach (2.5.1.2) and can be achieved by showing the quantified effect of PM actions on deterioration models (§ 2.3.1).

The reliability profile $\beta(t)$ is defined as the variation of the reliability index with time (Thoft-Christensen, 1996; Estes & Frangopol, 1996; Nowak et al., 1998). Frangopol & Das (1999) state that similar bridges, designed and constructed to the same requirements, have different reliability levels for different reasons. This variation of reliability (Figure 2.11) can be captured by eight random variables (Frangopol et al., 2000; Kong et al., 2000).

These eight random variables are: (i) the initial reliability index (i.e. at time $t=0$), $\beta_0$; (ii) the reliability index deterioration rate, $\alpha$; (iii) the time at which damage initiates, $t_i$; (iv) the improvement in $\beta$ immediately after the application of preventative maintenance, $\gamma$ (v) the deterioration rate of $\beta$ during the preventative maintenance.
effect, \( \theta \); (vi) the time of reapplication of preventative maintenance, \( t_p \); (vii) the duration of preventative maintenance effect on \( \beta \); \( t_{PD} \) (viii) the time of application of first preventative maintenance, \( t_{pi} \). Then Monte Carlo simulation is used to generate random samples from the probability density functions (§ 2.6.2) of these eight variables to incorporate the uncertainties during the service life of the bridge examined.

**Figure 2.11: Reliability Profiles and Associated Random Variables for the Options with or without Preventative Maintenance (Frangopol et al., 2000)**

The reliability-based maintenance management (§ 2.6) has the advantage that the reliability is explicitly taken into account. However, a disadvantage is that it is difficult to estimate the effect of maintenance on the reliability index.

Kong et al., 2000 used this model to show the effect of preventative maintenance on a group of deteriorating steel/concrete composite highway bridges considering bending and shear failure modes. A similar model was proposed by Thoft-Christensen (1999a) using artificial maintenance strategies to illustrate the effectiveness of PM and essential maintenance on the corrosion of reinforcement of 970 RC overbridges in the UK. It was assumed that a bridge undergoing preventative maintenance will remain in its current state but that the corrosion rate will be reduced (improved) by some amount.
As reliability theory has become better understood the research has moved toward more realistic and practical applications (Frangopol & Estes, 1997). However the reliability of the methods depends on the reliability of the input values (costs, random variables etc). Liu & Frangopol (2004) illustrate the effectiveness of silane as improvement of performance in term of condition and safety index. Two types of maintenance are applied: time based and performance based. The time based maintenance actions are applied at predefined time intervals and provide preventative improvement on the bridge performance where the performance based maintenance actions are applied when the target performance value is reached and therefore provide essential maintenance (Figure 2.12).

![Multi-linear performance profile model](image)

**Figure 2.12: Multi-linear performance profile model under no maintenance and under (a) Time based and (b) Performance –based maintenance interventions (Liu & Frangopol, 2004)**

Table 2.4 and 2.5 define the parameters of this approach. The data to derive these parameters were collected in the UK and forwarded by personal communication to the authors (Liu & Frangopol, 2004). Also the data are referred just to reinforced concrete crossheads. The variables were assumed to have a triangular distribution, characterised by their minimum, mode and maximum values. The multi-linear model (Frangopol et al., 2001) adopted in this approach assumes that the performance of the bridge deteriorates linearly with time.
Table 2.4: Parameters for condition and safety profiles under no maintenance strategies (Liu & Frangopol, 2004)

<table>
<thead>
<tr>
<th>Performance index</th>
<th>Initial value, $P_0$</th>
<th>Time of damage initiation (years), $T_i$</th>
<th>Deterioration rate (year$^{-1}$), $\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Condition</strong></td>
<td>Min = 0</td>
<td>0</td>
<td>Min = 0</td>
</tr>
<tr>
<td></td>
<td>Mode = 1.75</td>
<td></td>
<td>Mode = 0.08</td>
</tr>
<tr>
<td></td>
<td>Max = 3.50</td>
<td></td>
<td>Max = 0.16</td>
</tr>
<tr>
<td><strong>Safety</strong></td>
<td>Min = 0.91</td>
<td>0</td>
<td>Min = 0</td>
</tr>
<tr>
<td></td>
<td>Mode = 1.50</td>
<td></td>
<td>Mode = 0.015</td>
</tr>
<tr>
<td></td>
<td>Max = 2.50</td>
<td></td>
<td>Max = 0.035</td>
</tr>
</tbody>
</table>

Table 2.5: Parameters for condition and safety profiles under different maintenance strategies (Liu & Frangopol, 2004)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Silane</th>
<th>Do nothing but rebuild</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time of first application (years), $T_{pd}$</td>
<td>Design variable</td>
<td>When safety &lt; 0.91</td>
</tr>
<tr>
<td>Time interval of subsequent application (years), $T_p$</td>
<td>Design variable</td>
<td>When safety &lt; 0.91</td>
</tr>
<tr>
<td>Effects on condition index</td>
<td>0</td>
<td>Restored to condition = 0</td>
</tr>
<tr>
<td>Delay in deterioration (years), $T_d$</td>
<td>0</td>
<td>Min = 10</td>
</tr>
<tr>
<td>Deterioration rate during effect (year$^{-1}$), $\alpha - \delta$</td>
<td>Min = 0</td>
<td>Mode = 15</td>
</tr>
<tr>
<td></td>
<td>Mode = 0.01</td>
<td>Max = 30</td>
</tr>
<tr>
<td></td>
<td>Max = 0.03</td>
<td></td>
</tr>
<tr>
<td>Duration of maintenance effect (years), $T_{pd}$</td>
<td>Min = 7.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Mode = 10.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Max = 12.5</td>
<td></td>
</tr>
<tr>
<td>Improvement, $\gamma$</td>
<td>0</td>
<td>Min = 1.00</td>
</tr>
<tr>
<td></td>
<td>Mode = 1.25</td>
<td>Max = 1.50</td>
</tr>
<tr>
<td></td>
<td>While condition &lt; 1.0</td>
<td></td>
</tr>
<tr>
<td>Delay in deterioration (years), $T_d$</td>
<td>0</td>
<td>Min = 0.3</td>
</tr>
<tr>
<td>Deterioration rate during effect (year$^{-1}$), $\alpha - \delta$</td>
<td>Min = 0</td>
<td>Mode = 247</td>
</tr>
<tr>
<td></td>
<td>Mode = 0.007</td>
<td>Mode = 7410</td>
</tr>
<tr>
<td></td>
<td>Max = 0.018</td>
<td>Max = 28 898</td>
</tr>
<tr>
<td>Duration of maintenance effect (years), $T_{pd}$</td>
<td>Min = 7.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Mode = 10.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Max = 12.5</td>
<td></td>
</tr>
<tr>
<td>Unit cost (k$$$)</td>
<td>Min = 0.3</td>
<td>Min = 247</td>
</tr>
<tr>
<td></td>
<td>Mode = 39.0</td>
<td>Mode = 7410</td>
</tr>
<tr>
<td></td>
<td>Max = 77.0</td>
<td>Max = 28 898</td>
</tr>
</tbody>
</table>

Another well-known maintenance model is the replacement model (Van Noortwijk & Frangopol, 2004). The age replacement strategy involves replacement upon a failure or upon reaching a predetermined age, whichever occurs first (§ 2.3.1). This kind of model has been developed for maintenance strategy of bridge components but is based on condition state and assumed deterioration process.
2.5.1.1 Types of uncertainty

It is clear from the previous sections that it is difficult to estimate the effectiveness of PM measures. The reason is due to a range of uncertainties involved in its assessment. In Thoft-Christensen & Baker (1982) uncertainty is given with the following statement:

“All quantities (except physical and mathematical constants) that currently enter into engineering calculations are in reality associated with some uncertainty. This fact has been implicitly recognised in current and previous codes. If this were not the case, a ‘safety factor’ only slightly in excess of unity would suffice in all circumstances. The determination of appropriate standards of safety requires the quantification of these uncertainties by some appropriate means and a study of their interaction for the structure under consideration”.

The types of uncertainty that are usually considered are (Thoft-Christensen & Baker, 1982; Melchers, 1999):

- Physical uncertainty. It is identified with the inherent variability of physical quantities. Loads, material properties, dimensions will always be associated with a certain variability which is normally described in terms of probability distributions or stochastic process. The uncertainties, for example, of strength values and dimensions of concrete can be reduced, but not entirely eliminated, by the use of advanced production and quality control methods although this may cause an increase in production costs. The uncertainty of natural phenomena such as snow loading cannot however be eliminated.

- Statistical uncertainty. The physical uncertainty is estimated by examining sample data. From the sample data distribution parameters are estimated, but this estimate will depend on the size of the sample data set. Different sample data sets will usually produce different statistical results due to the sample size, the neglecting of possible correlations, or systematic variations of the
observed variable (e.g. climate variable). The corresponding uncertainties are called the statistical uncertainties.

- Modelling uncertainty. Mathematical models are used to describe the relationships between the basic variables. Models are often simplified due to a lack of knowledge of the detailed process being modelled. Modelling uncertainty concerns the uncertainty in describing physical phenomenon, such as the diffusion of chloride in concrete. This kind of uncertainty can be reduced with further research or with the availability of more data.

2.5.1.2 Measurement of uncertainty

Uncertainty is a description of the doubt about the properties of a parameter due to lack of knowledge and can be determined using qualitative or quantitative measurements.

- **Qualitative measurements of uncertainty.** Qualitative measures relating to, or expressed in terms of quality (use of words or phrases) descriptions. This approach can be used when the uncertainty of a given parameter in question cannot be written in number form. The information provides an understanding of how the described parameter behaves. The qualitative terms that are usually employed are given in Table 2.6.

<table>
<thead>
<tr>
<th>Table 2.6: Qualitative terms</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
</tr>
<tr>
<td>Medium</td>
</tr>
<tr>
<td>Low</td>
</tr>
</tbody>
</table>

- **Quantitative measurements of uncertainty.** Quantitative measurements are based on numerical data. The uncertainty of a specific parameter is given either as a number of standard units (e.g. 0.1 mm) or as a probability of failure (e.g. 3%). In the first case the value is based on previous studies or engineering judgement. In the second case the value is determined by probabilistic analysis with random variables.
If necessary, simple mathematical relations most probably assumed, can be used to convert the qualitative measurements into quantitative measurements.

2.5.1.3 Managing Uncertainty – Need for reliability

To understand better the importance of uncertainties in bridge management and how these affect the bridge condition, the following figure is considered. It illustrates the performance profiles of 15 concrete bridges, which were designed, and constructed, to the same requirements. The profiles have been calculated by Prof. P. Thoft-Christensen of Aalborg University, Denmark, and Cambridge University, in the course of a Highways Agency project.

![Figure 2.13: Whole life load capacity profiles of 15 concrete bridges. (Das, 1999a)](image)

Figure 2.13 illustrates that, even as constructed, the bridges had widely different load capacities, although they were designed to the same design requirements. Some of the bridges reached the critical/unsafe stage much earlier than others, which even after extensive deterioration they may remain safe (Das, 1999a).
The above, shows clearly the effect of uncertainties and the importance to be considered in bridge management decisions. The fact is that uncertainties cannot be avoided; therefore they must be specifically included in any analysis.

These uncertainties can be investigated using probabilistic methods. The use of reliability-based assessments provides a framework for taking into account the various uncertainties associated with particular assessment parameters. Furthermore it provides a basis for ‘managing’ uncertainty more effectively.

### 2.6 Reliability based bridge maintenance management

Although current BMSs base their maintenance optimisation on present condition states this can be said to be incorrect. According to Das (1998) the extent of bridge maintenance largely depends on the load carrying capacity of the bridge rather than on their condition alone. This means that the estimation of maintenance needs should be based on bridge reliability rather than condition states. In addition there are many uncertainties associated with bridge maintenance management decisions. The effect of these uncertainties can be wastage or inappropriate use of resources (Das & Onoufriou, 2000b). Within this context, the use of reliability techniques becomes very important. The use of these techniques provides a means of accounting for these uncertainties and assessing the level of confidence in the performance of the structures and the effectiveness of various actions. This provides a more rational basis for decision-making and strategy optimisation. The potential applications and benefits from the use of these techniques in bridge management were recognised by the UK HA several decades ago and a strategic research programme was commissioned over the last twenty years to develop improved methodologies and guidelines for use within the various bridge management activities (Das, 2000). The first comprehensive reliability-based bridge management system supported by the European Union is described in Thoft-Christensen (1995).

A reliability technique is based on probabilistic modelling which is briefly discussed below.
2.6.1 Probabilistic modelling

Probabilistic modelling is a technique that can assess the effect of uncertain input parameters and assumptions on the analysis model. An uncertainty or random quantity is a parameter for which it is not possible to identify the exact value at a given point in time. In a probabilistic analysis, these uncertain parameters are described by statistical distribution functions such as normal distribution, uniform distribution, weibull distribution, gamma distribution etc, where possible. It is best to use a probability distribution that can accurately describe the scattering of data but this requires a significant amount of data. Where great accuracy is not essential, and the data are limited, simple distributions can be utilised.

In bridge management development, probabilistic models have been used in areas such as: the prediction of whole-life costs (Rubakantha, 2001), the evaluation of the number of bridges requiring essential maintenance each year (Frangopol & Estes, 1997\textsuperscript{a}), estimation of time of initiation to corrosion (Thoft-Christensen, 1998), modelling the uncertainty inherent to chloride induced deterioration (Sterritt & Chryssanthopoulos, 1999), the calculation of the parameter uncertainties in reliability-based life-cycle maintenance cost optimisation of deteriorating bridges (Kong & Frangopol, 2003), the representation of the uncertainty in the lifetime of a bridge (Van Noortwijk & Klatter, 2004) etc.

2.6.2 Reliability analysis

Reliability is a probabilistic measure of the performance of an item (e.g. bridge element) or facility (e.g. bridge) with respect to its intended function under stated conditions and a specified period of time. In the case of structural reliability the strength of the structure (resistance) should be greater than the applied load (action effect), or, from the structural serviceability point of view, the serviceability parameter, such as crack width should not exceed a limit level. The resistance, the action effect and the serviceability parameter are associated with uncertainties. In reliability terms uncertain parameters are called random variable. Random variables may be described in terms of probability density and distributions functions. For a
random variable \( X \) the probability (cumulative) distribution function is defined as (Thoft Christensen & Baker, 1982):

\[
F_x(x) = P(X \leq x) \tag{2.1}
\]

Where \( F_x(x) \) is the probability that the random variable \( X \) takes on values equal or less than \( x \).

The probability density function is defined as (Thoft Christensen & Baker, 1982):

\[
f_x(x) = \frac{dF_x(x)}{dx} \tag{2.2}
\]

A schematic representation of those functions for a continuous normal variable is shown in Figure 2.14.

![Figure 2.14: (a) Normal Cumulative Distribution Function, CDF (b) Normal Probability Density Function, PDF (NIST/SEMATECH, 2004)](image)

Reliability is measured in terms of probability of failure \( (p_f) \) of a limit state function and can be expressed as:

\[
\text{Reliability} = (1 - p_f) \tag{2.3}
\]

A limit state function represents mathematically a boundary between desired and undesired performance. The simplest limit state function consists of two random
variables, for example resistance $R$ and action effect $S$, and the reliable state can be expressed as:

$$R \geq S$$  \hspace{1cm} (2.4)

The corresponding limit state (e.g. ultimate; Melchers, 1999) function would be:

$$G = R - S$$  \hspace{1cm} (2.5)

Failure is defined by the failure condition as:

$$G < 0$$  \hspace{1cm} (2.6)

The $p_f$ is thus equal to the probability that the event ($R - S > 0$) will occur. Assuming that $R$ and $S$ are continuous random variables and statistically independent (no correlation between the variables, Melchers, 1999), the $p_f$ can be expressed as:

$$p_f = P(R - S < 0) = \int_{-\infty}^{+\infty} f_s(x) F_R(x) \, dx$$  \hspace{1cm} (2.7)

Where $f_s(x)$ and $F_R(x)$ are the probability density function (PDF) and cumulative distribution function (CDF) for variables $S$ and $R$, respectively.

Figure 2.15: The fundamental case of structural reliability (Schneider, 1997)
As shown in Figure 2.15, the value of $p_f$ depends on the size of the overlapping region between the curves $f_S(x)$ and $f_R(x)$. The reliability is decreased when $f_S(x)$ and $f_R(x)$ are closer and the overlapping region is larger. Furthermore, with an increase in the dispersion of $f_S(x)$ and $f_R(x)$ due to the increase of uncertainties, the overlapping region is also enlarged.

![Figure 2.16: Three-dimensional representation of the design point (Bailey, 1996)](image)

Figure 2.16 illustrates the failure zone (the domain in which $R = S$) with respect to the joint PDF of $R$ and $S$. The limit state surface is defined by $R - S = 0$, and the $p_f$ is the volume integral over the failure zone. The point that gives the most likely combination of basic variable (e.g., $R^*$ and $S^*$) to cause failure, is defined as design point.

Reliability analysis with respect to the limit state function $G$ is very rarely just a two-dimensional problem. Both resistance and action effect are functions of many variables (material properties, section dimensions, environment, load effect, etc.). In this regard, a generalised formulation of the limit state function would be: $G = g(X) = g(X_1, X_2, \ldots, X_n)$ (2.8)

Where $X_1, X_2, \ldots, X_n$ are variables of the function.
In this case the limit state surface $g(X)=0$ is n-dimensional, but the principle of evaluation of the $p_f$ is the same. If $f_x(x)$ is the joint PDF of the n-dimensional vector $X$ of basic variables, then the $p_f$ is the volume integral over the failure zone:

$$p_f = P(G(X) \leq 0) = \int_{G(X) \leq 0} f_x(x) dx$$

(2.9)

Another way to measure the safety of the structure similar to the $p_f$ is the reliability index. The reliability index, $\beta$, is non-dimensional where the probability of failure is related to $\beta$ through: $p_f = \Phi(-\beta)$ or $\beta = -\Phi^{-1}(p_f)$.

Where $\Phi()$ is the standard normal distribution function (zero mean and unit variance, see Appendix A).

The relationship can be presented graphically as shown in Figure 2.17.

![Figure 2.17: Relationship $p_f$ with $\beta$ (based on the complementary standard normal table, Melchers, 1999)](image)

**2.6.2.1 Target reliability**

There is no meaning of calculating the probability of a function (e.g. $g(t)$) when there is no criterion of acceptance. For this reason the concept of target reliability was introduced. Target reliability is the minimum acceptable reliability that corresponds to the reliability based analysis (Figure 2.18). The minimum reliability is expressed either as minimum (target) $\beta$ or maximum $p_f$.
Appropriate target reliability values are difficult to evaluate since they depend on complex factors such as the structural design life and social impact. Eurocode BS EN 1990 (2002) Annex B provides some reliability differentiation. Based on three Consequence Classes (CC) that are produced in relation to the consequence of failure or malfunction of the structure, three reliability classes are defined and given in Table 2.7.

Table 2.7: Reliability classes and recommended minimum values for reliability index $\beta$ (Gulvanessian, 2004)

<table>
<thead>
<tr>
<th>Reliability Class</th>
<th>Ultimate limit states</th>
<th>Fatigue</th>
<th>Serviceability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 year reference period</td>
<td>50 years reference period</td>
<td>1 year reference period</td>
</tr>
<tr>
<td>RC3</td>
<td>5.2</td>
<td>4.3</td>
<td></td>
</tr>
<tr>
<td>RC2</td>
<td>4.7</td>
<td>3.8</td>
<td>1.5 to 3.8</td>
</tr>
<tr>
<td>RC1</td>
<td>4.2</td>
<td>3.3</td>
<td></td>
</tr>
</tbody>
</table>
The reference period given is 1 year and 50 years. For different reference period the values of $\beta$ can be calculated using the following expression:

$$\Phi(\beta_n) = [\Phi(\beta_1)]^n$$  \hspace{1cm} (2.10)

Where $\beta_n$ is the reliability index for a reference period of $n$ years and $\beta_1$ is the reliability index for one year.

Even though $\beta$ and its corresponding $p_r$ are only notional values they can be used as operational values for code calibration and comparison of the levels of structures (Gulvanessian, 2004).

### 2.6.2.2 Methods of analysis

In general, solving the integrals from Equations (2.5) or (2.8) is complicated and can be done in a closed form only for simple cases. This is because:

- Usually, there is hardly enough information to be confident about the marginal distributions (the distributions of the underlying random variables) and the covariances (the interdependence of two or several random variables) (Thoft-Christensen & Baker, 1982).

- The integration is numerically complex, extremely time consuming and is only feasible for less than about 5 basic variables (Melchers, 1999).

Therefore some mathematical tools are needed in order to evaluate the $p_r$. These methods can be either analytical or numerical (Ang & Tang, 1984; Schneider, 1997; Melchers, 1999). Analytical methods, such as the First Order Reliability Method (FORM) and Second Order Reliability Method (SORM) give approximate results (non-linear limit state function) and are based on determining the design point, the most probable point of failure on the limit state surface. On the other hand a numerical method such as Monte Carlo simulations (MCS) enables the uncertainty of an engineering problem to be statistically analysed. MCS can be visualised "as an experiment which is performed by a computer rather than in a laboratory" (Hart, 1982). The Monte Carlo method is easy to understand and very adaptable to a variety
of engineering problems (Schneider, 1997). In many engineering problems involving several random variables and/or complex non-linear failure margins MCS is frequently used (Ng & Fairfield, 2002; Liu & Frangopol, 2004). Furthermore advances in computer hardware and software have led to an increase use of the Monte Carlo approach. In cases where the amount of computer time needed to perform crude Monte Carlo simulation is relatively large there are available techniques that offer an increase in the efficiency and accuracy of the value of $p_f$ with a relatively small number of simulation cycles. Such techniques are: Latin hypercube Sampling, Importance Sampling, Stratified Sampling and Directional Simulation which are described in detail in standard textbooks. However is worth mentioning that the level of computational difficulty for each cycle using these methods will increase (Ayyub & McCuen, 1995) and therefore their computation time may increase too.

2.7 Whole life costing

Reliability based bridge maintenance management can provide the effectiveness of various maintenance options (e.g. preventative measure) as $p_f$ profiles in order to produce maintenance strategies. However, due to limited maintenance budget it is vital to estimate the whole life costing of these strategies.

The advantages of using Whole Life Cost (WLC) to appraise and compare alternative solutions based on their life cycle financial benefits are now widely accepted, although until the last decade, it was common practice to make decisions of an investment nature based on lowest initial cost. Bridge structures have a long life expectancy; therefore during their service life they will require maintenance, inspection, or monitoring in order to maintain safe operation and continuing serviceability. Consequently, it is more rational to base expenditure decisions on the total cost of a bridge structure (initial costs & ongoing maintenance costs) and consequently to consider future consequences of present actions (Vassie, 2000; Ryall, 2001). The most important features of whole life costing are which costs to include (De Brito & Branco, 1996), how to estimate them (Vassie & Rubakantha, 1996) and to identify the service lives.
Many researchers (Rigden, 1995; Jones, 1995; Wyatt, 1995; Palmer, 1995; Leeming, 1995; Frangopol et al., 2000; Rubakantha & Parke, 2000; Frangopol and Kong, 2001 & 2002; Val & Stewart, 2003) used whole life costing techniques in bridge management to aid decision-making. Some advanced BMSs such as Pontis (USA) and BRIDGIT (USA) use cost analysis to determine the best maintenance strategy in a fixed period. De Brito & Branco (1996) have undertaken a cost analysis that can be used for decision making in BMSs. They included a number of costs and benefits into the whole life costs. However the model fails to recognise the dependence between the costs, for example, increase in maintenance costs will reduce future repair costs.

In the UK procedures for WLC are provided in BD36/92 (1992) and BA28/92 (1992). BD 36/92 (1992) set out the basis for the assessment of maintenance costs when comparing alternative designs of highway structures either at the design or tendering stage or alternations to existing structures. Advice Note BA 28/92 (1992) supplies guidance, background information and examples. Work is underway to revise and extend the procedures contained within BD 36/92 (1992) and BA 28/92 (1992).

### 2.7.1 Principles of whole life costing

Whole life costing models can be deterministic or probabilistic. When there is uncertainty for the values of the input variables of the model, it is better to use probabilistic modelling instead of using a single value for a variable such a mean value (deterministic approach). Discount rates are usually utilised in such models and to define the time value money, consequently the cost of relative actions carried out at different times in future. This can be expressed as (Tilly, 1996):

\[
PV = \frac{C}{(1 + r)^t}
\]

(2.11)

Where \(PV\) is the present value cost; \(C\) is the current cost; \(r\) is the discount rate expressed as fraction and \(t\) is the time period in years between the present day (the year which the evaluation is undertaken) and the year in which costs arise.
The following parameters are required to perform the present value analysis of estimating the overall cost of a structure throughout its entire life:

- current cost
- discount rate
- life of structure.

2.7.1.1 Current cost

Current cost refers to all costs associated with the various stages of the bridge's life. The two most-known cost functions that can quantify the total cost are proposed by D.M. Frangopol (Frangopol et al., 1997) and by F.A Branco and J.de Brito (De Brito & Branco, 1996). In these functions, costs that can be included are: initial cost (design and construction), inspection cost, maintenance cost, repair costs, failure costs and for the latter cost function also the benefits during the life cycle.

In cases where only for a particular type of maintenance, for example, preventative maintenance the whole-life costing is needed, Equation 2.12 can be written as:

\[ C_{PM} = \sum_{i} \frac{C_{PM(i)}}{(1 + r)^{t_i}} \]  

(2.12)

Where \( C_{PM} \) is the present value sum of the discount cost of \( j^{th} \) PM applications options \((PM_{(j)})\) applied at time \( t_i \).

2.7.1.2 Discount rate

Discount rate is a base interest which is used to determine the present worth of a future value. The use of appropriate discount rate is essential as this affects the WLC profoundly as shown in Figure 2.19. Alternatives are therefore very sensitive to the discount rate therefore it is advisable to carry out a sensitivity analysis.
In the UK discount rates for infrastructures such as bridge have ranged between 6% (QUADRO 4 manual, 2002) and 8% (BD 36/92, 1992) per annum while other countries used discount rates ranging from 2% to 20% (Vassie, 2000).

![Discount factor Vs time](image)

**Figure 2.19: Discount factor Vs time**

### 2.7.1.3 Life of structure

Costing is to be carried out over the life of the structure therefore the end point should be defined. In the case of a bridge the design life is typically assumed in the UK as 120 years (Das, 1999). Though there are various notions of ‘bridge life’ such as structural life, service life etc (Appendix B). The life of the structure should influence the selection of the discount rate. However the use of high discount rate (i.e. 8%) is in contradiction with engineering judgement and bridge life since, for example the cost of replacement of the bridge at 60 years is approximately 1% of the initial cost (Jones & Cusens, 1997). Based on the Green Book (2003) for long term (beyond 30 years) assessment lower discount rate should be used.

### 2.7.2 Traffic cost

In the UK besides estimating the direct costs of construction (e.g. maintenance replacement) a computer program QUADRO 4 (Queues And Delays at Roadworks), released in 1982, is used to calculate other ‘invisible’ costs such as
(Chrysanthopoulos, 2001): Delay (i.e. extra time costs) due to slower running speeds; delay due to queues; delays due to incidents; increased vehicle operating costs.

QUADRO 4 to operate requires a network. A typical QUADRO 4 network is as shown in Figure 2.20 (Jones, 1995).

Figure 2.20: QUADRO 4 network (Jones, 1995)

The line segment $AB$ represents the main route where the arc $AB$ represents the diversion route. Point $C$ is the first upstream junction.

The QUADRO 4 system is generally for use in rural areas (Wyatt, 1995). The aim of this program is to minimise delay costs while achieving the best possible combinations of traffic management and when it is best to carry out the work (Ryall, 2001). It is a cost effective program that refers to the evaluation of alternatives according to their cost and their effect regarding their delaying schemes unlike COBA 11 (2002) which is a cost benefit analysis program that measures the benefits arising from maintenance works in monetary terms (Jones, 1995). Furthermore, QUADRO 4 appraises maintenance under uncertainty by incorporating two sets of national traffic forecasts ('high growth' and 'low growth') (QUADRO 4, 2002).

2.7.3 Limitations of WLC

Despite the advantages of using WLC to compare and appraise maintenance cost based strategies, there are a number of limitations (Ferry & Flanagan, 1991; Leeming, 1993; Das, 1999$b$; Vassie, 1999):

- Artificiality of method. It is difficult to justify the need to put aside sums of money for doubtful future expenditure. Also the maintenance expenses usually come from other budgets than the capital payments.
- Uncertainty of maintenance needs. With the design life of the bridge being 120 years it may be seen as a doubtful exercise to try to determine the maintenance needs of the bridges built today for the whole of their life. Furthermore, a large proportion of the bridge stock ceases to be functional long before the end of their design life.

- Need to consider different options. WLC should enable the choice of various options as long they can be justified through rational comparison. Also by considering the long life of the structure radical management options such as 'Do minimum' may provide better (minimum) cost based strategies.

- Long life structures. Using the present recommended discount rate for the bridges in the UK (6%; Tilly, 1996) all long term future costs become negligible. This gives an in-built advantage to short life or low initial maintenance options without necessarily to give the best whole life costing strategy.

- Bridge elements and components. Bridges are composed with different elements and components, which have different expected service life and different degree of difficulty for their replacement or repair. These features are not included in present procedures of WLC.

- Traffic delay costs. These costs tend to be so high that all other costs become unimportant by comparison. Furthermore the traffic delay cost is basically a 'notional' rather than a 'real' cost so it is questionable whether they should be included with real costs such as maintenance costs. Also if delay costs are included the bridge owner may have to choose an option with higher initial cost and lower user delay cost. The beneficiary of this decision will be only the road user while delay cost being a 'notional' quantity it is impossible to reimburse the spending agency directly for selecting such an approach. Finally traffic delay costs are very sensitive to specific bridge values (such as traffic flow rate) therefore the collection of particular data are necessary. On balance it maybe better to record and report delay costs separately from the engineering costs.
While the above criticisms are true in some degree, the method should not be dismissed but used to addresses the problem of maintenance in bridges. Sensitivity analyses can narrow the limitations. Also optimisation techniques should be considered to enable the applications of various option of maintenance when is needed while considering the long life of the bridges.

2.8 Bridge maintenance optimisation procedures

Optimisation is the process, act or methodology that searches the extremum (maximising or minimising) of particular properties that will give the best possible solution to make something more functional, effective or better but maybe within given criteria and constraints.

Optimisation can be used primarily as a project level management tool, even though the aggregation of the results of individual bridges can provide a network maintenance program, despite of some disadvantages (e.g. unable to combine pavement and bridge maintenance). Furthermore, optimisation algorithms can take into consideration predictable process as for example natural deterioration. In case of any unpredictable course of action such as accidental damage, vandalism, natural disasters or political factors these algorithms cannot respond therefore engineering judgment is necessary to deal with maintenance actions arising from such events (Woodward et al., 2001).

2.8.1 Optimisation techniques

Optimisation algorithms can be grouped into six categories (Figure 2.21) though neither of these categories are necessary mutually exclusive (Haupt R & Haupt S, 1998).
A brief description of Figure 2.21 is given below:

1. Trial and error optimisation is achieved by changing the parameters that affect the output without necessarily knowing the process. Experimentalists usually use this approach in contrast with the theoreticians that prefer to use mathematical functions to represent the optimisation.

2. When only one parameter is involved in the process then the optimisation is one-dimensional while if there are multiple parameters the optimisation is multi-dimensional. The degree of difficulty of the optimisation is proportional to the number of dimensions.

3. Dynamic optimisation refers to the process where the output is a function of time where in static optimisation the output is independent of time.

4. Optimisation can have either discrete or continuous parameters. Discrete parameters have a finite or countable infinite number of discrete values whereas continuous parameters have infinite number of values (Thoft-Christensen & Baker, 1982).

5. The parameters of the optimisation can be constrained meaning that the parameters can take values that are based on certain limits or conditions.
Unconstrained parameters are the parameters that can take any values. In cases where the optimisation process can not work with constrained parameters, they can be converted into unconstrained by transforming the variable of the parameters.

6. Traditional optimisation algorithms are generally based on calculus methods in contrast with natural algorithms that they represent process in nature. These two methods are the most commonly used for optimisation problems and are further presented in the following sections.

2.8.1.1 Traditional algorithms

Traditional or conventional optimisation algorithms are search procedures currently available to obtain optimum solutions. Their main disadvantage is that they can easily be trapped into some local minimum and so fail to find the minimum value. Some traditional optimisation algorithms are:

- Exhaustive search. This method tests each possibility sequentially in order to determine the solution. This exhaustive examination of all possibilities is known as exhaustive search, direct search, or the ‘brute force’ methods. However in order to find the optimum it requires fine enough sampling but it may take extremely long time to find the global minimum. Another drawback of this method is that the global minimum may be missed due to under sampling. Therefore exhaustive searchers are more practical for small number of parameter within a limited search space.

- Analytical optimisation. Analytical optimisation techniques that are based on the calculus approach are also referred to as gradient methods. They subdivided into two main classes (Goldberg, 1989): indirect and direct. Indirect methods set the gradient of the objective function equal to zero consequently; the analytical derivatives existence is a necessity. With two or more parameters in the function the roots of the gradient of the functions will give the minima. Direct methods hopped on the function and move in the direction related to the local gradient (hill climbing) and therefore to the
steepest direction. The major drawback of these methods is that they often find the wrong minimum (a local minimum rather than the global minimum). Even if hill climbing method is combined with random search (iterated hill climbing) can still fall into the local minimum since each of the random searches is carried out in isolation and the shape of the domain stay unknown and no mechanism is provided to focus the random search into high fitness regions. Therefore their robustness in unintended domains can be questioned.

- Optimisation based on line minimisation. At these methods an algorithm begins at some random point on the function surface, selects a direction to move and moves in that direction until the functions begins to increase. After that, the procedure is repeated in another direction. It is clear that the choice in which direction to move is critical for the algorithm to converge. Therefore a variety of approaches for deciding a sensible direction have blossomed like the quadratic programming. Despite successive improvements they manage to increase the speed of convergence but without improving the algorithm ability to find the global minimum instead of global minimum.

- Random search methods. According to these methods, points in the search space are selected randomly while saving the best solution. This is not a very intelligent strategy.

2.8.1.2 Natural algorithms

Natural optimisation algorithms use random search and probabilistic calculation to model the nature optimisation. They do not require the derivation of functions and can deal with discrete and continuous parameters. In some cases they can be slower than traditional optimisation methods but typically they can implement a large number of parameters with great success at finding the global optimum. These techniques have great success in a number of areas. Optimisation algorithms that fall into this category are:

- Simulated annealing. This technique was introduced in the early 1980s by Kirkpatrick & co-workers (Kirkpatrick et al., 1983). It draws its
inspiration from the annealing process, a technique involving heating and controlled cooling of a substance. When heated it gains energy and its molecules leave from the initial position (a local minimum of the internal energy) and start move freely to respect to one another. As it cools slowly, the thermal mobility is lost and the molecules try to find a different configuration with lower internal energy than the initial one. The essence of this process is the slow cooling, to allow sufficient time for redistribution of the atoms as they lose mobility. The algorithm analogue of this process involves the representation of the examined function as the energy level of the substance, that needs to be minimised, and the control parameter that sets the step size for the change in the parameter value as the temperature. The algorithm employs a random search which not only accepts changes that decrease objective function, but also some changes that increase it. Therefore the control parameter will force the algorithm to explore the space and reach to an optimum solution. Although is been proven that can reach global optimum (the travelling salesman problem; Kirkpatrick et al., 1983) it has some disadvantages like the difficulty to chose the rate of cooling and initial temperature as well it may need an infinite computer time since only one solution is carried over from one iteration to the next.

- Neural networks. The Artificial Neural Network (ANN) is an information processing paradigm that is inspired by the way the biological nervous systems and the brains work (Stergiou & Siganos, 1996). A neural network consists of a number of sub-units called neurons (N). These are interconnected in parallel to form a network as shown below (Figure 2.22). Neurons that perform similar functions are often grouped into layers. There are three types of layers. The input layer receives data from the user program where the output layer of neurons sends data to the user program. Connecting the input layer and output layer are hidden layers which only connect to other neurons and never directly interact with the user program.
Neural networks have the ability to learn how to perform tasks based on examples or initial experience without the need to be programmed based on algorithm. However, the individual relations between the input variables and the output variables are not developed by engineering judgment so that the model tends to be a black box or input/output table without analytical basis. Furthermore, it may require a large sample size that needs a great deal of computational effort and a lot of examples of the required behaviour. The disadvantages appear to outweigh the advantages, particularly in view of the black box effect.

- Genetic algorithms. Genetic algorithms are inspired by the Darwinian theory of evolution. A genetic algorithm is an optimisation algorithm where the solution is obtained through an evolutionary process that combines the mechanisms of natural selection and natural genetics with randomised genetic operators such as reproduction crossover and mutation (Haupt R & Haupt S, 1998; Davis, 1991). Holland (1975) proposed GA. Goldberg was the first to solve engineering optimisations problems using GA (Goldberg, 1983).
The genetic algorithm is almost the same in most applications only the functions are changing. The general algorithm is as follows (based on Davis, 1991):

1. Initialise a population of chromosomes.
2. Evaluate each chromosome in the population.
3. Create new chromosomes by mating current chromosomes; apply mutation and recombination as the parent chromosomes mate.
4. Delete members of the population to make room for the new chromosomes.
5. Evaluate the new chromosomes and insert them into population.
6. If time is up, stop and return the best chromosome; if not, go to 3.

Genetic Algorithm has many advantages. Some are: can work with both discrete and continuous parameters; no need for derivation of the function; can search simultaneously in a population of points not a single point, can be used for optimisation problems with complex functions, can incorporate a large number of parameters; etc (Haupt R & Haupt S, 1998; Falkenauer, 1998; Pezeshk et al., 1998). These advantages help to produce impressive results from suitable problems when the other optimisation methods fail. Its disadvantage lay to the speed of reaching an optimum solution but with the even more increasing speed power that currently personal computers have this problem tends to cease.

2.8.2 Existing optimum maintenance/repair/inspection strategies

From the early sixties the area of optimising maintenance through the use of mathematical models was exploited. This pioneering work is summarised in McCall (1965) and Barlow & Proschan (1965).

In current bridge management systems optimum maintenance strategies are determined by recording the present condition states of the bridges and their
elements and then using deterioration prediction models related to different maintenance regimes.

Some papers describing the application of traditional methods for the optimisation of repair/maintenance strategies in the field of civil engineering are presented further.

Thoft-Christensen (1995) proposed that to obtain the optimal repair type after a structural assessment it is necessary to consider for each repair action the following optimisation problem:

\[
\max_{T_R, N_R} W(T_R, N_R) = B(T_R, N_R) - C_R(T_R, N_R) - C_F(T_R, N_R)
\]

\[
\text{s.t. } \beta^U(T_R, T_L, N_R) \geq \beta^{\text{min}}
\]

Where \(N_R\) is the expected number of repairs; \(T_R\) is the time of the first repair; \(W\) is the total expected benefits minimum costs in the remaining lifetime of the bridge; \(C_R\) is the repair cost capitalised to time \(t=0\); \(C_F\) is the expected failure costs capitalised to time \(t=0\); \(T_L\) is the expected lifetime of the bridge; \(\beta^U\) is the updated reliability index and \(\beta^{\text{min}}\) is the minimum reliability index for the bridge (related to a critical element or the total system). The Bayesian statistical theory is used to update the basic variables. Therefore the final decision is not only influenced from the results obtained by solving the optimisation problem but also on expert knowledge.

Henriksen (1996) said that it may be possible through experience obtained from a large number of special investigations of RC structures to carry out qualified service life calculations and estimate the time for repair based on the deterioration rate. Combined with the relevant costs optimum repair strategies for RC bridges can be developed to such level that they will be sufficient for real-life bridge engineering application. Henriksen (1996) describes "the concept of the mobile laboratory, the testing procedures and calculation tools used, their practical background from several hundred bridge inspections", and how they are used in the process of optimisation of repair work. Even though he states that the final choice of strategy will not be based only on the traditional 'best and cheap' model but also on the risk involved in the individual repair options he does not give any further details on this issue.
Frangopol et al., 1997 used the event tree as a model to represent all possible events associated with repairs, or no repair actions, and obtained the optimum strategy by solving the following problem:

\[
\min C_{ET} \text{ subjected to } P_{f,\text{life}} \leq P_{f,\text{life}}^* \tag{2.14}
\]

Where \( C_{ET} \) is the total expected cost of the strategy; \( P_{f,\text{life}} \) is the lifetime failure probability and \( P_{f,\text{life}}^* \) is the maximum acceptable lifetime failure probability.

Frangopol & Estes (1997) proposes "a methodology for a system reliability-based condition evaluation of existing highway bridges". An optimum lifetime repair strategy can be developed based on minimum expected costs while maintaining an acceptable level of safety. The authors reported that with the use of biennial visual inspections and specific non-destructive evaluation testing can aid to update the initial optimum repair strategy.

Sorensen & Englelund (1997) as well as Englelund & Sorensen (1998) proposed to select the optimal strategy using a probabilistic approach. Probabilistic models for carbonation front and for chloride ingress are employed to determine the probability that a given repair strategy is used at a given time. Uncertainties related to inherent physical uncertainties, statistical uncertainties and model uncertainties are incorporated through the probabilistic models. Bayesian statistics can be used to obtain statistics for the parameters of the models from measurements. From different formulated strategies (Englelund & Sorensen, 1998) the optimum one is identified from the minimum expected costs and the ability to keep the probability of failure at acceptable level. An example case presents the determination of the optimal strategy for repair and maintenance of a coastal bridge pier. However, some prior beliefs of the decision-maker are incorporated in the reliability based analysis.

Estes & Frangopol (1999) proposes "a system reliability approach for optimising the lifetime repair strategy for highway bridges". The approach is demonstrated on an existing highway bridge. The bridge is considered as a series-parallel system than a collection of individuals components. The reliability of the bridge system is computed using time-dependent deterioration models and live load models. The
repair strategies are developed by testing all feasible combinations of the repair options for keeping the system reliability above target reliability. The optimum strategy is chosen based on its WLC. The conclusions reveal that there are some limitations in this study such as the analysis is related to strength-based consideration only and the minimum acceptable system reliability was arbitrarily established.

In the same context Frangopol et al., (1999) suggested obtaining optimum maintenance strategies for different bridge types by choosing the least expensive present value of the expected cumulative cost of predetermined strategies that are based on results from earlier research projects and operational experienced.

Estes & Frangopol (2000) summarise “a methodology for optimizing the timing, the frequency, and the type of inspection over the expected useful life of deteriorating structures”. For developing the inspection plans a decision tree analysis is used. The optimum strategy selected from the minimum WLC can be updated after inspections (either visual or non-destructive evaluation) to incorporate new data. This methodology is illustrated using only one inspection technique (a half-cell potential test) on a deteriorating concrete bridge deck. The study of several inspection techniques in combination will need further investigation.

The difficulty with these traditional optimisation algorithms lies in the formulation of the maintenance strategies, satisfying multiple and usually conflicting objectives and incorporating a large number of options. When it comes to searching the optimum strategy traditional methods use point-by-point basis therefore multiple algorithms are needed.

By reviewing the literature GA method seems to outweigh the other optimisation methods for multi-objective criteria with a large number of options. A brief review of some papers describing the application of GA in the field of bridge maintenance/repair/inspection/rehabilitation strategies is presented below.

Liu et al., (1996) employed GA to achieve optimisation of long term maintenance strategy of bridge decks. The maintenance methods used were routine, repair,
rehabilitation and replacement. Liu et al., (1996) stated that GA was effective in representing and processing the maintenance strategy optimisation problem.

Liu et al., (1997) developed a multiobjective optimisation GA method to solve the rehabilitation optimisation problem of bridge decks. The results of this work suggest that a multiobjective optimisation GA can be used effectively to achieve minimum cost program with minimum deterioration degree of bridge decks.

Furuta et al., (1997) have presented a decision-making supporting system for the paint maintenance program of existing bridge. This tool it was intended to be used by inexperienced engineers. Using the GA technique was possible to produce optimum repair program of a group of existing steel plate and box girders bridges which in the opinion of the authors of this paper are applicable for practical use.

Furuta et al., (1999) have presented an optimisation method using GA for the life-cycle cost design of RC T-girder that deteriorate over time to determine the time, quality and number of inspection and repair. The applicability of the proposed methodology is presented with a numerical example. Furuta et al., (1999) concluded that GA is “a promising technique to solve practical life-cycle optimisation problems for civil infrastructure systems”.

Miyamoto et al., (2000) employed GA to identify optimum maintenance plans as part of a new developed BMS for deteriorated concrete bridges. The effectiveness of the maintenance actions used was in relation with their effect on the load-carry capability and durability of the bridge. They concluded that “GAs are a powerful tool for obtaining an optimal maintenance plan”.

Suzuki et al., (2000) have considered the development of a tool capable of identifying the optimum plan that minimises the costs of inspection, repair and reinforcement for RC structures using GA. Even though they were successful in producing optimum plans they recommend that if the results are to be put to practical use, data for accurate quantitative evaluation of the effects of repair or reinforcement are essential.
Liu & Frangopol (2004) presented a multiobjective GA that takes into considerations the uncertainties associated with the deterioration process under no maintenance and under maintenance. Three objective functions, namely condition index, safety index and whole life maintenance costs were evaluated. However, only the use of one maintenance option (silane) was examined. Based on this paper the use of GA successfully aids to the development of optimal maintenance plan.

2.9 Summary of reviewed literature

The review study presented here was an attempt to gain an appreciation and understanding of current techniques and philosophies available for the estimation of the effectiveness of PM measures as well as for their sequence of application with the aid of optimisation methods. The conclusions drawn support the development of a methodology that will quantify the effectiveness of preventative maintenance measures and applied them in an optimum approach.

This chapter covers an introduction to bridge management (BM) and bridge management systems (BMS), briefly describing generic issues and identifying the maintenance optimisation procedures used. New and improved techniques for the development and enhancement of BMS are currently being developed. The evidence supports the view that the next generation of BMS will be based upon lifetime reliability and whole life costing. However sound engineering judgement is required to make a balanced final decision on what is reasonable and applicable. Current BMSs are not developed to manage bridges but rather to provide a tool to collect and process all the available information and to produce reports to aid the engineer in the decision-making.

Bridges of all types are associated with various forms of degradation. However, whilst the costs of maintenance are constantly increasing, funding is generally inadequate to allow indiscriminate repair of the entire bridge network in any one funding year (Cole, 2000). It has been suggested (Das, 1999a; Frangopol et al., 2000) that PM can help to reduce the total cost of maintenance needed for the bridge useful life. The thrust of this study is the need to evaluate the effectiveness of PM measures and their possible application as optimum strategies for maintenance of
bridges. The surveyed literature on the estimation of effectiveness of PM is deemed to remain limited. Some researchers (Vassie & Arya, 2000) attempt to produce the effectiveness qualitative, where others (Liu & Frangopol, 2004) prefer to show the effectiveness quantitative. Qualitative procedures content to state the particular PM measures than can protect against the chloride and carbonation mechanisms and give general but relevant information for PM measures though they miss the what-if analysis. On the other hand models which illustrate the effectiveness quantitatively can incorporate uncertainties corresponding to the process and use the what-if analysis. However, these models make certain assumptions regarding the deterioration process.

Uncertainties in processes lead to the need for use of reliability analysis to evaluate the performance of bridge or bridge elements. The literature review shows clearly that this technique is constantly gaining ground. Eurocode BS EN 1990 (2002) (Annex B and C) appreciates the importance of estimation of the reliability of structures and implements the reliability analysis not only for the calculation of partial factors but also provides a general framework for the differentiation of different acceptable reliability levels.

Furthermore whole life costing (WLC) is undoubtedly a very important tool for assessing the future work costs when there are limited budgets.

Optimisation procedures are available to provide a numerical tool for the identification of optimum strategies. In the last decade there has been a shift away from the use of traditional optimisation methods towards natural methods especially Genetic Algorithm (GA) for the optimisation of maintenance/repair/inspection/rehabilitation schemes. Even though several methods have been developed for the optimisation of strategies only a small fraction of papers is dealing with the optimisation of maintenance strategies. Nevertheless optimum maintenance strategies for bridges have been produced (Frangopol & Estes, 1997; Sorensen & Engelund, 1997) but it has not been attempted to produce a numerical tool for identifying optimum PM strategies to delay / prevent a specific deterioration mechanism.
The literature survey has clearly showed that PM is an essential part of the structural maintenance of bridges, which can provide reduced whole life maintenance cost while increasing the life of the bridges. To quantify their effectiveness of PM measures and enable the development of optimum strategies for maintenance of bridges it is necessary to develop a procedure to estimate the effectiveness of PM against the main deterioration process of specific bridge type. Due to the uncertainty associated with this procedure reliability techniques should be utilised. PM application can be focussed into groups of similar components or individual critical components since in BS EN 1990 (2002) "the requirements for reliability are related to the structural members of the construction works only, and not to the whole structure" (Gulvanessian, 2004). Whole life cost method can supply the total cost of the strategies developed. The limited budget allowance and the need for reliability indicate that there could be enormous benefit from optimising these strategies. Therefore there is a need for an optimisation methodology to link the effectiveness of PM measures with their associated costs and impact on reliability. Regarding optimisation techniques GA seems to be more promising than other traditional or natural optimisation methods.
Chapter 3

Deterioration of RC bridges and related PM measures

3.1 Deterioration in UK RC bridge stock

The aims, objectives and area of this research were introduced in Chapter 1 and 2. This chapter will identify the preventative maintenance (PM) measures that can be applied on RC bridges to prevent or delay the deterioration process. An understanding of the basic underlying causes of deterioration is necessary to point in this direction.

Eighty percent (≈8,790 bridges) and thirty four percent (≈43,860 bridges) of the bridge stock owned by HA and local authorities respectively, are built predominantly of concrete (Mallett, 1986; Department of Transport, 1987; Daly, 1999). This is to be expected since concrete is an effective structural material, relatively inexpensive, easily worked, and the materials from which it is made of can be found in most locations. Concrete has a generally high compressive strength, but relative low tensile strength. To overcome this deficiency when concrete is used as a structural material, metallic reinforcement (originally iron and then steel) is included in areas where tension occurs. As a consequence concrete is nearly always reinforced with steel embedded within it although other, non-metallic, reinforcement materials are available.

Consequently, the following sections will address the deterioration causes of conventional reinforced concrete (RC) bridges. Despite the complexity of several causes acting simultaneously it is possible to identify the primary cause of
deterioration of RC structures in the UK as resulting from chloride ions ingress due to the use of chloride based de-icing salts (Wallbank, 1989).

The last section of the chapter presents recommendation for preventing or delaying the initiation of corrosion of RC bridges with the use of specific PM measures.

### 3.2 Main causes of deterioration in reinforced concrete

The deterioration of reinforced concrete structures can be due to various degradation mechanisms, which affect either the concrete, or the steel reinforcement (Leeming, 1990; BA 35/90, 1990; CEB, 1992; Iffland & Birnstiel, 1993; Emmons, 1993 U.S. Army Corps of Engineers, 1995; Neville, 1995; BS EN 206-1, 2000; Duracrete, 1998, Mulheron, 2000a & 2000b; Pullar-Strecker, 2002) such as:

- **External (exposure) conditions:**
  - reinforcement corrosion induced by carbonation of the cover concrete
  - reinforcement corrosion induced by chlorides other than from seawater
  - reinforcement corrosion induced by chlorides from seawater
  - freeze/thaw attack
  - chemical attack such as sulphate attack, acid attack
  - fire damage
  - vibration.

- **Internal concrete conditions:**
  - alkali-silica reaction
  - soft water attack.

- **Mechanical damage:**
o accidental damage
o fatigue deterioration
o overloading
o malfunction of bearing and joints
o drying shrinkage/creep
o poor drainage.

• Human mistake:
  o design errors
  o construction errors
  o inappropriate, unsuccessful or lack of maintenance.

All of these forms of deterioration occur at different degrees and at various rates; therefore it is more practical to identify one dominant deterioration mechanism. Reinforcement corrosion due to chloride ingress (by de-icing salt) is generally accepted as the most dominant cause of RC highway bridge deterioration in the UK (Wallbank, 1989). It is estimated that the annual cost of UK concrete structures repairs, due to steel corrosion, costs over £500 million (Hobbs, 1996). Consequently, the developed methodology in this study (Chapter 4) will focus on this mechanism. If this proves successful then the methodology could be expanded subsequently to incorporate the remaining mechanisms.

3.3 Electrochemical principles of reinforcement corrosion

An electrochemical reaction involves a chemical reaction in the presence of an electrolyte, in which a transfer of electrons takes place between the reactants. Electrochemical principles of reinforcement corrosion in concrete have been described in detail in many textbooks such as Neville (1995), Mattsson (2001), Mulheron (2000b) etc. A brief description of the corrosion phenomenon is as follows.
The corrosion process initiates when the passive film (§ 3.4.1) breaks down (depassivation), and it can be destroyed by chloride attack (§ 3.9).

Once the protective oxide film has been impaired, in the presence of oxygen and moisture, corrosion of the reinforcement, usually mild steel, will occur. Considering steel to be primarily an alloy of iron and carbon, the general process of aqueous corrosion of steel will proceed, in simplified form, as shown in Figure 3.1.

![Figure 3.1: Corrosion process is an electrochemical reaction](image)

Anodic Reaction

\[ Fe \rightarrow Fe^{2+} + 2e^- \]

Cathodic Reaction

\[ \frac{1}{2} O_2 + H_2O + 2e^- \rightarrow 2OH^- \]

These two half-reactions work in tandem and can be written:

\[ Fe^{2+} + 2OH^- \rightarrow Fe(OH)_2 \]

In the presence of oxygen this further oxides to form:

\[ 4Fe(OH)_2 + O_2 + H_2O \rightarrow 4Fe(OH)_3 \]

which in the limit forms red rust:

\[ \rightarrow 2Fe_2O_3 \cdot H_2O (RUST) + 4H_2O \]

*Where: Fe: iron, Fe^{2+}: ferrous iron, e^-: free electron, OH^-: hydroxyl ion, Fe(OH)_2: ferrous hydroxide, O_2: oxygen, H_2O: water, Fe(OH)_3: ferric hydroxide*

In any electrochemical mechanism, an anodic (oxidation, emission of electrons) and cathodic (reduction, consumption of electrons) reaction must occur simultaneously. The electrons lost in the anodic reaction must be gained in the cathodic reaction so one reaction cannot proceed without the other (Figure 3.1). The oxygen and moisture...
availability allows these reactions (anodic and cathodic) to continue and as a result the anode will be subjected to oxidation while the cathode will remain intact.

It is worth adding that previous studies have shown (Al-Tayyib et al., 1990) that the pre-rusting of rebars in the atmosphere will not influence their corrosion behaviour when they are embedded in concrete, as long as there is no loose rust on the surface of the reinforcing steel (Neville, 1995).

### 3.4 Physical protection mechanisms of reinforcement corrosion

From experience it is known that when the reinforcement, usually mild steel, is left outside unprotected from water and air it will start to corrode. This has been attributed to the fact that metallic steel is thermodynamically unstable with respect to the original ion oxide products from which it was extracted and so it will react with the environment (Mulheron, 2000b). However, extensive corrosion will not be observed if the steel is embedded in concrete despite the fact that concrete is porous and contains moisture. But why is that? First, the concrete cover forms a physical protective barrier which significantly reduces the penetration rate of aggressive agents in the environment increasing the time before they can reach the level of the steel reinforcement. Consequently the protection level rises as the impermeability, thickness and integrity of the concrete cover increases. Generally the less permeable, more intact and thicker it is, the better the protection. Second, concrete, due to its highly alkaline composition, in most circumstances provides a good protection to the embedded reinforcing steel that is generally attributed to three physical mechanisms. These are the passivity of the steel surface, resistive control from the concrete cover and cathodic control, with the most important being passivity (Pullar-Strecker, 2002).

#### 3.4.1 Control of corrosion by passivity

Metals corrode in acid but Portland cement concrete is highly alkaline with a pH of 12 or more. The alkaline environment offered from concrete leads to the development of a ‘passive’ layer, on the steel surface. It is a complex iron
oxide/hydroxide which prevents oxidation (corrosion) (Broomfield, 1997). This film is strongly adherent to the surface and although thin is relatively impermeable to the passage of oxygen and water and acts to insulate the surface of the metal both physically and electrically. Although ordinary steel reinforcement in concrete is still thermodynamically not stable, the corrosion rate is depressed to an insignificant low level due to the formation of this barrier layer (Sandberg et al., 1995).

### 3.4.2 Resistive control of corrosion rates

The electrical resistance of the concrete termed resistivity is defined as the resistance between the opposite faces of a 1 cm cube. The corrosion current developed (Figure 3.1) is restricted by the limited pore fluid in the few and fine inter-connected capillaries. Even if passivity is lost in very dense, or dry, concrete, any corrosion of embedded steel will be slow. Corrosion is then said to be under resistive control. It is often due to this mechanism that permanently dry concrete structures are protected from corrosion. However, high resistivity can be observed even in saturated concrete if the capillaries are low in water. An increase in resistivity can be achieved with the application of surface treatments since they allow water vapour to escape but block the entrance of liquid (Pullar-Strecker, 2002).

Table 3.1 (cited in Pullar-Strecker, 2002) gives a widely accepted classification of the likelihood of corrosion occurred in relation to resistivity.

<table>
<thead>
<tr>
<th>Resistivity (ohm-cm)</th>
<th>Corrosion risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;5000</td>
<td>Very high</td>
</tr>
<tr>
<td>5000-10000</td>
<td>High</td>
</tr>
<tr>
<td>10000-20000</td>
<td>Moderate to low</td>
</tr>
<tr>
<td>&gt;20000</td>
<td>Insignificant</td>
</tr>
</tbody>
</table>
3.4.3 Cathodic control of corrosion

In order for the corrosion process to take place sufficient oxygen is necessary at the cathode. If oxygen access is very restricted at the part of the reinforcement that cathode would potentially form, the current will stop flowing. Consequently, since current at the anode can only flow when there is an equal flow at the cathode the overall corrosion reaction will stop. It is then said that corrosion is under cathodic control. However, this condition can usually only be seen in isolated units that are completely saturated (Pullar-Strecker, 2002).

3.5 Types of corrosion

There are two types of corrosion generally encountered in the deterioration of steel reinforcement: (a) general (uniform), and (b) localised (or pitting) corrosion (Broomfield, 1997). Uniform corrosion is characterised by corrosive attack under normal conditions, proceeding evenly over the entire surface area, or a large fraction of the total area. The anode areas are large and the cathode areas are relatively small so the local corrosion rate at the anode is slower compared to pitting corrosion. Pitting corrosion is a localised form of corrosion, observed in cases where only a small area of steel looses its passive layer, usually due to a significant local concentration of chloride ions. This type of corrosion is characterised by a large cathode area and a small anode area resulting in accelerated corrosion at the anode. Pitting corrosion produces cavities or 'holes' in the steel. This makes it particularly dangerous for prestressed concrete.

Corrosion of steel in concrete generally starts with the formation of pits. However these pits increase in number, expand and join up leading to generalised corrosion, usually seen on reinforcing bars exposed to chlorides ions (Broomfield, 1997). Accordingly, this research will focus on the effects of chloride ions induced general corrosion on the deterioration behaviour of the steel reinforcement in concrete.

3.6 Structural effects of corrosion

When corrosion is initiated, firstly loss of steel section and consequently change in material properties (strength, modulus) will be observed.
Secondly, since the products of the corrosion, rust, can occupy several times the volume of the original material (2-8 times), cracking and ultimately spalling and delamination will become evident on the surface of the concrete (Figure 3.2). Consequently, it will result in a reduced concrete cross-section. Also due to the presence of corrosion products, not only deterioration of concrete will take place but also the bond between the steel and concrete can be expected to deteriorate with time.

Figure 3.2: Example of deteriorating bridge element (Koch et al., 2001)

3.7 Consequences of corrosion – need for protection

There are many consequences of the corrosion of RC structures that an effective BMS must take into consideration. These include:

- Loss of structural capacity of RC elements that may lead to failure it is regarded as the most serious consequence. However, based on experience in RC bridges (Engelund & Sorensen, 1998), cracking and spalling (distinct signs of corrosion) will generally occur about 10-15 years after the initiation of corrosion giving enough warning before reaching the ultimate structural capacity limit of the structure.

- Increased economic and environmental cost caused by closures, and lane or/and weight restrictions due to inspections, maintenance, repair or replacement.
- The reduction of the service life of the bridge or bridge element.
- Cost of injury, or damage, caused by spalling.
- Aesthetics: Rust stain and loss of concrete cover can cause many people to think that the bridge is appalling or even dangerous to be used.

Therefore it is clear that a prevention of corrosion will gain tremendous benefits to the bridge owners as well as the public.

### 3.8 Modelling the reinforcement corrosion

The theoretical model shown in Figure 3.3 was first proposed by Tuutti (1982) and divides the reinforcement corrosion process into two periods: an initiation period and a propagation period. The initiation period consists of the time from the construction of the structure until aggressive agents (e.g. chlorides) reaches the rebars and depassivate the steel. The propagation period covers the time from the steel depassivation until the structure reached a certain unacceptable level of deterioration. The illustrated profile is adopted by many researchers such as; Mangat & Gurusamy (1987); Cady & Weyers (1992); Thoft-Christensen et al., (1996), Frangopol et al., (1997), Steward & Rosowsky (1998a) where in some cases additional stages were introduced to describe in more detail the deterioration process.

![Figure 3.3: Corrosion model (Tuutti, 1982)](image-url)
Also, an empirical corrosion model has been developed by the HA by applying statistical methods on results of field surveys of the Midlands Links motorways viaducts, as part of the new Midlands links risk-based repair strategy. The model was developed specifically for use on highway structure where the source of chloride is from de-icing salts. Empirical relationships such as between half-cell potential and corrosion rate, cover depth and corrosion initiation time have been developed. Furthermore the authors believe that with further calibration the model will form the basis for a more general-application model to provide information on the relative risk of corrosion (Robert et al., 2000; Cropper et al., 1999). A semi-empirical model has been proposed by Anstice & Roberts (2002) using as a basis the model developed by Roberts et al., (2000). “The deterioration model is used to predict the future deteriorated condition of the reinforced concrete bridge, or bridge element, in terms of reinforcement section loss and delamination of the cover concrete” (Anstice & Roberts, 2002).

3.9 Mechanism of corrosion: Chloride attack

3.9.1 Sources of chlorides

Chlorides can come from several sources. Existing chlorides in the concrete can be due to the use of seawater in the mix, the use of unwashed sea-dredged aggregates the use of additives/accelerators (calcium chloride, CaCl₂, was widely used until the mid-1970s; Broomfield, 1997) or from most cement (BS EN 197-1, 2000). BS 8110-1 (1997), states that chloride from all sources (i.e. cement, water, sand, filler, and admixture) should not exceed 0.1% by mass of the cement. In addition, chloride ions can diffuse into concrete from external sources, such as de-icing salts, exposure to seawater, use of chemicals or from airborne salt (Broomfield, 1997; Daly, 1999). For the UK concrete bridges, the primary source of chloride ions attack is due to the application of chloride based de-icing salts for winter maintenance (Wallbank, 1989) and this research will focus on this source. However, the cast-in chlorides should not be ignored, especially if they are part of the problem (Broomfield, 1997).
3.9.2 Need for de-icing salt

Some 1-2 million tons of chloride-based de-icing salt are used on roads, and road structures, each year in the UK, at a cost of around £40 millions on salt, £100m in environmental damage and a further £500m in traffic road user delay caused by winter conditions (Thornes, 1995; Burtwell, 1997; Broomfield, 1997). In the US about 15 million tons of salt are applied per year to highways (Broomfield, 1997; Salt institute, 2004). This practice provides an annual supply of chloride ions to contaminate RC highway bridges. In the UK alone, it is estimated that the annual cost of repairs of concrete structures due to steel corrosion exceeds £500 million (Hobbs, 1996). In the USA the annual direct cost of corrosion for highway bridges is estimated to be $8.3 billion (Koch et al., 2001). This raises the question, if salt can cause so much damage why has it been used so extensively for more than half a century?

Salt is a necessity to control snow and ice on roads. It provides safe personal and commercial mobility by keeping snow and ice from bonding to the pavement or even breaking the bond between pavement and ice. This is due to the fact that with the application of salt to ice and snow, brine is formed that has a lower freezing temperature (down to -21°C depending on the salt concentration in the solution) than the surrounding ice or snow (TRB, 1991). Research carried out by Marquette University (Kuemmel & Hanbali, 1992) established a link between salt application and accident reduction paralleling findings of an earlier German study (Hanke & Levin, 1988). In fact accident rates are reduced by 88.3% at spreading time as illustrated in Figure 3.4.
The primary type used is rock salt (sodium chloride, NaCl), which is plentiful and can be mined from the earth. Salt "is by far the most popular chemical deicer, because it is reliable, inexpensive and easy to handle, store, and apply" (TRB, 1991) although salt has temperature lower limit of -10 to -7 °C (15 to 20 °F). For lower temperatures more salting and time will be necessary than before for melting the ice (Wisconsin Transportation Bulletin No. 6, 1996). Furthermore, salt has been linked to many indirect costs including damage to the infrastructure, the environment along the roadside etc. New salt grading types (e.g. 6mm) and spreading techniques (e.g. pre-wet approach) are currently applied to reduce these negative effects (Lupton, 2004). These negative effects have lead to the search for alternatives de-icers that do not cause corrosion, have acceptable cost, are efficient in winter maintenance and may used at lower temperatures than salt can be applied. Unfortunately current alternative de-icers have some negative environmental impacts and most of them are most expensive than salt. For example urea or calcium magnesium acetate is ten and twenty times the cost of rock salt respectively (Vassie, 1996). This had lead to a slow pace of change of salt as the main de-ice even though data from UK and US studies, suggest that the true cost of salt is about 25 times its initial purchase price (Vitalliano, 1992; Thomes, 1995).
Therefore, the use of salt, currently, can not be avoided but its adverse side effects on the rebar of RC bridges can be prevented with the use of preventative maintenance measures that will delay/inhibit the chloride ingress.

The British Standard BS 3247 (1991) “specifies the essential properties of salt spreading on highway for winter maintenance and includes tests for certain of these properties”. Also it provides the types of salts (i.e. rock salt, vacuum salt and marine salt) that are used for winter road maintenance.

**3.9.3 Chloride ingress**

When chlorides ions in solution reach the concrete surface of a bridge structure, they can move into the concrete cover by various mechanisms. Capillary absorption and diffusion are the main mechanisms of chloride ingress into concrete. Capillary absorption is the process whereby the concrete takes in a fluid to fill spaces within the material through the capillaries. The movement of liquid, gas or ion under a concentration gradient, from a zone with high concentration to a zone with low concentration is called diffusion (Concrete Society, 1987). When concrete is fully saturated, chloride ions can only diffuse into the concrete. When the concrete is partially saturated, and the surface is subjected to successive cycles of wetting and drying (e.g. bridge piers), the chloride solution can be absorbed into the unfilled pores by capillary action (Bamforth & Pocock, 1990; Daly, 1999; Grace, 1994). Capillary absorption is a much more rapid penetration mechanism compared to diffusion but in most concrete structures is limited to the first few millimetres (e.g. 10-40mm) of concrete cover (Broomfield, 1997). In the case where there are surface defects /cracking or leaking joints, chlorides penetrate directly into the concrete. However if water is not present (concrete is dry) penetration is not possible by either mechanism (Daly, 1999).

Nevertheless, there can be principal mechanisms at different locations of a bridge. According to Vassie (1996), the most vulnerable locations to reinforcement corrosion are:

- Crosshead beams and piers under leaking expansion joints.
• The top surface of bridge decks if waterproofing membrane is defective.

• The lower parts of the vertical faces of piers, columns, and abutments that are in close proximity to the road due to salt borne in traffic spray.

Diffusion is the predominant mechanism in locations a) and b) whereas capillary action is dominant in location c). In this research, attention will be paid to the diffusion mechanism.

On the other hand, there are opposing mechanisms that decrease the chloride ingress. These include chemical reaction to form chloroaluminates and adsorption on the pores surfaces (Broomfield, 1997). These effects depend crucially on the cement type and degree of hydration of the concrete cover.

### 3.9.3.1 Corrosion model for chloride ingress to rebar level

The main mechanism discussed in this section will be the diffusion. To model the initiation stage (Figure 3.3) of chloride ingress (diffusion) in uncracked concrete, several mathematical equations have been developed. Some well-known models are:

- Fick’s law of diffusion. (Crank, 1975). Fick’s second law created in 1855 adopted Fourier’s equation of heat conduction, published in 1822, to model diffusion in liquids but not for chloride ions. Collepardi et al., (1970, 1972a, 1972b) was the first to realise that Fick’s law could be used to model chloride diffusion. Since then, Fick’s law has been used extensively as the basis for chloride ingress modelling. Due to the mathematical deficiency in finding a closed-form solution for chloride penetration is commonly evaluated using the error-function solution of Fick’s second law. When chloride transport into concrete is modelled it is assumed to be diffusion into a homogeneous and isotropic material in semi-infinite condition, with constant surface conditions. This is far from the reality of concrete cover material. However with these assumptions it is possible to express the consecration of chlorides in every point in concrete in future time.
- **DuraCrete model** (DuraCrete, 2000). The DuraCrete model is an empirical model for the prediction of ingress into concrete. The advantage of this model is that it is possible to directly use observations of chloride penetrations in predictions of future chloride penetrations. Also it considers the decreasing diffusivity of the concrete with increasing age. However, in order to solve the model it is required to have knowledge about initial and boundary conditions, both in laboratory and field conditions. Therefore specific data are required to be derived. More importantly, if no quality-check is made of the data used in the model, doubtful results maybe obtained (e.g. too large chloride penetration depths).

- **Mejlbro-Poulsen model** (Mejlbro, 1996). The model is a result of many comparisons and predictions made based on laboratory observations from marine exposure stations and investigations made on marine structures. The basis for the models is Fick’s second law of diffusion, thus the flow of chlorides is assumed to be proportional to the gradient of chloride concentration in the concrete. The main advantage of this model is that measured chloride profiles from field exposure can be used to quantify the parameters of the model. However there are some specific requirements for the evaluation of some parameters (i.e. chloride profiles exposed in the same environment and from the same concrete composition).

More chloride penetration models can be found in Duracrete (1998) report ‘Modelling of degradation’. However, the most commonly used model to describe chloride ingress is Fick’s law of diffusion.

### 3.9.4 Chloride attack mechanism

When chloride ions reach the reinforcement surface, they appear to act as catalysts to break down the passive layer. Verbeck (1975) describe chloride ions as ‘a specific and unique destroyer’. The reactions involved are as follows (Neville, 1975):

\[ Fe^{++} + 2Cl^- \rightarrow FeCl_2 \]  \hspace{1cm} (3.3)
FeCl₂ + 2H₂O → Fe(OH)₂ + 2HCl \hspace{1cm} (3.4)

Where Fe²⁺ is the ferrous iron, Cl⁻ is the chloride ions, FeCl₂ is the ferrous chloride, Fe(OH)₂ is the ferrous hydroxide and, HCl is the hydrochloric acid.

Thus, Cl⁻ is regenerated so that the rust contains no chloride, although ferrous chloride is formed at the intermediate stage.

Obviously a few chloride ions in the pore water will not break down the passive layer but a critical (threshold) concentration of chloride ions is needed (§ 3.9.5). After the initiation of corrosion, the local corrosion rate is not greatly affected by the chloride concentration but it only needs supply of oxygen and water (Roberts et al., 2000). However, it is noted that when the chloride concentration falls below a critical value, repassivation (restore of passivity) can occur (cited in Roberts et al., 2000).

### 3.9.5 Critical (threshold) concentration of chloride ions

The critical chloride concentration is the amount of chloride ions necessary for corrosion to commence. This threshold value depends on a number of factors including concrete quality (e.g. w/c ratio), chloride ions origin (added in the mix or penetrate by diffusion), environment (e.g. temperature, humidity), surface condition, and electrochemical potential of the embedded steel before corrosion initiation (Arup & Sorensen, 1995). The threshold value may be presented as a total chloride content, a free chloride content or a concentration ratio of free chloride ions to hydroxyl ions (Cl⁻ / OH⁻) (Glass & Buenfeld, 1995).

Each structure has its threshold value which when reached or exceeded, corrosion will commence. Engineers in the UK have suggested that an appropriate value of threshold chloride concentration should be in the range of 0.4-0.65% total chlorides by weight of cement or 0.06-0.1% total chlorides by weight of concrete (Middleton & Hogg, 1998).
3.10 Preventative Maintenance for corrosion due to de-icing salts

PM measures can play a significant role to inhibit, or delay, the transportation of chloride ions into the concrete. An investigation into the available PM for corrosion due to de-icing salts was carried out. A brief description of key PM measures (Figure 3.5) is given in the following sections.

![Figure 3.5: Key Preventative Maintenance measures](image)

PM is carried out usually at specified time intervals and can be either proactive or reactive. Proactive PM (waterproofing, surface treatment, etc.) involves carrying out works before a problem arises, while reactive PM (concrete replacement, cathodic protection, electrochemical chloride extraction, etc.) is undertaken after a deterioration cause (e.g. initiation of corrosion) is observed.

3.10.1 Waterproofing system

The waterproofing of bridge decks has been mandatory in the UK since 1965 (Pearson & Cuninghame, 1997), recognising that the provision of an effective waterproofing system (WS) on a bridge deck can prevent water and chloride ion ingress, and therefore protect the bridges from salt attack. BD 47/99 (1999) specifies
the requirements for the design materials and workmanship for waterproofing and surfacing of concrete highways bridge decks.

In the case of bridge decks, waterproof membranes comprise of sheet, or liquid, systems bonded to the concrete surface. Sheet systems are overlapping strips that together form a continuous membrane. These strips are bonded to the surface of the concrete. Liquid systems are applied by brush, squeegee or spray to form an integral coating on the concrete surface (Price, 1991). Further categorisation based on material composition can be found in Price’s 1990 UK publication ‘Laboratory tests on waterproofing systems for concrete bridge decks’ published by TRRL. The service life of the membrane depends on its installation and type. It has been found (Price, 1989) that a number of key factors, or usually their combination, influence the performance of waterproofing membrane and are listed below:

- method of application and compaction of hot asphaltic materials
- size and temperature of hot aggregate in the mix
- weather and temperature during laying
- moisture content of the concrete
- condition of the concrete substrate
- preparation, workmanship and site procedure
- degree of bond between concrete and asphaltic interfaces
- type of bonding adhesive and primer used
- durability of waterproofing systems
- thermoplastic properties of the membranes
- thickness of the membrane
- use of protection materials.
Therefore the life expectancy of the waterproofing membrane will vary. They usually have a life of about 10 to 30 years.

Waterproofing membranes are only one component in an overall WS, which can include protective layers, regulating layer and asphalt concrete surfacing (Figure 3.6). The performance of the overall WS and service life are dependent on the installation and type of WS, the provision of surface and sub-surface drainage, and the type and thickness of the surfacing. In this study, the service life of a typical WS is assumed to be 25 years (Pearson & Cuninghame, 1997). The removal of old waterproofing systems to be replaced with new ones has frequently proved to be a difficult task (Pearson & Cuninghame, 1997; Vassie & Arya, 2000).

![Figure 3.6: Typical waterproofing system](image)

However, membranes have been reported to fail (Broomfield, 1997). When membranes fail and water gets underneath, local corrosion can be severe and lead to heavy loss of section without any noticeable cracking and spalling on the concrete surface.

### 3.10.2 Concrete replacement

Concrete replacement (CR) involves the removal of all the chloride-contaminated concrete from those areas where corrosion caused disruption. The exposed area is filled with fresh concrete or an appropriate replacement material and sealed at the surface.
The successful performance of concrete replacements depends to a large extent on workmanship in the preparation, the application of the repair material and its compatibility with the original construction material. It has been estimated that a high-quality repair may have a service life of more than 15 years, while a low-quality repair is unlikely to last more than 2-5 years. Concrete repairs are in general expensive due to the labour intensive work involved. Furthermore, there is always an uncertainty, depending on the concrete amount that needs to be removed (Wallbank, 1989, Mallett, 1994; Vassie & Arya, 2000).

3.10.3 Surface treatment

Surface treatments can be classified into three groups as shown in Figure 3.7: coating, penetrant, and sealer (Buenfeld & Zhang, 1998). Coatings and sealers primarily act as physical barriers, whilst the most widely used penetrant, line the pores of the concrete to render it hydrophobic. BD 43/03 (2003) “provides criteria and background information for the impregnation of reinforced and prestressed concrete elements of highways structures, using hydrophobic pore-lining impregnants to help prevent and control chloride induced reinforcement corrosion from de-icing salts, and in marine environment”. Based on this Standard impregnation shall be carried out before new structures come into service otherwise if they built on existing trafficked carriageways impregnation shall be carried out at the earliest opportunity. Structures that are already in service and have not been impregnated previously, impregnation shall be carried out provided they comply with certain criteria described in BD 43/03 (2003). It should be noticed that it is not suitable to apply coating, penetrant, or sealer to concrete substructure or superstructure components that are corroding or already have a critical chloride-ion concentration (Weyers et al., 1993).

3.10.3.1 Coatings

A coating can be described as an additional dense physical layer on the concrete surface. “A coating is a one-or two-component organic liquid that is applied in one or more coats to a prepared concrete surface. Coating materials have a high solid content, have a surface film thickness of 25 to 76 μm, and usually do not contain
"aggregate" (Weyers et al., 1993). Permeability to both water vapour and liquid is usually very low. The service life of coating depends on its type and the exposed condition i.e. if the exposed condition is deicer salt the coating service life has been estimated to vary from 10 to 18 years (Weyers et al., 1993).

![Diagram of surface treatments](image)

**Figure 3.7:** Generic types of surface treatment (a) Coating (b) Sealer and (c) Penetrant
3.10.3.2 Penetrant

A penetrant (e.g. silane) penetrates the concrete, without leaving a significant surface coating. Depth of penetration will vary depending on the product, the properties of the concrete, the existence of contaminants, and to some extent on the surface preparation. “Penetrants are silanes and siloxanes (or combinations) that react with the pore walls of the hardened cement paste to create a non-wettable surface” (Weyers et al., 1993). They are generally clear and colourless so they do not prevent any following visual inspection for defects (Mulheron, 2000a). Since they are within the concrete they are protected from physical damage and degradation by ultraviolet light (Broomfield, 1997). Their service life, when applied to substructure and superstructure components has been estimated to be 5 to 7 years (Weyers et al., 1993). Although Vassie & Calder (1999) state that the effective life of a silane treatment is at least 10 years.

3.10.3.3 Sealer

A sealer acts as both a coating and a penetrant. It provides coating on the concrete surface, but also penetrates the concrete, lining or blocking pores. They are pore-blocking materials such as linseed oils or epoxy. “A sealer is a solvent- or water-based liquid applied to a prepared concrete surface” (Weyers et al., 1993). The effective service life of such sealers when applied to substructures and superstructure components has been estimated to vary between 1 -3 years depending on the environmental conditions (Weyers et al., 1993).

3.10.4 Cathodic protection

As stated before, the corrosion of steel in concrete is an electrochemical process. Therefore, electrochemistry can be used to mitigate corrosion. The science of cathodic protection (CP) was born in 1824 in a presentation to the Royal Society of London by Sir Humphrey Davy. Davy succeeded in protecting copper against corrosion from seawater by the use of iron anodes (Virmani & Clemena, 1998). CP of steel in concrete has been practiced for some 50 years (BA 83/02, 2002). There are two types of CP systems, impressed current and galvanic (sacrificial), although in the
UK galvanic anode systems are in experimental use for reinforced concrete (BA 83/02, 2002). Hence, CP is achieved by applying a low-voltage direct current, typically less than 20 mA/m², to flow from an anode system placed on the concrete surface through the concrete and onto the steel. The application of current will suppress the corrosion current of the reinforced steel (Figure 3.8) thereby stopping, or reducing to negligible values, the corrosion rate (Broomfield, 1997). CP requires that the impressed direct current (d.c.) flows either continuously or for the majority of the time.

Extensive research and trials have supported that this technique is efficient and reliable (BA 83/02, 2002). The Federal Highway Administration (FHWA), based on extensive government and private industry research concluded that CP is "the only rehabilitation technique that has been proven to stop corrosion in salt-contaminated bridge decks regardless of the chloride content of the concrete" (Barnhart, 1982).

However, a number of limitations are associated with the use of CP. CP systems require on-going maintenance, replacement at some stage, monitoring and permanent power supply (Broomfield, 1997). CP systems do not require additional preventative treatments to inhibit further chloride ingress such as in the case of electrochemical chloride extraction (§ 3.10.5) (Vassie & Arya, 2000). The life expectancy with satisfactory performance, but without maintenance, varies from 10 to 40 years depending on the system type (e.g. form of anodes, conductive coatings) used (BA 83/02, 2002; BS EN 12696, 2000).

With the use of whole life cost analysis, Robery et al., (1997) shown that CP is a cost-effective solution where chloride contamination of the concrete surface is extensive, yet little spalling and delamination has occurred.
3.10.5 Electrochemical Chloride Extraction

It seems that the use of electrochemical treatment of concrete is becoming more frequent. Electrochemical Chloride Extraction (ECE) is considered as a maintenance option that can stop the corrosion across the entire structure and therefore extending the service life of the structure.

ECE is a ‘one-off’ electrochemical treatment and is based on the principle that opposite electrical charges attract and like charges repel. A schematic diagram of a typical ECE installation is shown in Figure 3.9. A temporary anode is embedded in an electrolyte media (typically paper pulp mat; Mulheron, 2000) and placed to the surface of the concrete. The electrolyte media is saturated for the operational period using an appropriate electrolyte such as water and calcium hydroxide solution. Direct current, typical 1 to 5 A/m², is supplied through the connected anode and reinforcement in such a way that the anode is positively charged and the rebar is negative (Clemena & Jackson, 2000; Vassie & Arya, 2000).

Consequently, with chloride ions being negatively charged, they migrate to the positive charged anode. The anode is situated on the surface of the concrete therefore; all the chloride ions should leave the concrete and pass into the electrolyte.
media on the surface, which is replaced at intervals. Unfortunately, not all the chlorides are removed from the concrete and the duration of the effectiveness is unknown. However, the application of this technique results in the electrolytic production of hydroxyl ions at the reinforcing steel surface which increase the pH around the steel. The result is a strong protective passive film that will help to stop the corrosion process (Broomfield, 1997; Whitmore & Stewart, 1996; Clemena & Jackson, 2000).

ECE has many advantages such as: the typical treatment period is 4 to 12 weeks (Mulheron, 2000b) although difficult cases can take up to 3 months; it is easy to apply and no ongoing maintenance is required although monitoring is necessary to confirm the success performance of the treatment; no temporary structural support is required; and it is a silent process that normally requires less concrete breakout than concrete repair and therefore there is less noise, dust, vibration and environmental disruption (Vassie & Arya, 2000).

Figure 3.9: Illustration of CE setup on the 34th Street Bridge in Arlington, Virginia, USA (Clemena & Jackson, 2000)

Negative effects that can be induced by the ECE treatment can be some softening of the cement paste and etching at the concrete surface. Clemena & Jackson (2000) stated that this can be prevented by better control of the pH of the electrolyte. Surface coating follows ECE to prevent further ingress of chloride and to improve the appearance of concrete if it is affected by the ECE. Mulheron (2000b) and

Elena A. Tantele
Broomfield (1997) stated that there are some more side effects. One is the potential to initiating Alkali-Silica Reaction (ASR) after the treatment is completed. Another concern has been expressed that the high current employed can result in a subsequent increase in the permeability of the cover concrete. Also the thermal effects can lead to cracking of concrete and degradation of steel/concrete bond.

3.10.6 Effectiveness of Preventative Maintenance

The effectiveness of PM can be defined as the degree which PM measures are successful in achieving a specified goal. To establish the effectiveness of PM is a difficult task since it depends on a variety of factors such as their correct application. One likely method to identify the effectiveness of PM can be based on their ability to keep the reinforcement bar free from chloride ions. Therefore for this research PM measures are divided into 3 categories depending on the effect they have on delaying or ceasing chloride ingress and consequently corrosion (Figure 3.10).

Figure 3.10: Categories based on the Preventative Maintenance effectiveness mechanism

*Assuming waterproofing membrane is in perfect condition
The first category reduces the effective diffusion coefficient and can therefore achieve delay in the chloride ingress (less chloride ions will diffuse into the concrete at any given time). Surface treatments such as silane sealer and coating can be classified in this group. They also allow long-term drying of concrete surface, therefore the corrosion rates are reduced due to the increase in resistivity of the concrete. The second category prevents further chloride ingress by blocking the process. Waterproofing system and cathodic protection are able to accomplish this task. The third category simply removes the chloride ions from the reinforcement bar and the concrete cover; hence delaying the initiation of corrosion. PM measures that belong in this category here are electrochemical chloride extraction and cathodic protection.

However there is an uncertainty of the degree of effectiveness of these measures in keeping the reinforcement free from chloride ions. For this reason a probabilistic procedure will be developed to examine the different parameters that develop uncertainty. The procedure is presented in the following chapter.

### 3.11 Discussion and Conclusions

This chapter illustrates the wide range of deterioration types which affects bridges and which can have serious consequences on their behaviour and strength. However it is clear from examination of national bridge stocks that the dominant form of deterioration is corrosion due to chloride ions ingress that originate from de-icing salts.

It is widely accepted that chloride ion ingress can be described as the diffusion of chloride ions in solution in the concrete. Existing chloride ingress models may be used successfully to provide chloride profiles but its simplistic nature may introduce some limitations and approximations. However, a more complex model will need more refined data which are not currently available.

With the main deterioration mechanism defined relevant PM measures can be selected to prevent, or delay, the deterioration process. For non-critical chloride contaminated RC elements the use of surface treatment can delay only the chloride
ingress. There are three types of surface treatments namely coating, sealer and penetrant depending on their characteristic. Although surface treatments are considered effective to reduce the rate of chloride ingress their true effectiveness is hard to estimate. BD 43/03 (2003) states that impregnation with hydrophobic pore-lining impregnants, shall be carried out to new structures and to structures that are already in service provided they comply with certain criteria described in the standard. Moreover, waterproofing membrane should, and must be used, to prevent further chloride penetration. Its effectiveness can be drawn from experience in the UK and USA where, American bridges, due to the lack of waterproofing systems the bridges have suffered severe corrosion, e.g. the Florida Keys Bridges (Sagiéés et al., 1994). However, failures of waterproofing systems to stop water and chlorides have been noted. For existing chloride contaminated RC elements (e.g. bridge decks), impressed-current cathodic protection can provide the ultimate permanent solution as long as appropriate maintenance is carried out on the rectifiers and electrical wiring and permanent power supply is provided. An alternative rehabilitation method for stopping steel corrosion in contaminated concrete is the electrochemical chloride extraction (ECE). ECE is less permanent, it cannot remove all the chloride from concrete and its long-term effectiveness is not known. On the other hand, it has the advantage of having no rectifiers or wiring required after the short time of treatment. Using concrete replacement method the chloride contaminated concrete is removed. Its successful performance depends on many factors such as its correct application. Furthermore it is in general expensive with uncertain service life.

It is clear that there is an uncertainty as to what extent these measures will achieve their goal. For this purpose a probabilistic procedure needs to be developed to model the effectiveness of various PM measures and obtain predictions of the probability of failure in the presence of various PM measures. This procedure is presented in the following chapter.
Chapter 4

Development of probabilistic analysis – Effectiveness of PM

4.1 Introduction

This chapter presents in detail the procedure developed, using probabilistic techniques, to enable the prediction of the effectiveness of different PM actions. The chloride ingress model used permits the comparison of different PM interventions with respect to their effectiveness to reduce or stop the probability of initiation of corrosion. Uncertainties that influence the degree of PM action’s effectiveness are incorporated. The outcome is expressed as a probability of failure ($p_f$) with respect to the proposed limit state.

A parametric study is performed to assess the sensitivity of the various model parameters and their relative contribution to the overall uncertainty of the degree of effectiveness of PM measures.

4.2 Modelling of corrosion deterioration due to de-icing salt

As stated before numerous studies have found that the penetration of chlorides through concrete is best represented by the diffusion process. The most commonly used model for diffusion of chlorides, Fick’s second law (Equation 4.1) is adopted here.

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2}$$  (4.1)
where \( C \) is the chloride penetration into concrete; \( x \) is the distance from the surface; \( t \) is the time and \( D \) is the chloride diffusion coefficient.

The exact solution of this equation depends on the boundary condition. The standard solution was presented by Crank (1975) as

\[
C(x,t) = C_s \left\{ 1 - \text{erf} \left( \frac{x}{2\sqrt{D \cdot t}} \right) \right\}
\]

(4.2)

Where \( C(x,t) \) is the chloride concentration at depth \( x \) after time \( t \) (kg/m\(^3\)); \( C_s \) is the chloride concentration at the surface (kg/m\(^3\)); \( x \) is the distance from the surface to the reinforcement (m); \( D \) is the effective rate of chloride ion diffusion through concrete (m\(^2\)/yr), \( t \) is the time of exposure to surface chlorides (years) and \( \text{erf}(z) \) is the error function for variable \( z \).

The solution is applied to homogeneous and isotropic material, in semi-infinite stable conditions. Therefore, this deterioration model assumes that: (Crank, 1975; Daly, 1999; Stewart, 1999):

- The diffusion coefficient \( D \) is constant and takes place perpendicular to the concrete surface.
- The initial chloride concentration is zero throughout the medium.
- The surface chloride concentration, \( C(0,t) \) is uniform all over the surface and does not change with time.

However this is not the case in concrete structures. For example:

- Concrete is a heterogeneous anisotropic material that may have cracks, so its properties will vary with distance from the surface (Neville, 1995).
- The chloride concentration at the surface of the concrete is known to vary with time and is influenced from location and environmental factors (Bamforth et al., 1997).
- Where alternating wetting and drying occurs, no stable conditions can be observed.
The diffusion coefficient can be affected from environmental conditions (e.g. temperature and relative humidity (Val and Stewart, 2003). Despite these issues Fick's second law of diffusion model is generally accepted to predict the chloride contamination of concrete bridges at different depths and time.

4.3 Deterministic deterioration model- Fundamental model form

Deterministic procedure can be employed to solve Equation 4.2, when a conservative solution is needed due to high safety factors. In addition, when large quantities of site specific location data are given, a credible outcome using this procedure is possible. However, a large quantity of data means a high investment cost is involved.

Unfortunately while this model is simple and straightforward it fails to recognise that RC bridge deterioration due to chloride ions involves many uncertainties caused by internal and external factors such as the concrete quality or the environment. These uncertainties can have significant effect in the deterioration analysis outcomes.

Therefore, a model capable of incorporating the inherent variability of the input parameters and the model's limitation to describe the complex chloride ingress mechanism is more preferable, such as a probabilistic model.

4.4 Probabilistic deterioration model

A probabilistic analysis using for example Monte Carlo simulation (MCS) can simulate the deterioration process by defining: (i) the input as random variables and (ii) a limit state. Consequently:

- Instead of one deterministic outcome, a number of possible solutions are produced.
- It can take into account some of the model and input data uncertainty.
- Having a number of solutions a distribution can be fitted.
Better use of small quantity of data can be achieved since more reliable and quantifiable outcomes will be possible. Therefore, the simulations can predict the probability of an event occurring, i.e. corrosion initiation. The input data and output results are described by two (or more) parameters, normally mean and standard deviation, to incorporate the uncertainty involved, as an alternative to the single parameter used in a deterministic approach. The output results can then be defined based on levels of confidence, for example 0.1 probability of structural failure after 50 years.

4.4.1 Limit state selection

Limit state margins need to be defined in order to perform probabilistic analysis. Considering a RC element, for example a beam or a slab, emphasis is given on satisfying the criteria, which will enable the element to maintain its service life as much as possible.

As Stewart et al., (2003) reported most reliability-based techniques usually consider ultimate limit state since loss of strength capacity is often assumed as the main criterion for decision-making (Frangopol et al., 1997; Stewart & Rosowsky, 1998b). Some studies have considered the interaction of multiple limit states (ultimate, serviceability) for the time-dependent reliability of the structure (Stewart & Rosowsky, 1998b; Stewart & Val, 2003). Only relative few studies have considered serviceability limit state in terms of corrosion initiation, cracking, spalling etc (Hoffinan & Weyers, 1996; Gehlen & Schiessl, 1999; Faber & Sorenson, 2002; Stewart, 2001).

Using proactive procedures, the main concern is to postpone and/or inhibit the corrosion initiation using PM measures. Hence, the limit state (margin) is set to satisfy the condition, that the chloride concentration at the surface of the embedded reinforcement does not exceed the threshold value, i.e. there is no corrosion initiation. Mathematically, this can be expressed as:

\[ M = C_{th} - C_{(x,t)} \]  

(4.3)
Where $C_{th}$ is the threshold chloride concentration (kg/m$^3$); obtained from literature; $C_{(x, t)}$ is the chloride concentration at reinforcement (kg/m$^3$); calculated using Fick's second law of diffusion and M is the margin.

Throughout this study, this margin will be examined for violation using probabilistic methods. It needs to be emphasised that failure is defined in this research as the probability of initiation of corrosion due to chloride ingress at the top level of the reinforcement in the concrete.

### 4.4.1.1 Computational procedure

The failure probability at time intervals, $p_f(t)$, can be expressed in terms of the margin as

$$p_f(t) = p(M < 0) = p(C_{th} - C_{(x,t)} < 0) \quad (4.4)$$

However with complexities such as non-linear state function, non-normal random variables and time-varying quantity ($C(x,t)$), closed form solutions are not tractable (Stewart & Rosowsky, 1998). Hence, Monte Carlo simulation (Appendix A) has been used herein to evaluate limit state probability defined in Equation 4.4 Therefore an estimate of the $p_f(t)$ can be found by

$$p_f(t) = N_f(t) / N(t) \quad (4.5)$$

Where $N_f(t)$ is the number of simulation cycles at a reference time, $t$, in which $M<0$, and $N$ is the total number of simulation cycles at reference time, $t$.

### 4.4.2 Probabilistic analysis procedure

The following probabilistic procedure is adopted to show the effectiveness of various PM measures under the action of chloride ions, while allowing uncertainties to be incorporated into the chloride ingress model.

1. A bridge structural RC (made of PC) element (e.g. slab, beam), is considered for examination.
2. The mechanism of corrosion is identified. For modelling diffusion of chloride ions, Fick's second law (Equation 4.2) is used. It is assumed that the routine work is carried out successfully, therefore no local deterioration issues will arise.

3. The limit state equation (Equation 4.3) is examined for violation using crude MCS hence all the basic variables are randomly generated. The input values of the parameters can be chosen based on the quality of concrete, the chloride environment condition, measured magnitude of specific quantity (e.g. $D_c$) and the construction quality where applicable.

4. The probability of failure ($p_f$) and the reliability index ($\beta$) profiles ($p_f$ and $\beta$ as function of time respectively) are estimated through the simulations. Failures are counted depending on the resulting sign of Equation 4.4. The $p_f$ is estimated as the ratio of the number of failures to the total number of simulation cycles (Equation 4.5). It is important to ensure that convergence of the simulation process with an appropriate number of simulations is achieved, so that the estimated $p_f$ is within an acceptable level of statistical error.

5. The effectiveness of various PM measures based on their ability to reduce/inhibit the corrosion initiation is incorporated in the deterioration model and corresponding $p_f$ are produced.

Steps, 1-4, of the described procedure are illustrated with the following example. To perform the probabilistic analysis using simulations a program was developed using Excel XP and Visual Basic Programming (VBA) 6.

Only physical uncertainties without modelling uncertainties are assumed in this study, i.e., the probabilistic model used to describe the deterioration process is assumed 'exact' and only the relevant parameters in this model vary randomly according to designated probability distributions, respectively. The corresponding statistical information of the random variables (Appendix C) required to perform the probabilistic analysis are given in Table 4.1 and are based on related published work.

For the selection of these values, concrete quality, construction quality and chloride environment conditions are assumed to be average. The examined service life is assumed to be 120 years.
To overcome some of the shortcomings of Fick's diffusion model (Equation 4.2) an approach that combines the use of Fick's second law of diffusion with in-situ measurements has been widely adopted to predict the chloride ingress in the concrete structures (Tuutti, 1982; Cady and Weyers, 1992). The base of this approach is to refer to the surface chloride content (C₀) and diffusion coefficient (D), calculated by fitting Equation 4.2 to measured chloride profiles, as the 'notional' surface chloride content (C₀) and the 'effective', or 'apparent' diffusion coefficient (Dₑ). A few researchers considered the diffusion coefficient being as time dependent and proposed different modification, e.g. Mangat & Molloy (1994), Stewart & Rosowsky (1998). However, Bamforth et al., (1997) have observed that there is very little time-dependent change in chloride diffusion coefficients for PC based concrete (until 100 yr old); consequently Dₑ here will be assumed constant for all the reference time.

Furthermore the concrete cover (x; Equation 4.2) in this research is taken as the 'effective' concrete cover (x_{cover}). The x_{cover} is the concrete cover less the depth of the concrete cover that absorption may occur (§ 3.9.3). Therefore the chloride ingress is modelled as a Fickian process only over the effective concrete cover depth.

In the following, whenever surface concentration, diffusion coefficient and concrete cover are mentioned it will imply that these are the notional near-surface (due to the effective x_{cover}) chloride content, effective diffusion coefficient and effective concrete cover.

**Table 4.1: Values and distribution type of basic random variables**

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Distribution</th>
<th>Mean</th>
<th>CoV</th>
<th>Units</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>C₀</td>
<td>Uniform* [1.2,2.4]</td>
<td>1.8</td>
<td>0.19</td>
<td>kg/m³</td>
<td>Middleton &amp; Hogg (1998)</td>
</tr>
<tr>
<td>Dₑ</td>
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<td>3.15 E-05</td>
<td>0.2**</td>
<td>m²/yr</td>
<td>Middleton &amp; Hogg (1998)</td>
</tr>
<tr>
<td>x_{cover}</td>
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<td>0.25</td>
<td>m</td>
<td>Chryssanthopoulos &amp; Sterritt (2002)</td>
</tr>
<tr>
<td>Cₜh</td>
<td>Uniform [0.6,1.2]</td>
<td>0.9</td>
<td>0.19</td>
<td>kg/m³</td>
<td>Hoffman &amp; Weyers (1996)</td>
</tr>
</tbody>
</table>

*assumed distribution; **assumed CoV based on Vu & Stewart (2000)
The element to be examined is a RC beam that deteriorates from chloride ingress. Probabilities of failure for the top reinforcement as obtained from MCS analysis are presented in Figure 4.1. To determine the appropriate number of iterations to illustrate convergence of the simulation process the \( p_f \) was estimated for numbers of iterations ranging from \( 3 \times 10^5 \) to \( 3.6 \times 10^6 \) cycles with steps of \( 3 \times 10^5 \).

![Figure 4.1: Probability of failure versus time of exposure (SLS: depassivation of the reinforcing steel)](image)

**Figure 4.1:** Probability of failure versus time of exposure (SLS: depassivation of the reinforcing steel)

### 4.4.2.1 Statistical accuracy of the estimated probability of failure

It is evident from Equation 4.5 that as \( N \) approaches infinity, the \( p_f \) approaches the true \( p_f \). It is recommended (Ayyub & McCuen, 1995) to measure the statistical accuracy of the estimated \( p_f \) by computing its Coefficient of Variation (CoV) as

\[
CoV(p_f) \equiv \sqrt{\frac{(1 - p_f)p_f}{Np_f}}
\]  

(4.6)
The smaller the CoV is, the better the accuracy of the estimated \( p_f \). Figures 4.2-4.4 illustrate the convergence behaviour of the simulation process while the number of iterations is increasing (from \( 3 \times 10^5 \) to \( 3.6 \times 10^6 \)). Three cases were chosen; for time equal to 1, 60 and 120 year so to cover all the range of magnitude of \( p_f \).

Figure 4.2: (a) Estimated probability of failure for time=1\textsuperscript{st} year, b) Coefficient of variation of estimated failure probability for time=1\textsuperscript{st} year

Figure 4.3: (a) Estimated probability of failure for time=60\textsuperscript{th} year, b) Coefficient of variation of estimated failure probability for time=60\textsuperscript{th} year
Figure 4.4: (a) Estimated probability of failure for time=120\textsuperscript{th} year, b) Coefficient of variation of estimated failure probability for time=120\textsuperscript{th} year

The CoV for all the three cases was approaching zero though for small \( P_f \) \( (\approx 10^{-4}) \) (Figure 4.2) it was necessary to increase the number of simulation cycles more than in the case where \( P_f \) was in the magnitude of \( 10^{-1} \) (Figure 4.3 & 4.4).

Based on the results of these trails, 3.6 million iterations were chosen for the iteration process because of the good balance between precision of the estimate and the time to run the simulation.

4.2.2.2 Reliability index profile

The results of the simulations can also be expressed as reliability index rather than \( P_f \). Figure 4.5 shows the reliability index profile obtained from the previous example. The number of simulation cycles was again ranging from \( 3 \times 10^5 \) to \( 3.6 \times 10^6 \).
4.4.3 Validation of the probabilistic model

A validation of the $p_f$ results, presented in Figure 4.1, is carried out using the commercial reliability program PROBAN 4.4-0.2 (PROBAN, 1996). The initiation of corrosion is examined as in the previous example. The MCS method is applied with 3.6 millions sample values. Figure 4.6 shows the results of the analysis expressed as $p_f$ profiles, obtained from Excel and PROBAN.
Figure 4.6: Validation of the probabilistic modelling using Excel and PROBAN

To compare the $p_f$ profiles, boxplots (Appendix D) are produced (Figure 4.7). The box plot is based on the following five summary measures:

1. The lowest (minimum) value

2. The value of Q1 (the lower or first quartile; twenty-fifth percentile)

3. The value of Q2 (the median; mid-point of the data set)

4. The value of Q3 (the upper or third quartile; seventy-fifth percentile)

5. The highest (maximum) value

Good agreement was obtained from their comparison of the five summary measures. Furthermore, negative skewness (Appendix C) is evident for both set of data in the boxplot since the median is closer to Q3 than to Q1. That implies that there is a greater dispersion of values to the left of the $p_f$ mean value.
Furthermore, the \( p_f \) profile of Figure 4.1 is in line with observations obtained from 40 concrete bridges in the UK (Figure 4.8). As Figure 4.8 suggests, an early start of initiation of corrosion is being recognised. In addition similar \( p_f \) profiles were obtained by Gaal et al., (2002) and Hoffman & Weyers (1996).
4.4.4 Incorporating the effectiveness of PM in the probabilistic model

The final step (5) of the probabilistic procedure incorporates the effectiveness of PM action and expresses it into probabilistic failure profiles for the initiation of corrosion. As introduced in Section 3.9.6 the effectiveness of PM is divided into 3 categories.

4.4.4.1 Decelerate chloride ingress

Surface treatments belong to this category.

- Surface treatments

When surface treatments are applied, there is a change in the diffusion coefficient ($D_e$) of the concrete. The new combined diffusion coefficients (surface treated specimen) are derived using Buenfeld & Zhang (1998) equation:

$$D_e = \frac{T_c}{\left(\frac{T_{st}}{D_{st}} + \frac{T_{su}}{D_{su}}\right)}$$

(4.7)

Where $T_{su}$ is the thickness of the specimen (concrete cover depth); $D_{su}$ is the diffusion coefficient of the specimen (concrete); $T_{st}$ is the thickness of the surface treatment layer; $D_{st}$ is the diffusion coefficient of the surface treatment layer; $T_c$ is the thickness of the specimen and the surface treatment, which is equal to $T_{st} + T_{su}$.

The aim of the surface treatments is to reduce the value of diffusion coefficient and the effectiveness of each surface treatment depends on the new combined diffusion coefficient. Lower values of $D_e$ indicate a higher effectiveness.
Figure 4.9: Deceleration of the diffusion process

Figure 4.9 shows schematically the effect of different surface treatment on the $p_f$ profile. When surface treatments are applied, because the rate of chloride ingress is reduced, the probability of initiation of corrosion decreases too. The decrease depends on the diffusion coefficients of the specific PM action examined. Surface treatment should be applied before the initiation of corrosion, i.e. when the $p_f$ is below a specified allowable value ($p_f$ target).

**4.4.4.2 Prevent chloride ingress**

Cathodic protection and waterproofing membrane fit in this category.

- **Cathodic protection**: Cathodic protection (CP) removes the chloride ions from the surface of the bar, using an electrochemical process, for as long as it is operational. Consequently, it excludes the event of initiation of corrosion. It is assumed that it is 100% effective in keeping the reinforcement bar free from chlorides.

- **Waterproofing system**: The waterproofing system (WS) is capable of preventing the chloride ingress when in perfect condition. However, this is not always the case. Due to bad construction or bad materials used, a small degree of chloride ingress may occur. A simple modelling approach is adopted here which takes into account the probability of WS failing. The
latter is assumed to be 10%. Within the context of this study, waterproofing is considered to be an option when the structure is not corroding, i.e. \( p_{f\text{(target)}} \) is not reached (Figure 4.10).

The effect of CP and WS on the \( p_f \) can be seen schematically in Figure 4.10.

\[ 
\text{Figure 4.10: Prevention of the diffusion process (a) Cathodic protection (b) Waterproofing system} 
\]

Figure 4.10 shows that when CP is applied the \( p_f \) reduces to zero, as long as it is operational. In the case of the WS if it was in perfect condition, the \( p_f \) should stay constant but due to the probability of the WS failing, a \( p_f \) profile is assumed.
4.4.4.3 Remove Chlorides from reinforcement bar

In this category, electrochemical chloride extraction (ECE) and concrete replacement (CR) are placed.

- **Electrochemical Chloride Extraction**: ECE removes a part of the amount of chloride in the concrete cover. However, a percentage remains in the concrete and chloride ingress continues after the application is completed. This can be shown schematically in Figure 4.11. The reduction of chloride assumed here, when ECE is applied is based on recent studies by Clemena & Jackson (2000) and Sharp et al., (2002). Based on these findings it is assumed, that the reduction of chloride concentration throughout the concrete cover is 40%.

![Figure 4.11: Electrochemical chloride effectiveness in chloride ions removal in concrete](image)

**Figure 4.11:** Electrochemical chloride effectiveness in chloride ions removal in concrete

Figure 4.12 shows schematically the effect of ECE on the $p_f$ that is a significant $p_f$ reduction.

![Figure 4.12: Effect of electrochemical chloride extraction on $p_f$](image)

**Figure 4.12:** Effect of electrochemical chloride extraction on $p_f$
- **Concrete replacement**: CR means that part of a concrete element that has critical concentration of chloride can be replaced with a new one. This has to be achieved before the initiation of the corrosion on the reinforcement bar.

![Graph showing probability of failure with and without concrete replacement](image)

**Figure 4.13: Removal of chloride ions from the reinforcement bar with the use of concrete replacement**

Therefore when CR is applied, there is 100% efficiency for removing the chloride ions consequently the $p_f$ is approaches zero and set to zero value. Cathodic protection can also be described under this category but the modelling will be the same in Figure 4.10.

### 4.4.5 Sensitivity analysis of the probabilistic modelling

To interpret the results of the simulation it is useful to understand the sensitivity of the various input parameters on the $p_f$. Therefore a sensitivity analysis is conducted to ascertain the effect of different parameters. Only one variable is isolated each time from Equation 4.4. A substantial investigation of the factors that affect these parameters is given by Bamforth et al., 1997.

#### 4.4.5.1 Influence of $x_{cover}$ on the $p_f$ profiles

Sensitivity analysis was conducted to assess the influence $x_{cover}$ on failure probabilities (Figure 4.14). The nominal values for the $x_{cover}$ examined are: 35mm,
40mm and 45mm. Cover depth distribution can be chosen based on the construction quality (Table 4.2). The construction quality adopted in this study is average.

Table 4.2: Influence of construction quality on concrete cover depth (Chryssanthopoulos & Sterritt, 2002)

<table>
<thead>
<tr>
<th>Construction quality</th>
<th>Cover depth distribution (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good quality finish, no/ few signs of poor workmanship</td>
<td>N(nominal, 5)</td>
</tr>
<tr>
<td>Average quality workmanship</td>
<td>N(nominal, 10)</td>
</tr>
<tr>
<td>Poor quality workmanship</td>
<td>N(nominal, 15)</td>
</tr>
</tbody>
</table>

Where N: normal distribution

Both the standard deviation and the $x_{\text{cover}}$ of the beam are constant as seen in Table 4.1. Therefore, when the mean value of the $x_{\text{cover}}$ increases CoV is reduced.

Figure 4.14: Effect of $x_{\text{cover}}$ on the probability of failure
Figure 4.14 signifies the very important role of concrete $x_{\text{cover}}$ on the service life extension of RC bridge elements exposed to chloride contamination. For example, if it is assumed that a $p_f$ of 10% specifies the time to first PM measures applications, then for concrete $x_{\text{cover}}$ 35mm will require PM after about 14 years in service while with concrete $x_{\text{cover}}$ 40mm and 45mm will require maintenance after 20 and 28 years of service of the steel, respectively. Therefore increasing $x_{\text{cover}}$ further delays the diffusion of chlorides to the surface of the steel.

### 4.4.5.2 Influence of $D_e$ on the $p_f$ profiles

Typical values for high, medium and low $D_e$ measured from UK’s bridge structures have been suggested by Middleton & Hogg (1998). The mean value of $D_e$ utilises herein are illustrated in Table 4.3. The standard deviation is constant and the distribution is normal for all the cases (Table 4.1). In addition the suggested $D_e$ is assumed that it can be related to the level of concrete quality and is given in Table 4.3.

**Table 4.3: Determination of $D_e$ based on the concrete quality**

<table>
<thead>
<tr>
<th>Concrete quality</th>
<th>$D_e$ 3.15m$^2$/yr (Low)</th>
<th>$D_e$ 31.5m$^2$/yr (Average)</th>
<th>$D_e$ 315m$^2$/yr (High)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Good x 10$^{-6}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average x 10$^{-6}$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poor x 10$^{-6}$</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

To determine the concrete quality Table 4.4 can be utilised. If more than one parameter is known about the concrete quality then the worst condition is to be used.
Table 4.4: Determination of concrete quality (Anstice & Roberts, 2002)

<table>
<thead>
<tr>
<th>Concrete quality</th>
<th>Good</th>
<th>Average</th>
<th>Poor</th>
</tr>
</thead>
<tbody>
<tr>
<td>w/c</td>
<td>&lt;0.4</td>
<td>0.4-0.5</td>
<td>&gt;0.5</td>
</tr>
<tr>
<td>$f_{cu}$</td>
<td>&gt;50N/mm$^2$</td>
<td>40-50N/mm$^2$</td>
<td>&lt;40N/mm$^2$</td>
</tr>
</tbody>
</table>

Where w/c is the water-cement ratio and $f_{cu}$ is the concrete strength.

The results of the sensitivity analysis (Figure 4.15) demonstrate that the $p_f$ increases more rapidly for high $D_e$ since $D_e$ is within the square root function in the denominator of the statistical error function. The dramatic change in the $p_f$ indicates that the diffusion coefficient is a very important variable that influences significantly the outcome of the analysis.

![Graph showing the effect of diffusion coefficient on the probability of failure](image)

**Figure 4.15: Effect of diffusion coefficient in the probability of failure**
4.4.5.3 Influence of standard deviation \( (D_e) \) on the \( p_f \) profiles

The influence of standard deviation \( D_e (s_{De}) \) was also evaluated using time-dependent probabilistic analysis. If standard deviation increases and the mean value remain the same, the CoV will increase too. Three different CoV values are studied; 10%, 20% and 30%. It is evidenced from Figure 4.16 that change of the CoV has modest impact on the \( p_f \) profiles. At the beginning (until 32nd year) of the \( p_f \) profile, reduced CoV leads to a decrease in \( p_f \) and then to an increase of \( p_f \).

![Figure 4.16: Effect of CoV of De in the probability of failure profile](image)

4.4.5.4 Influence of \( C_o \) on the \( p_f \) profiles

Figure 4.17 shows how the \( C_o \) influences the \( p_f \). Different \( C_o \) ranges, shown in Table 4.5 were suggested by Middleton & Hogg (1998) and herein are assumed to be related to the level of the corrosive environment (Table 4.5). To identify the corrosive environment Table 4.6 can be utilised. Considering the 10% fractile of the \( p_f \), this can be reached at the 10th and 20th year of service with high and medium range of \( C_o \) respectively, but with low range of \( C_o \) it is possible that it will not be reached at all during the examined here service life (120 years).
Table 4.5: Determination of $C_o$ based on the environment

<table>
<thead>
<tr>
<th>Corrosive environments</th>
<th>Low risk</th>
<th>Medium risk</th>
<th>High risk</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$kg/m^3$</td>
<td>$kg/m^3$</td>
<td>$kg/m^3$</td>
</tr>
<tr>
<td>$C_o$</td>
<td>$U(0,1.2)$</td>
<td>$U(1.2,2.4)$</td>
<td>$U(2.4,3.6)$</td>
</tr>
</tbody>
</table>

Where $N$ is normal distribution

Table 4.6: Determination of corrosive environment (Anstice & Roberts, 2002)

<table>
<thead>
<tr>
<th>Corrosive environment</th>
<th>High risk</th>
<th>Medium risk</th>
<th>Low risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chlorides at the level of the steel (% by weight of cement)</td>
<td>$&gt;1%$</td>
<td>$0.3-1.0%$</td>
<td>$&lt;0.3%$</td>
</tr>
<tr>
<td>Half-cell potential readings(copper/copper sulphate electrode)</td>
<td>More negative than $-350mV$</td>
<td>Between $-200mV$ and $350mV$</td>
<td>Less negative than $-200mV$</td>
</tr>
<tr>
<td>Exposure to de-icing salts</td>
<td>High</td>
<td>Medium</td>
<td>Low</td>
</tr>
</tbody>
</table>

Figure 4.17: Effect of $C_o$ on the probability of failure profile
As expected an increase in the chloride concentration at the surface has results on the increase of $p_f$ (Figure 4.17).

### 4.4.5.5 Influence of mean values and distribution of $C_{th}$ on the $p_f$
profiles

A comparison of different chloride threshold levels is illustrated in Figure 4.18. The statistical information of the first two cases [$C_{th}: U(0.6,1.2)$ and $U(1.44,2.4)$] were taken from Hoffman & Weyers (1996) and Middleton & Hogg (1998) respectively. The third case [$N(0.9,0.17)$] and the last case [$N(1.4,0.175)$] assumed to have the same mean and CoV as the first and the second case, respectively but different distribution (i.e. normal rather than uniform).

![Figure 4.18: Effect of threshold value to the probability of failure profile](image)

It is clear from Figure 4.18 that when threshold chloride ranges increase the $p_f$ decreases. It can also be seen that that the effect of changing the distribution of $C_{th}$ from uniform to normal or vice versa is negligible.
4.4.5.6 Discussion of the results of the sensitivity analysis

A sensitivity analysis was performed for the variables of the analysis (concrete cover depth ($x_{cover}$), threshold chloride concentration ($C_{th}$), chloride concentration at the surface ($C_o$) and diffusion coefficient ($D_e$)) to establish how a change in one parameter will affect the $p_f$ profiles. The result of the present analysis suggests that the mean value of $x_{cover}$, the $D_e$, the $C_o$ and the $C_{th}$ affect significantly the $p_f$ profile. The standard deviation of $D_e$ ($s_{De}$) and the change of distribution from normal to uniform of the $C_o$ were found to have negligible effect on the $p_f$. Therefore, with an increase in $C_o$ and $D_e$, the failure probabilities will significantly increase while an increase in $x_{cover}$ and $C_{th}$ will resulted in a significantly reduction in the failure probabilities. This information is extremely useful not only for identifying the effect of the parameters on the $p_f$ profiles but also for prioritising which parameters need to be more accurately described.

However these results are obtained based on the parameter configuration (e.g. distribution) assumed in the analyses.

4.4.6 Probability of failure profiles based on the effectiveness of different surface treatment

The selection of appropriate values for the computation of the probability of failure (Equation 4.4) is necessary. An investigation of the relevant literature reveals that there is generally only very limited data regarding the effectiveness of PM measures. The parameter values for the combined diffusion coefficient (Equation 4.7) adopted in this study, are based on relevant work by Buenfeld & Zhang (1998) and Middleton & Hogg (1998). The remaining data are taken from Table 4.1. Clearly, there is a need for better quality and more quantity of data, to improve predictions in this area. Low $p_f$ values mean high reliability and high effectiveness.

Three surface treatments (silane, sealer, polymer modified cementitious coating) were selected to represent the categories of surface treatment products available to protect concrete structures from chloride penetration. The combined diffusion coefficients derived for the different surface treatments are shown in Table 4.7
together with the values from plain concrete. Plain concrete is referred to as ‘control’.

**Table 4.7: Surface treatments and related parameters** (Buenfeld & Zhang, 1998; Middleton & Hogg, 1998)

<table>
<thead>
<tr>
<th>PM</th>
<th>Description</th>
<th>$D_{st}$ m²/y</th>
<th>$T_{st}$ mm</th>
<th>$D_{su}$ m²/y</th>
<th>$T_{su}$ mm</th>
<th>$D_{e}$ m²/y</th>
<th>$T_{c}$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-m coating</td>
<td>Polymer modified cementitious coating</td>
<td>$6.3 \times 10^{-15}$</td>
<td>1.5</td>
<td>40</td>
<td>$4.71 \times 10^{-6}$</td>
<td>41.5</td>
<td></td>
</tr>
<tr>
<td>Sealer</td>
<td>Polyurethane Sealer</td>
<td>$1.4 \times 10^{-15}$</td>
<td>0.02</td>
<td>40</td>
<td>$2.32 \times 10^{-5}$</td>
<td>40.02</td>
<td></td>
</tr>
<tr>
<td>Penetrant</td>
<td>Silane</td>
<td>$2.5 \times 10^{-13}$</td>
<td>-2</td>
<td>40</td>
<td>$2.74 \times 10^{-5}$</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>Control</td>
<td>Concrete with no PM measures; average quality</td>
<td>$3.15 \times 10^{-5}$</td>
<td>40</td>
<td>$3.15 \times 10^{-5}$</td>
<td>40</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* depth below the surface of the concrete cover

The probability of corrosion was found different when surface treatments are applied (Figure 4.19). The treated concrete provides more resistance to chloride diffusion than the concrete itself therefore has more effectiveness. Even though $D_{st_{p-m}}$ is higher than $D_{st_{polyurethane sealer}}$ the diffusion resistance of the former is higher due to its greater thickness.
Corrosion initiation is very likely for a PC concrete element (assumed $p_f$ 10%) after 20 years of service. The same element when silane and sealer is applied has a 10% probability of corrosion initiation after 22 and 28 years respectively. Corrosion of with P-m coating concrete is unlikely to occur after 120 years.

In addition, the effect of different $D_e$, concrete $x_{cover}$ and sd (i.e. change in CoV) of $D_e$ (Figure 4.20) was investigated with concrete subjected to surface treatment. As expected the pattern of the $p_f$ profiles match the $p_f$ profiles with concrete untreated.
Figure 4.20: (a) Effect of $D_e$ on surface treated concrete (Sealer); (b) Effect of $x_{cover}$ on surface treated concrete (Silane) (c) Effect of CoV ($D_e$) on surface treated concrete (P-m coating)
4.4.7 Electrochemical chloride extraction with different surface treatments

The effectiveness of electrochemical chloride extraction (ECE), based on the assumptions made in Section 4.4.4.3, is shown as $p_f$ profile in Figure 4.21. Figure 4.21 illustrates the reduction of $p_f$ based on what action was applied before setting ECE. For example, if an element is deteriorating without any surface treatment ($p_f$ profile of control) at time 80, the $p_f$ is the value at A but if at that time ECE is applied its $p_f$ will be equal to the value B. From the figure below it can also be noticed that the impact of ECE is proportional with the initial value of $p_f$. Therefore the higher the $p_f$ is before the application of ECE the higher the reduction of $p_f$ is when ECE is applied.

![Figure 4.21: Effect of ECE on the probability of failure profiles on a 40 mm concrete cover (with and without surface treatments)](image)

1: Plain concrete, 2: Silane, 3: Polyurethane sealer, 4: P-m coating, ECE: Electrochemical Chloride Extraction

Figure 4.21: Effect of ECE on the probability of failure profiles on a 40 mm concrete cover (with and without surface treatments)
4.5 Discussion and Conclusions

A probabilistic methodology was introduced to provide a tool for relating and model the uncertainties associated with the effectiveness of different preventative maintenance (PM) measures such as the limitation of the deterministic chloride ingress model (Equation 4.2) to describe the complex chloride ingress mechanism and the large scatter and variability of field data. A limit state (margin) is set to satisfy the condition, that the chloride concentration at the surface of the reinforcement should not exceed the threshold value, so that the corrosion process is delayed or prevented. Probability of failure profiles, indicate the violation of the limit state, i.e. the probability that corrosion initiation will occur.

Although the required data for the methodology are limited, some data were identified and used to perform probabilistic analysis in order to demonstrate the methodology. Clearly there is a need to improve the database of various PM measures to obtain better predictions of probability of failure. As more refined data become available this methodology can be used to compare the effectiveness of different PM measures and generate profiles for subsequent selection of PM strategies.

A sensitivity analysis was conducted to ascertain the effect and identify significant trends of the parameters of the analysis on probability of failure for 120 years service life with, and without, PM applied on the concrete. These parameters are the mean value of concrete cover depth ($x_{\text{cover}}$), threshold chloride concentration ($C_{\text{th}}$), surface chloride concentration ($C_o$) and diffusion coefficient ($D_e$), the distribution of $C_o$ and the standard deviation of $D_e$ ($s_{De}$). As expected these were found to significantly affect the failure probabilities for initiation of corrosion in both cases (with or without PM) except for the change of $s_{De}$ and the distribution of the $C_o$ that had a negligible effect on the $p_f$. An increase in $C_o$ and $D_e$ will result in a significant increase in failure probabilities. Though, an increase in $x_{\text{cover}}$ and $C_{\text{th}}$ results in a significant reduction in failure probabilities. This information can be used for identifying the effect of the parameter but also for prioritising which parameters need to be more accurately described. However, these results are obtained based on the parameter configuration (e.g. distribution) assumed in the analyses.
Furthermore, it should be emphasised that the proposed analysis (i) ignores system effects and examined only bridge elements, (ii) assumes all reinforcement corrodes uniformly across the entire steel area of the top reinforcement, (iii) assumes the absence of inspection and repairs. Also, the values assigned to the parameters of the corrosion models are subjected to considerable uncertainty while the model of corrosion initiation is just an assumption. Therefore the failure probabilities calculated herein should be considered ‘notional’ and therefore the results should be used for comparison purpose only.

Nevertheless, the findings presented in this chapter can be used to provide a tool that will facilitate the creation of successfully optimised PM measures strategies. An expansion of the above methodology is described in Chapter 5, where the probability of failure profiles are related to cost and whole life planning to enable various possible strategies to be compared.
Chapter 5

Methodology for optimum whole life PM strategy using GA

5.1 Introduction and overview

Optimisation is the art of producing the best alternative among a given set of options and constraints. Every optimisation problem has an objective function. Herein the objective is to plan the PM actions in such a way that the $p_r$ (i.e. initiation of corrosion) stays below a minimum acceptable level during the service life of the bridge element at minimum possible WLC. Based on the literature review, an evolutionary technique, named Genetic Algorithm (GA) is selected to achieve this optimisation.

The GA method was first proposed by John Holland (1975) and subsequently developed by him, his students and colleagues. A GA is an optimisation algorithm where the solution is obtained through an evolutionary process that combines nature’s mechanisms (e.g. natural selection) with randomised genetic operators such as crossover and mutation (Goldberg, 1989; Haupt R & Haupt S, 1998; Davis, 1991).

management systems for the optimisation of maintenance or repair strategies or inspection schemes.

GA has many advantages against traditional optimisation methods i.e. analytical optimisation. These include the ability to search simultaneously in a population of points and not a single point, use complex functions for the optimisation problems, incorporate a large number of parameters, work with a coded set of the variables and not the variables themselves, use objective functions information without any derivatives or other auxiliary knowledge, apply probabilistic transition scheme compared with deterministic that traditional methods employ, etc (Goldberg, 1989; Haupt R & Haupt S, 1998; Falkenauer, 1998; Pezeshk et al., 1998).

However, GA also has a number of drawbacks. The great advantage of GA is that the solution is found through evolution. However this can be considered also as its biggest disadvantage. In nature, life does not always evolve to the optimum solution. It may evolve to tackle bad circumstances. Consequently this can lead to an evolutionary dead end and risking finding the local optimum solution rather than the global optimum. Such a barren situation can be avoided with the introduction of some techniques and constraints in the GA process. Also since GA is heuristic (it estimates a solution) the exact solution is not known. However, most real life problems are like that: the solution can be estimated and not calculated exactly. Another disadvantage of GA is its computational time. GA can be slower than other methods but the process can be terminated at any acceptable time and with the increasing power of the available software and computers this does not now seem a weakness anymore.

Therefore, a GA based methodology has been developed to combine the findings of Chapter 4 (probability of failure profiles) with cost, so that optimum whole life PM measures strategies can be developed. To implement this methodology a spreadsheet-based program in Excel XP with the aid of VBA 6 has been developed.
5.2 **Basic outline of the genetic algorithm**

Initially GA produces a population of 'individuals'; all representing a possible solution to a given problem. During each successive generation every member is tested according to a function. This function provides a measure of how good a solution is to the problem in relation to another solution. Highly 'fit' individuals are ranked at the top of the pool and given the opportunity to mate with other high fit members to produce new individuals. The new individuals contain portions of the features of both old individuals. Furthermore, in a few randomly selected individuals some arbitrary changes are made in their characteristics to add some diversity to the population. The least fit members of the population have less probability to be selected for reproduction so they die out. At this point the whole new population is constituted by selecting the best individuals from the initial population and mating them to produce a new set of members. Therefore the new generations have in higher proportion the good characteristics needed to be a good member. With the execution of many generations it is expected that the good characteristics will spread throughout the population. So, although GA is randomised it efficiently incorporates information from previous generations to create new search points. In addition by focusing on the 'fit' individuals the most promising areas of the domain are being explored, assuming that 'fit' can be identified.

Hence, based on the nature of the problem (i.e. complex problem) and with the use of a well designed GA, it may be possible to converge to an optimal solution of the problem where other optimisation methods may fail (§2.8).

However, it must be noticed that there is no mathematical proof that GA will converge or find the global optimum. Holland (1975) presents a handwaving proof for the genetic algorithm, called Schema Theorem (Appendix E; Goldberg, 1989), where schema is a string represented by three characters: the binary digit 1 and 0, and an additional 'don’t care character', #. The Holland's Schema Theorem is widely accepted as the foundation for explanations of the power of GA. The Schema Theorem says:
"A schema occurring in chromosomes with above-average evaluations will tend to occur more frequently in the next generation, and one occurring in chromosomes with below-average evaluations will tend to occur less frequently." (Davis, 1991)

A typical path through the components of the GA is shown as a flow chart in Figure 5.1. Each block of this figure is discussed in detail in this section.

![Flow Chart of GA Generation](image)

**Figure 5.1: Basic outline of GA generation (based on Haupt R& Haupt S, 1998)**

### 5.2.1 Basic Principles of GA

Some biological process and the related terminology should be introduced since they are the basis of GA and this would be helpful in understanding the procedure. More complete guide for learning how GA works can be found in many textbooks such as 'Genetic Algorithms in search optimisation and machine learning', Goldberg, D; 'Practical Genetic Algorithms', Haupt, R & Haupt, S H; 'Handbook of Genetic Algorithms', Davis, L and many more.
5.2.1.1 Chromosomes and genes

All living organisms consist of cells (Figure 5.2). Almost all cells have nucleuses. These nucleuses contain a number of chromosomes specific to the organism. The word ‘chromosome’ means coloured body (in Greek chroma = color and soma = body). The name was given because of its ability to be stained with a certain dye. Each chromosome is composed of a long, continuous strand of DNA.

![DNA: The Molecule of Life](image)

Figure 5.2: DNA: The Molecule of Life (courtesy of U.S. Department of Energy Human Genome Program)

Arranged along the DNA strands are the genes. Genes are physical and functional units of heredity that carry information from one generation to the next. It can be said that each gene encodes a trait, for example colour of eyes. A complete set of genetic material (all chromosomes) is called genome.

5.2.1.2 Definition of parameters and functions (Block 1)

The GA begins by defining the optimisation parameters and functions of a problem. The parameters representing the problem are expressed as genes. A string of genes, called chromosome, represent a possible solution of the problem. Therefore the chromosome can be displayed as an array of parameter values. If a chromosome has $N_{par}$ parameters then it can be presented as an $[1*N_{par}]$ element array (Haupt R & Haupt S, 1998):

$$\text{Chromosome} = [p_1, p_2, p_3, \ldots, p_{N_{par}}]$$  \hspace{1cm} (5.1)

Where $p_1, p_2, p_3, \ldots, p_{N_{par}}$ are genes of the chromosome.
Each chromosome has an objective (fitness) (Goldberg, 1989) function, $F$ where:

$$F = f (\text{chromosome}) \quad (5.2)$$

This function can have any form to enable the description of the optimisation problem; for example a solution that gives minimum cost.

### 5.2.1.3 Parameters representation – Encoding system (Block 2)

GA works with coded parameters (genes) and not the parameters themselves. Choosing the right representation (i.e. constructing the chromosome) can be a difficult task. Many representations are possible but only some can successfully lead to good solutions. The reason is that the specific representation chosen will inevitably limit the search space to certain areas. The representation is based on different encoding systems.

- **Encoding systems**

The parameters of the genetic algorithm process are represented using an encoding system. The type of encoding scheme is vital for the success of the GA since different encoding schemes have different characteristics. The most common encoding systems are the binary encoding (use of binary digit 1 and 0 only) and the value encoding. In this study, real (integer) value encoding is used since it offers the advantage of requiring less storage than the binary genetic algorithm (Haupt R & Haupt S, 1998) and therefore, is very useful for chromosomes with many genes (§ 5.3.2). Furuta et al., (1997 & 1999), applied real value encoding with integer variable to determine a maintenance (repainting) program and an inspection scheme respectively.

In the real value encoding (Table 5.1), every chromosome is a sequence of some values (e.g. real numbers, letters) so real value encoding is a chain of real numbers such as integers.
### Table 5.1: Example of chromosomes with value encoding

<table>
<thead>
<tr>
<th>Chromosome 1</th>
<th>1.858</th>
<th>6.265</th>
<th>5.636</th>
<th>0.526</th>
<th>3.029</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chromosome 2</td>
<td>1 2 1</td>
<td>3 4 1</td>
<td>5 7 8 9</td>
<td>4 3 2 1</td>
<td></td>
</tr>
</tbody>
</table>

#### 5.2.1.4 Creation of Initial population (Block 3)

The GA process generates successive populations of trials referred as chromosomes that form the initial population (ipop). The initial population is formed as a matrix with \( N_{ipop} \) chromosomes. Consequently each row in the matrix represents a particular chromosome of \( N_{par} \) genes.

\[
\text{ipop} = \text{random} \\
\begin{bmatrix}
\text{Chromosome} & 1 \\
\text{Chromosome} & 2 \\
\text{Chromosome} & 3 \\
\vdots & \vdots \\
\text{Chromosome} & N_{ipop}
\end{bmatrix}
\]

As Haupt R & Haupt S (1998) state, “deciding upon the size of the initial and generation population is very difficult. There are some trade-offs between population size and the number of generations needed to converge”.


Syswerda (1991) offers some advice in choosing population size: “General wisdom dictated that a larger population will work more slowly but will eventually achieve better solutions than a smaller population. Experience indicates, however, that this rule of thumb is not always true, and that the most effective population size depends on the problem being solved, the representation used, and the operators manipulating the representation”. Therefore a sensitivity analysis should be carried...
out to specify an effective generation size. However, \( N_{\text{ipop}} \) is considered to be constant for all the required successive simulations (Miyamoto et al., 2000).

### 5.2.1.5 Evaluate fitness – Natural selection - Mating pool (Block 4)

The population is tested based on the fitness function. This function is evaluated independently for each chromosome by inserting the numerical values of its genes. The outcome reflects the way each chromosome compared to the other chromosomes in the population is near to the optimum. However, the initial population is too large to undertake the whole process of the genetic algorithm (Haupt R & Haupt S, 1998). Thus, a large number of the chromosomes in the initial population are discarded through natural selection. According to Darwin’s theory of evolution, the ‘fittest’ survive to create new offsprings (new solutions). Therefore, the fitter chromosomes in the population are chosen so that their offsprings will have higher fitness.

The chromosomes (\( N_{\text{ipop}} \)) of the initial population are ranked from lowest to highest fitness. After that only the fittest \( N_{\text{pop}} \) members of the population are kept. In general, \( N_{\text{pop}} \) can be equal or less than \( N_{\text{ipop}} \) (Haupt R & Haupt S, 1998). Haupt R & Haupt S (1998) state that often only 50% of the initial population is kept and this is adopted here, so:

\[
N_{\text{pop}} = 0.5 \times N_{\text{ipop}} \quad (5.3)
\]

The mating pool (parent’s pool) contains the \( N_{\text{good}} \) chromosomes that are the fittest chromosomes from the earlier selected \( N_{\text{pop}} \). The cross over rate (\( X_{\text{rate}} \)) will decide the number of \( N_{\text{good}} \) chromosomes that will enter the mating pool.

\[
N_{\text{good}} = X_{\text{rate}} \times N_{\text{pop}} \quad (5.4)
\]

High crossover rates introduce many new chromosomes into the population. A low \( X_{\text{rate}} \) on the other hand, does not do much exploring the population. From the literature the crossover rate often is taken from 60%-100% (Jenkins, 1997; Obitko, 1998; Koumousis and Arsenis, 1998, Miyamoto et al., 2000). The \( X_{\text{rate}} \) can be constant throughout the generations or vary from generation to generation (Haupt R
& Haupt S, 1998). Here the X_rate is selected as constant 100% (Miyamoto et al., 2000).

5.2.1.6 Selection of parents (Block 5)

As stated before, the parents that will produce the offsprings are the N_good chromosomes in the mating pool. There are many methods for selecting the pair of parents. Some methods are: roulette wheel selection, tournament selection, random pairing, pairing from top to bottom and some others (Falkenauer, 1998). All techniques are effective and aim to generate random pairs. For this study the commonly used roulette wheel selection technique (Davis, 1991; Suzuki et al., 2000) using the rank weighting (Haupt R & Haupt S, 1998).

In roulette wheel selection each chromosome represents a compartment on the roulette wheel. The bigger the compartment is, the bigger the probability to choose the corresponding chromosome. Selecting N chromosomes from the population is equivalent to playing N games on the roulette wheel; therefore, each candidate is drawn independently. The following Figure (5.3) gives an example.

![Figure 5.3: Distributions of fitness of chromosomes](image)

The process used here can be described by the following algorithm:

1. Chromosomes are ranked in terms of their fitness and assigned an integer number from 1 to N_good with 1 being the fitter chromosome.

2. Each chromosome is assigned a probability (P_n):
\[ P_n = \frac{N_{good} - n + 1}{\sum_{n=1}^{N_{good}} n} \]  

(5.5)

Where \( n \) denotes the number of the chromosomes in the ranking order.

3. The cumulative probability of each chromosome is calculated:

\[ \sum_{i=1}^{n} P_i \]  

(5.6)

4. A random number between 0 and 1 is generated.

5. The first chromosome with a cumulative probability greater than the random number is selected to be a parent.

6. The procedure is repeated \( N_{good} \) times and pairs are chosen based on Step 5. If a chromosome is about to be paired with itself it is better to randomly choose another chromosome to be paired.

5.2.1.7 Crossover (Block 6)

An important operator of the GA methodology is the crossover. The crossover operator is used to generate the offsprings. Crossover is inspired by the role of sexual reproduction in the evolution of biological organisms where the offsprings are formed by combining elements of existing solutions (e.g. DNA strands) in order to have some of the features of each parent. If no crossover takes place the offsprings are exact copies of their parents.

There are many possible ways to perform a crossover operation and in this development, single point crossover is chosen (Liu et al., 1996; Miyamoto et al., 2000; Liu & Frangopol, 2004). In the single point crossover (\( \downarrow \); Figure 5.4) the beginning of the chromosome to the crossover point is copied from the first parent; the rest is copied from the second parent.
For the real value encoding system the genes at the crossover point (point a) are changed into new genes (Haupt R & Haupt S, 1998), and the genes to the right of point a, are swapped as in Figure 5.

So if the parents are:

$$\text{parent}_1 = [p_1-1, p_1-2, \ldots, p_1N_{par}]$$ \hspace{1cm} (5.7)

$$\text{parent}_2 = [p_2-1, p_2-2, \ldots, p_2N_{par}]$$ \hspace{1cm} (5.8)

The offspring will be:

$$\text{offspring}_1 = [p_1-1, p_1-2, \ldots, g_{new1}, \ldots, p_2N_{par}]$$ \hspace{1cm} (5.9)

$$\text{offspring}_2 = [p_2-1, p_2-2, \ldots, g_{new2}, \ldots, p_2N_{par}]$$ \hspace{1cm} (5.10)

To evaluate the new genes at the point of the crossover the following equations are used (Haupt R & Haupt S, 1998):

$$g_{new1} = g_{p1a} - y (g_{p1a} - g_{p2a})$$ \hspace{1cm} (5.11)

$$g_{new2} = g_{p2a} + y (g_{p1a} - g_{p2a})$$ \hspace{1cm} (5.12)

Where $y$ is a random number between 0 and 1; $g_{new}$ is the new offspring gene at point a, rounded down to the nearest integer; $g_{p1a}$ is the gene at point a of parent 1 and $g_{p2a}$ is the gene at point a of parent 2.

"This method does not allow offspring parameters outside the bounds set by the parent unless $y>1$" (Haupt R & Haupt S, 1998).
5.2.1.8 Mutation (Block 7)

The newly created offspring can then be mutated. Mutation is a simple operator consisting of random alternations in the value of genes in a chromosome that are mainly due to malfunction in the copying genes process from parents. While crossover is supposed to exploit greedily the generated solutions to find better ones, mutation is supposed to aid for the exploration of the whole search space. Mutation is the only way to bring new genes to be tested into a population. As a result of that genetic diversity can be maintained in the generated population and local minimum to be avoided. For this reason, in this study, mutation will be applied to the whole population (parent and offspring chromosome) to account also for mutation that parent chromosomes can be subjected in their lifetime and enhance the chances to converge to optimum solution.

There are many different forms of mutation for the different kinds of representation. A simple mutation for real value encoding is adopted here and is consisting of randomly changing the value of a gene in the chromosome. A visual for mutation with real encoding system, is shown in Figure 5.4. After mutation has taken place the value of the 7th gene in the chromosome is altered from 5 to 7.

Table 5.2: Mutation process in chromosome with genes represented as integers

<table>
<thead>
<tr>
<th>Chromosome before mutation</th>
<th>1 2 3 4 5 7 8 9 4 3 2 1 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chromosome after mutation</td>
<td>1 2 3 4 7 7 8 9 4 3 2 1 4</td>
</tr>
</tbody>
</table>

The points to be mutated are selected using a random number generator. The number of points that will be mutated depends on the mutation rate (Haupt R & Haupt S, 1998).

\[
\text{Number of points to be mutated} = (N_{\text{pop}} \times N_{\text{par}}) \times \text{M}_{\text{rate}} \quad (5.13)
\]

Where \(\text{M}_{\text{rate}}\) is the mutation rate.
Mutation rate usually varies from 1% to 20% (Haupt R & Haupt S, 1998). If the mutation rate is too high, the search becomes effectively random since a lot of random changes will occur with a lot of offsprings which do not resemble their parents. If the mutation is too low many useful genes will never be tested. As with the case of $X_{\text{rate}}$, $M_{\text{rate}}$ can be constant or vary during the GA process. In this research mutation rate is taken as constant (Miyamoto et al., 2000).

However, it not wise to perform mutation to already good solutions. For this reason elitism selection is often embedded (Furuta et al., 1997; Miyamoto et al., 2000) to enforce the preservation of the best individual chromosomes of the current generation to the next. With elitism, a number of good solutions are designated as elite solutions and are destined to propagate unchanged. Elitism can very rapidly increase the performance of GA, since it prevents losing the best found solution to date.

5.2.1.9 Convergence (Block 8 and 9)

Through the generations the chromosomes will evolve and ultimate converge to a best solution. The number of generations that will be performed depends on whether a converged solution/plan is reached or an arbitrary number of iterations are exceeded (Haupt R & Haupt S, 1998). The convergence in this study is reached when the parent population has the same chromosomes and cost and therefore even after crossover there is no change in the genes.

Unfortunately, GA may fail because of a convergence to an unacceptable local optimum. Furthermore, it is time consuming to investigate each time key parameters (e.g. mutation rate, population size) of the GA process for various actions-scenarios that need to be examined. To improve the chances to obtain optimum solution and also to get an indication that GA process may not function properly a new GA methodology is proposed and outlined in the following diagram (Figure 5.5).

As Figure 5.5 illustrates, at the beginning ‘GA part A’ is executed one time through mutation ignoring the outcome of the convergence test (step 5) so mutation will be incorporated into the process. Then ‘GA part A’ is performed until convergence occurs not only one time but $N$ times to enhance the probability of reaching an
optimum solution. N here is equal to 48. Next the N optimum plans obtained from the latter procedure are collected and named as genome. If the genome does not consist of the same solutions then the genome is used as the initial population of 'GA part B' therefore further exploiting the converged solutions of 'GA part A'. The analysis will terminate until the genome consists of the same plans and the recommended solution is accepted as the optimum PM strategy. In the case where the optimum solution obtained from 'GA part B' was not one of the solutions of 'GA part A' that means that the GA process did not search the domain properly and alternation of some key parameter of the GA methodology (e.g. mutation rate or generation size) may need to be considered. The idea is based on nature that from 'optimum' parents it should not be possible to obtain 'more optimum' offsprings.

However is has to be noticed that the improvement obtain from 'GA part B' is not guarantied to lead to the optimum solution but to ascertain whether there are flaws in the selection of key parameters of the GA process (§ 5.3.11).
**Methodology for optimum whole life PM strategy using GA**

**GA methodology**

a. Execute "GA part A" until convergence, \( N \) times

b. Collect \( N \) optimum plans, called genome

c. Genome consist of same optimum plans

d(2). Execute "GA part B" until convergence, \( N \) times

d (1). Obtain optimum plan as PM strategy

---

**GA part A**

1. **Selection:** Parameters, Objective (Fitness) Function
2. Parameters representation, Encoding system
3. Creation of initial Population
4. Evaluation of Fitness, Natural Selection, Mating pool
5. Selection of parents
6. Crossover
7. Convergence is reached or the max no of generations is exceeded?
8. Mutation, next generation

**GA part B**

1. **Genome**
2. Evaluation of Fitness, Natural Selection, Mating pool
3. Selection of parents
4. Crossover
5. Convergence is reached or the max no of generations is exceeded?
6. Mutation, next generation

*The first generation of the GA process is performed through mutation even if the solution converges.*

**Figure 5.5: Proposed GA methodology for better convergence**

Elena A. Tantele
5.3 Incorporating GA approach to PM strategy optimisation (GA methodology)-An illustrating example

GA principle have not yet been applied in finding the optimum PM plan specifically to stop /delay a deterioration process (i.e. corrosion) for RC bridge elements. Furthermore, previous studies were based on mainly hypothetical values about the effect of PM action on the life of the bridge and speculative deterioration profiles (Kong & Frangopol, 2004; Neves et al., 2004; Liu & Frangopol, 2004). This section describes in detail the methodology developed, using GA principles to find optimum PM strategies for RC bridge elements prone to corrosion. The effect of PM on the probability of corrosion initiation is taken from Chapter 4 and is based when possible on actual data. Although clearly there is a need for more refined and improved data that will reflect more accurately the reality.

Microsoft Excel software is selected for the implementation of the proposed methodology because of its ease of use and powerful programming features. Macro Language of Excel is used to facilitate the users input and activate the GA process. An example case illustrates the GA methodology. However, in the following sections only some of the Excel tables developed herein are demonstrated.

5.3.1 Definition of the parameters and objective function

In this application each chromosome represents a PM measures strategy and each gene (parameter) represents an action within the strategy. The actions used in this study are:

- do nothing
- silane
- polyurethane sealer
- p-m coating
- cathodic protection
- electrochemical chloride extraction
- concrete replacement

PM measures
An optimum PM strategy should include the key objective of bridge management such as functionality, safety, sustainability, environmental effect, aesthetics, best monetary allocation etc. In this research the objective (objective/fitness function) is to identify a PM strategy that has WLC as low as possible, while maintaining the lifetime $p_f$ of the element examined (under the actions of PM) lower than the target lifetime $p_f$. It is important to recall $p_f$ means the probability of corrosion initiation on the top level of the reinforcement (Chapter 4).

**Objective:**

$$F = \sum_{t=t'}^{T-1} C_{PM_{ij}} \rightarrow \text{minimum cost} \quad (5.14)$$

**Subject to:**

$$p_f(PM_{ij}) < p_f(\text{target}) \quad (5.15)$$

Where $F$ is the total cost of PM measures; $C_{ij}$ is the cost of PM measure $j$ carried out in year $t$; $j$ is the type of PM measure chosen for year $t$; $p_f(PM_{ij})$ is the probability of failure for PM measure $j$ carried out in year $t$; $p_f(\text{target})$ is the maximum acceptable value of $p_f$; $T$ is the expected service life of the bridge element (set here equal to 120 years); $t$ is the bridge element age (years) and $t'$ is the present age of bridge element (set here equal to 0).


**5.3.1.1 Service life and cost of PM measures**

The service life and the costs of PM measures used in the methodology are shown in Table 5.3 (Weyers et al., 1993; HA, 1999; and Pearson & Cuninghame, 1997). Although some are approximate based on the literature, and some are assumed, they are used here for illustration purposes.
Table 5.3: Service life and cost of PM measures (Weyers et al., 1993; HA, 1999; and Pearson & Cuninghame, 1997)

<table>
<thead>
<tr>
<th>PM action</th>
<th>Time (years)</th>
<th>Cost (£/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silane</td>
<td>5</td>
<td>4</td>
</tr>
<tr>
<td>Polyurethane sealer</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>P-m coating</td>
<td>10</td>
<td>30</td>
</tr>
<tr>
<td>Cathodic protection</td>
<td>As long as in service</td>
<td>Installation cost: 220</td>
</tr>
<tr>
<td>Chloride extraction</td>
<td>≈6 weeks</td>
<td>50</td>
</tr>
<tr>
<td>Concrete replacement</td>
<td>N/A</td>
<td>1940</td>
</tr>
<tr>
<td>Waterproofing system</td>
<td>25</td>
<td>25</td>
</tr>
</tbody>
</table>

*the maintenance cost includes maintenance of the anodes and inspection costs for every 5 years.

Herein the whole life PM plan cost is the sum of all PM applications costs applied during the 120 years assumed service life, and are respectively discounted to the present value using the following equation:

\[
C_{PM} = \sum_{i} \frac{C_{PM(i)}}{(1 + r)^{t_i}}
\]

Where \(C_{PM}\) is the sum of the discount cost of \(j^{th}\) PM applications options \(PM_{(i)}\) applied at time \(t_i\) and \(r\) is a constant discount rate of money. Here \(r\) is assumed to be 3% (Tilly, 1997; Green Book, 2003) and can easily be alternated since its value is an input in the program developed.

### 5.3.1.2 Estimation of the \(p_f\) at any reference time

To calculate the \(p_f\) at any time under the action of PM (service life of PM) the \(p_f\) profiles produced in Chapter 4 are employed. By performing best fit on the profiles, the equations obtained using MATLAB 6, are given in Table 5.4. The concrete cover used to obtain these profiles is 40 mm.
### Table 5.4: Equation of $p_f$ of different PM measures

<table>
<thead>
<tr>
<th>PM action</th>
<th>Probability of failure- Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain concrete-Control</td>
<td>$-6.2639\times10^{-20}t^0+5.2362\times10^{-17}t^2-1.8639\times10^{-14}t^5+3.7062\times10^{-5}$</td>
</tr>
<tr>
<td></td>
<td>$t^2-4.5111\times10^{-10}t^6+3.4328\times10^{-8}t^5-1.5649\times10^{-6}t^4+3.5394\times10^{-4}$</td>
</tr>
<tr>
<td></td>
<td>$t^3+7.2955\times10^{-5}t^2+0.00053504t$</td>
</tr>
<tr>
<td>Silane</td>
<td>$-5.3334\times10^{-21}t^{10}+2.5748\times10^{-18}t^2-3.0394\times10^{-15}t^8+9.8177\times10^{-12}t^6$</td>
</tr>
<tr>
<td></td>
<td>$-1.6243\times10^{-10}t^6+1.5531\times10^{-8}t^5-8.521\times10^{-7}t^4+2.2521\times10^{-4}$</td>
</tr>
<tr>
<td></td>
<td>$t^3-4.3991\times10^{-5}t^2+0.0004t$</td>
</tr>
<tr>
<td>Sealer</td>
<td>$-2.7265\times10^{-20}t^8-1.5349\times10^{-16}t^6+3.3151\times10^{-15}t^8$</td>
</tr>
<tr>
<td></td>
<td>$2.8649\times10^{-13}t^7-7.8186\times10^{-12}t^6+3.8924\times10^{-9}t^5$</td>
</tr>
<tr>
<td></td>
<td>$3.4034\times10^{-7}t^4+1.1777\times10^{-5}t^3-1.5508\times10^{-2}$</td>
</tr>
<tr>
<td></td>
<td>$t^2+0.00029636t$</td>
</tr>
<tr>
<td>P-m coating</td>
<td>$-4.6619\times10^{-21}t^8-2.5526\times10^{-18}t^9+5.7275\times10^{-16}t^8-6.7532\times10^{-15}$</td>
</tr>
<tr>
<td></td>
<td>$t^9+4.4489\times10^{-12}t^6-1.6006\times10^{-10}t^5+2.6233\times10^{-4}$</td>
</tr>
<tr>
<td></td>
<td>$t^9+3.9229\times10^{-8}t^3+1.1969\times10^{-6}t^2+3.1456\times10^{-5}t$</td>
</tr>
<tr>
<td>Control-ECE</td>
<td>$-6.3252\times10^{-21}t^8-4.1850\times10^{-18}t^9+1.1822\times10^{-15}t^8-1.8446\times10^{-12}$</td>
</tr>
<tr>
<td></td>
<td>$t^9+1.6917\times10^{-11}t^6-8.5456\times10^{-10}t^5+1.4821\times10^{-8}t^4+4.0067\times10^{-4}$</td>
</tr>
<tr>
<td></td>
<td>$t^7+8.672\times10^{-6}t^2+2.7923\times10^{-5}t$</td>
</tr>
<tr>
<td>Control-Silane</td>
<td>$-1.9619\times10^{-22}t^9+9.7597\times10^{-20}t^9-6.1423\times10^{-18}t^8-3.8542\times10^{-15}$</td>
</tr>
<tr>
<td></td>
<td>$t^9+7.4442\times10^{-13}t^6-1.5217\times10^{-11}t^5-3.7275\times10^{-10}t^4+6.1936\times10^{-8}$</td>
</tr>
<tr>
<td></td>
<td>$t^7+2.8326\times10^{-6}t^2+4.1498\times10^{-5}t$</td>
</tr>
<tr>
<td>Control-Sealer</td>
<td>$-1.3271\times10^{-21}t^9-1.4052\times10^{-18}t^9+5.3888\times10^{-16}t^8-1.0605\times10^{-13}$</td>
</tr>
<tr>
<td></td>
<td>$t^9+1.2023\times10^{-11}t^6-7.9882\times10^{-10}t^5+2.8177\times10^{-8}t^4-3.5211\times10^{-6}$</td>
</tr>
<tr>
<td></td>
<td>$t^7+9.9534\times10^{-6}t^2+3.3802\times10^{-5}t$</td>
</tr>
<tr>
<td>Control-P-m coating</td>
<td>$-2.1893\times10^{-21}t^9+1.2254\times10^{-18}t^9-2.8485\times10^{-16}t^8+3.5526\times10^{-14}$</td>
</tr>
<tr>
<td></td>
<td>$t^9-2.5644\times10^{-12}t^6+1.0821\times10^{-10}t^5-2.6242\times10^{-8}t^4+4.0216\times10^{-6}$</td>
</tr>
<tr>
<td></td>
<td>$8t^5+2.8362\times10^{-7}t^2+9.4879\times10^{-6}t$</td>
</tr>
<tr>
<td>Waterproofing system</td>
<td>$-0.10\times6.2639\times10^{-20}t^9+5.2362\times10^{-17}t^9-1.8639\times10^{-12}t^8+3.7062\times10^{-10}$</td>
</tr>
<tr>
<td></td>
<td>$t^9+4.3123\times10^{-8}t^5-1.5649\times10^{-6}t^4+3.5394\times10^{-4}t^3-7.2955\times10^{-2}$</td>
</tr>
<tr>
<td></td>
<td>$t^2+0.00053504t$</td>
</tr>
</tbody>
</table>

Where $t$ is the reference time.

### 5.3.2 Representation of genes as PM measures

Based on the real value encoding system actions are described by an integer number (Table 5.5), which in turn becomes one of the genes chromosomes.
Table 5.5: Parameter representation

<table>
<thead>
<tr>
<th>Application of PM action</th>
<th>Genetic Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>Do nothing</td>
<td>1</td>
</tr>
<tr>
<td>Silane</td>
<td>2</td>
</tr>
<tr>
<td>Polyurethane sealer</td>
<td>3</td>
</tr>
<tr>
<td>P-m coating</td>
<td>4</td>
</tr>
<tr>
<td>Cathodic protection</td>
<td>5</td>
</tr>
<tr>
<td>Electrochemical chloride extraction</td>
<td>6</td>
</tr>
<tr>
<td>Concrete replacement</td>
<td>7</td>
</tr>
<tr>
<td>Waterproofing system</td>
<td>8</td>
</tr>
</tbody>
</table>

The PM plan is represented as an array of integer numbers such as:

\[
\text{PM plan} = [1 \ 5 \ 3 \ 2 \ 5 \ 6 \ \ldots \ 5 \ 7 \ 1]
\]

The interval for PM maintenance action is assumed to be 5 years so PM is to be carried out after a principal inspection (BD 63/94, 1994; Thoft-Christensen, 1995) is performed. The total genes of the chromosomes are 25. With seven possible options (PM) the possibilities are equal to \(7^{25}\) which are equal to \(1.34 \times 10^{21}\). With the addition of one more PM the combinations are dramatically increase by \(3.64 \times 10^{22}\). From this it is clear that a pure random search or identification of all possible combinations (event tree; Vassie 2000) is not practical or possible.

### 5.3.3 Parameters of the genetic operators used in this GA process

The parameters of the genetic operators used in this GA methodology are given in Table 5.6. Selection method, crossover method and crossover rate were selected based on literature to achieve efficiency while satisfying the convergence requirements (§ 5.2). Population size and mutation rate were chosen based on a sensitivity analysis (§ 5.3.11).
Table 5.6: Parameters of the genetic operators

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Parameter value or method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial population size</td>
<td>48</td>
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<tr>
<td>Population size in every generation</td>
<td>Same as initial population size</td>
</tr>
<tr>
<td>Selection method</td>
<td>Roulette wheel selection and elitism selection</td>
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<tr>
<td>Crossover method</td>
<td>Single point crossover</td>
</tr>
<tr>
<td>Crossover rate</td>
<td>Constant 100%</td>
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<tr>
<td>Mutation rate</td>
<td>Constant 6%</td>
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<tr>
<td>Maximum generation</td>
<td>Until convergence or 2000 generations</td>
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</table>

5.3.4 Initial Population

The initial population was generated using Excel’s random number generator. In this stage, only the ‘do nothing’ options and proactive maintenance can be considered such as: silane, sealer and P-m coating. Therefore, one possible solution contains numbers from 1-4 only. For 48 initial chromosomes (PM plans) a table with 48 rows and 25 columns was created. Each column represents a 5-year period of bridge element service life. Part of the Excel table is shown in Figure 5.6, for the first 85 years.
Table 1: Random numbers generation between 1 and 4. Each random number corresponds to a preventative maintenance measure.

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<td>2</td>
<td>2</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>
The next step is to convert each PM plan to the associated PF based on the PF profiles produced in Chapter 4 (Figure 5.8).

**Figure 5.7: Constraint application for the service life of different PM actions**

**5.3.5 Fitness function –Natural selection-Mating pool**

The next step is to convert each PM plan to the associated $p_r$ based on the $p_r$ profiles produced in Chapter 4 (Figure 5.8).

**Figure 5.8: Conversion of PM actions into $p_r$ values**

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Methodology for optimum whole life PM strategy using GA

However, this is not a straightforward procedure due to the interaction of different PM measures. To overcome this, the following procedure is developed (Figure 5.9):

1-2: At year 0, the $p_{00}$ (initial $p_f$) is assumed. From 0 to 5 year, the $p_f$ profile corresponding to the action applied at year 0 is used.

2-3: At year 5, the $p_{05}$ is matched to the corresponding $p_f$ of the profile of the selected PM ($PM_3$).

3-4: Between 5 and 10 year the profile corresponding to the action applied at year 5 is used.

5 - End of service life: The same process (step 2-3 and 3-4) is repeated for the remaining life of the bridge element.

$p_{0x}$: probability of failure at time $x$, $PM_x$: probability of failure profile of PM applied at time $x$

![Figure 5.9: Estimation of $p_f$ at different time intervals](image)

The fitness function is subjected under the condition to keep the $p_f$ associated with every PM action lower that a target. To achieve that, when $p_f$ is exceeded (red colour in Figure 5.8), reactive PM measures are applied such as concrete replacement, electrochemical chloride extraction and concrete replacement. Special
attention needs to be made to ‘jump’ to the proper PM profile when ECE is applied. This is due to the fact that the p_f profile of ECE depends on the preceded actions.

The process consumes substantial Excel space (tables and sheets) but at the end 48 PM plans are produced all of which maintain the p_f under the target p_f. Then the fitness function is evaluated by computing the costs of each PM plan (Figure 5.10) and ranked based on their overall cost from the lowest to the highest cost. Hence maintenance number 1 may be ranked as 26th (Figure 5.11).

**Figure 5.10: Cost estimation**

After a number of generations, convergence starts to occur and some of the maintenance plans are the same with the exact cost. Since Excel gives the same ranking number to the same cost, the program is developed so that to be able to rank plans with the same cost in ascending order. The correct ranking is essential for the selection of plans for reproduction.

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Figure 5.11: Ranking of PM options based on their total cost

For reproduction, half of the initial population is selected based on Haupt R & Haupt S (1998). From the 48 ranked chromosomes only the 24 with the lowest cost continue to the next step of GA (Figure 5.4). With a crossover rate equal to 100% the survived 24 PM plans will form the mating pool for the selection of the parents.

5.3.6 Selection of parents

To select the combination of the chromosomes that will become parents of the next generation the roulette wheel method is used which was explained in Section 5.2.1.6. The 24 plans are ranked from the lowest to the highest cost. Each PM plan is assigned a probability computed using Equation 5.5. Then the cumulative probabilities are calculated (Equation 5.6). Using Excel random generator 24 random numbers between 0 and 1 are generated and compared with the cumulative probabilities respectively for each PM plan. If the cumulative probability is higher...
that the random number the PM plan is selected to be a parent (Figure 5.12). In case where the program chooses the same parent to form a pair, automatically it will select a second parent with the directly higher probability. This will enhance the genetic diversity since two offsprings will not be produced from the same parent.

5.3.7 Crossover

Crossover takes place using the single crossover method (§ 5.2.1.7).

A random number gives the position where crossover is to take place. In Figure 5.13, year 15 is the crossover point. The values to the left of this point stayed the same while the values to the right are swapped. The values at the crossover point are changes as explained in Section 5.2.1.7.

The old population (parents) and the new population (offspring) are each composed of 24 chromosomes and are as shown brown and green in Figure 5.13 respectively. These two populations combined, after they are subjected to mutation, will comprise

Figure 5.12: Selection of parent
the next generation. Therefore, the number of chromosomes in each new generation is always the same (48 plans).

![Random crossover point: 15th year. The values at this column are changed. The values in the right column are swapped](image)

**Table 26: Crossover**

**Figure 5.13: Crossover operator**

### 5.3.8 Mutation

The mutation is applied to the whole population (old and new). The mutation rate here is 6% (§ 5.3.11.2). That means that from the 48 chromosomes and 25 time intervals (48*25 = 1200 values), 6% will change i.e. 72 values will be replaced by new ones. However, since elitism is embedded two ‘parent’ chromosomes with the least overall PM cost are not subjected to mutation.

In Figure 5.14 the 12th row specify the number p_i points to be mutated. The 13th and the 14th row display the randomly selected plans and time respectively that mutation will take place. For example the first point to be mutated is on the 23rd PM plan at the 10th year and will change to value 3. The mutated point is circled on the Figure 5.14.
Figure 5.14: Mutation

After carrying out the same procedure for all of the mutate points the formed PM plans structure the new population for the next generation (Figure 5.15).

Figure 5.15: Next generation
From this point, the procedure is repeated until convergence is achieved.

5.3.9 Convergence

When convergence is achieved for each generation whole life cost is plotted against the optimum 24 PM plans proposed (§ 5.2.1.9). Figure 5.16 illustrates one of the converged plans. After 41 generations convergence to the lowest cost option is reached and the optimum plan is produced.

![Figure 5.16: Convergence to minimum cost](image)

Table 5.7 presents the different actions selected at 5-year intervals as established by the GA methodology.
Table 5.7: Optimum PM Plan

<table>
<thead>
<tr>
<th>Year</th>
<th>0</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
<th>45</th>
<th>50</th>
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<tbody>
<tr>
<td>Encoded actions in GA</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>4</td>
<td>-</td>
<td>6</td>
<td>1</td>
<td>3</td>
<td>6</td>
<td>1</td>
<td>4</td>
<td>1</td>
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<td>95</td>
<td>100</td>
<td>105</td>
<td>110</td>
<td>115</td>
<td>120</td>
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</tr>
<tr>
<td>Encoded actions in GA</td>
<td>4</td>
<td>-</td>
<td>6</td>
<td>1</td>
<td>3</td>
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<td>1</td>
<td>4</td>
<td>-</td>
<td>4</td>
<td>-</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

Decoded Actions: 1: Do nothing, 3: Sealer, 4: P-m Coating, 6: Electrochemical chloride extraction, -: P-m coating still effective due its service life (10 years)

The element performance when this recommended optimum PM plan is applied can be seen in Figure 5.17 which shows the $p_f$ profile with time. At no time frame the $p_f$ exceeds the target $p_f$ failure.

![Figure 5.17: Probability of failure profile](image)

Based on the input data the program chooses to utilise electrochemical chloride extraction rather than cathodic extraction or replacement when critical concentration
of chloride was expected on the rebar. This makes sense since the use of electrochemical chloride extraction has a dramatic impact on the \( p_r \) profile at little cost compared to the other reactive options. The application of PM measures is not the same throughout the lifetime of the element since the costs are discounted and cheaper solutions were sought at the beginning of the strategy (0-20 years).

5.3.10 Key parameters affecting the program efficiency

5.3.10.1 Population size

A sensitivity analysis was carried out to investigate the population size by running ‘GA part A’ (Figure 5.5) for the example case. In accordance with Haupt R & Haupt S (1998) a population size of 24 chromosomes was first examined in this research. To determine the effect of the population size on the GA process also a population size with 48 chromosomes (number of human chromosomes) and 6 chromosomes was tested. In the case with the 24 chromosomes, in order to achieve convergence and obtained the plan with the minimum cost more generations were needed (less efficient for the program) rather than in the case with 48 chromosomes. Using 6 chromosomes as the initial population, convergence occurred earlier but the minimum cost obtained for this case was higher than in the cases where population size was 48 and 24. Therefore the population size in every generation was set equal to 48 chromosomes.

5.3.10.2 Mutation rate

The GA is sensitive to the choice of mutation rate. A mutation rate between 1 and 10 % often works well but needs to be specifically checked to examine its effect on the program efficiency. An analysis using different \( M_{\text{rate}} \) (1\%, 2\%, 3\%, 4\%, 5\%, 6\%, 7\%, 8\%) was carried out. Figure 5.18 illustrates that 6\% and 7\% \( M_{\text{rate}} \) give the lowest overall PM cost.
Figure 5.18: Selection of mutation rate

In order to identify if the appropriate mutation rate is being used, ‘GA part B’ process is performed on the example case. Table 5.8 shows that the cost of the converged solutions obtained from ‘GA part A’ process and using M_{rate} equal to 4%, 5%, 8% was not the minimum costs (£79.33). However, an improvement was noticed when ‘GA part B’ process was carried out. For example, with M_{rate} =4% the cost from ‘GA part A’ process was £79.56 but from ‘GA part B’ process was £79.33.

In some cases this improvement lead to the minimum solution (M_{rate} = 8% and 4%) but sometimes the improvement was not enough (M_{rate} = 5%). The cost of the converged solution obtained from ‘GA part A’ process and M_{rate} = 6% and 7% was the minimum therefore no improvement was noticed while carried out ‘GA part B’ process. In present study 6% M_{rate} was used than 7% since it gave less number of generations while carry out ‘GA part A’ process. As a consequence it is concluded that the selection of M_{rate}= 6% is a reasonable value.
Table 5.8: Mutation rate Vs minimum cost

<table>
<thead>
<tr>
<th>Mutation rate (%)</th>
<th>Minimum cost from ‘GA part A’, (£/m²)</th>
<th>Minimum cost from ‘GA part B’, (£/m²)</th>
</tr>
</thead>
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<tr>
<td>4</td>
<td>79.56</td>
<td>79.33</td>
</tr>
<tr>
<td>5</td>
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<tr>
<td>8</td>
<td>79.40</td>
<td>79.33</td>
</tr>
</tbody>
</table>

It is worth mentioning here that even though the ‘GA part B’ in the cases shown in Table 5.8 gave a small reduction in the final cost (≈0.09%-0.3%), in other cases a bigger reduction was achieved. For example in Section 6.3.3 and 6.3.7, despite the fact that the $M_{\text{rate}}$ was taken as 6%, ‘GA part A’ did not reach the minimum cost and ‘GA part B’ gave a higher reduction in cost (≈1.6%). This may be justified with the fact that the initial parameters for choosing $M_{\text{rate}}$ as the optimum discount rate were altered in the cases presented in Section 6.3.3 (i.e. target pf) and 6.3.7 (i.e. modified ECE). Therefore the use of ‘GA part B’ proved to be very useful for the parametric studies presented in Chapter 6.

5.4 Discussion and Conclusions

GA is a powerful tool that simulates the logic of Darwinian selection for solving various optimisation problems. A methodology based on the GA principles is developed to obtain optimum or near optimum PM strategies. It relates the uncertainties associated with the effectiveness of PM (Chapter 4) with their costs.

The methodology developed was implemented using Excel XP with Macro Language to utilise its familiar interface and powerful functions. Excel provides visualisation of the results in every step of the GA process so the whole procedure is clear to the user.

From an initial population of random generation solutions, GA can drive the near to optimum solution towards the objective set (e.g. minimum cost). This is achieved by generating and evaluating tens or even hundreds of thousands of different combinations. As David Goldberg, a pioneer of GA said “Three billion years of evolution can’t be wrong. It’s the most powerful algorithm there is” (Naik, 1996).
From the example case study illustrated in this chapter the following conclusions can be drawn:

- The GA methodology showed that PM measures can help to secure the safe and efficient operation of bridge elements. The optimum PM strategy proposed kept the probability of corrosion initiation below a critical value (0.1) at all times with minimum cost.

- The GA methodology showed sensitivity on the selection of key parameters of GA such as mutation rate and population size, however, improved GA methodology has been proposed to enhance the probability of reaching an optimum PM strategy and also to examine whether key parameters of the methodology were correctly chosen. The real (integer) value encoding method was efficient in representing and processing the PM strategy.

- The results show that the use of reactive measures such as electrochemical chloride extraction has a strong impact on the reliability performance of the element examined.

- To maximise the benefits from the use of this methodology whether it is for showing the trend of the main parameters or for practical use, there is a need to enhance the quantity and quality of input data of the methodology.

It is not the intention to give the impression that GA is a magic tool. There are some crucial decisions (e.g. representation of the problem, implementation of basic steps of the GA process) to be made when casting a problem. The example case highlighted not only how the production of PM strategy was dealt with but also a number of issues that are extremely important for the efficient operation of GA methodology. It is the author's opinion that GA has fascinating possibilities and will see increasing use due to the increasing power of computing hardware and constant demand for solution of complex optimisation problems.

The next chapter will present various case studies to examine the proposed GA methodology of obtaining near optimum PM strategy for RC bridge element.
Chapter 6

Case studies examined

6.1 Case studies examined using the GA methodology

The GA methodology developed in Chapter 5 is used to examine a number of cases. In total 24 case studies are reported here. In each case study, a number of PM actions are defined as options and the methodology is used to identify the most optimum whole life strategy (e.g. Table 5.7). The PM actions include representative surface treatment measures such as silane, sealer, coating, cathodic protection (CP), electrochemical chloride extraction (ECE), concrete replacement (CR), and waterproofing system (WS). Based on the opinion of the author these were considered to be the most frequently used in the UK.

Unless specified otherwise, the input data for the GA methodology (Figure 5.5) of the Pilot case presented in Chapter 5 are used. Therefore, the effectiveness of PM actions is based on the models and data derived in Chapter 4. The service life and cost of PM measures, their p_r’s equation for any reference time as well as their encoding representation used to obtain the following cases are taken from Tables 5.3 -5.5. Also the parameters of the genetic operators used for the GA process are obtained from Table 5.6.

The Pilot case is utilised for the comparison with the other cases to identify trends and to study the sensitivity of different input data of the GA methodology.

Table 6.1 presents the PM options defined in the various cases examined. The case studies, examined a beam with average quality PC and 40 mm cover with the
exception of case 24, which investigates a deck with similar quality concrete, and depth of cover.

Table 6.1: PM measures used in case studies 1 to 24

<table>
<thead>
<tr>
<th>Case studies Pilot case</th>
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<th>2</th>
<th>3</th>
<th>4-7</th>
<th>8-15</th>
<th>16-18</th>
<th>19-20</th>
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<td>0.1</td>
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<td>Service life of the bridge element</td>
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<td>120</td>
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<td>60</td>
<td>120</td>
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<td>120</td>
</tr>
</tbody>
</table>

* Different effectiveness profile from Chapter 4

The optimum strategies obtained for the cases of Table 6.1 are listed for comparison purposes in Appendix F.
6.2 Validation cases

The first two cases were selected to illustrate the reliability of the methodology as they provide useful validation results.

6.2.1 Effectiveness of concrete replacement: Case study 1

For this case only 2 actions are considered: 1) Do nothing and 2) Concrete replacement. At the end of every GA process the WLC is plotted for the 24 PM plans in order to show the convergence of the program to the lowest cost option.

Using the proposed GA methodology (Part A, Part B; Figure 5.5) an optimum plan is proposed after 8 generations, with a WLC equal to £2281.4/m². Figure 6.1 shows one of the generations during which the GA process achieved convergence.

![Graph showing convergence to the lowest cost](image)

**Figure 6.1: Convergence to the lowest cost – Case study 1**

The program performs as expected and convergence to the optimum PM plan is achieved. This works as a useful validation case since the outcome can easily be estimated by other methods.
The optimum PM plan proposed by the GA methodology, showing the various actions selected at 5 year intervals, can be seen in Table 6.2. Every twenty years the option ‘Apply concrete replacement’ was chosen since the target $p_f$ was reached at that time interval.

Table 6.2: Optimum PM Plan of Case 1

<table>
<thead>
<tr>
<th>Year</th>
<th>0</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
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<th>45</th>
<th>50</th>
<th>55</th>
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</tr>
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<tbody>
<tr>
<td>Encoded actions in GA</td>
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<td>1</td>
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<td>110</td>
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</tr>
<tr>
<td>Encoded actions in GA</td>
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<td>1</td>
<td>1</td>
<td>7</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>7</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

Decoded Actions: 1: Do nothing, 7: Apply CR

To visualise the element performance when this optimum maintenance plan is applied the $p_f$ profile as a function of the bridge element age is plotted (Figure 6.2).

Figure 6.2: Probability of failure profile – Case study 1
6.2.2 Effectiveness of P-m coating: Case study (2):

In Case 2, more measures were studied than Case 1, as illustrated in Table 6.1. The measures used here are the same as in Pilot case but without the options CP & ECE. Figure 6.3 shows one converged solution of the GA methodology that was achieved after thirty generations with a minimum WLC equal to £113.9 /m².

Figure 6.3: Convergence of optimum PM plans – Case study 2

This case can be estimated without the use of the program, since the Pm-coating is by far the most preferable action. The estimated solution is a good validation for the program, since the program identified that the use of the P-m coating is the most effective solution from the given options.

The effectiveness of the P-m coating is significantly higher than the sealer and the silane (Figure 4.19) while the relative increase in cost is small (Table 5.3). The GA optimum solution shows that by selecting to apply the P-m coating every 10 years (Table 6.3) the \( p_f \) remains below this critical level for the period of 120 years (Figure 6.4).
### Table 6.3: Optimum PM Plan of Case study 2

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</tr>
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</table>

Decoded Actions: 4: P-m coating, -: P-m coating still effective during its service life (10 years)

![Figure 6.4: Probability of failure profile – Case study 2](image)

**Figure 6.4: Probability of failure profile – Case study 2**

### 6.3 Results of remaining case studies

#### 6.3.1 Effect of ECE Case study (3) + Pilot case

In this case the effect that a different combination of PM options has in the formation of an optimum strategy is investigated. The P-m coating is not used as an option; therefore a more frequent use of silane and sealer is applied to ensure than the $p_f$ remains lower than the target $p_f$ (Figure 6.16).
Figure 6.5: Convergence to the lowest cost – Case study 3

The proposed optimum PM plan is shown in Table 6.4. From this table it can be seen that the program selected to do nothing for the first 20 years and then use ECE, silane, sealer at frequent intervals. This can be justified due to the discount rate that is applied on the cost of the different PM measures. Therefore, the program applies PM measures as late as possible, for the discounted WLC to be minimised.

Table 6.4: Optimum PM Plan of Case study 3

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Decoded Actions; 1: Do nothing, 2: Silane, 3: Sealer, 6: ECE
Figure 6.6 show the effect of these actions on the $p_r$ profile of the examined element.

Figure 6.6: Probability of failure profile – Case study 3

6.3.2 Effect of different discount rates: Case studies 4-7 + Pilot case

Cases 4-7 examine the sensitivity of the discount rate. As shown in Figure 6.7 a higher WLC is reached when the discount rate is 0% where the lowest cost is given when the rate is 8%. The outcomes have the same trends as in Figure (2.19).

Figure 6.7: Effect of discount rate on the WLC
The optimum maintenance plans proposed for different discount rates can be seen in Tables 6.5-6.8.

**Table 6.5: Optimum PM Plan with discount rate = 0% - Case study 4**

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

**Table 6.6: Optimum PM Plan with discount rate = 3% - Pilot case**

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

**Table 6.7: Optimum PM Plan with discount rate = 3.5% - Case study 5**

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)
Table 6.8: Optimum PM Plan with discount rate = 6% - Case study 6

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

Table 6.9: Optimum PM Plan with discount rate = 8% - Case study 7

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

Table 6.5 shows that when the discount rate is 0% a P-m coating is utilised in most PM intervals. However, Tables 6.6-6.9 show that when the discount rate is higher than 0% (e.g. 3%, 3.5%, 6%, 8%) the program proposed for the first 20 years less costly solutions such as ‘Do nothing’ and then the use of ECE and surface treatment at different PM intervals for the remaining life of the element. With high discount rates such as 6% and 8% the costs tend to reduce with time (Figure 2.19). Therefore the use of ECE is employed more frequently, almost double the times used in the cases with lower discount rate (0%, 3%, 3.5%).

Figures 6.8 -6.9 illustrate the $p_f$ profile under the action of the proposed PM plans. At no point in time does the $p_f$ exceed the target $p_f$. 

Elena A. Tantele
Figure 6.8: Probability of failure profile – Case studies 4-5 +Pilot case

Figure 6.9: Probability of failure profile – Case studies 6-7
6.3.3 Effect of different target $p_f$: Case studies 8-15 + Pilot case

Case studies 8-15 investigate the effect of different target $p_f$ failures. From the results of these cases it is obvious that the target reliability influences highly the outcome of the WLC. Figure (6.10) shows that when the target $p_f$ increases the cost decreases. This was actually expected since increased reliability (less $p_f$) is immediately likely to be related to higher general cost.

However the use of same intervals of target $p_f$ does not necessarily mean that the effect in the WLC will be the same between the cases. This is due to the availability of the options to fit a strategy to ensure that the $p_f$ remains below a target $p_f$. As more options are available the difference in WLC between the cases is expected to be more uniform.

![Figure 6.10: Effect of target $p_f$ on the WLC](image)

As mentioned in Chapter 5 (§ 5.3.10.2), by using the ‘GA part B’ of the proposed methodology (Figure 5.5) it enabled the reduction of the minimum WLC obtained from ‘GA part A’ by approximately 1.6%.

The optimum plans obtained from the GA methodology for each examined target $p_f$ are given in Tables 6.10-6.15.
Table 6.10: Optimum PM Plan with target $p_f = 0.80$ – Case study 8

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</table>

Encoded actions in GA | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 105 | 110 | 115 | 120 |

Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

Table 6.11: Optimum PM Plan with target $p_f = 0.85$ – Case study 9

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Encoded actions in GA | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 105 | 110 | 115 | 120 |

Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

Table 6.12: Optimum PM Plan with target $p_f = 0.90$ – Case study 10

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</table>

Encoded actions in GA | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 105 | 110 | 115 | 120 |

Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)
### Table 6.13: Optimum PM Plan with target $p_f = 0.95$ – Case study 11

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

### Table 6.14: Optimum PM Plan with $p_f = 0.10$ – Pilot case

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

### Table 6.15: Optimum PM Plan with $p_f = 0.105$ – Case study 12

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)
Table 6.16: Optimum PM Plan with target $p_f = 0.11$ – Case study 13

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

Table 6.17: Optimum PM Plan with target $p_f = 0.115$ – Case study 14

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

Table 6.18: Optimum PM Plan with target $p_f = 0.120$ – Case study 15

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

The $p_f$ profiles of these cases are shown in Figures 6.11-6.13. Despite some differences in the choice of PM actions, a pattern can be identified from these figures. The element is left to deteriorate with no action taken, approximately for the
first 20 years. Then the use of some surface treatments for a period of 20-30 years follow by the use of ECE is repeated throughout the lifetime of the bridge element.

Figure 6.11: Probability of failure profile – Case studies 8-10

Figure 6.12: Probability of failure profile – Case studies 11-12 + Pilot case
Figure 6.13: Probability of failure profile – Case studies 13-15

6.3.4 Effect of initial $p_f$: Case studies 16-18 + Pilot case

The following case studies are examining the effect of initial $p_f$ on the WLC. As expected with higher initial $p_f$ the WLC also increases (Figure 6.14). This is due to the fact that more money was needed to restore the initial condition of the bridge element to safe levels (concerning its probability of failure) at an early stage.
The optimum plans for different initial $p_f$ are given in Table 6.19-6.22.

### Table 6.19: Optimum PM Plan with initial $p_f = 0$ – Pilot case

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

### Table 6.20: Optimum PM Plan with initial $p_f = 0.025$ – Case study 16

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

### Table 6.21: Optimum PM Plan with initial $p_f = 0.05$ – Case study 17

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)
Table 6.22: Optimum PM Plan with initial $p_f = 0.10$ – Case study 18

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

Figure 6.15 illustrates the corresponding $p_f$ profiles of these cases.

Figure 6.15: Probability of failure profile – Case studies 16-18 + Pilot case

From Table 6.19 and the Figure 6.15 it can be seen that when the initial $p_f$ is equal to 0 the program selects the use of some PM measures such as sealer, P-m coating and ECE, but almost in the 1/3 of the maintenance intervals (especially in the first fifteen years) the use of ‘Do nothing’ is preferred.
With the increase of initial $p_f$ to 0.025 (Table 6.20) the more frequent use of PM measures such as sealer and P-m coating is observed to restore the $p_f$ to acceptable levels while the option ‘Do nothing’ is chosen less frequently. Furthermore, while the initial $p_f$ increased to 0.05 (Table 6.21) and 0.1 (Table 6.22) the use of ECE is increased too. However it is noticed that the use of sealer is less frequently selected, but P-m coating and ‘Do nothing’ option are more often used.

In all cases when the $p_f$ is reaching 0.1 the use of ECE is employed to lower the $p_f$ level.

### 6.3.5 Effect of different service life: Case studies (19-20) + Pilot case

Case studies 19-20 show that the program can propose optimum PM plan for different service life scenarios. As expected the cost will be reduced as the service life is reduced too (Figure 6.16).

![Bar chart showing the effect of service life on WLC](chart.png)

**Figure 6.16: Effect of service life on the WLC**

Tables 6.23 and 6.24 show the optimum plan proposed with service life 60 years and 100 years respectively.
Table 6.23: Optimum PM Plan with service life = 60 years – Case study 19

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

Table 6.24: Optimum PM Plan with service life = 100 years – Case study 20

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

Table 6.25: Optimum PM Plan with service life = 120 years – Pilot case

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)
Case studies examined

Figure 6.17: Probability of failure profile – Case studies (19-20) + Pilot case

From the optimum plans (Table 6.23-6.25) and from the \( p_f \) profiles (Figure 6.17) it can be seen that the same pattern is kept for the first 30 years. The plan for the remaining service life of the element is not the same in the three cases since the element has different service life and therefore the program is selecting PM actions with criterion the optimum selection of the measures based on the whole service life of the element.

6.3.6 Effect of modified P-m coating: Case study 21 + Pilot case + Case 3

This case was chosen to show how the different effectiveness of available PM measures affect optimum PM strategy and consequently the WLC. When the P-m coating is 50% less effective that which was assumed (Figure 5.19), the cost increases. However, the cost is still less than without the use of P-m coating (Figure 6.18). The use of ‘GA part B’ of the proposed methodology (Figure 5.5) resulted to approximately 0.7% reduction of the minimum WLC obtained from ‘GA part A’.
Figure 6.18: Effect of the effectiveness of P-m coating on the WLC

The optimum strategy proposed with 50% effective P-m coating is shown in Table 6.26 and it consists of PM measures such as silane, sealer, P-m coating and ECE. With 50% effective P-m coating the use of silane is observed in comparison with the case where the P-m coating is 100% effective (Table 2.27). There is also an increase in the use of sealer but a reduction of the application of P-m coating. The use of ECE is the same in both cases. When there is not an option of P-m coating (Table 2.28) the program select the use of silane, sealer and much more use of ECE.

Table 6.26: Optimum PM Plan with P-m coating 50% effective – Case study 21

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6:ECE, -: P-m coating still effective due its service life (10 years)
Table 6.27: Optimum PM Plan with P-m coating 100% effective – Pilot case

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, - : P-m coating still effective due its service life (10 years)

Table 6.28: Optimum PM Plan with no P-m coating- Case study 3

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 6: ECE

The $p_f$ profiles (Figure 6.19) show the effect that different effectiveness of the same measure has in the performance of the element. The $p_f$ profile of both 50% and 100% effective P-m coating is almost the same despite the fact that there is a different selection of PM measures. When the $p_f$ is reaching 0.1, the use of ECE is favourably selected.
6.3.7 Effectiveness of modified ECE: Case study 22

In this case ECE is assumed to have different effect than that estimated with the probabilistic methodology in Chapter 4. When ECE is applied the $p_f$ is assumed to reduce to zero and then for the next 5 years to follow the $p_f$ profile of 'no treated' concrete. One of the converged solutions can be seen in Figure 6.20. Here, with the use of the 'GA part B' of the proposed methodology (Figure 5.5), a reduction of approximately 1.6% of the minimum WLC, obtained from ‘GA part A’, was achieved.

Figure 6.19: Probability of failure profile – Case study 3 + Case study 21 + Pilot case
Figure 6.20: Convergence to the lowest cost – Case study 22

As it can be seen from Table 6.29 the element is left to deteriorate untreated and when it reaches target $p_f$, ECE is applied. The same pattern can be seen in Case 2 (Table 6.3) but with higher cost since the ECE has the same impact on the performance of the element as the concrete replacement with the only difference being that ECE is less expensive.

Table 6.29: Optimum PM Plan of Case study 22

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, - : P-m coating still effective due its service life (10 years)

Figure 6.21 shows the $p_f$ profile for this case.
6.3.8 Effectiveness of modified ECE including P-m coating: Case study 23

In this case ECE has the same $p_f$ profile as in Case 22 (Table 6.29) but this time the P-m coating (which is considered the most effective surface treatment proactive measure) is included in the PM options.
The pattern of the optimum PM plan (Table 6.30) is the same as in Case 22 (Table 6.29) since the use of the P-m coating could not provide lower cost PM strategy. Therefore the program was capable of ignoring options that will not have any beneficial effect on the optimum PM plan.

Table 6.30: Optimum PM Plan of Case study 23

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, -: P-m coating still effective due its service life (10 years)

The $p_f$ profile for this case is illustrated in Figure 6.23

![Figure 6.23: Probability of failure profile – Case study 23](image-url)
6.3.9 Effect of Waterproofing System: Case study 24

A waterproofing system (WS) is examined in this case. The element in examination here is a deck rather than a beam that was in the previously mentioned cases. The use of this action is mandatory therefore WS is applied at year 0. After the first 25 years (service life), the measure chosen by the program is based on its objective and fitness function. The cost of WS is taken as £25/m² (HA, 1999) and its assumed service life to be 25 years (Pearson & Cuninghame, 1997). Figure 6.24 shows one converged solution of the GA methodology that was achieved after six generations with minimum WLC equal to £43.7/m².

![Figure 6.24: Convergence to the lowest cost – Case study 24](image)

Table 6.31 gives the optimum plan based on the action used to ensure that the $p_f$ during the 120 years stay lower than 0.1 (Figure 6.25).
Table 6.31: Optimum PM Plan of Case study 24

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Decoded Actions; 1: Do nothing, 8: waterproofing system, - : waterproofing system still effective during its service life (25 years).

Figure 6.25: Probability of failure profile – Case study 24

The use of WS is selected throughout the 120 years and not CP or CR. The WS is applied every 25 years (service life of WS) with the exception of a 5-year gap (at the 25th year) during which the deck can be left to deteriorate with no action applied. Overall, the \( p_f \) remains below the critical \( p_f \) of 0.1 for the whole 120 years and the WLC of the plan is noticeably less than if CR (Figure 6.3) or CP (Table 5.3) is used.
6.4 Discussion and Conclusions

A number of case studies were analysed using the methodology developed in this project. The objective of this part of the study was to demonstrate the application of the program and examine how the resulting optimum strategy and costs may be affected when different sets of PM options are included. A number of the cases selected also serve as useful validation for the program as the optimum plans for these cases can also be identified manually. In total 24 case studies were examined, of which 23 are related to various PM actions applied to a beam of a RC bridge and one case (no. 24) is related to a deck of RC bridge where waterproofing would be applicable.

Cases 1 and 2 are useful validation cases and the results obtained confirm the accuracy of the program.

In Case 3, P-m coating measure is excluded from the options and that gives higher cost than when this is an option (Pilot case; Chapter 5). The program was capable of recognising that and providing the appropriate results.

Cases 4-7 examine the effect of different discount rate. With the actions used in the Pilot case the program proposed different plans depending on the discounted rate used. Lower discounting rate gives high WLC.

Cases 8 – 15 use the same actions as in the Pilot case (but with different target \( p_I \)). The results of these cases provide a feel for the sensitivity of the results to the target \( p_I \) and confirm that the program provides the correct trend in the results. As expected higher target \( p_I \) gives the lower cost

Cases 16-18 examined the effect of different initial \( p_I \). A higher initial \( p_I \) gives higher WLC.

Cases 19-20 show that the GA methodology can be implemented for different service life. As expected lower service life will gives lower WLC.
Case 21 investigates the effect that different effectiveness of PM that initially assumed in the Pilot study has on the optimum PM plan. In this example the effectiveness of P-m coating is assumed to be 50% than that calculated in Chapter 4.

Case 23 uses the same actions as Case 22 with the addition of P-m coating. Also the \( p_r \) profile of ECE is different than the \( p_r \) profile estimated in Chapter 4. Both of these cases provide the same output. This shows firstly that P-m coating can not bring any additional benefit with this group of actions and secondly that the program was capable of rejecting P-m coating to reach the optimum PM plan with the minimum cost.

Case 24 examines the efficiency of the WS on a deck. From Figure 6.18, it can be seen that when the WS is included as an option this is selected throughout the 120 years with the exception of one 5-year interval during which no action is applied. The overall profile remains below the 0.1 critical value for the 120-year period.

All of the optimum PM plans obtained from these cases maintain their \( p_r \) lower than the target \( p_r \) at any time. Where feasible, the program chooses PM actions as late as possible—usually after the 20th year (Table 6.2, Table 6.4-6.7, and Table 6.9). This is due to the discount rate. If the same actions were applied in a different order, while keeping the \( p_r \) lower than 0.1 at any time, this would have resulted in higher cost.

Overall, the results of these case studies confirm the accuracy and efficiency of the program and comparison of the different cases shows some interesting trends in the results. When sensitivity studies were carried out, the proposed GA methodology (Figure 5.5; Chapter 5) proved to give with the use of ‘stage B’, plan with even lower WLC than with the use of ‘stage A’.

Furthermore, this study confirms that the proposed methodology can be used effectively to examine various strategies for PM measures in order to identify the optimum course of action. Improved quality of data would be required to maximise the benefits from the use of this methodology.
Chapter 7

Conclusions, guidelines and recommendation for future work

7.1 Introduction

The overall objective of this research is to develop a methodology that will provide a tool for relating the uncertainties associated with the effectiveness of PM measures and their cost in order to develop optimum PM plans for RC highway bridge elements exposed to chloride ions. This key aim has largely been achieved. The specific conclusions and observations derived in each area considered have been given in detail at the end of each chapter. As such, this chapter aims to reiterate the main discussion points and to clarify the main conclusions of the research work described in this thesis.

This chapter concludes the work in two parts. The first part summarises the findings of the probabilistic methodology developed to evaluate the effectiveness of PM measures. The second part reviews the conclusions related to the GA methodology developed for obtaining optimum PM strategies. Suggested guidelines for the probabilistic analysis procedure and for the application of GA methodology, recommendation for future work and the original contributions of the work are presented at the end of the chapter.

7.2 Conclusions from the probabilistic methodology for evaluating the effectiveness of PM measures

Preventative maintenance time-application and effectiveness contain uncertainties that may lead to waste of valuable time and inappropriate use of resources. It is very
important that uncertainties are identified and reduced as far as possible to ensure that the needs of bridge maintenance are met in a cost-effective manner. To achieve this, proactive (action taken before failure occurs) bridge maintenance approach is adopted in this research.

In Chapter 3, the causes of the RC bridge deterioration problem in the UK are examined briefly. The dominant deterioration mechanism identified and adopted in this study, is the diffusion of chlorides due to de-icing salt. Existing chloride ingress model, Fick's second law of diffusion, is normally used to characterise the diffusion process but its simplistic nature may introduce some limitations and approximations. However, a more complex model involves the utilisation of complex relations and therefore more refined data will be needed, which in most cases are currently unavailable. Consequently, the simple diffusion model (Fick’s law of diffusion) is used in this study.

PM measures that affect the process of diffusion of chlorides were identified and a review was undertaken to find relevant data on their effectiveness (Chapter 3). For non-critical chloride contaminated RC elements the use of surface treatment can delay the chloride ingress. Three types of surface treatments are currently available, namely coating, sealer and penetrant. Surface treatments are considered effective to reduce the rate of chloride ingress; however their true effectiveness is hard to be estimated. Based on BD 43/03 (2003), impregnation with hydrophobic pore-lining impregnants, shall be carried out to new structures and to structures that are already in service provided they comply with certain criteria. Waterproofing system (WS) can also be applied to non-critical chloride contaminated RC elements. They can prevent further chloride penetration, so in the UK their application is mandatory since 1965 (Pearson and Cuninghame, 1997). Lack of use of WS on the bridge decks can lead to severe corrosion, e.g. the Florida Keys Bridges (Sagüés et al., 1994). Nevertheless, failures of waterproofing systems to stop water and chlorides were noticed (Broomfield, 1997). For existing chloride contaminated RC elements (e.g. bridge decks), impressed-current cathodic protection can provide the ultimate permanent solution as long as appropriate maintenance is carried out on the rectifiers and electrical wiring and permanent power supply is provided. An alternative
electrochemical repair method is the electrochemical chloride extraction that can also stop steel corrosion in contaminated concrete. This method is less permanent (4-12 weeks) and has the advantage of having no rectifiers or wiring required after the short time of treatment. However, it had been reported that it cannot remove all the chloride from concrete and its long-term effectiveness is not known. Another method for critical chloride contaminated RC elements is concrete replacement method where the chloride contaminated concrete is removed. Its successful performance depends on many factors such as its correct application. Furthermore, it is in general expensive with uncertain service life.

With the main deterioration mechanism defined and relevant PM measures identified, a probabilistic analysis procedure was developed to illustrate the effectiveness of PM as probability of failure \( (p_f) \) profiles (Chapter 4). With the use of the probabilistic methods uncertainties associated with the PM measures are incorporated in the procedure. Furthermore, the uncertainties that have arisen from the limitation of the deterministic chloride ingress model (Equation 4.2) to describe the complex chloride ingress mechanism and the large scatter and variability of field data are also incorporated. The method was applied to elements of RC bridge (e.g. beam, slab etc). A limit state (margin) is set to satisfy the condition, that the chloride concentration at the surface of the reinforcement should not exceed the threshold (critical) concentration value (corrosion initiation). Probability of failure profiles, indicate the violation of the limit state i.e. the probability that corrosion initiation will occur. To evaluate the probabilities of failure Monte Carlo simulation method was used. Excel XP and Visual Basic 6 were employed for the development of a program to run the simulations. A validation of the results was carried out using a commercial program, PROBAN, and showed a good agreement in the results.

Although the required data for the probabilistic analysis procedure are limited, some data were identified and used to perform the probabilistic analysis. Probability of failure profiles were produced illustrating the effectiveness of PM on the chloride ingress. The degree of effectiveness reduces as the rate of increase of \( p_f \) increases. Representative proactive and reactive PM measures were examined and their degree of effectiveness was established. Clearly, there is a need to improve the database on
the PM characteristic's to obtain better predictions of $p_f$ profiles. As more refined data become available, this procedure can be used to compare the effectiveness of different PM measures and generate profiles for subsequent selection of PM strategies. Nevertheless, the findings presented in Chapter 4 can not only provide information on the relative importance of the parameters but can be used to provide a tool that will facilitate the creation of successfully optimised PM measures strategies.

From the results of the sensitivity analyses, it was possible to identify which of the PM measures examined were more effective and to what extent these were able to protect the rebar from chloride ions and consequently to prevent corrosion initiation. Significant trends of the parameters of the analysis on probability of failure ($p_f$) with and without PM applied on the concrete were identified. The parameters examined were the mean value of concrete cover depth ($x_{cover}$), threshold chloride concentration ($C_{th}$), surface chloride concentration ($C_o$) and diffusion coefficient ($D_e$), the distribution of $C_o$ and the standard deviation of $D_e$ ($s_{De}$). As expected most of these parameters were found to have a significant effect on the failure probabilities for initiation of corrosion in both cases (with or without PM) except for the change of $s_{De}$ and the distribution of the $C_o$ that had a minor effect on the $p_f$. From the results of the sensitivity analysis was noted than an increase in $C_o$ and $D_e$ resulted in a significant increase in failure probabilities. Increase in $x_{cover}$ and $C_{th}$, resulted in a considerable reduction of the failure probabilities. The information obtained from these analyses can be used to identify the effect of the parameter but also for prioritising which of these parameters need to be more accurately described. However, these results are obtained based on the parameter configuration (e.g. distribution) assumed in the analyses.

Furthermore, it should be emphasised that the proposed analysis (i) ignores system effects and examined only bridge elements, (ii) assumes all reinforcement corrodes uniformly across the entire steel area of the top reinforcement, (iii) assumes the absence of inspection and repairs. Also, the values assigned to the parameters of the corrosion models are subjected to considerable uncertainty while the model of corrosion initiation is only approximate. Therefore the failure probabilities calculated
Conclusion, guidelines and recommendation for future work

herein should be considered ‘notional’ and therefore the results should be used for comparison purposes only.

7.3 Conclusion from the genetic algorithm methodology for identifying optimum PM strategies

An expansion of the above probabilistic procedure is described in Chapter 5, where the $p_r$ profiles are related to cost and whole life planning to enable various possible PM strategies to be compared. PM actions need to be planned in such a way that on the one hand the probability of failure (i.e. initiation of corrosion) is kept below a minimum acceptable level and on the other hand the cost is kept to the minimum possible value. Therefore, it is necessary to optimise their use. To achieve the optimisation, a GA based methodology was developed which was shown to be very effective.

The methodology developed was implemented in a spreadsheet program (Excel XP) using Macro Language to utilise its familiar interface and powerful functions. Excel provides visualisation of the results in every step of the GA process so the whole procedure is transparent to the user.

From the example case study illustrated in Chapter 5 the following conclusions can be drawn:

- The GA methodology showed that PM measures can help to secure the safe and efficient operation of bridge elements. The optimum PM strategy proposed kept the probability of corrosion initiation below a critical value (0.1) at all times with minimum overall cost.

- The GA methodology showed sensitivity on the selection of key parameters of GA such as mutation rate and population size, however, improved GA methodology has been proposed to improve the changes of obtaining optimum PM strategies and also to examine whether key parameters of the methodology were correctly chosen. The real (integer) value encoding method was efficient in representing and processing the PM strategy. A population size equal to 48 and a mutation rate 6% was
proved to be satisfactory for reaching convergence in an acceptable time period.

- The results of this example case study show that the use of reactive measures such as electrochemical chloride extraction has a strong impact on the reliability performance of the element examined.

- To maximise the benefits from the use of this methodology whether it is for showing the trend of the main parameters or for practical use, there is a need to enhance the quantity and quality of input data of the methodology.

The GA methodology and the Excel program have been demonstrated and validated in Chapter 6. The methodology was validated using two cases for which the outcome could easily be estimated outside the methodology. Overall, twenty four case studies have demonstrated the potential benefits of the developed GA methodology for the identification of optimum PM strategies.

From the case studies it was possible to identify a trend for an optimum PM strategy based on the PM measures examined. Generally, the program selects ‘Do nothing’ option for approximately the first 20 years. When the \( p_f \) was reaching target \( p_f \) the reactive measure ECE (electrochemical chloride extraction) was chosen in favour of the cathodic protection and concrete replacement, as it was considered to be the more economical solution. For the remaining life of the element examined (i.e. beam) the program selected the reapplication of proactive measures such as P-m coating, silane and sealer. Different combinations of the proactive measures were produced in the optimum strategies depending on their availability. That gave an indicator on their effect on the WLC of the element. For the beam the most optimum PM strategy for the same examined initial constraints (i.e. effectiveness of PM measures, discount rate, target \( p_f \), initial \( p_f \), and service life of the bridge element) was the use of sealer, P-m coating, and ECE. For the deck the optimum strategy was composed of reapplication of the waterproofing system.

It was also clear that the input data (i.e. effectiveness of PM, target \( p_f \) etc) had a great impact on the reliability of the results. However, it has been established that the
methodology developed can be used effectively to examine various possible combinations of PM measures and identify optimum PM strategy.

In addition, the results have also highlighted the need for more data in the area of PM effectiveness for reliable outcomes. Nevertheless, when the data are limited, the method and outcomes can still be very useful in gaining a better understanding of the parameters that influence the results. The methodology developed here is a valuable tool for engineers, enabling them to gain better understanding of the relative effectiveness of PM and applying them more successfully.

7.4 General guidelines

From the findings of the research a number of guidelines relating to the factors that control the probabilistic analysis method and to the implementation of the GA methodology are highlighted in the following section

7.4.1 General guidelines for the probabilistic analysis procedure

- Methods of analysis. Probabilistic analysis is best used when there are uncertainties associated with the parameters of the analysis performed. For computing the $p_f$, either analytical methods (FORM, SORM) or simulation methods can be used. This depends on the complexity of the functions used for the analysis. Simulation methods are more easily utilised for complex function (e.g. error function). For detail of these methods reference should be made to standard textbooks, see for example Ditlevsen & Madsen (1996), Melchers (1999).

- Method of simulation. The Monte Carlo simulation method can be used. It involves simulating a large number of numerical experiments and using the results to estimate $p_f$. The number of simulations required for good estimation of $p_f$ using crude Monte Carlo simulation can be very large for low $p_f$. For $p_f$ values higher than $10^{-4}$ four million simulations have found to be sufficient. If it is not possible to run with such large number of simulations more advanced Monte Carlo techniques can reduce the number of simulations required to compute the $p_f$. Four techniques are commonly used in reliability analysis.
These are Latin hypercube Sampling, Importance Sampling, Stratified Sampling and Directional Simulation which are described in detail in standard textbooks.

- Limit state function. To show the effectiveness of PM measures it is important to state what condition is required to be maintained in order to consider the PM effective. In this study this condition is the delay of the initiation of corrosion due to chloride ingress.

- Deterioration model. Depending on how effective the PM measures are, this will delay/stop certain deterioration process. After establishing the process the appropriate model to be incorporated into the analysis is ascertained. For diffusion of chloride ions Fick's second law is commonly used. Even though its simple nature may introduce some limitations and approximations more complex models will need more refined data, which are probably not readily available.

- Data. The data are used to examine the limit state function for violation. If the required data are limited, the available data must be identified and applied to perform the probabilistic analysis. As more refined data become available it will be possible to calculate more reliable results. Therefore, the final use of the results depends on the reliability of the data. If the data are not consistent, the results can be utilised to provide some guidelines.

- Sensitivity analysis. To ascertain the effect and identify significant trends of the parameters used for the probabilistic analysis, a sensitivity analysis must be conducted. A sensitivity analysis will allow a range of plausible inputs to be considered when there is uncertainty about the true value of an input. It will also identify which parameter's representation require the most attention.

- Programming language. Such analysis can be performed either using commercial software or by developing specific programs. In this case Excel XP and Visual Basic (VBA) 6 were used to develop a program where the commercial software, PROBAN was used to perform some validation work.
The advantage of using Excel with VBA rather than a commercial program is that it provides more flexibility to build the program based on the requirements of the specific problem examined. Furthermore, Excel provides the visualisation of the results in every step of the analysis, which is very useful for checking intermediate results.

7.4.2 For the application of GA methodology

A GA methodology can be used for optimisation purpose. When a program is developed for realisation of the methodology the following points should be considered:

- Initial population size. It should be chosen based on a sensitivity analysis on the specific problem. There is no recipe to give the number of chromosomes needed for efficient programming. However there is a connection between the number of chromosomes used and the reliability of the results.

- Encoding. It depends on the problem and on the size of the chromosome. For maintenance activities real value encoding can be used efficiently. With this encoding a large number of activities can successfully be incorporated without taking too much space in the algorithm.

- Selection. It is clearly an important genetic operator, that parents are selected Roulette wheel selection works well with real value encoding. Also elitism method should be used for saving the best found solution to the next generation.

- Crossover and mutation type. Operators depend on the chosen encoding and on the problem. For real value encoding one point-crossover and single mutation work well. In this study mutation was applied to the whole population (parents and new offsprings) to enhance the chances of finding an optimum solution.
• Crossover rate. This rate should normally be high (80%-100%). In this study a crossover rate 100% is used and proved to be sufficient for the generation of optimum PM strategies.

• Mutation rate. Can also influence the efficiency of the program and the reliability of the results. Therefore a sensitivity analysis is a necessity for choosing the appropriate mutation rate for the specific problem examined. For real value encoding 1-10% often works.

• The quality of input data will clearly have a directed effect on the results of the GA methodology. When sufficient data is unavailable, the methodology can be used as a tool for proving general guidelines.

• Validation cases and parametric analysis are an important part of such a development. Validation cases can be used to evaluate the integrity and correctness of the program, since these cases can be estimated with the use of other methods. Therefore a confirmation of the correct execution of the methodology can be achieved. Parametric analysis can present the sensitivity of the parameters and identify the values of the parameter that can be used in the methodology, to provide ‘optimum’ results.

• Programming language. In this study Microsoft Excel software is selected for the implementation of the proposed methodology because of its ease of use and powerful programming features. It may be possible to use commercial software (Evolver, Generator, ActiveGA, MATLAB Genetic Algorithm Toolbox, etc), which have some GA capability but their applicability to the problem examined here needs to be investigated.

7.5 Original contribution and main conclusion

The research presented here has provided valuable information and a unique insight into the following key areas:
• Development of a probabilistic methodology for the prediction of the effectiveness of various PM measures than can be applied on RC bridge elements.

• Development of a genetic algorithm methodology for developing a tool for obtaining optimum PM strategies.

• Presentation of the current status of maintenance strategies of bridges particularly in the UK.

• Identification of areas where difficulties still remain with the proposed methodology which prevent more consistent reliability prediction being made.

• Assessment of the relative significance of and sensitivity to main parameters of the probabilistic procedure and the genetic algorithm methodology.

• Useful insight into the relative effectiveness of different PM strategies based on the results of the case studies examined.

• Some guidelines on the application of the probabilistic procedure and the GA methodology developed for obtaining optimum PM strategies has been introduced.

• Recommendations on areas that require further research to improve the developed probabilistic procedure and GA methodology towards optimum maintenance strategies for bridge elements.

This work has generated knowledge about:

• The applicability of probabilistic methods for assessing the effectiveness of PM measures.

• The influence of the uncertainties on the parameter of the diffusion model.

• The applicability of GA methods for identifying preventative maintenance strategies.

• The influence of various GA operators for the successful implementation of GA methodology.
The principal conclusions of this thesis are:

- The probabilistic methodology developed to incorporate the uncertainties associated with the need and effectiveness of PM measures is shown to be an efficient way to estimate quantitatively the degree of effectiveness of PM measures in delaying/inhibiting the corrosion initiation of reinforcement in concrete bridge elements due to chloride ingress. The limited availability of data was highlighted by this study which needs to be addressed in future.

- An optimisation methodology using the GA principles was developed linking the cost with the probabilistic effectiveness of PM in order to identify optimum PM strategies. The implementation of the GA methodology on a typical bridge element showed that it is an efficient approach for identifying optimum PM strategies. These developments need to be viewed within the context of the specific constraints and assumptions made, the limitations of the statistical methods used and the limited availability of data.

- An improvement on the developed GA methodology was made to increase the users' confidence in finding optimum solutions and assess the efficiency of the parameter values used in the process. The methodology developed was validated using two simple case studies though a direct comparison with other optimisation methods was outside the scope of this study.

- The various case studies performed demonstrate the applicability of the methodology and highlight the efficiency and consistency in the results produced. The precise mixture and timing of MP selected within optimum strategy results depends on the relative effectiveness and cost of the measures as well as the range of available options. In general there is a need to combine both proactive and reactive PM measures to obtain optimum strategies and it is shown from the results that as the range of available proactive and reactive options increases, optimum strategies with reduced WLC can be obtained.

- The study shows that the GA methodology can provide a useful comparison of optimum PM strategies and highlights the possible benefits from introducing this approach to bridge maintenance in terms of safety and cost.

- The methodologies developed are generic and can be adapted for use in different deterioration mechanisms, structures or networks. Furthermore, they can also be extended for application in design for maintenance.
7.6 Recommendations for future work

The developments presented here provide a good basis for the examination and rational selection of optimum preventative maintenance (PM) strategies by incorporating the uncertainties associated with the effectiveness of PM and the related costs. A number of areas have been identified where further work is needed to refine the methodology, improve the relevant databases, and extend the application of these methods. These are briefly discussed below.

The deterioration model can be refined further using more complex models. However, at present the lack of good quality data in the area of deterioration mechanism and effectiveness of PM limits the benefits from using models that are more refined. As more data becomes available, improved models can be beneficially employed. This highlights the need for consistent data collection to enable more effective use of these methods, which in turn will lead to better use of limited resources.

Absorption due to capillary action can also be examined. Studies carried out by TRL (UK) showed that absorption is an important mechanism of chloride transportation. Therefore, it can be incorporated into the proposed methodology.

The methodology developed addresses problems on elements of a specific type of bridge. However, given the generic nature of the approach developed here the methodology can be developed further for application to other types of bridges or for a network of bridges. This type of methodology can also be extended for the application to other types of civil infrastructure.

The limit state assumed here relates to corrosion initiation. The methodology can be extended further to include the corrosion phase of the first and subsequent bars and relate it to appropriate essential maintenance measures. Such a methodology will enable a combination of essential and PM measures to be examined. Furthermore, the effectiveness of routine maintenance can be investigated within this content.
A further development can examine the strategy from the system point of view as opposed to element and incorporate spatial variability of various parameters within the bridge.
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Appendix A: Monte Carlo Simulation

A.1 History

Monte Carlo simulation (MCS) was named after the city in Monaco, where the primary attractions are casinos containing games of chance (roulette wheel). The name and systematic development of Monte Carlo methods date from about 1944 by early pioneers (Stanislaw Marcin Ulam, Enrico Fermi, John von Neumann and Nicolas Metropolis). The real use of Monte Carlo methods as a research tool stems from the work on atomic bomb design (random neutron diffusion in fissile material) during the Second World War.

A.2 The Monte Carlo method

A Monte Carlo method is a stochastic technique that by using random numbers and probability gives us a way to model complex systems that maybe hard to investigate with other types of techniques. Monte Carlo methods are now used routinely in, any fields including engineering, economics, finance, physics and chemistry.

A random number generator produces a random number between 0 and 1. Each such number is interpreted as a value of the cumulative distribution function $F_X(x)$ and delivers the associated realisation $x$ of the variable $X$. Thus set of variables is fed into the limit state function and by repeating this process many times it is possible to simulate the probability distribution for the safety margin by progressive building up a larger sample. Next, the number of failure $n_o$ i.e. the number of all realisations for which the limit state function is $<0$, is counted and the probability of failure $p_r$ can be calculated according to the frequency of failure from:

$$p_f \approx \frac{n_o}{n}$$

In the above expression $n$ is the total number of all realisation and the greater the number of $n_o$, the more reliable is the prediction of $p_r$. This can be shown through the coefficient of variation of $p_r$, which is defined as:
In order to achieve an acceptably low CoV, say for example 10% for a probability of failure in the region of $10^{-4}$ as many as $10^{-6}$ simulations would have to be produced. For more complicated limit state functions and lower probabilities of failure this process can, even on today’s computers, be slow. In order to reduce the computational effort a method called ‘importance sampling’ has been developed whereby the realisations can be focused on the failure region. Another technique, which improves the efficiency of the simulation, especially for small failure probabilities, is ‘directional simulation’ (Ditlevsen and Madsen, 1996).

An alternative method to determine $p_f$ is to statistically analyse the set of realisations by determining the mean value, $\mu_M$ and standard deviation $\sigma_M$ of the resulting failure function $M$ (i.e. the function of all realisation). From these two values the safety index $\beta$ can be calculated as:

$$\beta = \frac{\mu_M}{\sigma_M}$$

And the probability of failures can be estimated as:

$$p_f \approx \Phi(-\beta)$$

### A.3 Limitation of MCS

The Monte Carlo simulation has only become feasible in the last decade or so due to the increasing availability of powerful computers, but the technique is computationally very cumbersome for very small failure probabilities such as they are typical of bridge elements. Hence, the potential disadvantage of MCS is the amount of computing time needed, especially when very small probabilities of failure are being estimated. Furthermore simulations could be misleading if the user utilise the wrong assumptions; for example giving the wrong distribution of the variables of the model. Therefore a proper modelling of the variable is essential.
Appendix B: Typical bridge life definitions

The following figure presents various notions of ‘bridge life’ (OECD, 1992).

<table>
<thead>
<tr>
<th>Year</th>
<th>Event</th>
<th>Structural life</th>
<th>Service life</th>
<th>Economic (&quot;optimal&quot;) life</th>
<th>Functional life</th>
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<tbody>
<tr>
<td>1950</td>
<td>End of construction (prestressed bridge)</td>
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<td></td>
<td>Normal inspections and maintenance</td>
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<tr>
<td>1965</td>
<td>Problems with cables protection</td>
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</tr>
<tr>
<td></td>
<td>More regular inspections, special maintenance</td>
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<tr>
<td>1968</td>
<td>Road is widened, bridge is posted</td>
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<tr>
<td>1972</td>
<td>Some cables are replaced</td>
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<td></td>
<td>(restricted budget excludes replacement)</td>
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<td>1975</td>
<td>New problems, bridge replaced</td>
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<tr>
<td></td>
<td>Bridge failure</td>
<td></td>
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</tbody>
</table>

Figure B.1: Typical bridge life definitions

Where the design life of a structure is the projected life (in years) of a new structure under normal loading and environmental conditions before replacement or major rehabilitation is expected.
Appendix C: Moments of Random Variables

C.1 First Moment - Mean or Expected Value

This is a ‘weighted average’ of all the values that a random variable may take:

\[
E(X) = \mu_X = \int_{-\infty}^{\infty} x f_X(x) \, dx \quad \text{for continuous variable}
\]
\[
= \sum_{i} x_i p_X(x_i) \quad \text{for discrete variable}
\]

C.2 Second moment – Variance and standard deviation

The variance of a random variable is a measure of the degree of the randomness about the mean:

\[
Var[X] = E(X - \mu_X)^2 = E(X)^2 - (\mu_X)^2
\]
\[
= \int_{-\infty}^{\infty} (x - \mu_X)^2 p_X(x_i) \quad \text{for continuous variable}
\]
\[
= \sum_{i} (x_i - \mu_X)^2 p_X(x_i) \quad \text{for discrete variable}
\]

The standard deviation is defined as

\[
\sigma_X = \sqrt{Var(X)}
\]

The coefficient of variation (CoV) is defined as

\[
CoV_X = \frac{\sigma_X}{\mu_X}
\]
C.3 Third Moment-Skewness

A measure of skewness or lack of symmetry of a distribution is given by the third central moment about the mean:

\[ E(X - \mu_X)^3 = \int_{-\infty}^{\infty} (x - \mu_X)^3 f_X(x) \, dx \quad \text{for continuous variable} \]

\[ = \sum_{i} (x_i - \mu_X)^3 p_X(x_i) \quad \text{for discrete variable} \]

Hence

\[ \text{Skewness}(\gamma_1) = \frac{E(X - \mu_X)^3}{\sigma_X^3} \]

As Figure C.1 shows a positively skewed distribution has a longer tail to the right. A negatively skewed distribution has a longer tail to the left. A distribution with no skew (e.g. a normal distribution) is symmetrical.

\[ \gamma_1 < 0 \quad \gamma_1 = 0 \quad \gamma_1 > 0 \]

(a) (b) (c)

Figure C.1: Skewed Distributions (a) Negative skewness; (b) zero skewness; (c) positive skewness
C.4 Fourth Moment – Coefficient of kurtosis

A measure of the flatness of distribution is given by the fourth central moment

\[ E(X - \mu_X)^4 = \int_{-\infty}^{\infty} (x - \mu_X)^4 f_X(x) \, dx \]

\[ = \sum_{i} (x_i - \mu_X)^4 p_X(x_i) \]

Hence

\[ \text{Kurtosis}(\gamma_2) = \frac{E(X - \mu_X)^4}{\sigma_X^4} \]

Figure C.2 illustrates the notion of kurtosis. The PDF on the right has higher kurtosis than the PDF on the left. It is more peaked at the centre and it has thicker tails.

Normal distributions have a kurtosis of 3 (mesokurtic) irrespective of their mean or standard deviations, if the kurtosis is greater than 3 it said to be leptokurtic (peaked, long thicker tails). If the kurtosis is less than 3 is said to be platykurtic (less peaked; smaller thinner tails).
Appendix D: Box Plots

D.1 Introduction

A box plot is a graph that is useful for very large data. A box plot summarises the data to only five numbers: the median, upper and lower quartiles, and minimum and maximum values. It provides a quick visual summary that easily shows centre, spread, range and any outliers, from two or more samples.

D.2 Parts of the box plot

Below is a sample of vertical box plot showing its’ different parts.

Definitions:

- Maximum is the largest value in the data set.
• **Minimum** is the smallest value in the data set.

• **Median** ($Q_1$) is a measure of the centre that is the middle value. If the list has an odd number of entries, the median is the middle entry in the list after sorting the list into increasing order. If the list has an even number of entries, the median is equal to the sum of the two middle (after sorting) numbers divided by two. The crossbar indicates this value. When the crossbar is off-centred in the box, it indicates that the data is skewed in the middle 50% of the data. If the crossbar is closer to the bottom of the box and if the whisker is shorter on the lower end of the box than on the upper end, then, the distribution is positively skewed. When the cross bar is in the middle of he box, and the spread of the values are almost the same range on both sides of the curves then the distribution is symmetric.

• **Lower or first quartile** ($Q_1$) refers to the value which exceeds no more than 25% of the data, and which is exceeded by no more that 75% of the data. **Upper or third quartile** ($Q_3$) refers to the value which exceeds no more than 75% of the data, and which is exceeded by no more than 25% of the data.

• **Outliers** are the extreme values in the dataset. They fall outside the whiskers on both sides of the box.

• **Whiskers** are the lines that extend out from the bottom of the box (and from the top of the box) to the smallest (or larger value), or to the value that is one half times the values of the IQR if that is sorted.
Appendix E: Genetic Algorithms

E.1 The genetic algorithm (GA) used in this study

1. The first step of the GA is to identify the objective function that expresses the main goal of carrying out the process and indicates the fitness of the potential solutions.

2. Then an initial population of chromosomes is randomly generated.

3. Based on the objective/fitness function, the fitness of the chromosomes is evaluated.

4. The fitter chromosomes will form the parent (mating) pool.

5. The parents are combined using a genetic operator called crossover to produce the offspring. Two parents will produce two offsprings. The parents and the offsprings will form the new generation.

6. The new generation will be subjected to random mutation i.e. random changes in few of the genes of individual chromosomes.

7. Then the fitness of the chromosomes of the new generation is evaluated.

8. Finally the outcome of the fitness evaluation is subjected to constraints. Firstly if convergence is reached or secondly if the maximum generation is exceeded. If the answer of any of this is positive then the algorithm will stop. Otherwise the algorithm will be repeated from Step 4 until the final constraints are met.
E.2 Schemata theorem

The schema theory says that "short, low order schemata are given exponentially increasing or decreasing numbers of samples depending on a schema's average fitness" (Goldberg, 1989). This theorem can be expresses by the following equations:

\[ m(H, t+1) \geq m(H, t) \times \frac{f(H)}{f_{avg}} \left[ 1 - p_c \left( \frac{\delta(H)}{L-1} - O(H)p_m \right) \right] \]

Where \( m(H, t+1) \) and \( m(H, t) \) are the number of schema \( H \) in generation \( t+1 \) and \( t \), respectively; \( f(H) \) is the average fitness values of strings that include \( H \); \( f_{avg} \) is the average fitness value of the whole population, \( \delta(H) \) is the length of schema \( H \); \( L \) is the total length of string; \( O(H) \) is the order of schema \( H \) and \( p_c \) and \( p_m \) are the probabilities of crossover and mutation, respectively.

Interested readers are referred to Goldberg (1989) for details of the theorem and a simple example of schema processing.
## PM plans

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Decoded Actions: 1: Do nothing, 2: Silane, 3: Sealer, 4: P-m Coating, 6: ECE, - : P-m coating still effective due its service life (10 years), 7: Concrete replacement, 8: Waterproofing system