OPTIMISATION OF INSPECTION PLANNING FOR STRUCTURES

by

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DECLARATION

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SUMMARY

The main aim of this thesis was the development of a maintenance management methodology which incorporates two aspects of a structural inspection, i.e. (i) detection of a defect and (ii) sizing of a defect. Markov transition matrices were used to simulate the growth, inspection, repair and failure of a population of defects over time. The range of damage extent was subdivided into groups. Since the methodology was developed to be material independent, the ability to simulate many forms of deterioration was considered important. Therefore, two parameters representing (i) the growth rate and (ii) the deterioration kinetics (i.e. abruptness) of the growth were introduced to simulate the growth of a defect. In addition, it was recognised that some materials can demonstrate an initiation phase and a propagation phase of deterioration so the ability to simulate both phases of deterioration was incorporated into the methodology. By specifying deterioration parameters for each group independently it was possible to simulate non-linear deterioration propagation. Alternatively, the deterioration parameters from the model can be determined by fitting to results of laboratory or on-site testing for the rate of deterioration, as demonstrated in the thesis.

In relation to inspections, parameters were introduced for both stages of an inspection to allow the optimum inspection quality for detection and sizing to be determined for a particular structure or group of structures taking deterioration characteristics, environment and limit state into account. The Probability of Detection and Probability of False Alarm were used to quantify the quality of an inspection for detection and two new parameters, termed the Probability of Good Assessment and Probability of Wrong Assessment, were introduced in this thesis to quantify the quality of an inspection technique for defect sizing. By separating these procedures the interaction between both stages of an inspection could be studied, allowing the optimal inspection quality for detection and sizing to be determined and the sensitivity of the optimal inspection qualities to be investigated.

Considering the repair of defects, initially it was assumed that defects in a structure were repaired using the original material with which it was constructed. This methodology was further developed to simulate the repair of defects using a material which was different to the original construction material, allowing the efficiency of different repair materials to be compared, facilitating the optimisation of the repair material and NDTs. To further

study this, an experimental program was carried out to investigate the rate of deterioration in different concrete repair materials. According to the literature, corrosion is the most common form of deterioration in reinforced concrete. It was therefore decided to study the rate of crack growth in reinforced concrete due to chloride-induced corrosion using an accelerated corrosion procedure. For this thesis, three common repair mixes were studied, OPC, OPC+PFA and OPC+GGBS. Adding PFA or GGBS to a concrete mix is an alternative to using OPC alone, and is reported in the literature to result in an increase in the durability of a new structure, or of a repair of an existing structure. The results of the experimental study demonstrated that the rate of crack propagation in OPC+GGBS is actually faster than in OPC and that OPC+PFA has the slowest rate of crack propagation of the three mixes considered. By analysing the mass loss of the reinforcing bars at different points in the experiment, it was concluded that the mass loss needed to produce a particular crack width is significantly larger for an OPC+PFA concrete. This superior performance of OPC+PFA in relation to concrete cracking is considered to be due to its higher porosity.

Using these experimental results, the capabilities of the developed methodology were demonstrated using a practical example. Since the results of the study were from an accelerated corrosion test, they first had to be converted to real time before being used in the maintenance management model. Following this, the deterioration parameters for each group were determined based on the converted experimental results. The initiation time of the concrete mixes was estimated based on information from the literature. Using the maintenance management model developed, the optimal inspection interval, optimal inspection qualities for detection and sizing, the expected costs and the expected number of failures could be determined for each repair material. For the reinforced concrete example considered, OPC+GGBS resulted in the lowest cost for an inspection interval of up to 4 years and the lowest number of failures for an inspection interval of up to 2 years, even though it had the highest rate of crack propagation. OPC+GGBS did however have the longest initiation time. Otherwise, OPC+PFA had the lowest cost ($\Delta T \geq 5$years) and the lowest number of failures ($\Delta T \geq 3$years). In addition, the sensitivity of the optimum inspection interval and optimum combination of inspection qualities to the estimated initiation time and rate of crack propagation was studied. Other sensitivity studies were also performed in the context of the derived methodology to investigate the effect of changes in the input parameters (e.g. growth parameters, modelling parameters, mode of failure, limit state) on the resulting maintenance strategies.

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NOMENCLATURE

A = surface area of reinforcing steel (cm^2)

AASHTO = American association of state highway and transportation officials

BMS = bridge management systems

CF = cost of failure for an individual defect

CI1 = cost of an individual inspection for detection

CI2 = cost of an individual inspection for sizing

CoV = coefficient of variation

C_o= initial cost of construction

CR = cost of an individual repair

d = actual size of the defect

d_c = critical defect size

$\bar{d}_i$ = mean defect size of a group i

d_{min} = detection threshold

d_{ref} = reference defect size

d_{ref_pf}= reference defect size for the probability of failure, Weibull law parameter

d_l = limit defect size, Weibull law parameter

$\hat{d}_1$ = size of the detected defect (from inspection 1)

$\hat{d}_2$ = size of the defect from inspection (from inspection 2)

$E(C_{F\_TOTAL})$ = expected mean annual total cost of failure (sum of all groups)

$E(C_{F1})$ = expected mean annual cost of failure for each group due to no detection at an inspection year

$E(C_{F2})$ = expected mean annual cost of failure for each group life due to no repair at an inspection year

$E(C_{F1}^{+})$ = expected mean annual cost of failure for each group following a correct inspection for detection (leading to no sizing assessment)

$E(C_{F1}^{-})$ = expected mean annual cost of failure for each group following an incorrect inspection for detection (leading to no sizing assessment)

$E(C_{F2}^{+})$ = expected mean annual cost of failure for each group following a correct sizing assessment (leading to no repair)

$E(C_{F2}^{-})$ = expected mean annual cost of failure for each group following an incorrect sizing assessment (leading to no repair)

$E(C_{F3})$ = expected mean annual cost of failure for each group between inspections

$E(C_{I\_TOTAL})$ = expected mean annual total cost of inspections (sum of all groups)

$E(C_{I1})$ = expected mean annual cost of inspections for detection for each group

$E(C_{I2})$ = expected mean annual cost of inspections for sizing for each group

$E(C_{I2}^{+})$ = expected mean annual cost of inspections for sizing due to correct inspection results for each group

$E\left(C_{I2}^{-}\right)$ = expected mean annual cost of inspections for sizing due to incorrect inspection results for each group

$E\left(C_{R}\right)$ = expected mean annual cost of repairs for each group

$E\left(C_{R}^{+}\right)$ = expected mean annual cost of repairs due to correct inspection results for each group

$E\left(C_{R}^{-}\right)$ = expected mean annual cost of repairs due to incorrect inspection results for each group

$E\left(C_{R\_TOTAL}\right)$ = expected mean annual total cost of repairs (sum of all groups)

$E\left(C_{R\_TOTAL}^{+}\right)$ = expected mean annual total cost of repairs due to correct inspection results (sum of all groups)

$E\left(C_{R\_TOTAL}^{-}\right)$ = expected mean annual total cost of repairs due to incorrect inspection results (sum of all groups)

$E\left(C_{TOTAL}\right)$ = expected mean annual total cost (sum of all groups)

E(MeanAnnualFailures) = expected mean annual number of failures

E(MeanAnnualRepairs) = expected mean annual number of repairs

E_{1A} = event 1 for sizing assessment – good sizing, no repair

E_{2A} = event 2 for sizing assessment – wrong sizing, repair

E_{3A} = event 3 for sizing assessment – wrong sizing, no repair

E_{4A} = event 4 for sizing assessment – good sizing, repair

E_{1D} = event 1 for detection – no defect, no detection

E_{2D} = event 2 for detection – no defect, detection

E_{3D} = event 3 for detection – defect, no detection

E_{4D} = event 4 for detection – defect, detection

E(# Failures) = expected mean annual number of failures in each group between inspections

E(#(Failures @ Detection)) = expected mean annual number of failures due to missed detection for each group at an inspection year

E(#(Failures @ Assessment)) = expected mean annual number of failures due to incorrect sizing assessment leading to no repair for each group at an inspection year

E(#Repairs) = expected mean annual number of repairs for each group at each inspection year

E(#1st Inspections) = expected mean annual number of first inspections for each group at each inspection year

E(#2nd Inspections) = expected mean annual number of second inspections for each group at each inspection year

FHwA = federal highway administration

FLS = fatigue limit state

FORM = first order reliability analysis

G = inverse power equation which is used to calculate the values in the upper triangular portion (growth part) of the transition matrix

GA = genetic algorithm

GGBS = ground granulated blast-furnace slag

g = deterioration kinetics parameter

ISO = international standards organisation

ISTEA = intermodal surface transportation efficiency act

i_{corr} = corrosion rate ($\mu A/cm^2$)

$i_{corr(exp)}$ = experimental corrosion rate ($\mu A/cm^2$)

$i_{corr(real)}$ = corrosion rate in a real structure ($\mu A/cm^2$)

K = constant for calculating experimental corrosion rate ($8.76x10^4$)

k_F = failure impact coefficient

k_I = inspection cost coefficient

k_R = repair cost coefficient

LCC = life-cycle cost

LVDT = linear variable displacement transducer

MC = Monte Carlo

MDP = Markov decision process

m = Weibull exponent (to calculate p_f) which determines the spread of the curve

N = total number of groups

NDT = non-destructive technique

n = positive integer

$\bar{n}$ = mean of the noise distribution

OPC = ordinary Portland cement

PDF = probability density function

PFA = probability of false alarm

P(Failure | No Assessment) = probability of failure given that no sizing assessment has been carried out (at an inspection year)

P(Failure | No Repair) = probability of failure given that no repair has been carried out (at an inspection year)

$P_{f\_allowable}$ = allowable probability of failure

PGA_{NR} = probability of a good assessment resulting in no repair

PGA_R = probability of a good assessment resulting in repair

P_{ij} = probability that a defect will move from group i to group j in one time step (also the term in the i^{th} row and the j^{th} column of the matrix)

P_{ij_GROWTH} = probability that a defect will move from group i to group j in one time step in the matrix which simulates growth only

$P_{ij\_MARKOV\_1}$ = probability that a defect will move from group i to group j in one time step in the complete Markov matrix for an inspection year

$P_{ij\_MARKOV\_2}$ = probability that a defect will move from group i to group j in one time step in the complete Markov matrix for a year between inspections

PoD = probability of detection

POT = modified trimmer potentiometer

P(Repair) = probability of repair

PWA_{NR} = probability of a wrong assessment resulting in no repair

PWA_R = probability of a wrong assessment resulting in repair

p_F = annual probability of failure

$p_{R\cup F}$ = probability of repair or failure at an inspection year

P(2nd Inspection) = probability that a second inspection is carried out for each group

Q_{ref} = reference inspection quality

Q_1 = quality of the inspection method for defect detection

Q_{1opt} = optimum inspection quality for detection

Q_2 = quality of the inspection method for sizing assessment

Q_{2opt} = optimum inspection quality for sizing

R^2 = coefficient of determination

SLS = serviceability limit state

SD = standard deviation

$\bar{s}_i$ = mean of the "signal+noise" distribution for group *i*

T = total experimental time (hours)

Ti = initiation time

$t_{(exp)}$ = experimental time to a particular crack width

$t_{(real)}$ = estimated real time to a particular crack width

ULS = ultimate limit state

USDOT = United States department of transport

W = mass loss of steel (g)

Y = year

ΔT = inspection interval in years

ΔT_{opt} = optimum inspection interval in years

α = growth rate of a defect

γ = probability that the actual defect size is greater than d_{min}

η = ratio of cost coefficient for the second inspection (k_{i2}) to the cost coefficient of the first inspection (k_{i1})

λ = probability that the size of the actual defect is greater than d_c

ρ_s = density of steel (7.86g/cm^3)

σ_d = standard deviation of the defect size in a group

σ_{NA} = standard deviation of noise distribution (for assessment)

σ_{ND} = standard deviation of noise distribution (for detection)

defects in group = stabilised number of defects in each group at an inspection year after the growth for that year but before the inspection

$(\#\,\text{defects in group})_t$ = stabilised number of defects in a group at the t^{th} year between inspections after the growth for that year but before failure

$\#d_{j@Y_k}$ = number of defects in group *j* at year Y_k

0* = start time of experimental results

CHAPTER 1 - INTRODUCTION

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1.1. INTRODUCTION

Due to the extent of deteriorating infrastructure in the US (about 5,000 bridges become classed as deficient each year), the estimated cost of rehabilitation and repair has been estimated at $1.3 trillion (Enright and Frangopol, 1999). As well as deterioration, the number of vehicles on the road infrastructure is continuously increasing, and bridge management and inspection procedures must be developed and implemented to cope with this increasing demand (Minchin et al., 2006; Liu and Frangopol, 2005b). An efficient road network is necessary to facilitate a functioning and healthy economy (Frangopol et al., 1997).

Bridges provide connectivity between places of interest, along with other areas of civil engineering infrastructure, which are essential for modern society (Liu and Frangopol, 2005b). Therefore, bridge maintenance is essential to detect deterioration, damage and loss of strength in critical members, and to maintain safety above an acceptable level. In order to efficiently plan inspections, maintenance and repair, it is essential that some form of deterioration modelling is used to predict future deterioration (Chryssanthopoulos and Sterritt, 2002; Sterritt et al., 2002). Inspections and resulting bridge maintenance can extend the lifetime of a structure and result in fewer failures (Frangopol et al., 1997). When investing in bridge maintenance, the areas of interest are remaining lifetime, load capacity, serviceability and durability, where the lifetime depends on the application of regular and appropriate maintenance. However, in most cases, financial resources for maintenance activities are inadequate to perform all maintenance that is required (Liu and Frangopol, 2005b). In addition, "The federally mandated biennial inspection interval is not the most cost-effective maintenance strategy for bridges (Soltani, 1995), and bridge repairs are not always performed with life-cycle cost effectiveness in mind" (Enright and Frangopol, 1999). Therefore, maintenance management strategies must be developed to optimise the use of restricted maintenance budgets and prioritise maintenance to obtain maximum benefits (Geier and Reiter, 2008; Liu and Frangopol, 2005b; Neves et al., 2004).

As a result, a lot of research has been conducted into the optimisation of existing infrastructural resources over the last decade to develop new bridge management systems

(BMS) which consider the dual constraints of optimisation of the available maintenance budget spend whilst maximising the efficiency of the infrastructural element/network for the required remaining service life (Lauridsen, 2007; O'Connor and Enevoldsen, 2007; Lauridsen et al., 2006; Stewart and Mullard, 2006; Kong and Frangopol, 2005; Stewart, 2005; Kong and Frangopol, 2004a; Stewart et al., 2004; Kong and Frangopol, 2003; Chryssanthopoulos and Sterritt, 2002; Faber and Sorensen, 2002; Radojicic et al., 2001; Stewart, 2001; Estes and Frangopol, 1999). The main objective of these systems is to develop a methodology to establish the optimal maintenance management plan as a function of the structural condition, thereby optimising the whole life cost of the structure. Monitoring and inspections are key aspects in this process (Corotis et al., 2005) as the information from these tests can be used to update deterioration models (thereby reducing systemic uncertainty) and to derive the optimal economic maintenance strategy for the remaining lifetime of the structure. Many of these methods rely on quantitative data from inspections, rather than qualitative and subjective data. Therefore, the main focus of this thesis is on the development of an inspection based maintenance management methodology, incorporating analysis on the effect of the cost and quality of NDT tools to assess the condition of a structure over its lifetime. The objectives and organisation of this thesis will be discussed further in Section 1.2 and Section 1.3, respectively

1.2. OBJECTIVES OF THESIS

This thesis details the development of a maintenance management model which, as part of a BMS, aims to optimise maintenance strategies by providing owners/managers of structures with a rational tool to allocate restricted maintenance budgets. More specifically, the objectives of this thesis are outlined below, and are illustrated in Figure 1.1:

1. The development of a maintenance management methodology which optimises inspection planning by taking into account the two aspects of an inspection, detection of a defect and sizing of a defect. Since each stage of an inspection is carried out for a distinct purpose, different parameters should be used to represent each procedure. This has not been previously considered in the literature. By separating these two procedures, an optimal maintenance management plan can be developed by choosing the most suitable inspection technique for each stage of the

inspection, whether it is for detection or sizing, rather than using the same inspection technique for both procedures or selecting different techniques without a rational, robust and repeatable decision basis.

2. To develop a method which simulates the deterioration, inspection, repair and failure of structures over time using Markov matrices, with the ability to consider many different forms of defect growth and deterioration kinetics (i.e. gradual and abrupt growth, linear and non-linear), allowing for different materials and environments (i.e. passive and aggressive) to be studied.
3. To carry out an experimental study to investigate the efficiency of different repair materials such as OPC, OPC+PFA and OPC+GGBS in relation to the rate of crack growth in concrete due to chloride-induced corrosion of the reinforcing bars.
4. To incorporate the ability to simulate both the initiation phase and the propagation phase of deterioration using a Markov decision process. Materials such as reinforced concrete demonstrate a two stage deterioration process, initiation and propagation. In order to model the deterioration behaviour of a material, both phases must be considered.
5. To incorporate the ability to simulate the repair of a structure using a different material, since repairs of structures are often carried out using more durable/'newer' materials than that from which the structure was originally constructed, which have different deterioration characteristics to the original construction material.
6. To demonstrate the capabilities of the developed maintenance management model using a practical example. The objective is to use the results of the experimental study (objective 3) and information from the literature to compare the efficiency of different repair materials, considering the initiation phase (from literature) and propagation phase (from experimental results) of deterioration of reinforced concrete due to chloride-induced corrosion.
7. To perform sensitivity studies in the context of the derived methodology to investigate the effect of changes in the input parameters on the resulting maintenance strategies. By performing a sensitivity analysis, it is possible to see how the input parameters, such as growth parameters, initiation phase, inspection qualities, modes of failure, limit states or repair materials, influence the overall results of the maintenance management strategy.

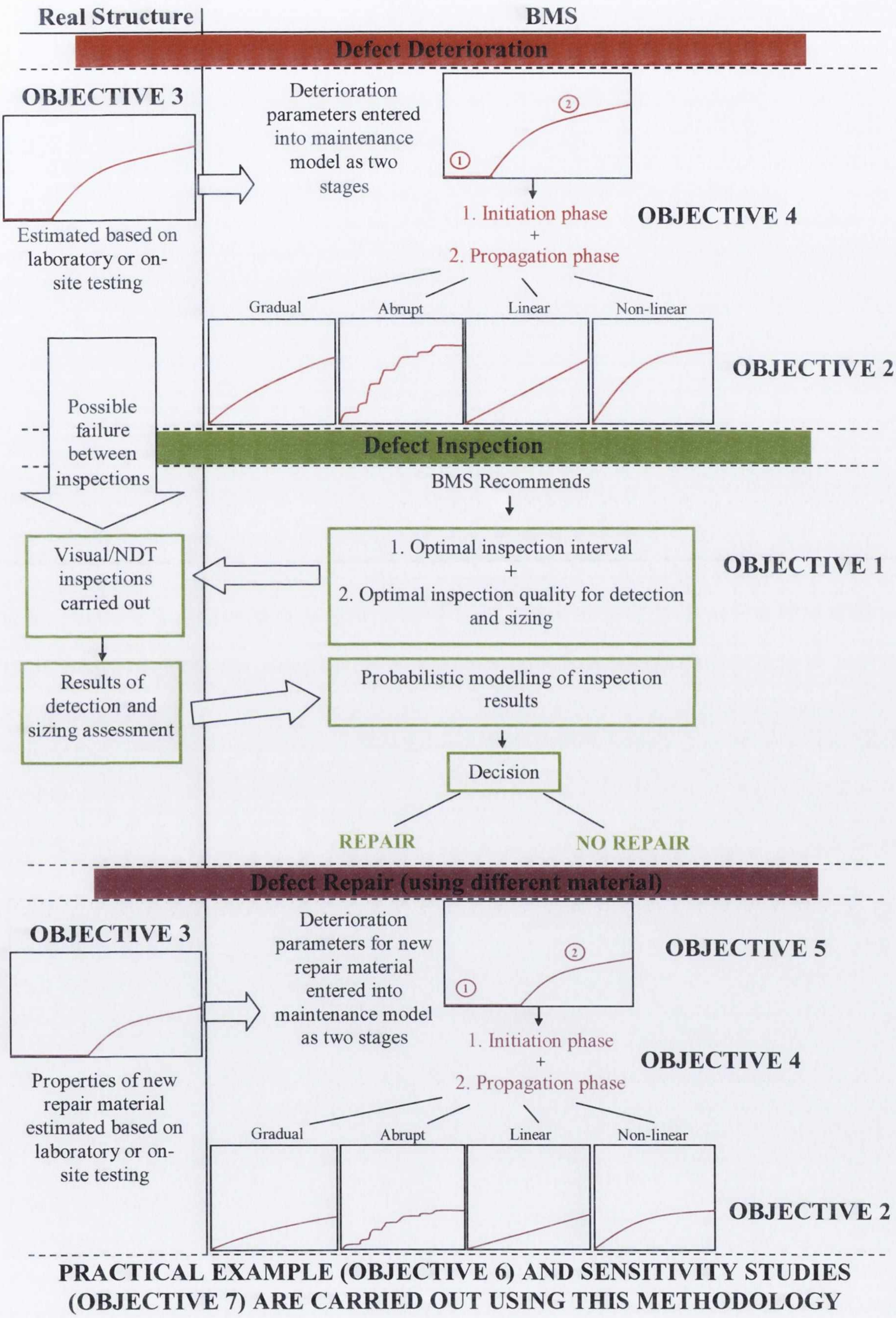


Figure 1.1. Illustration of relationship between the different objectives of this thesis

1.3. ORGANISATION OF THESIS

Chapter 1 presents an introduction to the topic of bridge maintenance and why there is a need to optimise maintenance budgets. The objectives of this work and the outline of this thesis are also presented in Chapter 1.

Chapter 2 presents a state of the art review of current research in the area of bridge management systems (BMS). The literature review also highlights specific areas of BMS which require further research. In this regard, the objectives of this thesis are given context.

Chapter 3 details the theoretical development of the maintenance management model, including probabilistic modelling of a two stage inspection process, an event based decision theory to determine the ratio of statistical correct/incorrect inspection results and deterioration modelling. Originally, the two aspects of an inspection, detection and sizings are considered here to accurately simulate the inspection process which is carried out as part of a maintenance management strategy. It is considered important and logical to distinguish between these two stages of an inspection to obtain optimal maintenance strategies, yet this has not been previously considered in the literature. This chapter also describes the development of Markov transition matrices to simulate the deterioration, inspection, repair and failure of a population of defects over time.

In Chapter 4 results are presented using the developed methodology which is outlined in Chapter 3. This chapter demonstrates how the optimum maintenance strategy is determined based on the minimisation of expected total costs. On this basis, the optimum combination of inspection techniques for both detection and sizing assessment are determined. A comparison is also made between the developed two stage inspection process and the tradition single stage inspection process, which considers detection only. In addition, the variation in the number of failures and percentage of repairs carried out due to correct/incorrect inspection results is investigated. The sensitivity of the results to changes in the input parameters, such as the growth parameters, modelling parameters, modes of failure and limit states is also considered.

Chapter 5 outlines the experimental study which is carried out as part of this thesis to compare the efficiency of different repair materials (OPC, OPC+PFA and OPC+GGBS in this case) with regard to the rate of crack propagation in reinforced concrete due to chloride-induced corrosion of the reinforcing bars. The experimental setup and procedure is outlined and the results of the experimental study (which are subsequently fed into the maintenance management model) are presented.

Chapter 6 outlines the revised maintenance model which incorporates the ability to study the efficiency of different repair materials, and the ability to model the initiation phase of deterioration and a non-linear propagation phase. This chapter brings together all aspects of the thesis by carrying out a practical example using the developed methodology and the results of the experimental study. The experimental results are used to model the propagation phase of deterioration of reinforced concrete due to chloride-induced corrosion and information is taken from the literature to model the initiation phase. In addition, sensitivity studies are also carried out to investigate the effect of the initiation time and the propagation phase of different repair materials on the results of the optimal maintenance strategy in terms of the expected cost, number of failures and optimum combination of inspection techniques.

Chapter 7 concludes the thesis by outlining how the objectives which are outlined in Chapter 1 have been achieved. In addition, future work to further develop the maintenance management methodology which was developed in this thesis is also recommended.

CHAPTER 2 - LITERATURE REVIEW

CHAPTER 2 - LITERATURE REVIEW

2.1. INTRODUCTION

As bridges approach an end to their design lives, and volumes of traffic continue to increase, bridge deck maintenance has become a big challenge for governments around the world who are managing ageing infrastructure (Scott et al., 2003). “A BMS is defined as a rational and systematic approach to organising and carrying out all the activities related to managing a network of bridges” (Scherer and Glagola, 1994). Two of the main objectives of BMS are the prediction of the future state of structures, and the prioritisation of maintenance and rehabilitation. It is used to optimise the inspection, maintenance and rehabilitation of a network of structures with respect to minimising the total cost, and achieve the best performance from the network of structures (Minchin et al., 2006; Rens et al., 2005). Czepiel (1995) also emphasises that it is necessary to optimise the maintenance of a network of bridges by ensuring that components are repaired before the structure reaches an unsafe condition, and to minimise the whole life cost of the structure.

In a life cycle costing analysis, the decisions made at some point in time affect the decisions which will be taken in the future. Therefore owners/managers of structures must not only make a decision based on budgets available now, but must also consider the consequences and costs of these decisions in the future. Therefore, management systems are developed to provide them with rational tools to use when choosing maintenance interventions, which are based on minimising the life cycle cost rather than just a point in time cost of maintenance (Corotis et al., 2005). However, for engineers to have confidence in a BMS, it must be able to reliably and uniformly rank and prioritise the inspections, maintenance and repairs in accordance to what is needed within a bridge network in addition to predicting when it is needed (Gattulli and Chiaramonte, 2005).

The need to monitor bridges became apparent after the spectacular collapse of the Point Pleasant Bridge in the US in 1967 (Onoufriou and Frangopol, 2002; Frangopol et al., 2001; Czepiel, 1995). In 1968 the Federal Highway Act was passed ordering all states in the US to monitor and record the condition of their bridges according to the National Bridge Inspection Program and in 1991 the Intermodal Surface Transportation Efficiency Act (ISTEA) was passed, which stated that all states in the US must adopt a BMS (Frangopol et al., 2001; Czepiel, 1995). The guidelines accompanying this act and the AASHTO Guidelines for Bridge Management Systems both identify 4 distinct features of a BMS. These are (i) data archive (information from inspections), (ii) models for deterioration and cost analysis, (iii) optimisation of maintenance strategies (on the basis of minimising the whole life costs for a specified serviceability level) and (iv) updating of models using new information (Czepiel, 1995). In 1995, The Federal Highway Administration (FHwA), under the USDOT, published guidelines for the inspection and management of bridges in the US (The National Bridge Inspection Standards). In this document there are requirements laid out for the frequency of inspections, the required qualifications of personnel managing the bridge inspection/inventory and requirements for the inspection report and bridge inventory (Minchin et al., 2006).

Since then, BMS have been developed and are currently being used to manage infrastructure maintenance in many countries (e.g. US, Mexico, Japan, South Africa, France, Denmark, Italy, Ireland), and there is ongoing research in this area to improve these methods and provide rational tools for the management of ageing structures considering uncertainties associated with deterioration, inspection and repair processes. A review of current developments in the area of BMS is presented in this chapter.

To facilitate practical demonstration as to how the theoretical methodology which is developed in this thesis may be employed as part of a BMS, it was necessary to perform an experimental study to investigate the rate of deterioration in reinforced concrete due to chloride-induced corrosion. On this basis, some of the factors affecting the rate of deterioration in reinforced concrete are also discussed in this chapter.

2.2. CURRENT DEVELOPMENTS OF BMS

When developing a new framework for maintenance management of structures, it is important to consider not only current practice in the area, but also new developments that have been achieved. This informs researchers of areas where further research needs to be conducted. Therefore, to determine the current needs in relation to maintenance management, a literature review of past and present studies was carried out. In this regard, the objectives for this thesis were outlined based on the current needs in the area and on recommendations of further work which was suggested in the literature by other researchers.

For the purpose of this literature review, the proposed management systems have been categorised into four headings, based on the main development in each of the studies. These headings are (i) Markovian based maintenance management, (ii) reliability based maintenance management, (iii) cost-benefit based maintenance management and (iv) inspection modelling within a BMS. It is recognised that some of the reviewed studies may relate to two or more of these headings, but in this case they are categorised based on the main methods and developments described in the publication. Again, by highlighting areas where further research is required within each of these categories, the main objectives of this thesis were defined.

2.2.1. Markovian Based Maintenance Management

Markovian based maintenance management is used in practice as part of existing BMS such as Pontis and BRIDGIT (Bakht and Mutsuyoshi, 2005; Rens et al., 2005; Adey et al., 2003; Frangopol et al., 2001; Czepiel, 1995). As well as these existing BMS, research is continuing in this area to further develop these Markov Decision Process (MDP) models. According to Scherer and Glagola (1994), "MDPs are a powerful and useful technique for bridge management systems". A Markovian model can be used to predict the deterioration of a structure from one condition state to another over time (Rens et al., 2005; Roelfstra et al., 2004; Frangopol et al., 2001). With the development of Markov chains (Markov transition matrices) predictions can be made on the level of future deterioration of structures. Given the control variables (e.g. a repair threshold), and the costs associated with different actions, an objective function can be developed to optimise maintenance (Scherer and Glagola, 1994).

A Markov process has the property that the probability of an event occurring (i.e. moving to the next condition state) is independent of past states and only depends on the condition state it is in at the present point in time and the action applied to it (Adey et al., 2003; Scherer and Glagola, 1994; Cesare et al., 1992; Ang and Tang, 1975). This is known as the Markovian property. Also, using a Markov process, the deterioration is modelled as discrete condition states rather than a continuous deterioration process (Adey et al., 2003). Although the condition of the structure is recorded using discrete states and at discrete times using a Markovian based methodology (unlike a reliability based approach), the physical process of deterioration of a structure is relatively stable, and inspections and maintenance are carried out at discrete times, therefore, the state of a structure is only required at these discrete times when decisions are made (Corotis et al., 2005).

In a study by Cesare et al. (1992), transition matrices were developed using a database of deterioration information for 850 bridges in New York State to simulate deterioration of the overall condition of bridge structures. Based on this information transition matrices were developed for the individual components of a bridge. Using these transition matrices, the expected condition rating for an individual structure and the average condition rating for a network of structures was computed. This study used 7 condition states to model the deterioration of bridge structures, based on the 7 condition states which are used by the New York State Department of Transport, where condition state 7 represents the 'as new' structure, and condition state 1 represents a potentially hazardous bridge. The historical data used in this study indicated that a bridge element does not deteriorate by more than one state in a two year period (inspections are carried out every two years). Therefore, it was assumed in this study that a bridge element either stays in the same state or deteriorates to the next lowest state in a one year period. This is illustrated in Figure 2.1. The probability of deteriorating to the next condition state is 1.0 minus the probability of staying in the same condition state (T_{ii}) over a one year period. To compare the deterioration rate of different structures in this study, the probability that the condition rating was less than a threshold state after a certain period of time was computed, or the expected value of condition rating over time was plotted graphically.

$$\mathbf{T} = \begin{bmatrix} T_{77} & 1 - T_{77} & 0 & 0 & 0 & 0 & 0 \\ 0 & T_{66} & 1 - T_{66} & 0 & 0 & 0 & 0 \\ 0 & 0 & T_{55} & 1 - T_{55} & 0 & 0 & 0 \\ 0 & 0 & 0 & T_{44} & 1 - T_{44} & 0 & 0 \\ 0 & 0 & 0 & 0 & T_{33} & 1 - T_{33} & 0 \\ 0 & 0 & 0 & 0 & 0 & T_{22} & 1 - T_{22} \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

Figure 2.1. Transition matrix to predict future deterioration (Cesare et al., 1992)

Correlated components within a bridge were also considered. If elements within a structure were correlated a non-stationary Markov transition matrix was used. In the example presented, it was assumed that the condition of one element (A) depended on the state of another element (B). This was simplified by defining the state of element B, in terms of its functionality, as good (above a threshold condition rating) or bad (below a threshold condition rating). Element A had a different Markov matrix depending on the functionality of element B (good or bad). In addition, the repair of bridges was also considered, assuming that a certain percentage of bridges below a threshold condition rating were repaired to the 'as new' condition (i.e. to condition state 7). This paper studied the effect of the percentage repair (10% or 20% of the structures below the threshold condition rating) on the overall condition rating of a network of bridges, and was used to find the percentage repair needed to keep the bridge stock at the same average condition rating over time.

In relation to costs for an individual bridge, each condition rating was assigned a maintenance cost. Using this and the number of elements in each condition state, the total expected cost was determined by summing the cost of each element. It was recognised by the authors that to carry out a risk-based analysis, the importance of each element must be considered (possibly using a series-parallel system), although it was not discussed any further in this paper. In relation to repair, uncertainty was considered by introducing a 90% confidence that they were repaired to the 'as new' condition (while the other 10% were repaired to condition state 6). Using this approach, the total expected cost of an element was determined. It was stated that the optimum repair strategy could be optimised by finding the minimum expected total cost (or risk), over a period of time, although optimisation methods were not detailed in this paper.

Following this, in 1994 a paper was published by Scherer and Glagola studying the effectiveness of Markov models to be used as part of BMS. In this paper it was demonstrated that Markov models could be used where uncertainty was present, to predict the deterioration of a network of bridges over time. To develop a Markov chain, states first have to be developed which can clearly describe the condition of the system, and decisions can be made based on the state a structure is in. The transition states of a Markov chain represent the probability of moving between condition states due to deterioration or maintenance actions. In this study the states were chosen in accordance with the Virginia

Department of Transport. This consists of 7 condition states ranging from 9 to 3, where 9 represents the 'as new' condition and 3 represents a poor condition.

Rather than defining a vector which records the state of all bridges in the network being considered (which would result in a very large number of computations, namely 7^N, where N is the number of bridges), a record was kept of the number of bridges in each state. To further reduce the number of possible computations, the bridges were classified and grouped (according to bridge type, age, environment, loading etc.). On this basis, a separate Markov model and maintenance management policy was determined for each class of bridge. It was assumed that bridges within a class would deteriorate at the same rate, given the same maintenance interventions.

In this case, transition matrices for each class were developed using information on the actual number of bridges moving between condition states in Virginia. The probability distribution for each row in the matrix was developed. Given the initial condition state of a structure, the row in the transition matrix can indicate the possible future condition states. Poisson probability mass functions were fitted to the historical data and were used to model the probability that a bridge moves from one condition state to the next over a specific time period. The developed Markov matrices were then used to probabilistically predict deterioration over time, providing a range of possible condition states for a bridge at any given time, with an associated probability. In addition, the authors carried out an important study to validate the Markovian property for this case (using data from Virginia). The results of the study indicated that there were no significant differences in the probabilities obtained whether past data was considered or not (thereby verifying the Markovian property).

The Markov based maintenance methodology was further developed by Jiang et al. (2000) and Corotis et al. (2005). These studies detailed the development of a model for the optimisation of maintenance policies using a partially observable Markov decision process, which take into account the uncertainties associated with inspection techniques. Therefore, as well as determining the optimum inspection interval, this method could also be used to determine the type of single stage inspection process to be used and the extent of the repair to be carried out. However, no experimental studies were carried out to calibrate the repair parameters and different repair options.

Different forms of structural deterioration such as fatigue and corrosion were considered. Deterioration parameters were modelled as random variables. The state of a structure could be estimated based on deterioration models (from historical information) or from observations and inspection results. The authors acknowledged the different sources of uncertainties which are associated with inspections, such as variations in the skill of the operator, environmental conditions, inspection instruments and even on the interpretation of the inspection results. These uncertainties were taken into account in this study using a partially observable Markov decision process.

It was assumed that inspections were carried out to estimate the deterioration of a component or the condition state of a component at regular intervals. The condition of the structure was recorded in one of 5 predefined condition states. Since the results from inspection techniques are uncertain, the present state of a structure or structural component cannot be known with certainty. On this basis, a partially observable Markov decision process was developed to take into account the uncertainty associated with inspections due to incomplete or imperfect information from NDTs or visual inspections. Using additive and multiplicative bias parameters and a random error parameter associated with the inspection technique, a relationship was developed between the true state of a structure and the results of the inspection of the same component. The probability density function (PDF) of the true state of the structure could be predicted using deterioration models, and since the properties of the single stage inspection technique were known, the PDF of the inspection result could be estimated.

There is also uncertainty associated with each type of maintenance intervention, due to variability in materials, environment, equipment, workmanship skill etc. To take this into account, the probability that a maintenance intervention improves the condition state of a structure from one state to another was determined and used to develop the transition matrix describing the effect of a particular maintenance intervention. In this case, the maintenance intervention did not have to restore the component to the original condition. A joint transition matrix was calculated using the transition matrix describing the deterioration and the transition matrix describing the maintenance interventions. It was assumed in this study that Markov processes are stationary.

Due to uncertainties associated with inspections and maintenance actions, the true state of the system cannot be known with certainty. Therefore, the probability of a structure being in a condition state was determined. The process was broken into time steps, and at each time step an inspection and a maintenance intervention could take place. When an inspection was carried out, the observed state of the structure was determined, although this may not have been the same as the true state of the structure. At each time step, the information known about the structure was stored, such as maintenance carried out and inspection results. Using this information and the relation between the true state of the structure and the observed state of the structure, the transition matrix of the true state of the structure could be estimated. Bayesian updating was used after each inspection result to update the information on the true state of the structure. At each time step, the decision maker could choose the maintenance action to be taken at the time, and the inspection to be carried out at the next time step.

The cost model used in this study was based on micro-economic concepts of production consisting of a fixed and variable component. Using this form of cost model, the quality of the inspection technique and maintenance intervention were taken into account (using the variable component of the cost model). In addition, failure costs and penalty costs (which could include costs such as user delay) were included in the analysis. Optimisation of the maintenance strategy was based on the minimisation of the life cycle costs of the structure, considering inspection, maintenance and reduced service penalty costs.

A study modelling the deterioration of concrete pipes due to reinforced concrete corrosion using a Markov process was published by Adey et al. (2003). The optimal repair strategy was determined by considering the minimisation of both risks and costs. Deterioration was modelled using 5 condition states, where condition state 1 represented the 'as new' structure, and condition state 5 represented failure. Figure 2.2 illustrates the possible transition of components between different condition states due to deterioration or repair (considering 5 condition states).

Failure was assumed to occur if the deterioration of a component was in condition state 5. Four repair options were considered, one for each condition state (i.e. one was to replace pipes in condition state 4, another was to replace the pipes in condition state 3 etc.). In this study, repaired or replaced pipes were assumed to return to the initial condition state.

Optimisation was performed by minimising the sum of the agency cost of repairs and the failure (risk) costs.

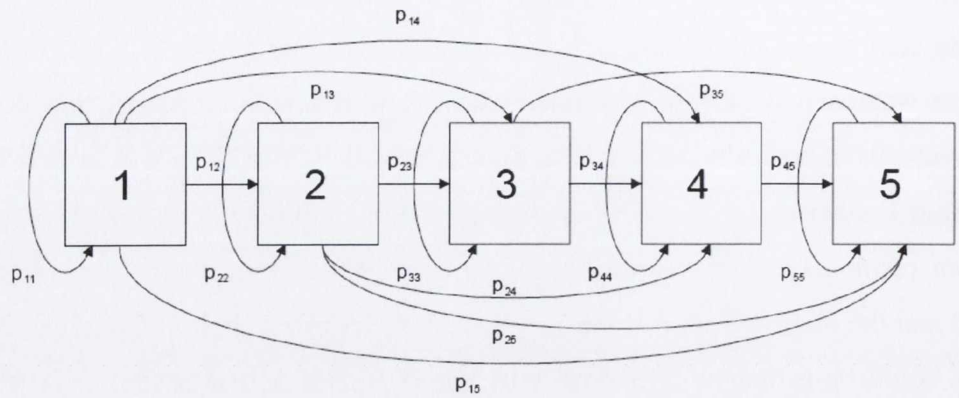


Figure 2.2. Illustration of the possible transition of components between condition states (Adey et al., 2003)

Roelfstra et al. (2004) published an alternative approach to Markovian based maintenance management, which proposed an improvement to the BMS that is currently in place in Switzerland. The Swiss bridge management system, KUBA-MS, was developed in 1998. Under this system, the structural assessment of a bridge is performed on an elemental level. Currently a Markov model is used to predict the deterioration of a structure from one condition state to another over time. Five condition states are defined, and the transition probabilities are estimated based on previous information from inspections, and the change in condition states between successive inspections. During a visual inspection, the elements are assigned one of the 5 condition states (from 1-5) based on the observed level of deterioration, where condition state 1 represents a good condition and condition state 5 represents an alarming condition. These condition states are presented in Table 2.1. Based on the condition of elements in the structure, maintenance interventions are recommended, with associated costs necessary to carry out these maintenance operations.

However, there is not much information available from inspections of condition states 4 and 5, which are the two worst states, and therefore the Markov transition probabilities that are calculated on this basis may not be very reliable. In addition, it was recognised that the condition states are based on visual observations of damage, and therefore are not directly related to the safety of the structure. It is assumed that in order to ensure safety of the structure, all elements must be repaired before they reach condition state 5 (even if the element is redundant). For large elements, they are broken into segments according to homogeneous environmental conditions and loading. Therefore, it is appropriate to assume uniform deterioration over the element or segment.

Condition state		Description
1	Good	No visible damage; only thin superficial cracks; no signs of corrosion
2	Acceptable	Visible spots of rust and/or local spalling; thin cracks due to corrosion of the reinforcement and/or humid zones; insignificant mechanical damage
3	Damaged	Spalling with visible reinforcement, insignificant loss of section, less than 10% than visible reinforcement; cracks and/or humid zones
4	Bad condition	Spalling with visible reinforcement, significant loss of section, more than 10% of reinforcement visible; cracks and/or humid zones
5	Alarming	The structure is in danger, measures are necessary before next principal inspection; immediate measures

Table 2.1. Condition states defined by KUBA-MS (Roelfstra et al., 2004)

Roelfstra et al. proposed another approach to calculate the transition probabilities between condition states for the Markov process, using the physical mechanisms which cause the deterioration. In this study, deterioration due to the chloride-induced corrosion of reinforcing bars in concrete was considered. The physical process of deterioration caused by chloride-induced corrosion of reinforced concrete is well known. The model developed by the authors to describe this deterioration incorporated chloride penetration, initiation and a simplified model for propagation. Using this model, simulations were carried out to predict the deterioration of a structure. The transition probabilities for the Markov matrix were then calibrated to fit this data. However, the simulated results using prediction models differ from the results using the corresponding Markov chains, as the initiation period was not considered in the Markov process. A recommendation of this paper was that the Markov chain models need to be improved to take initiation time into consideration. Therefore, the incorporation of an initiation phase into the deterioration modelling of structures using Markov chain models is developed as part of this thesis.

Another problem with KUBA-MS is that the parameters that are used to model the physical process due to deterioration, such as chloride-induced corrosion, cannot be determined from available onsite inspection techniques (in Switzerland). For this reason, a deterioration model of this form cannot be automatically incorporated into the BMS model. Therefore, a model was developed by the authors that takes information from visual inspections and relates this to the level of deterioration in the structure due to physical processes such as reinforced concrete corrosion.

In this study, a relationship was developed between the level of reinforcement corrosion and the actual visual appearance of the element (which is recorded during a visual inspection according to the 5 condition states defined by KUBA-MS), based on the expected level of visual deterioration for different reinforcement section losses. The 5 condition states which are defined as part of the KUBA-MS system are presented in Table 2.1. The corresponding states which were defined in this study to relate the deterioration due to loss in reinforcement to the condition states according to KUBA-MS, are presented in Table 2.2, although the level of uncertainty associated with this prediction was not discussed in this publication.

Transition	Condition state
CS 1	<0.2% free Cl^{-}/mass of cement
CS 2	<50 μm of reinforcement radius loss
CS 3	<10% of reinforcement section loss
CS 4	<25% of reinforcement section loss
CS 5	>25% of reinforcement section loss

Table 2.2. Condition states developed in this study, which can be mapped to KUBA-MS condition states (Roelfstra et al., 2004)

Orcesi and Cremona published a study in 2006 examining the deterioration of the French reinforced concrete bridge stock. The bridge stock in France is assessed using the IQOA programme, which provides an assessment each year of the condition of the overall bridge network. This system provides data to develop probabilistic deterioration models for a network of bridges. According to IQOA, bridges are assigned one of 5 overall condition ratings. These 5 condition states are presented in Table 2.3. When carrying out an inspection of a bridge element, the defects are noted and classified and a condition rating is assigned to the overall structure based on the worst defect present in each element. Similar to the IQOA programme, in this study bridge data were stored according to their age and their condition state.

State	Meaning
1	Good overall state
2	Equipment failures or minor structure damage. Non urgent maintenance needed.
2e	Equipment failures or minor structure damage. Urgent maintenance needed.
3	Structure deterioration. Non urgent maintenance needed.
3u	Serious structure deterioration. Urgent maintenance needed.

Table 2.3. Condition states defined according to IQOA (Orcesi and Cremona, 2006)

Using this information Markov chains were developed in this publication to assess the probability that the bridge would deteriorate from one state to another over time. It was assumed that Markov chains are time independent. An optimisation procedure was used to find the Markov transition matrix from an historical database. Repair probabilities were added to the Markov transition matrix in accordance with maintenance procedures carried out by the French Highway Agency. With the Markov matrix developed describing how the condition of real bridges evolve over time, the aim of this study was to optimise the maintenance strategies in relation to minimising costs while keeping the bridges serviceable. Binary transition matrices were developed representing different maintenance alternatives. Figure 2.3 illustrates the matrix representing the formulation of one of the maintenance strategies. The original transition matrix which described the evolution of real bridges (which was developed using information from IQOA) was altered to describe deterioration only. This matrix was then multiplied by the maintenance matrix, to investigate the effect of alternative maintenance strategies.

The state of the bridge stock each year was determined by summing up the bridges in each of the condition states for each maintenance strategy. The optimal combination of maintenance techniques each year was found by minimising the costs of maintenance strategies each year while maintaining the condition state of the bridge stock above a certain threshold level. Alternatively, the budget could be fixed and the maintenance plans could be optimised using this as a constraint.

$$S_3 = \begin{pmatrix} 1 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 \\ 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 \\ 1 & 0 & 0 & 0 & 0 \end{pmatrix}$$

The bridge moves from 2e to 1
(equipment maintenance)

The bridge moves from 3u to 1
(demolition - rebuilding)

Figure 2.3. Example of a binary maintenance matrix used to assess the effect of alternative maintenance strategies (Orcesi and Cremona, 2006)

The literature discussed above has demonstrated that many advances have been made in recent years in the development of Markovian based maintenance management models. Although there are limitations associated with a Markovian based maintenance management system (e.g. discrete condition states at discrete points in time), it does provide a rational framework for the prediction of deterioration and maintenance of a network of structures over time. In practice, many BMS use Markov chains to predict future deterioration and repair (Bakht and Mutsuyoshi, 2005; Rens et al., 2005; Adey et al., 2003; Frangopol et al., 2001; Czepiel, 1995). It is therefore useful to continue this progress in relation to the development of Markovian based maintenance management, which can lead to an improvement in BMS which are already in place. The reviewed studies demonstrate how uncertainties due to deterioration properties (Scherer and Glagola, 1994) and single stage inspection information (Corotis et al., 2005; Jiang et al., 2000) can be incorporated, how the issue of correlated components can be addressed (Cesare et al., 1992), how transition matrices can be developed based on historical data (Orcesi and Cremona, 2006; Roelfstra et al., 2004; Scherer and Glagola, 1994; Cesare et al., 1992) and how they can be updated using inspection information (Corotis et al., 2005; Roelfstra et al., 2004; Jiang et al., 2000). However, it was recommended that Markov chain models be further improved to take initiation time into consideration (Roelfstra et al., 2004). On this basis, addressing this issue is one of the aims of this thesis. In addition, it was assumed in Cesare et al. (1992) that a bridge element either stays in the same state or deteriorates to the next lowest state in a one year period. Since this deterioration process is quite limited another objective of this thesis is to develop a deterioration process whereby defects can move to any condition state over a 1 year period. The intention is to facilitate the simulation of many forms of deterioration within a maintenance management model (e.g. gradual to very abrupt deterioration). In addition, since the discretisation of the condition of a structure into only 4 or 5 condition states is seen as a limitation of Markovian based management systems (Kong and Frangopol, 2005), in this thesis the deterioration of a structure is broken into 10 groups or states to minimise the inaccuracy caused by discretisation. The sensitivity of the results of the maintenance model (i.e. optimum inspection interval) to the selected number of groups is also studied.

2.2.2. Reliability Based Optimisation of Maintenance Interventions

Reliability based methods of optimising the maintenance and repair interventions of bridges and networks of bridges began in the 1990s. "In simple terms, the reliability of a structure or a system is the probability of achieving a particular performance level" (Sterritt et al., 2002). It was recognised that the simulation of the deterioration and repair of structures using reliability methods has advantages over other methods such as Markov processes. Markov processes can have limitations. For example, much of the information used to develop the transition matrices are based on subjective data from inspections (Kong and Frangopol, 2004b). Also, in most BMS that use Markov chains (including BRIDGIT and Pontis), only four or five condition states are used to describe the condition of a structure. This can be quite limiting (Kong and Frangopol, 2005).

Using a reliability based approach allows for the incorporation of uncertainties (as parameters can be modelled as random variables) and it also allows different limit states to be considered (Kong and Frangopol, 2004b; Sterritt et al., 2002). Reliability methods are used to take into account the uncertainty associated with deterioration, inspections, repair, and by quantifying this uncertainty, decisions on repair optimisation can be taken with greater confidence (Estes and Frangopol, 2001). The advantage of a reliability based method is that many different factors can be considered which affect the optimal maintenance strategy and life-cycle costs of a structure, for example, different failure modes, loading and deterioration, the cost-reliability interaction of different maintenance interventions, minimising cost and maintaining reliability above a certain level (Kong and Frangopol, 2004b).

Frangopol et al. (1997) proposed a reliability based methodology for the optimal maintenance management of a structure, by minimisation of the life-cycle costs while maintaining the safety of the structure above a critical level. As well as the initial construction cost, the cost of preventative maintenance, inspection, repair and failure were considered in the life-cycle cost analysis. The inspection interval, the quality of repair, the life-cycle cost and target reliability of the structure were all taken into account in the optimisation procedure. The aim was to minimise the expected lifetime costs while ensuring that the reliability of the structure remained above a target level. The methodology considered both preventative (routine) and repair (essential) maintenance. The former was considered to be minor maintenance interventions such as cleaning, painting, repairing cracks, patch repairs, etc., whereas, essential maintenance was assumed to consist of repairs such as deck resurfacing, replacing a bearing, etc. A costing model was proposed for preventative maintenance, which assumed that the cost of preventative maintenance was linearly proportional to the age of the bridge. It was recognised that further research was needed to improve on the accuracy of this model. Routine maintenance was assumed to be carried out every 2 years and, on this basis, the service life costs of the structure were calculated.

A decision on whether to repair was based on the results of an inspection. Inspections in this case were considered to be based on NDTs for the detection of corrosion damage due to chlorides, although it was recognised by the authors that most inspections are visual. A detectability function was defined as the probability of detecting damage, given the damage intensity and was modelled as a cumulative normal distribution function. It was assumed that a high quality inspection leads to a high quality repair. For a defect less than the detection threshold of the inspection technique, it was assumed that no repair was carried out. It was also assumed that the extent of the repair was related to the detection capabilities of the inspection method.

A deterioration model was presented where corrosion propagated after an initiation phase, although it was assumed that after repair had been carried out that there was no further initiation phase. An event tree was used to combine all possible repair methods, and the cost of each method was evaluated. This process is illustrated in Figure 2.4. At each inspection there were two options. To repair (denoted 1) or not repair (denoted 0). The more inspections there were, the more branches there were in the event tree. In this paper, the probabilities of each event were calculated according to the detectability function of the inspection method chosen. The probability of failure was also calculated at each branch in the event tree.

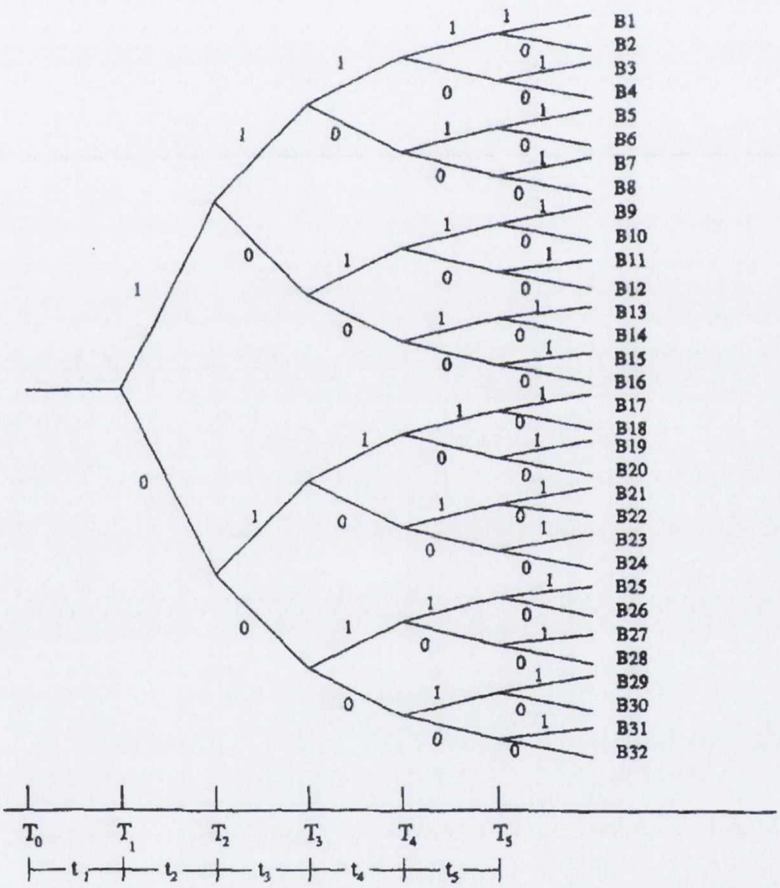


Figure 2.4. Event tree for all possible repair options (Frangopol et al., 1997)

The expected cost of each event was calculated by summing the initial cost, the preventative maintenance cost, the inspection cost, the repair cost and failure cost associated with each branch. The number of branches was 2^m, where m is the number of inspections. In the example illustrated in Figure 2.4, the number of inspections is 5 and the final number of branches in the event tree is 2^5=32. The optimisation procedure minimised the lifetime total cost, with a constraint that the probability of failure must remain below a specified allowable probability of failure, while also taking into account the inspection quality, deterioration evolution over time, the increase in structural capacity due to repair, and a range of possible repair options in an event tree. Monte Carlo simulations were carried out to calculate the probability of failure before inspections and at the end of the lifetime of a structure. Uniform and non-uniform inspection intervals were considered. It was recognised by the authors that uniform inspection intervals are easier to manage, but that lower costs may be obtained if non regular inspection intervals are considered. Additional constraints were added into the optimisation function to ensure the inspection interval was within specific bounds, and that the sum of the inspection intervals was less than the service life. As with regular inspections, the optimum number of inspections was found by carrying out different simulations with a fixed number of inspections and selecting the optimal scenario. It was recognised by the authors that this was an initial maintenance management model, and further research was needed for improvement.

An improvement of this methodology was published by Estes and Frangopol in 1999, which detailed the development of a reliability based repair optimisation procedure, considering a variety of failure modes which were combined in this study using a series-parallel system. In this case, deterioration and loading models were considered over time, although only deterioration due to corrosion of reinforced concrete was considered. The proposed method allowed for updating, based on results from visual inspections (which were assumed to be carried out every 2 years) and NDTs.

Firstly, a limit state equation (in terms of random variables) was developed for each failure mode based on the relevant parameters. The reliability was calculated in relation to each failure mode using FORM and all failure modes were combined in a series-parallel system, to compute the overall reliability of the bridge (at a point in time). The correlation between different failure modes was considered. Using deterioration models and loading models, the time dependant reliability of the structure was calculated. Different repair options were computed, each with an associated cost. For this study, repair options and costs were formulated in association with the Colorado Department of Transport. Repair was carried out when the reliability of the system fell below a specified target level.

The repairs that were considered were based on the overall reliability of the structure, rather than the reliability of the components (i.e. repairs were carried out when the overall reliability of the system fell below a specified target level). For this study, all repair options that were considered were essential maintenance (i.e. there was no minor maintenance or preventative maintenance). The effect of the repair on the increase in reliability of a specific component, and the overall structure was considered. It was noted in this paper that using this system reliability approach, a component with a reliability index lower than the target reliability may not necessarily be replaced, if it does not lead to the overall reliability of the structure to fall below the target reliability level.

Considering all repair options, the optimum repair strategy was determined by minimising the total costs over the lifetime of the structure while ensuring that the reliability of the structure remained above a target reliability.

Thus far, only the ultimate limit state had been considered when optimising maintenance interventions. It was recognised that for a maintenance management framework to be effective, both failure due to ultimate limit state conditions (structural failure due to moment, shear) and serviceability limit state conditions (serviceability failure due to cracking, spalling, deflection) must be considered. On this basis, Estes and Frangopol (2001) further developed the previous model (Estes and Frangopol, 1999) to take serviceability constraints into consideration when managing a structure. A system reliability approach was adopted again for the strength limit state (ULS) of the system. In this paper, however, the importance of serviceability (concrete cracking or spalling, painting, potholes etc.) on the optimal maintenance strategy of a structure was identified, and serviceability flags were incorporated into the system approach to take this into account. It was noted by the authors that it is difficult to incorporate serviceability failure into a system reliability approach, as the consequence of failure is much lower, so the incorporation of serviceability flags was proposed. Their purpose was to override the system reliability approach (ULS), when issues other than strength limit state were addressed. Based on these serviceability needs, repairs were carried out when necessary, regardless of whether they were needed to increase the overall reliability of the system. The decision on when to carry out these repairs was based on engineering judgement or historical inspection data and developed deterioration models.

According to the Federal Highways Agency (FHwA) in the US, bridges must be inspected every 2 years, and each element is assigned one of 10 of the National Inventory Condition Ratings, as detailed in Table 2.4. Based on this data, deterioration models are being developed for different bridge elements, and many are expressed as a linear reduction in condition state over time. For this study, these models were used to estimate the deterioration rate for the elements associated with the serviceability flags. Markov chains were used to describe the transition of an element from one condition state to another over time. Using this formulation, an element could only move one condition rating in a year (i.e. the probability of staying in CR 9 was some probability, and the probability of moving to CR 8 was 1 minus the probability of staying in CR 9). These transition probabilities were used to simulate the serviceability deterioration of an element over time, which were used to decide when serviceability flags should be inserted. These could be updated over time using information from inspections. All repair options were optimised similar to the previous studies carried out by the authors (in Estes and Frangopol, 1999).

NBI rating	Description	Repair action
9	Excellent condition	None
8	Very good condition	None
7	Good condition	Minor maintenance
6	Satisfactory condition	Major maintenance
5	Fair condition	Minor repair
4	Poor condition	Major repair
3	Serious condition	Rehabilitate
2	Critical condition	Replace
1	Imminent failure condition	Close bridge and evacuate
0	Failed condition	Beyond corrective action

Table 2.4. National Bridge Inventory Condition Ratings (Estes and Frangopol, 2001)

In 2004, a paper was published by Stewart et al. describing the implementation of this methodology to a specific reinforced concrete example. For this specific example, the effect of the chosen limit state on the optimum maintenance strategy and the resulting life cycle costs was demonstrated. The purpose of the study was to investigate the importance of each limit state and determine which should be used. Monte Carlo simulations were carried out to find expected replacement times for a reinforced concrete bridge deck. The SLS and ULS were considered, and it was assumed that a violation of either limit state resulted in replacement of the bridge deck. The results of this study indicated that in this case, when both limit states were considered, the SLS was the critical limit state, resulting in more frequent replacements than for ULS. It demonstrated that SLS must be considered or the analysis would lead to an underestimation of costs. Although inspections were not considered in this methodology, it was recognised that inspections are vital to verify or update model predictions of deterioration.

A new methodology of maintenance optimisation was presented by Kong and Frangopol (2003), which improved on previous studies. Within this methodology, a new procedure for estimating the effect of maintenance interventions on the reliability index of a structure was introduced. In this study deterioration was not considered, just the effect of maintenance on the reliability profile. Using superposition, the effect of maintenance actions on the reliability profile was added to the deteriorating reliability profile to produce the reliability profile of the system over time. Figure 2.5 illustrates this method, where the reliability index profile for a particular failure mode was presented with and without a maintenance intervention. The associated cost of each maintenance action was also computed.

The proposed methodology used a computer program developed in the University of Colorado (Life-Cycle Analysis of Deteriorating Structures, LCADS) which calculates the time dependent reliability of a structure. The initial reliability index profile of a structure (or small group of similar structures) with no maintenance was first inputted into the program. The effect of different maintenance interventions on the reliability index profile was also inputted. Any action which changes the reliability profile could be inputted (such as inspection and repair). These changes in reliability profile could be based on experimental data or inspection results. Using Monte Carlo simulations, the purpose of this program was to compare the effect of different maintenance strategies, and find an optimal maintenance strategy subject to constraints. A maintenance intervention was carried out if the reliability index fell to or below a certain threshold level. This study also took into account the cost of different maintenance interventions and used life-cycle cost analysis to optimise maintenance.

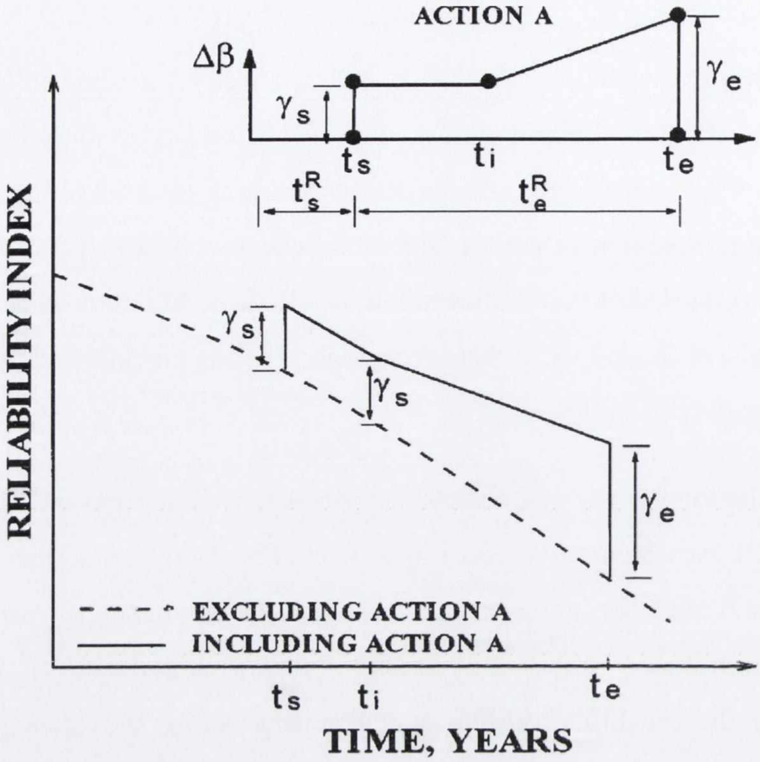


Figure 2.5. Effect of maintenance intervention on reliability profile (Kong and Frangopol, 2003)

Preventative and essential maintenance were considered in this study. Maintenance actions could be time controlled (for example, preventative maintenance such as painting may be carried out after a set number of years) or event controlled (essential maintenance may be carried out due to the reliability of a component or the whole system falling below a target reliability level). These actions could be carried out once or cyclically. The reliability index profile for a given failure mode was assessed using the inputted initial reliability profile for the failure mode plus the effect of any actions on the reliability index profile associated with the failure mode. The probability of the reliability index falling below a target level was estimated using Monte Carlo simulations.

The inspection, maintenance and rehabilitation costs were assumed to be associated with the reliability profile of a structure over time. In relation to a maintenance interaction, the cost of maintenance was assumed to depend on the condition of the structure when the action was applied, the change in reliability index due to the maintenance intervention, and the duration of the effect of the maintenance action. The user delay cost (or cost due to loss in service time during inspections and maintenance) was also considered.

It was recognised that the optimum maintenance scenario chosen by BMS is not always within budgetary constraints, and that the maintenance scenario may have to be altered based on this. In this study the present value of the expected cost over the lifetime of the structure was evaluated based on stated objective functions, constraints and design variables. In the examples which were presented, it was assumed that the reliability index returned to the initial reliability index when essential maintenance was carried out.

More detailed cost models to be included as part of this methodology were proposed by Neves et al. (2004). Since there is little information provided on the relation between cost of maintenance actions and their effectiveness, the authors proposed a new method which related the cost of a maintenance action to the increase in the reliability index. Any action which increased the reliability of the structure (by γ), reduced the rate of deterioration of a structure (by δ) for a period of time (t_{pd}) or delayed the initiation of the deterioration process for a period of time (t_p) was considered a maintenance intervention. The effect of different maintenance interventions on the reliability profile of a deteriorating structure is illustrated in Figure 2.6.

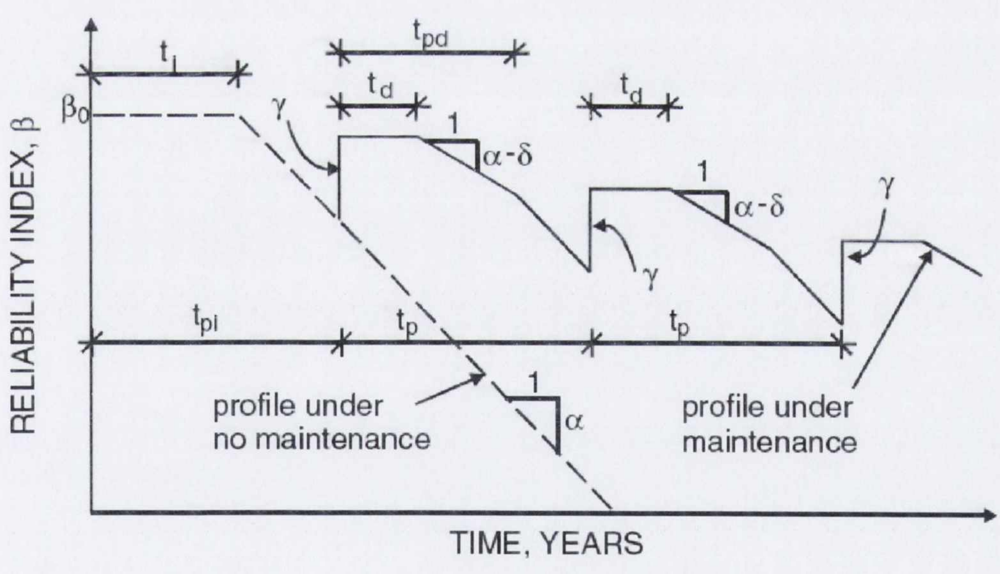


Figure 2.6. The effect of maintenance interventions on the reliability profile of a deteriorating structure (Neves et al., 2004)

In this study, the cost of a maintenance action depended not only on the maintenance being performed but also on the state of the structure before and after the maintenance action (in relation to any of the three options mentioned above). In addition, it was assumed that the cost was a fixed cost plus a cost associated with reducing the deterioration rate plus a cost associated with increasing the reliability. Sensitivity studies were carried out to demonstrate the sensitivity of the total cost to variations in the chosen parameters. It was recognised by the authors that further work needed to be performed in quantifying the parameters used to relate the costs and reliability improvements of different maintenance interventions for real structures.

Further cost models were provided in Kong and Frangopol (2004a), where the cost of a maintenance intervention was related to the quality of the intervention. According to the authors, using cost functions like these allowed for more flexibility in the optimisation of maintenance interventions using life-cycle cost analysis. As well as the direct maintenance costs associated with an intervention, this study also considered indirect costs, such as the cost of user delay due to traffic disruption. It was recognised by the authors that there is more uncertainty associated with the cost of user delay as it is more difficult to quantify, therefore it is important to account for uncertainties in the analysis.

The maintenance cost in this case was related to the reliability of the structure. Similar to Kong and Frangopol (2003), the cost of a maintenance intervention was a function of the reliability of the structure before the maintenance intervention, the increase in reliability due to the maintenance, and/or the reduction in deterioration rate when the maintenance action was applied. Improved cost functions for each of these scenarios were presented. It was also noted that other issues could be addressed as part of the cost functions, that were not considered in this study, such as the detectability of the damage, and the probability of detection (similar to Frangopol et al., 1997).

Using this methodology, the reliability of a component or the whole structure could be considered. In this case the effect of the maintenance intervention on the whole structure was the main focus. When considering the cost of maintenance, which resulted in an increase in the reliability of the structure or a reduction in deterioration rate, there were other parameters included in the cost function that could take into account the importance of structural members. This provided flexibility to develop cost models for many different situations. This study looked at the effect of these different parameters associated with the increase in the reliability index ($\Delta\beta$) on the cost of maintenance interventions.

Within each cost function there was a fixed cost and variable cost. Preventative and essential maintenance were both considered. It was assumed that preventative maintenance caused an increase in reliability of less than 0.5 and essential maintenance caused an increase in reliability index of greater than 0.5. The ratio of fixed cost of maintenance to total cost of maintenance was assumed higher for preventative maintenance than essential maintenance. It was also assumed that the cost of increasing the reliability index by $\Delta\beta$ (or reducing the deterioration rate by $\Delta\alpha$) also depended on the initial reliability index of the structure before the maintenance intervention (i.e. the cost of increasing β from 2-3 would be less expensive than increasing it from 4-5). Therefore, an additional multiplication factor was added to the cost function to take this into account. Using the multiplier function it was possible to define a range where a maintenance intervention was effective. For this study, the parameters chosen for the cost functions were based on engineering judgement. Using the developed LCADS software (discussed previously in Kong and Frangopol, 2003), the optimum mean application time of each maintenance intervention was determined. The authors recommended further work to develop cost functions that were not based on expert judgement (to reduce uncertainties), and to include the effect of inspections on the life cycle cost analysis. On this basis, a rational approach is proposed as part of this thesis to incorporate inspection results into a maintenance management model.

Kong and Frangopol (2004b) used Monte Carlo simulations to demonstrate the developed methodology using the proposed models (in Kong and Frangopol, 2004a; Kong and Frangopol, 2003) for preventative and essential maintenance, including the effect of uncertainties. Also, the reduction in the time-variant reliability of a structure due to increased loads and deterioration were considered simultaneously.

This methodology was further developed by Kong and Frangopol (2005), by introducing new optimisation and parametric analysis tools. The proposed methodology optimised the time for preventative maintenance and the time to essential maintenance in relation to minimisation of life-cycle costs. Alternatively, essential maintenance could be reliability controlled. The optimisation procedure was controlled by the expected total costs, and the cost of failure acted as a penalty to ensure the reliability remained above a target level. This was not considered directly in the optimisation procedure by adding a constraint on the reliability, which resulted in a single optimisation problem. It was recognised by the authors that the failure cost coefficient could depend not only on the direct failure cost (dismantling and rebuilding), but also indirect costs (user delay due to diversions), and a target reliability index could be chosen based on these factors. When looking for the optimal maintenance strategy for a structure, all maintenance options must be considered, in addition to the times to maintenance interventions. The optimal solution could be found by considering all possible interventions within constraints of application times, and choosing the scenario which results in the lowest expected total cost.

Due to the discrete nature of sampling of variables when using the MC simulation method, crude MC is an inefficient way to perform the optimisation. To increase the efficiency of the optimisation procedure, the branching technique and multidimensional interpolation methods were used. Since the objective function was discrete (as costs were evaluated annually at discrete points in time), the multidimensional interpolation method was used to artificially smooth the objective function. The traditional branch and bound technique was used to find discrete values of design variables at each step in the optimisation process (e.g. the time to a maintenance intervention). This process is illustrated in Figure 2.7.

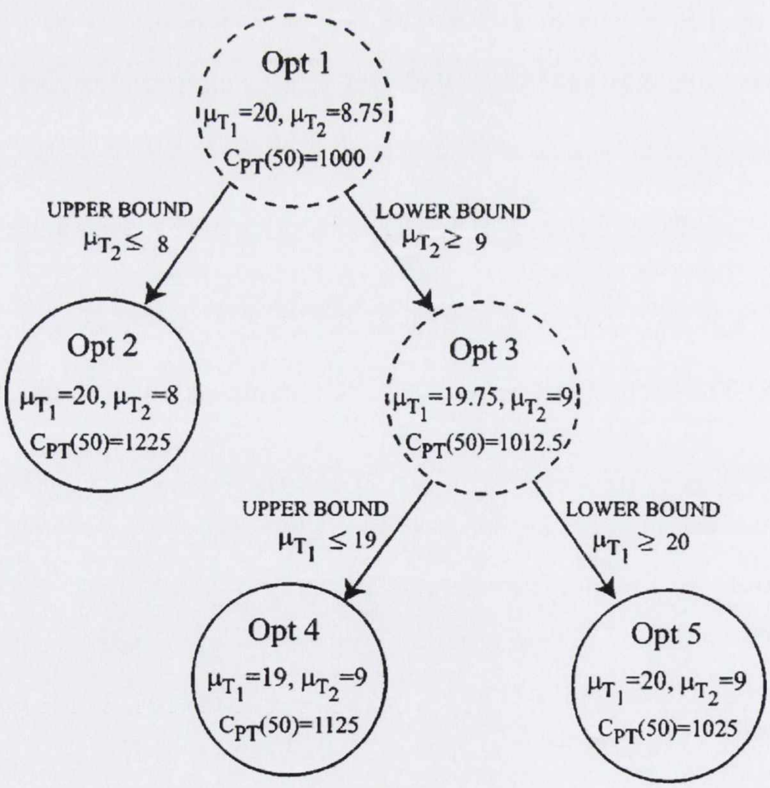


Figure 2.7. An example of the traditional branch and bound technique (Kong and Frangopol, 2005)

Using this method, upper and lower bounds were obtained for each design variable at each stage in the optimisation, which resulted in integer values of the maintenance application time. Using this method the number of solutions was 2^N, where N is the number of design variables. All feasible solutions were obtained and the optimal solution was obtained by comparing the expected total costs of each solution.

In this case, however, the design variables were continuous random variables, and the objective function was discrete. Therefore, the branching method was used to find costs associated with upper and lower bounds of the design variables that were then used in the multidimensional interpolation procedure to find the annual cost associated with the original design variables. Therefore, using the initial variables from the optimisation (before branching), the objective function was solved using multidimensional interpolation and was used to find a search direction for the next optimisation step. This methodology provided the ability to carry out parametric studies, to investigate the influence of different parameters on the optimal maintenance result (e.g. time horizon, failure cost, discounting).

A new optimisation procedure was proposed by Liu and Frangopol (2005a). Unlike many other bridge management systems, the methodology proposed in this paper did not look for one single optimal maintenance plan which aims to minimise the expected total costs subject to a minimum safety requirement. Instead this approach provided the owner of a structure with a set of different maintenance strategies. The condition index, the safety index of the structure, and the life cycle maintenance cost were all treated as separate optimisation functions. A range of possible maintenance scenarios, which comprised the best trade-off between all three objectives, were computed using a multiobjective genetic algorithm. This would allow the owner to choose the most suitable maintenance scenario, based on the set of maintenance plans provided and expert judgement. This is considered to be an important factor, and is therefore considered as part of this thesis as will be discussed further in Chapter 6.

The condition index of the structure was assumed to be assigned to an element of a structure during a visual inspection in accordance with many BMS. The safety index was defined as the ratio of available load carrying capacity to what is required. Constraints were placed on the minimum safety index and the maximum condition level (higher condition level indicates more deterioration). The effect of maintenance interventions on the reliability profile were also calculated using the superposition method described previously (Kong and Frangopol, 2003). Maintenance interventions were carried out when the reliability index profile reached a specific target level for condition or safety.

A new method of optimisation had to be adopted as the methods in place for previous studies could not handle multiple and conflicting objective functions. In this case, genetic algorithms (GAs) were the tool chosen for guiding the search process in the optimisation. The method behind it was based on Darwin's 'survival of the fittest' theory (Darwin, 1859). Using GAs and Pareto optimality, a set of solutions were determined which provide an optimal trade-off between all three objective functions. Examples were carried out for four specific maintenance options on reinforced concrete, and they demonstrated that a set of alternative solutions to the multi-objective problem could be determined, which provide the owner with the decision on which objective is most important at any given time (e.g. for a fixed cost there can be various options for a balance between condition and safety, and one can be chosen which best suits the owner). It was demonstrated that uncertainties must be taken into account to ensure that condition and safety levels remain within the target levels.

An alternative method for optimisation was proposed by Liu and Frangopol (2006a). In this study dynamic programming was used to find the optimal maintenance management plan for a structure, which minimised the net present value of expected maintenance costs, while satisfying minimum constraints for condition and safety levels. To carry out this optimisation procedure a new computer program BMS-DP was developed. A range of feasible maintenance options were identified using a dynamic programming procedure. Uncertainties were taken into account and Monte Carlo simulations were carried out as part of the procedure. Like previous studies (Kong and Frangopol, 2003), the superposition method was used to investigate the effect of maintenance interventions.

Dynamic programming is a recursive optimisation method which breaks the problem into smaller stages. The optimal solution for this smaller problem is identified, and the problem is enlarged. In the paper, using this procedure and recursive methods, the optimal solution for the entire problem was found. Within the method there were three main subroutines, MCS (Monte Carlo simulation), FMP (feasible maintenance plans) and OFP (optimal feasible plans). A flow chart representing this procedure is presented in Figure

2.8. For this study four maintenance actions were considered. It was stated that for any combination of maintenance actions, the order of the maintenance actions did not affect the resulting service life of the structure. On this basis the optimisation was performed.

In the MCS subroutine, values were generated for the random variables according to predefined probability functions. In the FMP, initially, it was assumed that only one maintenance action was applied over the lifetime of the structure (N=1), and the resulting maintenance scenarios where the lifetime of the structure exceeded the prescribed lifetime (without reliability falling below certain level) were considered feasible. Otherwise, they were considered infeasible, and were stored. Next, it was assumed that two maintenance actions were carried out (N=2). In this case all infeasible scenarios from the previous simulation (where N=1) were considered with an additional maintenance action, and again, the feasible options and infeasible options (now less) were updated.

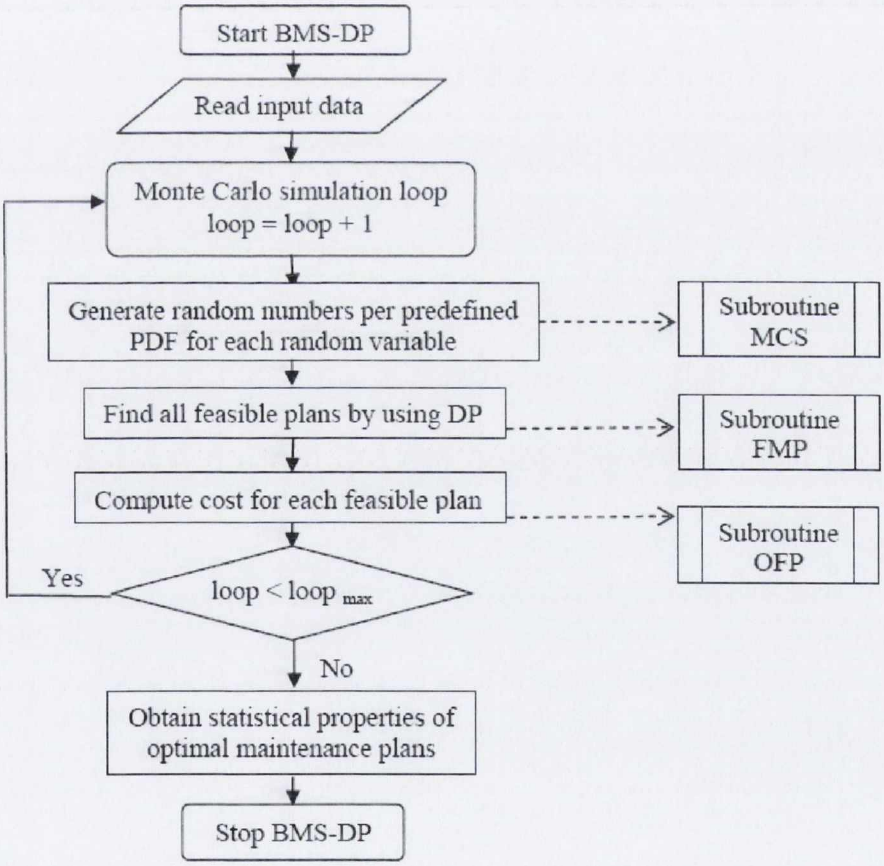


Figure 2.8. Flow chart illustrating BMS-DP optimisation procedure (Liu and Frangopol, 2006a)

This process was repeated until there were no more infeasible options to be considered. Following this the OFP searched for the optimal application times of maintenance for each feasible plan such that the present value cost of the maintenance plan was minimised. The flow chart for each of the three subroutines is illustrated in Figure 2.9. When considering the application times, the safety index must remain above a target level and the condition index must remain below a specific level. This was achieved using GAs. However, it was assumed in this publication that no inspections were carried out to update the condition of the structure or to identify if repairs were actually necessary or not. Contrary to this, this thesis integrates inspection results as part of the developed maintenance management plan and the probability of repairs being carried out due to correct/incorrect inspection results is also considered.

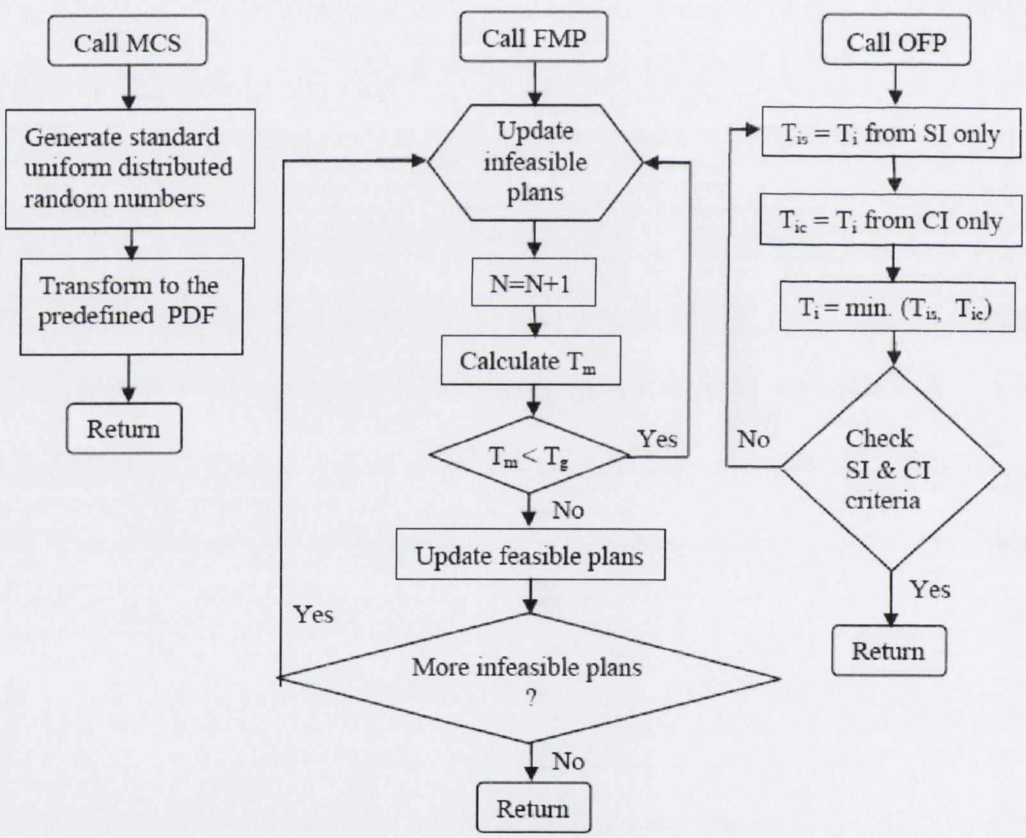


Figure 2.9. Flowchart for subroutines (MCS, FMP and OFP) of the BMS-DP computer program (Liu and Frangopol, 2006a)

A novel method of optimisation was also proposed by Bucher and Frangopol (2006). In this study the objective function for optimising maintenance was formulated based on the probability of exceeding certain threshold values. As well as calculating the probability of violating safety levels and the threshold condition of the structure, the probability of exceeding the allowable total cost was also determined. This study investigated the effect of using different objective functions which were based on the probability of exceeding a threshold value. These threshold values could be specified by the owner of a structure.

The deterioration of the condition of the structure and the safety of the structure were both considered over time. Preventative maintenance (time based) and essential maintenance (condition based) were both considered. For preventative maintenance it was assumed that a maintenance action reduced the deterioration rate for a certain period of time (a random variable). For essential maintenance, once the safety of the structure fell below the minimum β value, the structure was assumed to be rebuilt. This increased the safety index (by a random amount) and the condition index was assumed to be 'as new'. Subsequent deterioration of the rebuilt structure was delayed by a certain time (random variable). The time delay was different for condition and safety.

Using Monte Carlo simulations the cost of maintenance was calculated over the lifetime of the structure. The expected probability of exceeding certain thresholds and the expected number of failures (and cost of failure) were determined using the results of these simulations. The optimisation function was formulated to minimise the expected total service life cost, with a constraint on the probability of exceeding the cost threshold (must be less than a specific value), the probability of violating the safety threshold (must be less than a specific value) and the probability of violating the condition threshold (must be less than a specific value).

Up to this point, the reliability based optimisation of maintenance interventions was carried out considering only one structure. For a maintenance management methodology to be effective for bridge owners or managers (such as a highway agencies), the maintenance of a network of bridges must be considered. Liu and Frangopol (2005b) introduced a methodology for maintenance optimisation of a network of structures. This methodology used genetic algorithms (GAs) to optimise maintenance management of a network of bridges, ensuring that the bridges on the most important routes were highly serviced.

When considering a network of bridges, the overall reliability of the system, or the connectivity of the system can be considered. The connectivity is concerned with the ability to travel from an origin to a destination. In this case the reliability of travel time from an origin to a destination was of interest. This takes into account the importance of different routes and bridges along these routes. It was noted that in this study correlations between the behaviour of bridges within a network were ignored (e.g. due to similar environment, loading etc.), allowing failures to be considered as independent.

Using an event tree, an expression for the network connectivity reliability was developed. Bridges which were purely in series were grouped. All possible combinations of functioning and failed bridges and groups were considered. The probability of occurrence of each branch was determined and the resulting connectivity was determined (connected or disconnected route). This event tree process for the connectivity of a group of bridges between origin (denoted O) and destination (denoted D) is illustrated in Figure 2.10.

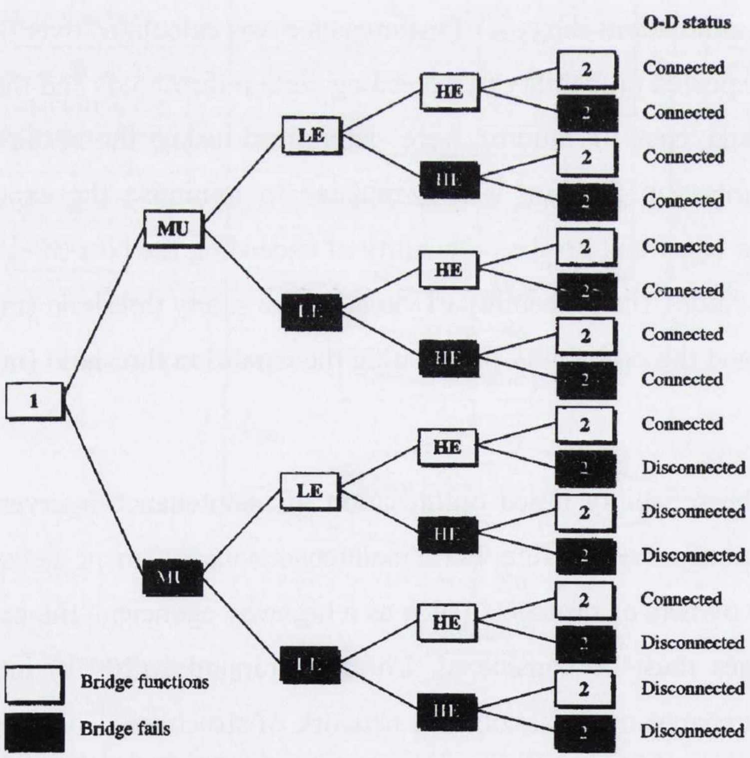


Figure 2.10. Event tree process for the connectivity of a group of bridges (Liu and Frangopol, 2005b)

The total probability of disconnectivity is the sum of the probabilities of disconnectivity for each branch. These probabilities were obtained from the probability of failure for individual bridges or groups of bridges, using the individual time dependent reliability profiles of the bridges. Therefore, the bridges considered could be different structural systems in different environments, with different ages. In addition, a system reliability model (series and parallel) could be adopted like previous studies (Estes and Frangopol, 1999) to calculate the overall reliability of a bridge given the reliability of different components (for different failure modes). Over time, the overall network connectivity reliability deteriorates with bridge deterioration but the extent of this decrease due to each bridge was different, depending on the location and arrangement of bridges in the network. The sensitivity of an individual bridge or group of bridges on the overall network reliability connectivity could be investigated.

In this case only two objective functions were considered. One was the net present value of the total expected maintenance cost of the network, and the other was maintaining the performance of different routes (connectivity between origin and destination) using time dependent reliability analysis. Like previous studies (Liu and Frangopol, 2005a), this resulted in a range of maintenance options which were within a set of optimum tradeoffs between the two objective functions. This problem was solved using GAs and Pareto analysis.

In Liu and Frangopol (2006b), this proposed methodology was further improved by evaluating the performance of a bridge network in terms of bridge connectivity, user satisfaction (using a new cost model which was based on demand and capacity) and structural reliability of critical bridges within the network.

A very detailed model to evaluate user satisfaction was proposed, which was based on the probability that a specific route between two points of interest could meet a specific traffic volume. A traffic flow model was developed to estimate the unit traffic costs between two points (origin and destination). This was based on the sum of the travel time cost and the vehicle operation cost. The travel time cost took into account the speed of the traffic on a route, which depended on the traffic flow and the capacity of the route. Using this traffic flow cost model, the most cost effective route between two points was evaluated. The performance of a link in the network was assumed to be a function of the difference in the traffic flow capacity and demand of the link. Using this method, the probability of traffic congestion could be determined for each link. On this basis, the traffic flow cost model and the probability of unsatisfactory performance for each link (and hence the bridge network) were determined.

To evaluate the structural reliability of the bridge network, a minimum-weight spanning tree (MST) was developed, which consisted of the shortest path between any two points in the network (in terms of most cost efficient route). Only the bridges along these shortest paths (referred to as critical bridges) were considered in the structural reliability of the bridge network. These critical bridges were assumed to be in a series system to ensure reliability of the MST. The overall performance of the network took these three factors into account in a series system model. Since some of the random variables associated with these three factors were correlated, the upper and lower bounds of the performance index of the network were evaluated.

In Frangopol and Liu (2007), an alternative optimisation procedure was proposed, which consisted of a multiobjective optimisation of a network of structures with different remaining service lifetimes, with different reliability importance factors using a stochastic dynamic programming procedure. Unlike the previous studies, this method used a two stage process of optimisation, which consisted of firstly finding the optimum maintenance strategy for each individual bridge within the network, followed by the second stage which optimised the network maintenance strategy by using a weighting function which included reliability importance factors of the structure, whilst allocating funds such that the optimal maintenance plans for as many bridges as possible could be realised.

The multiobjective formulation considered four different factors:

1. The safety index of a structure must remain above a target level
2. The condition index must remain below a certain target level
3. The life-cycle maintenance costs must be minimised
4. The annual total costs must be less than a pre-defined budget.

The first stage of the optimisation procedure identified the optimum maintenance strategy for each bridge which minimised life-cycle maintenance costs while satisfying condition and safety limit states. For the second stage a single objective function was defined to maximise the probability that that the optimal maintenance strategy for each of the bridges would be carried out, with a constraint to ensure the total maintenance cost remained below a pre-defined maintenance budget. Reliability importance factors were used to weight the importance of different bridges to the overall connectivity of the bridge network. Like previous studies (Liu and Frangopol, 2006a) dynamic programming and Monte Carlo simulations were used to carry out the optimisation.

The reviewed studies have demonstrated that a lot of progress has been made in the area of reliability based maintenance management in the last two decades. Methodologies which take condition and safety, life cycle costs, uncertainties, multiple failure modes and networks of structures into account have all been developed. Although a lot of research has been carried out in this area, there are areas which require further development.

The effect of inspections on life cycle costs and the incorporation of inspection results into the maintenance management framework have not been considered in detail in the studies outlined and needs to be developed in future research (Kong and Frangopol, 2004a; Kong and Frangopol, 2003). Stewart et al. (2004) also highlighted that that inspections are vital to verify or update model predictions of deterioration, although inspections were not considered as part of that study. Therefore, a rational approach for the incorporation of inspection results into the maintenance management framework is addressed as part of this thesis. The probability of repairs being carried out due to correct/incorrect inspection results is also considered.

Also according to Estes and Frangopol (2001), deterioration models are being developed to simulate the deterioration of bridge elements based on information provided from different US states, where each element is assigned one of 10 of the National Inventory Condition Ratings. However, many of these deterioration models are expressed as a linear reduction in condition state over time. This limits the ability to accurately model the stochastic nature of the deterioration of components. On this basis, a method for simulating non-linear deterioration of structures is developed as part of this thesis, using Markov matrices to describe the deterioration between condition states/groups.

2.2.3. Cost-Benefit Based Maintenance Management

An alternative approach to the minimisation of whole life costs using either reliability analysis or a Markov process is a cost benefit analysis, which aims to minimise costs and maximise benefits to provide an optimal maintenance strategy. A cost-benefit analysis provides a rational approach to allocating limited resources efficiently (Higuchi and Macke, 2007). This is carried out by considering the cost and benefits associated with time-dependent deterioration and maintenance interventions over the remaining lifetime of a structure (Radojicic et al., 2001).

Radojicic et al. (2001) presented a probabilistic method for the maintenance management of existing structures using a cost benefit analysis. In this case both safety and serviceability were considered. This approach provided a rational framework for the incorporation of serviceability and safety limit states, by considering the costs and benefits associated with both limit states. Uncertainty was taken into account in this methodology using probabilistic models to calculate expected costs, and the implications of delaying maintenance interventions were also considered.

In this study, cost models were developed for routine maintenance and repair considering the cost and benefits associated with each intervention, for each limit state over the remaining service life of the structure. The benefits associated with a maintenance intervention were formulated in terms of the reduction in the cost of inadequate serviceability of the structure for each limit state being considered in relation to a set of parameters describing the structure being considered (e.g. material, dimensions, importance of the structure within the network and traffic loads). It was assumed that the cost of a maintenance intervention included direct costs (i.e. material and labour costs) and user delay costs. When considering the cost of failure, the failure-time probability (i.e. failure over a period of time $[t_{i-1}, t_i]$) was considered, where $\Delta T=[t_{i-1}, t_i]$. The product of the failure-time probability and costs associated with failure was integrated to calculate the expected cost of failure over a given time period (e.g. $[0, t_i]$). The cost of inadequate serviceability was calculated in terms of the loss of service to users and the loss of benefits to the owner due to a deteriorated structure. The level of inadequate serviceability was measured by a factor between 0 and 1, where 0 represented a structure which was fully serviceable and 1 represented a structure which could not be used. Thresholds for this factor (associated with 0 and 1) could be set for each limit state being considered.

It was assumed that a maintenance intervention had the effect of reducing the future probability of structural failure, therefore decreasing the costs associated with failure. In relation to structural failure, the benefit associated with the maintenance intervention (B_{fail}) was assumed to be the difference in the expected cost of failure (C^f) with and without the maintenance intervention over a specific reference period, T_{ref}. In relation to serviceability, an intervention would also increase future serviceability. On this basis, the benefit of a maintenance intervention was assumed to be the difference in the expected cost of inadequate serviceability (c_{ser}) with and without the maintenance intervention. Figure 2.11 illustrates the benefits of maintenance interventions at a specific reference period, T_{ref}, in relation to structural failure (Figure 2.11a) and serviceability (Figure 2.11b).

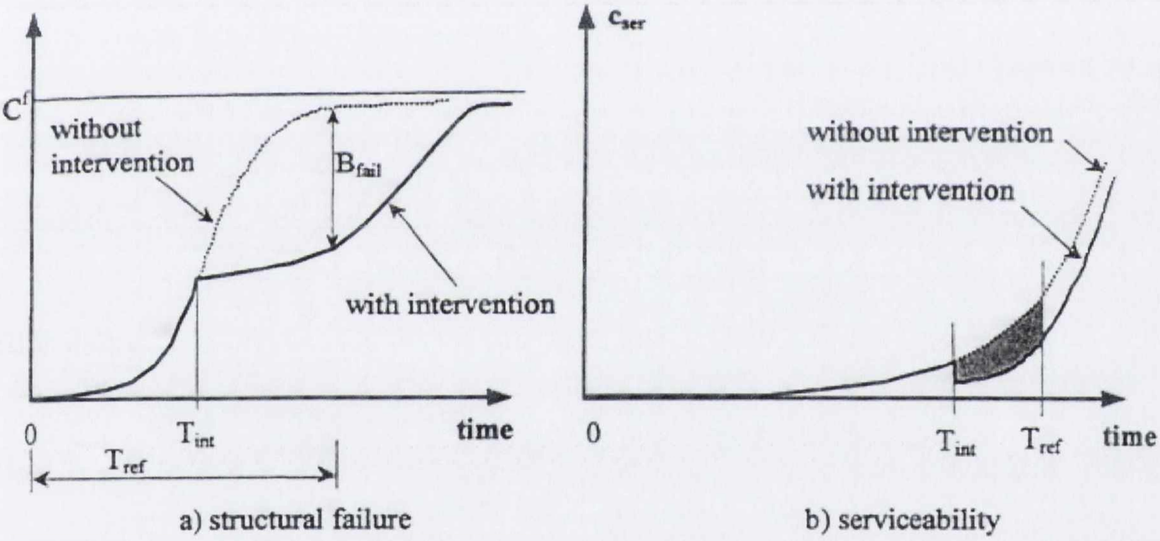


Figure 2.11. Illustration of the benefit of maintenance interventions in relation to structural failure and serviceability (Radojicic et al., 2001)

An example was presented which simulated the loading (load Q at angle θ) and deterioration of a simple three bar structure over time (Figure 2.12). A time dependent reliability analysis and an event tree model were used to determine the reliability of the structure and the failure-time probability for all elements considered. Different repair strategies were considered as part of this example. This approach was used to determine the optimum maintenance strategy over the remaining lifetime of the structure, by minimising the costs and maximising the benefits. Considering all maintenance interventions, at different times over the lifetime of the structure, a series of points were obtained, which were plotted in this case on a graph of total benefits against total costs, as illustrated in Figure 2.13. From this graph, the efficient line was determined, defining the most efficient maintenance strategies. This provides the owner of the structure with a range of efficient maintenance strategies to choose from, depending on the available budget and the structure being considered.

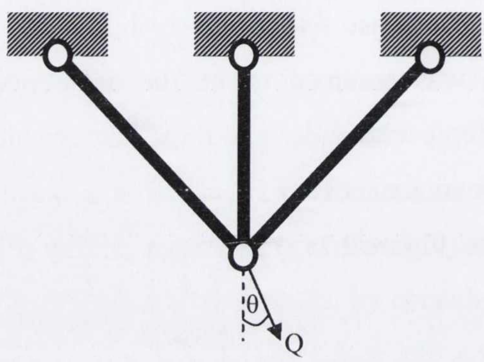


Figure 2.12. Schematic of simple three bar structure (Radojicic et al., 2001)

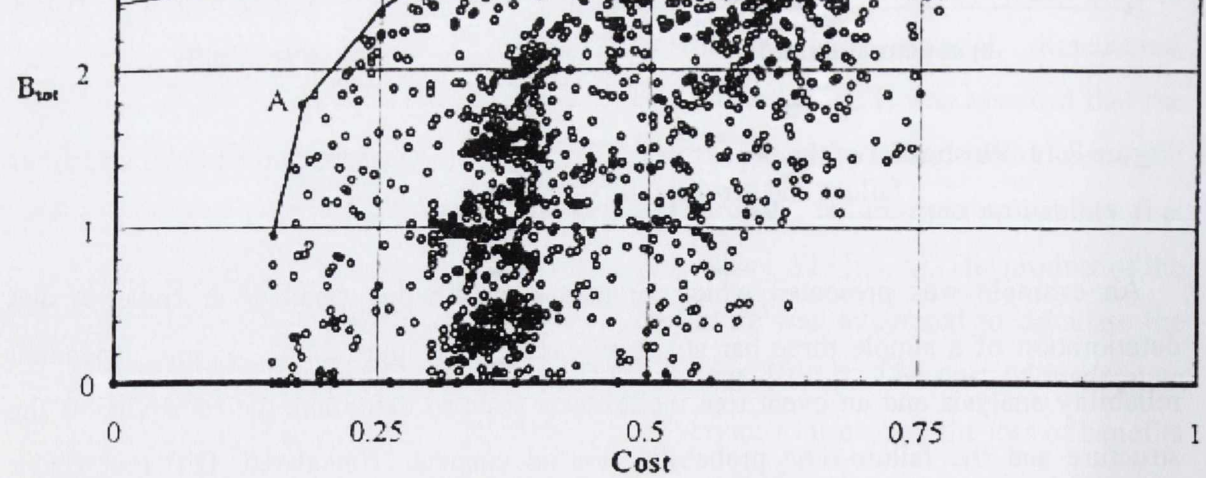


Figure 2.13. Plot of total benefits against total costs to determine efficiency line (Radojicic et al., 2001)

Vassie and Arya (2006) also used a cost benefit analysis to determine long-term maintenance strategies for highway bridges. It was recognised by the authors that the optimal maintenance management plan is not always affordable due to limited maintenance budgets, so a prioritisation system was adopted in this study ranking bridge maintenance in terms of their cost/benefit ratio of delaying the recommended maintenance work. Ideally, maintenance work should be carried out soon after it is recommended, but this is generally not the case, which results in a need for prioritisation of maintenance actions for a bridge network. Therefore, this paper introduced a long term plan to eliminate the maintenance backlog that exists due to long term inadequate budgets.

Two types of maintenance were considered here, steady state maintenance and essential maintenance. Steady state maintenance was assumed to consist of preventative maintenance and maintenance where a delay in carrying it out would not lead to a reduction in structural performance. However, this may lead to a reduction in the lifetime of the structure or increased maintenance costs in the future. The optimum maintenance time was found by considering the effect on the lifetime costs of the structure. Essential maintenance was carried out where the safety of the structure was compromised and it needed to be strengthened. Delaying essential maintenance could result for example in weight restrictions being introduced and an associated environmental cost of traffic delays and traffic jams. It was recognised by the authors that direct and indirect maintenance costs, and environmental damage, must be taken into account when deciding when an essential maintenance action should take place. In addition a predictive model for deterioration must also be considered to estimate the effect of delaying maintenance.

Considering a network of new structures in the present time, it is likely that the optimal maintenance strategy can be found and implemented. In practice, however, when optimal maintenance strategies are being considered for a network of bridges, there is already a backlog of maintenance which needs to be carried out. This results in very high maintenance costs in the first few years of an optimal maintenance program. Therefore, prioritisation procedures must be adopted. According to the authors of this study, the ratio of essential and steady state maintenance should be constant with time but due to maintenance underfunding a higher ratio of maintenance budget goes to essential maintenance.

The authors proposed to eliminate this backlog with an increased budget, a long term maintenance plan (about 10 years) and prioritisation of the maintenance within the long term plan. A formula was presented to estimate the required annual budget to clear the backlog. A prioritisation process for this long term plan was presented. Table 2.5 presents the problem where the number of bridges requiring maintenance is larger than the number of bridges that are actually maintained. This leads to an increase in the number of bridges in the backlog. The authors presented a 10 year plan to eliminate this backlog, Table 2.6, allowing the optimal maintenance strategy to be adopted once the backlog has been cleared. In this study, the maintenance of bridges was prioritised based on a cost benefit analysis considering the cost of maintenance for each bridge and the consequences of not carrying out the maintenance. A simplified flow chart illustrating this process is presented in Figure 2.14.

Year	Bridges needing maintenance *	Bridges actually maintained †	No. of bridges in the backlog
1	100	20	80
2	180	20	160
3	260	20	240
4	340	20	320

* The number of additional bridges needing maintenance each year is 100
† The budget is sufficient to maintain 20 bridges per year

Table 2.5. An example of the increase in the backlog of bridges needing maintenance due to underfunding (Vassie and Arya, 2006)

Year	Bridges needing maintenance *	Bridges actually maintained †	No. of bridges in the backlog
0	-	-	500
1	100	150	450
2	100	150	400
9	100	150	50
10	100	150	0
11	100	100	0

* The number of additional bridges needing maintenance each year is 100
† The budget is sufficient to maintain 150 bridges per year

Table 2.6. An example of how to eliminate the backlog of bridges needing maintenance (Vassie and Arya, 2006)

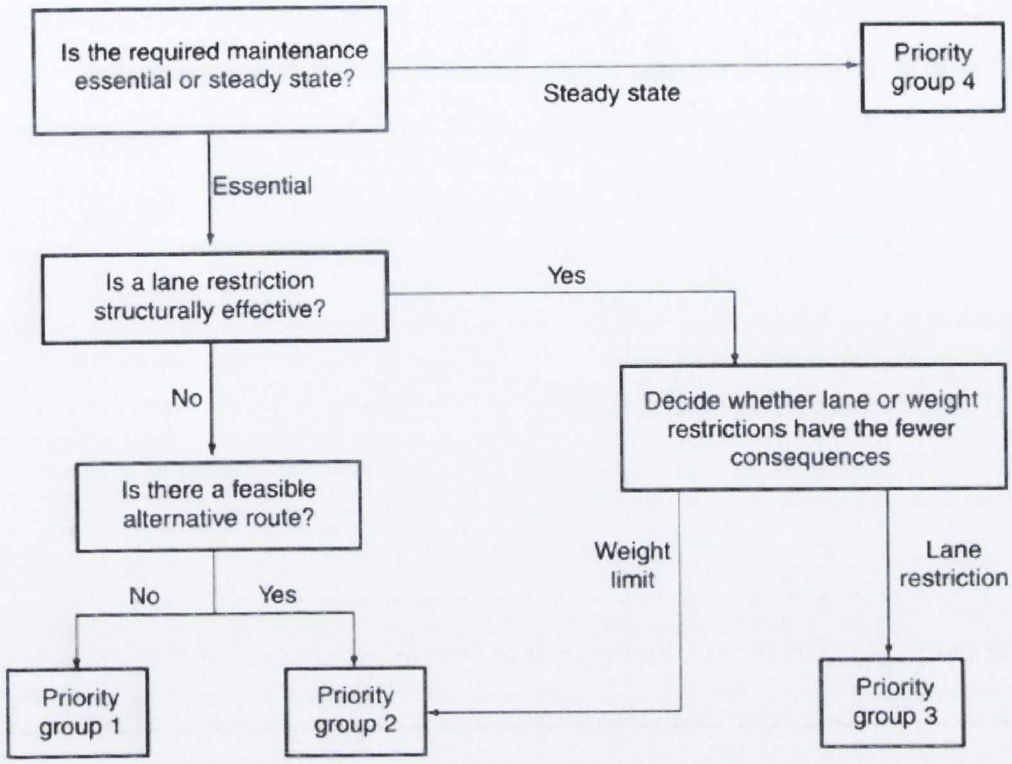


Figure 2.14. A simplified flow chart illustrating the method for the prioritisation of maintenance for each bridge (Vassie and Arya, 2006)

An alternative cost-benefit analysis was presented in Higuchi and Macke (2007) and Macke and Higuchi (2007). These papers detailed the development of a model to optimise the maintenance interventions on a structure over the whole life of the structure by considering a cost-benefit approach. If the cost of inspections and repair/rehabilitation interventions outweighed the benefits of the interventions, then the structure was considered no longer economically serviceable. To optimise maintenance, this study not only considered the life cycle costs associated with construction, inspection, maintenance and failure of a structure, but also the associated benefits of these actions. Using this approach, it was possible to optimise the maintenance times, level of repair and inspection quality which would lead to the optimal service life of the structure. It was also possible to determine the optimal service life of the structure.

To simulate the deterioration of a structure, the deterioration was described by one of m condition states (ranging from 1 to m, where 1 related to no deterioration, and m related to structural failure). The deterioration of the structure was simulated using a continuous time

Markov chain. This method was independent of material, so any deterioration process could be inputted by choosing appropriate transition probabilities to fit the type of deterioration. The entries in the transition matrix could be formulated from analytical models or experimental data.

A probability distribution was used to determine the probability that the structure was in one of m condition states. This probability distribution was updated after an inspection or repair was carried out, depending on the quality of repair (Q), extent of repair (C) and the quality of the single stage inspection (D), where Q, C and D were matrices. The probability of detection was also considered, and was related to the inspection quality and the condition state being considered. Inspection intervals were not regular and repair could be carried out at any time over the service life of the structure. The extent of the repair was assumed to depend on the minimum rehabilitation level (any states larger than this were repaired). The quality of the repair dictated how many states the structure was repaired by. A perfect repair restored the structure to the initial condition state. It was assumed that the maintenance action did not affect the deterioration mechanism of the structure (i.e. the repair material was the same as the original material).

A net present benefit function was defined, considering benefits, losses, maintenance interventions (inspection and repair) and construction costs. The expected benefit was calculated using the benefit rate associated with being in a particular state and the probability of being in the state. The expected loss was assumed to depend on the number of times a structure deteriorated to the failure state (the last state, m). The maintenance cost was calculated based on the inspection quality and the extent of repair.

It was assumed that for acceptable structural performance the expected net benefit rate must be greater than zero (i.e. the benefits outweigh the losses). On this basis, the acceptable failure rate was determined for a specific service life (i.e. once the expected net benefit rate was positive, the failure rate was less than the acceptable failure rate). In relation to the service life, it was assumed that the net present benefit had to be positive at the end of the service life (i.e. the construction cost, maintenance costs, benefits and losses). Also, it was assumed that for a maintenance intervention to be reasonable, the benefits of the maintenance intervention (over the time period until the next intervention) must outweigh the costs of the intervention. On this basis, for a given number of maintenance interventions, the optimal maintenance sequence, rehabilitation levels and inspection qualities were determined over the remaining lifetime of the structure.

These studies highlighted the benefits of using a cost benefit analysis. Different limit states could be considered as part of the analysis (e.g. strength and serviceability), incorporating uncertainties. Cost benefit analysis provides a rational approach for the optimisation of maintenance interventions over the remaining lifetime of a structure. As presented by Higuchi and Macke (2007), and Macke and Higuchi (2007), this approach can also be used to find the optimum service life of the structure, or an acceptable failure rate for a given service life. The deterioration process can be simulated using a Markov process or a reliability analysis, with the optimisation of the maintenance interventions depending on the cost-benefit analysis. Higuchi and Macke (2007) and Macke and Higuchi (2007) highlighted that using Markov transition matrices allowed the deterioration of any material to be simulated by choosing the entries in the transition matrix to fit experimental data or on site data. This is considered to be a useful quality and is therefore adopted as part of this thesis. However, it was assumed in these studies that the maintenance action did not affect the deterioration mechanism of the structure. This is considered to be a limitation of this methodology, since repairs of structures are often carried out using a material which is different to the construction material. Therefore, as part of this thesis, the ability to simulate the repair of a structure using a different material (i.e. with different deterioration characteristics) is incorporated into the methodology.

2.2.4. Inspection Modelling within a BMS

The optimisation of inspection, maintenance and repair strategies for governments/authorities managing ageing infrastructure is an ongoing challenge. To carry out accurate and cost effective maintenance on a bridge deck, it is useful to accurately determine the condition of the bridge deck and the materials involved (Scott et al., 2003). It is information from inspections (visual and NDT) that allows managers to estimate the condition of a structure, and predict future deterioration (Chryssanthopoulos and Sterritt, 2002; Faber and Sorensen, 2002). To develop an effective maintenance plan for a structure, an accurate prediction must be made of the future deterioration of the structure (Rafiq et al., 2005b). This must be based on inspections since the as-built structure may differ from the design drawings due to errors in construction (Kato and Uomoto, 2005). Therefore, inspections are carried out to gain partial information on the state of the structure (Rouhan and Schoefs, 2003). On this basis, many studies have been conducted to optimise maintenance and repair strategies based on inspection results, for both visual and NDT.

In the US a survey was carried out in 1996 to determine the extent of NDT usage when carrying out inspections. The survey was sent to state agencies in the US and 86% of these states (43 states) responded. When asked "Do you perceive a need to assess concrete (beyond visual inspection)?", 39 out of the 43 responded yes (Rens and Transue, 1998). Therefore when developing a BMS it is important to incorporate the ability to process results from NDTs, and to have an informative process which enables the best NDT combination to be selected. NDT can prove very useful in deciding which repair/rehabilitation option to choose (Rens and Kim, 2007). Some studies carried out consider inspection updating (Rafiq et al., 2005b; Rafiq et al., 2004; Zhang and Mahadevan, 2000), whereas others focus on probabilistic inspection modelling, and NDT evaluation within BMS (Chung et al., 2007; Chung et al., 2006; Straub and Faber, 2005; Straub and Faber, 2004; Rouhan and Schoefs, 2003; Faber and Sorensen, 2002; Straub and Faber, 2002; Rouhan and Schoefs, 2000; Mori and Ellingwood, 1994a; Mori and Ellingwood, 1994b). Also, it is recognised that there are two types of uncertainties where inspections are concerned, detection and sizing (Zhang and Mahadevan, 2000). A review of these studies is presented in the following section of the literature review.

Some studies have been carried out which focus on the updating of structural performance based on information from inspections and structural health monitoring. In relation to updating maintenance strategies, Zhang and Mahadevan (2000) proposed a method of using inspection results to update the model uncertainties associated with inspections and maintenance. It was recognised that there is uncertainty in the models which are chosen to represent components (e.g. the deterioration characteristics), and it is not always clear which distribution should be used to represent the variable being considered. Therefore, this approach used all models available for prediction rather than just use one model where the uncertainty may be higher. The probabilities that each of the models is more accurate were set, depending on how accurate the models were thought to be. When new information became available from inspections, the probabilities associated with the accuracy of each model were updated. For example, different distributions were used to model a parameter and then, as it was updated with new information, it became more apparent which distribution described the behaviour of the parameter most accurately. Based on the results from inspections (e.g. detection or no detection, detection of a particular size crack), the model uncertainties were updated, and an updated maintenance strategy was determined.

In relation to reinforced concrete, studies were published by Chryssanthopoulos and Sterritt (2002) and Sterritt et al. (2002), which focused on the integration of deterioration modelling and inspection results into a reliability analysis to facilitate maintenance planning. The authors recognised that it is not only the structural reliability that must be considered, but also the reliability of other aspects which could influence maintenance planning, such as inspection results and the probability of a particular level of deterioration occurring (i.e. corrosion initiation, time to critical crack width). Furthermore, more accurate deterioration modelling and inspection results could lead to more accurate maintenance planning. In this paper, a deterioration model was developed by the authors which considered (i) the ingress of chlorides into concrete due to diffusion and absorption, (ii) random input parameters which varied spatially and could be updated based on inspection results and (iii) environmental conditions which could vary over time. Using this model, the time to a particular level of deterioration (or milestone) could be estimated and used to plan inspection and maintenance activities. It was noted by the authors that deterioration data from laboratory and on site tests is essential to develop and update deterioration models.

It was concluded that when updating deterioration models, information from inspections was more useful when a structure or component was divided into zones and deterioration was modelled using spatial random fields. In addition, it was highlighted that when the actual performance of a structure was found to be different to the predicted performance the application of Bayesian statistics to update the deterioration rate may not always be the most accurate option. It may be more suitable to reassess the initial assumptions of the condition of the structure such as construction quality and cover depth. The authors noted that such decisions and alterations could significantly influence the resulting maintenance strategy and hence the whole life costs of a structure.

Subsequently, Rafiq et al. (2004) proposed a method to reduce the uncertainties associated with the prediction of structural deterioration using information from structural health monitoring. By using this information to update the structural reliability of deteriorating structures, decisions taken on maintenance actions could be made with a higher degree of confidence. Alternatively, information from health monitoring or inspections could be used to update the success of a maintenance intervention, or to estimate the time to failure or probability of failure at a specific time.

Due to uncertainties and the random nature associated with many parameters of deterioration, probabilistic models were used to predict the deterioration behaviour of a structure. It was recognised by the authors that information from health monitoring can be inputted into deterioration models directly (a parameter that is monitored is incorporated directly into the deterioration model) or indirectly (a parameter monitored is used to estimate the behaviour of another modelled parameter using physical relations). In this case indirect health monitoring of initiation of corrosion was considered and was used to update the condition of the structure using Bayesian updating methods. The predictive results, from the developed deterioration model, indicated that the probability of failure and time to failure were strongly influenced by the data inputted from the health monitoring system. This illustrated the importance of accurately determining the state of the structure when using predictive deterioration models to optimise the maintenance strategy of a structure.

In addition, Rafiq et al. (2005a) presented a methodology to update maintenance planning using information from sensors within a structure or within a particular component of a structure. Using this information from sensors on the actual condition of a structure, greater confidence could be placed on maintenance decisions since uncertainties had been reduced. It was recognised by the authors that the deterioration of a structure may not be uniform due to spatial and temporal variations. Therefore the structure was divided into zones where corrosion was considered to be uniform. The size of the zones was chosen so that they were (i) large enough to ensure that there was no spatial correlation of the outputs from the sensors and (ii) small enough to ensure that there was uniform deterioration within the zone. Two approaches to updating were proposed depending on the situation which was considered. The first was updating using sensors on different locations on the structure, with one sensor in each zone, while the second approach was updating the condition of one critical zone of the structure using information from multiple sensors within that particular zone. Using this updated information, more accurate and robust predictions could be made on the future deterioration of the structure.

In this paper, the time to corrosion initiation in reinforced concrete was considered using the two approaches which were outlined. Based on the first approach, the information from the sensors was used to update the time to corrosion initiation for the whole system. When all sensors resulted in the same output (i.e. all indicated corrosion initiation or no corrosion initiation) there was a reduction in the uncertainty associated with the results, which resulted in a lower coefficient of variation. When the second approach was considered, the information from the sensors was used to update the time to corrosion initiation of the critical zone. Based on the results of this study, it was concluded that increasing the number of sensors within a zone would lead to a reduction in the uncertainty associated with the results, leading to higher confidence when predicting the future deterioration of the structure. However, in this study it was assumed that the performance of the sensors was perfect, and the effect of incorrect indications from the sensors was not considered.

It was recognised by the authors that in some cases, both approaches may be used simultaneously, firstly to determine the updated condition of each zone, and secondly to determine the updated condition of the entire system. On this basis, the maintenance and repair of a structure could be updated and optimised.

In addition to this work, an interesting study was carried out in Rafiq et al. (2005b), which compared different maintenance strategies in relation to safety and life cycle costing over a 30 year period. Three types of BMS were considered, the first was regular inspections, the second was predictive models which were updated using information from inspections, and the third was predictive models which were updated using information from structural health monitoring systems. Deteriorating bridges subject to chloride-induced corrosion were considered in this study. It was recognised by the authors that life-cycle costs include the expected net present costs of design, construction, inspection, repair and failure; however, in this case it was assumed that the structures had been constructed, so only inspection and repair costs were considered. As in the UK, regular inspections (principal inspections) were assumed to be carried out every 6 years regardless of the age and condition of the structure. If the results of the inspection indicated that the level of corrosion was near to or close to the threshold level then repair was carried out. It was assumed in this case that repair returned the damage to the 'as new' condition. Results indicated that the limit state could be exceeded between inspections.

It was also considered that the deterioration of a structure could be predicted using developed deterioration models. The information that was inputted into one of these models was assumed to be updated after an inspection was carried out, and partial information of the condition of the structure had been obtained. The time to repair was rescheduled based on the new predicted deterioration rate. Repairs were carried if it was predicted that the level of corrosion would exceed a particular threshold level before the next inspection. After the scheduled repair the next principal inspection was 6 years later. Since the repairs were decided upon based on predictive deterioration models which were updated using inspection results, the limit state was not exceeded.

Similarly, the extent of deterioration, which again was predicted using deterioration models, could be regularly updated using information from structural health monitoring, and repair was carried out before the level of corrosion exceeded a threshold level.

Regular inspections and repair resulted in the lowest LCC (inspection and repair cost over a 30 year period was considered), but the limit state may have been exceeded. Using predictive models which were updated using information from inspections and structural health monitoring resulted in higher LCC (inspection and repair cost over a 30 year period was considered), but the limit state was not exceeded. The maintenance strategy using information from structural health monitoring was found to be the most effective of the three methodologies considered here.

Other studies were also carried out using an alternative approach to modelling, which focused on probabilistic modelling of inspection results as part of a BMS. Mori and Ellingwood (1994a) presented the first part of a two part study in which a conceptual maintenance management framework was developed, which relied on additional information from inspections to be provided. A growth model for an individual damage was developed which could be fitted to data from experimental results. In this model, homogeneous initiation of damage was assumed over the surface of the structure or component being considered. Updating of the damage intensity was carried out using Bayesian methods after information was obtained from inspections. Using this damage model and the initial condition of the structure, the probability of failure over time was estimated.

However, it was recognised by the authors that defects on a structure cannot be detected with certainty during inspections (using visual inspections and NDTs). Therefore, it was assumed in this case that the probability of detecting a defect depends on the threshold of the method and increases with defect size until, for a certain defect size, the probability of detection is 1.0. This detectability function, d(x), is illustrated in Figure 2.15. In this methodology it was assumed that detection of damage would lead to automatic repair of the damage to the original state. Therefore, the structural state of the system would depend on the detectability function of the NDT.

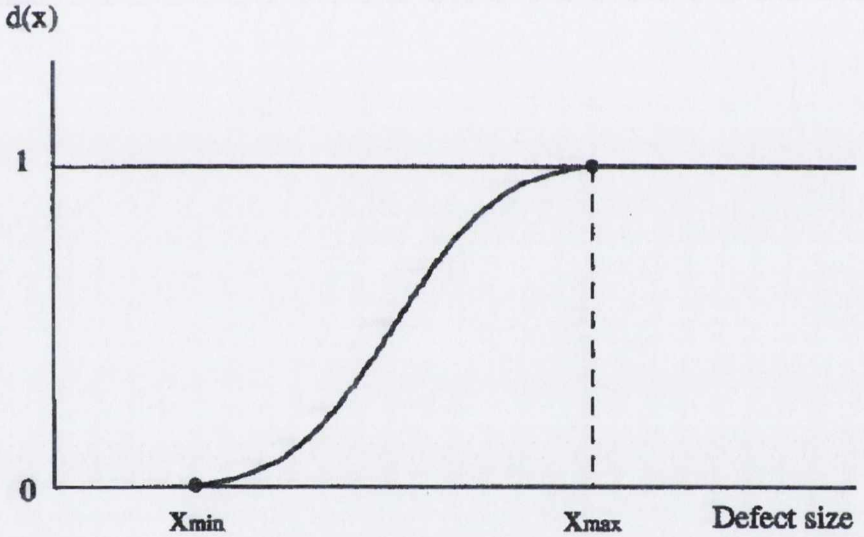


Figure 2.15. Detectability function (Mori and Ellingwood, 1994a)

In this paper, the initial study assumed that the whole of a component or structure was inspected and all damages that were detected were automatically repaired. Following on from this, the idea of partial inspection was introduced, where only part of a structure or component was inspected. This is more likely the case in practice due to the associated inspection costs. A decision on whether to repair the whole component was based on the information from the partial inspection, as the information from this partial inspection was assumed to represent the condition of the whole component. If, when inspected, the damage was less than a critical value, then no further action was taken. If the intensity of the damage was greater than the critical value, another percentage of the structure was inspected. If after the second inspection the intensity of the damage was less than the critical value, then nothing was done until the next planned inspection. If the damage intensity was greater than the critical threshold value, then the whole component was inspected and all damages that were detected were then repaired. If this was the case, and repairs were carried out on the entire component, all further inspections were full inspections and all damages that were detected were repaired.

The second paper in this study (Mori and Ellingwood, 1994b) detailed the development of the time dependent reliability based methodology which was used to determine the optimum inspection and maintenance options, while still maintaining the safety of the structure below the specified allowable probability of failure for the limit state which was considered. This approach acknowledged the importance of inspections as part of this process and that a balance must be found between the extent, cost and quality of inspections that are carried out. In this case, reliability based optimisation techniques were used to find the optimum balance between extent and quality of inspections, the required safety level and the costs involved. Inspection, repair and failure costs were considered here.

In relation to deterioration, this paper assumed that damage growth was a linear function of time, that only defects over a certain threshold were detected, and that all damage detected was subsequently repaired. The inspection cost was assumed to be a linear function of the number of inspections which were carried out, but a non-linear function of the inspection quality. The repair cost was assumed to be linearly proportional to the area of the structure that was repaired, but a non linear function of the maximum damage intensity. In this case, discounting was not considered for simplicity and because the future discount rate was unknown.

The variables which were used in the model for optimisation were the inspection/repair times (for a specified number of inspections over the lifetime of the structure), and the minimum detection threshold for damage for each inspection. A constraint in the model was that the maximum inspection interval was 4 years. It was found that the optimum inspection/repair intervals were almost uniform in all examples considered in this study.

A detailed probabilistic approach to modelling inspection results was proposed in Rouhan and Schoefs (2000) and Rouhan and Schoefs (2003). The authors presented an approach for the optimisation of inspection, maintenance and repair of structures using probabilistic decision and detection theories, including the probability of detection and the probability of false alarm. According to the authors, "the optimal inspection plan would be to inspect at the right place, at the right time, and with the right tool, at the lowest cost". It was recognised that the safety of the structure must also be maintained above a predefined level.

Since uncertainties exist as part of the inspection process, a probabilistic approach was adopted to characterise the inspection quality and inspection results. Work was carried out by the ICON project (Rouhan and Schoefs, 2003) to calibrate inspection quality of different NDTs in realistic in-situ conditions, which led to the development of an NDT database. In this paper, the probability of detection (PoD) was defined as the probability of detecting an existing crack, greater than the detection threshold of the NDT. The probability of false alarm (PFA) was defined as the probability of detecting defects that were not present, which could occur due to noise from sources such as environmental conditions and human interference. This could lead to unnecessary repairs and cost overrun. The PoD was therefore assumed to depend on the signal size, the detection threshold and the noise, whereas the PFA was assumed to depend only on the detection threshold and the noise. The size of the signal was not considered, and the PoD and PFA were defined in relation to a binary variable which represented the presence of a crack. PoD was the probability of detecting a crack, given a crack was present, and PFA was the probability of detecting a crack given that none was present. The authors recognised that there are two aspects to an inspection, detection and sizing, although only detection was considered in their work. The PoD and PFA will be discussed further in Chapter 3, and based on the recommendations by the authors, both detection and sizing will be considered as part of this thesis.

Receiver Operating Characteristics (ROC) curves were used in this study to compare the quality and performance of different NDTs. This is a curve of PFA plotted against PoD. A perfect inspection has the coordinates (PFA,PoD)=(0,1), as illustrated in Figure 2.16. For a particular signal and noise distribution, an ROC curve could be achieved by changing the detection threshold of the NDT being used. Using the PoD, PFA, and another parameter which was defined as the probability of presence of a crack at the inspected area, an event based decision process was developed to estimate the probability of different inspection events. Four events were defined, E_1, E_2, E_3 and E_4, and are presented in Table 2.7. E_1 represented no defect present and the inspection had not detected a defect, E_2 represented no defect present and the inspection had detected a defect, E_3 represented a defect present and the inspection had not detected a defect and E_4 represented a defect present and the inspection had detected a defect. This will be discussed further in Chapter 3.

The expected cost of an event was calculated using the cost associated with the event multiplied by the probability of the event occurring. Four inspection events were considered, consistent with the inspection events outlined above. It was assumed that no detection led to no further action, and that detection led to repair. Using this methodology, the effect of inaccurate inspection results could be used to determine the cost overrun due to unnecessary repairs. The cost of failure due to missed detections could also be determined. This method could be used to rank inspection techniques to optimise the inspection, maintenance and repair of a structure.

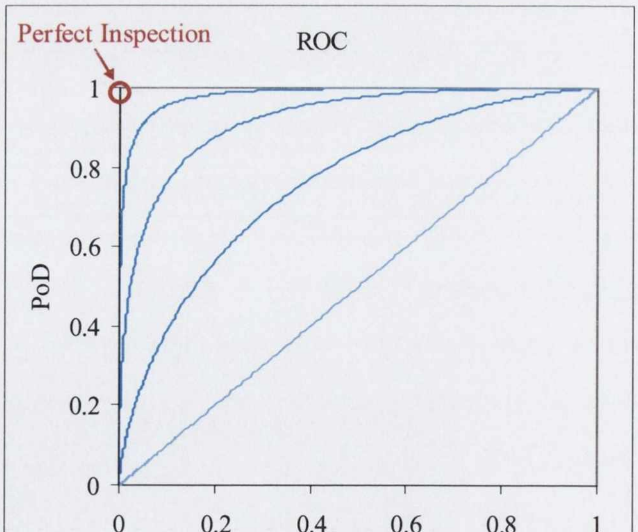


Figure 2.16. Example of an ROC curve (Rouhan and Schoefs, 2003)

	Defect Detected (by inspection)	Defect Present (in structure)
E_1	×	×
E_2	✓	×
E_3	×	✓
E_4	✓	✓

Table 2.7. Four events which were defined in Rouhan and Schoefs (2003)

An alternative probabilistic approach was outlined in Faber and Sorensen (2002). This paper proposed a method of relating the indirect information from inspections to update the condition of a structure or component, incorporating the expert judgement of inspection personnel. Inspections were carried out on condition indicators rather than the condition of interest directly. It was shown how this approach could be used to develop an optimal maintenance strategy by minimising costs.

In this study, it was assumed that the defect rate was an indication of the general condition of a series of components, assuming that all components had the same defect rate. Based on inspection results and expert judgement, the probability that a component was defective (or the defect rate) could be updated. Due to a uniform defect rate it was assumed that inspections were carried out on a random sample of the components. Sampling based on expert judgement considering factors such as criticality, inspectability, expected condition and experience was also included in the methodology. The information from all inspections could be used to update the defect rate of a population of components using Bayesian statistics.

Two additional probabilities were introduced to update the defect rate after an inspection, p_i and q_i, where p_i was defined as the probability that the component was defective given that the inspection (of a particular condition indicator i) had indicated that a defect was present. q_i was defined as the probability that the component was defective given that the inspection (of a particular condition indicator i) had indicated that no defect was present. The results of the inspection could also be used to update these probabilities. This method, which took inspection, repair and failure costs into account, could be used to optimise the inspection and maintenance of structures. Using a risk based approach, the costs were calculated based on the probability of an event occurring, multiplied by the consequence of the event. The variables in the optimisation were the inspection method, the indicator of damage, the inspection coverage, the criteria for repair and the repair method.

In the example presented, preventative and essential repair were considered in relation to localised and uniform deterioration of reinforced concrete. This study assumed that repair was perfect (i.e. the component returned to the 'as new' condition). As well as visual inspections (which were carried out to detect visual corrosion), NDT tests were also carried out to detect corrosion before it became visible, which allowed preventative repair to be carried out. The quality of the NDT was taken into account using the probability that the NDT indicated that corrosion had initiated given that corrosion had initiated, and the probability that the NDT erred and indicated that corrosion had initiated given that corrosion had not initiated. The inspection and repair process is illustrated in Figure 2.17.

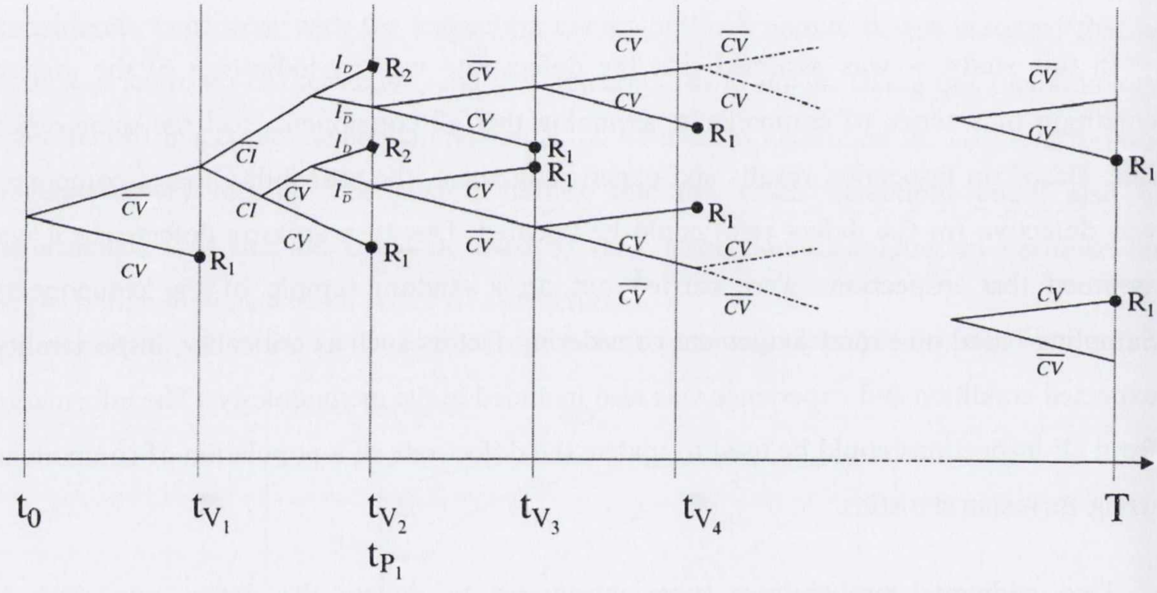


Figure 2.17. Event tree for inspection and maintenance of structures (Faber and Sorensen, 2002)

CV is the event of visual corrosion (detected by visual inspection at time t_V), CI is the event of corrosion initiation and I_D is the event of positive indication of corrosion initiation (detected by NDT at time t_P). The complimentary events are denoted, $\overline{CV}$, $\overline{CI}$ and $I_{\overline{D}}$. A major repair (R_1) was assumed to be carried out when corrosion was detected by visual inspection and a preventative repair (R_2) was assumed to be carried out when corrosion initiation was detected by NDT. Based on this event tree, the service life costs could be evaluated for all scenarios and the optimal strategy could be determined based on the minimisation of the service life costs.

Another approach of probabilistic inspection planning was proposed in Straub and Faber (2002), whereby, the optimum inspection coverage was determined based on the spatial variability of the deterioration mechanism which was considered and the quality of the inspection techniques which were used. Deterioration modelling was not considered in this study, therefore, a point in time assessment of the structure was assumed. A defect was assumed to be critical when its dimensions were greater than a specified critical defect size, which was taken to be a deterministic value in this case. Only the defects which were indicated to be larger than the critical defect size were repaired. For each element considered, the largest defect was described by a marginal distribution, and a covariance matrix described the correlation of the largest defects in the elements, as illustrated in Figure 2.18.

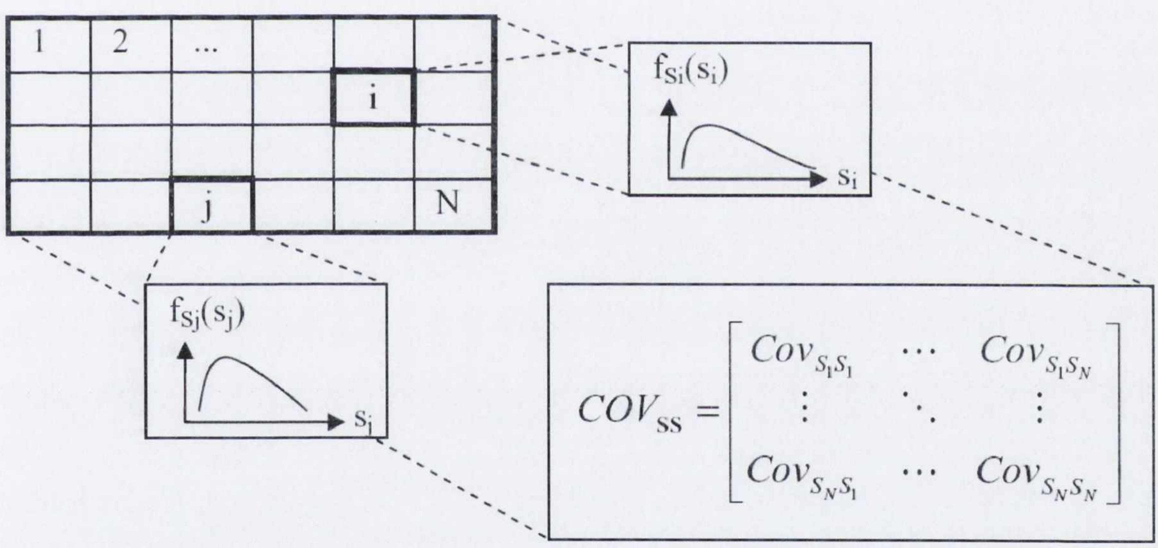


Figure 2.18. Illustration of covariance matrix which describes the correlation of the largest defects in the elements (Straub and Faber, 2002)

The decisions and events associated with an inspection and maintenance scheme were described by a set of variables, which are illustrated in Figure 2.19. In this figure E={e} was the set of all possible inspections, Z={z} was the set of inspection outcomes (e.g. detection, no detection), A={a} was the range of possible actions following an inspection (e.g. repair, do nothing), Θ={θ} was the unknown true state of the structure and u(e,z,a,θ) was the assigned utility, which was decided upon by the decision maker depending on the other variables (which may be a monetary value). A decision rule a=d(e,z) was defined to relate the inspection chosen and the inspection results to the action taken following the inspection. The inspection strategy was then described by (e,d). The expected utility corresponding to this inspection strategy could then be determined and the inspection strategy was optimised based on the maximisation of the utility function.

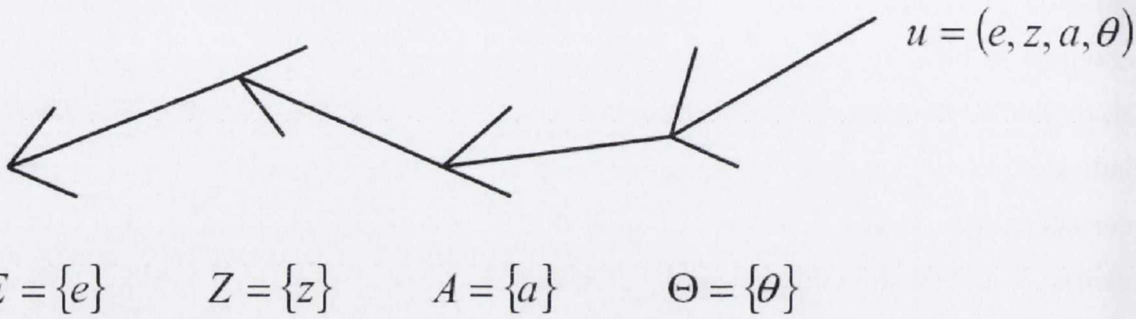


Figure 2.19. Decisions and events associated with an inspection and maintenance scheme (Straub and Faber, 2002)

An adaptive inspection strategy was proposed to incorporate the ability to carry out further inspections (i.e. carry out inspection on other elements) based on the results of the first inspection. The decision rule d* related the results of the inspection of the first element to the inspection strategy chosen for the second element (i.e. e*=d*(e,z)). The original decision rule which related the inspection results to the action taken (i.e. repair) then used the outcomes of all inspections (i.e. a=d(e,z,e*,z*)). Therefore, the extent of the repair to be carried out depended on the results of all previous inspections. The probability of detection (PoD) related the quality of the inspection to the inspection outcome (i.e. z=detection or z=no detection). For correlated elements, the PoD for the inspection of the second element (i.e. the quality of the second inspection) depended on the outcome of the inspections of the first element. This process is illustrated in Figure 2.20.

The inspection quality in relation to individual defects was modelled using a PoD curve which was related to the size of the defect. In this case the PoD of the system was not defined in terms of the defect dimensions. Since this study was concerned with many possible defect locations, the PoD of the system was defined as the expected proportion of identified critical defects to the expected total number of critical defects. An additional probability, the Probability of False Indications (which was the probability of indication of a defect when none was present) was also used to take account of unnecessary repairs. On this basis, the PFI was estimated as the fraction of elements that were repaired (or inspected) unnecessarily.

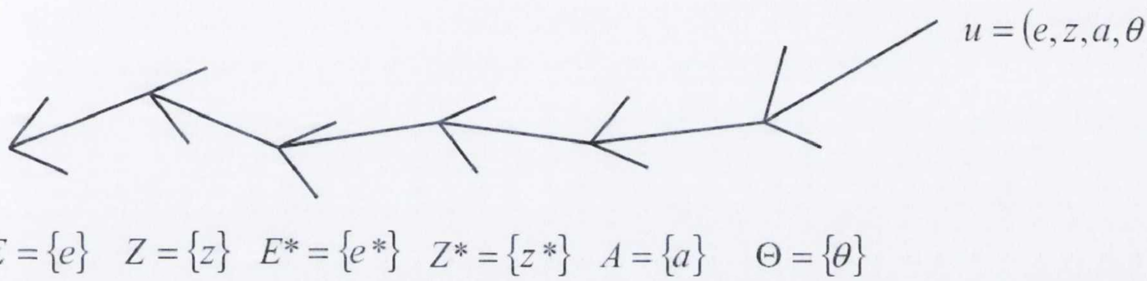


Figure 2.20. Adaptive inspection strategy (Straub and Faber, 2002)

A decision rule was also proposed to update the inspection plan (i.e. increase or decrease the percentage of inspected elements) after the first round of inspections, based on whether the fraction of inspections resulting in detection was greater or less than a certain value, q. The application of this methodology was demonstrated using a costing model, which was used to determine the expected total cost of the inspection based maintenance strategy, considering the inspection cost, repair cost and failure cost. The optimal initial inspection coverage and the optimum q value could be determined based on the minimisation of the expected total costs.

Similarly, Straub and Faber (2004) and Straub and Faber (2005) presented a detailed approach for the inspection and maintenance planning of an entire system, in which the dependency and correlation of the elements within the system was taken into account explicitly by considering the dependency of different variables between elements. All of the critical areas of the structure which may have been subject to deterioration were first identified (known as hot spots). Using this system model, the authors proposed how to estimate the condition of a hot spot, given the inspection results obtained for hot spots in different locations.

In Straub and Faber (2002), the system consisted of N elements (hotspots), and the dependency between the largest defects in the elements was represented by a covariance matrix. This was due to common factors such as loading, material properties, environment etc. The level of dependency between hotspots was estimated based on these factors. In these papers, the dependency was not modelled using one covariance matrix, but rather, by explicitly modelling each of the common influencing variables, and the correlation between hotspots for each of these variables. To reduce the number of possible outcomes of the decision rule, it was assumed that when repair was carried out, the hotspot was repaired to its original 'as new' condition.

The inspection cost was assumed to have a fixed and variable component, with the variable component depending on the number of hotspots to be inspected. However, the variable part was assumed to be only a minor part of the total repair cost. The fixed component could arise from the cost of accessing the hotspots to carry out inspections and possible temporary closure of the structure. Dependency between the inspection performance at different locations was also considered, although, it was stated in the study that the inclusion of this dependency had little effect on the system reliability. This dependency could be due to environmental factors, or inspector performance. However, it was recognised that the results of an inspection at one hotspot could provide information on the state of other hotspots in the system, due to the dependency in deterioration between hotspots. Therefore, in a large structural system, not all hotspots require inspection, and a method for determining the optimal inspection coverage as a function of these dependencies was proposed by the authors. On this basis, using the method which was outlined in Straub and Faber (2002), the optimum inspection coverage could be determined.

Chung et al. (2006) and Chung et al. (2007) proposed a probabilistic approach to inspection based maintenance, to optimise the inspection process (using the most suitable NDT) and inspection frequency, resulting in the minimum total cost, while maintaining the structure above a threshold safety level. The discussion in Chung et al. (2006) was specific to fatigue deterioration, and fracture critical members in steel bridges, whereas Chung et al. (2007) considered deterioration due to corrosion and fatigue.

Monte Carlo simulations were used to carry out the analysis. Crack growth of a structural member due to fatigue deterioration was simulated. The results of the analysis recommended a single NDT and an inspection interval which was the optimal solution from the Monte Carlo simulations. The probability of detection was used as a measure of detection accuracy of an NDT. In relation to detection of cracks, four parameters were used to describe the outcome of an inspection, true positive (defect present, defect detected), false negative (defect was present but not detected), false positive (defect detected, none present) and true negative (no defect present, no detection). Table 2.8 also provides an explanation of these four parameters. In this case the probability of detection was defined as a true positive result.

Does crack exist?		Is crack detected by NDT?	
Yes	No	Yes	No
True positive (hit or correct rejection)	False positive (false call)	True positive (hit or correct rejection)	False negative (miss)
False negative (miss)	True negative (correct accept)	False positive (false call)	True negative (correct accept)

Table 2.8. Explanation of four possible inspection outcomes of hit/miss method (Chung et al., 2006)

The authors described how the PoD for different inspection techniques could be estimated by introducing cracks of known length, and using the NDTs to assess the size of the defects. The hit/miss method or signal response method could be used to fit the PoD to the inspection data from the tests. The authors outlined that a log-logistic function or normal distribution function (for a large number of inspections) could be used to represent the PoD when data is recorded in the hit/miss format (for a particular crack size). In addition, the signal response method could be used when the results of the inspection are recorded in relation to the amplitude of the signal. If the signal is less than the threshold, then no signal is recorded by the NDT (i.e. only signal responses greater than a specified decision value are recorded), as illustrated in Figure 2.21. Using regression parameters and a term for error associated with imperfect inspections (assumed normally distributed, with some standard deviation and a mean of 0), the recorded signal response could be related to the actual crack size. In this study, only the probability of detection and the probability of not detecting (1-PoD) were considered to quantify the detection quality of an NDT.

Fixed inspection intervals were assumed in this study for practicality. Using a crack growth model to estimate the crack size at each inspection, the PoD at each inspection was estimated for a particular NDT tool, as illustrated in Figure 2.22. The material properties, traffic loading and the initial crack size were modelled as random variables, and the crack growth was modelled using MC simulations. Based on the crack growth model, the expected probability of reaching a critical crack size without being detected, and the expected number of inspections before fracture occurred were determined for a particular inspection interval without repair.

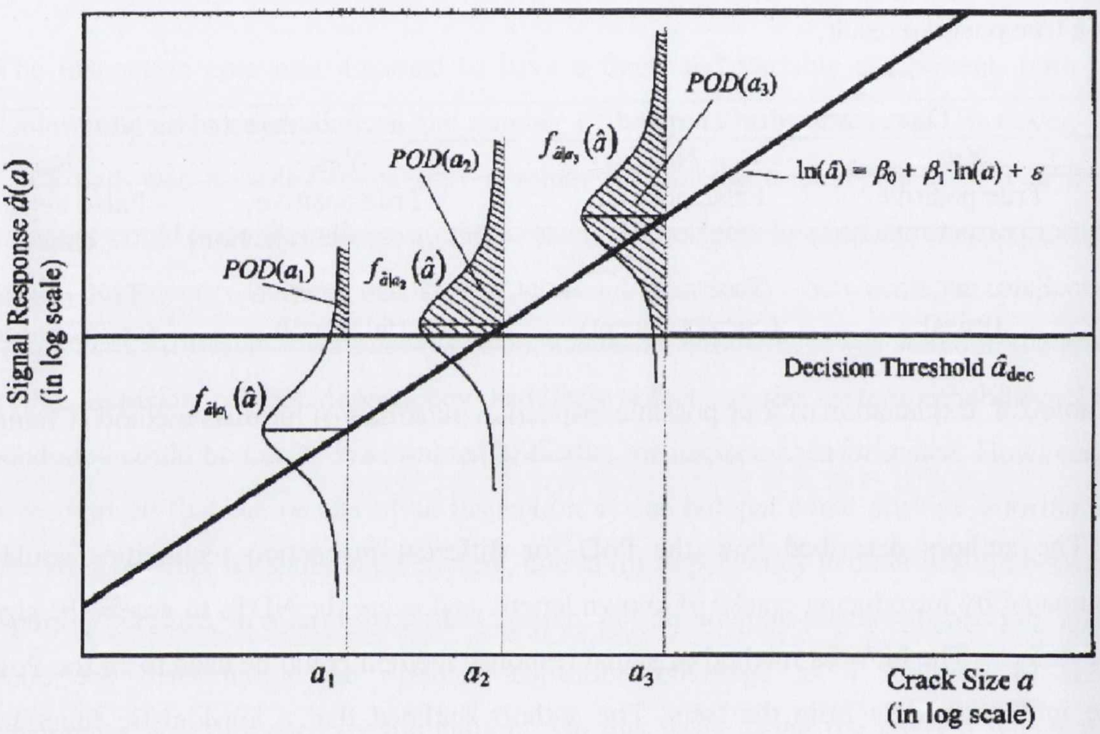


Figure 2.21. Illustration of PoD using signal response method, which relates the actual crack size to the signal response (Chung et al., 2006)

To carry out optimisation, a cost function was developed, considering the cost of inspections and the cost of failure. The PoD function (i.e. quality of NDT) and the inspection interval were used as optimisation parameters, which were varied independently to find the minimum total cost. A constraint was added to limit the allowable probability of not detecting a crack. Restrictions were also placed on the inspection interval so it was not too large or too small (i.e. eliminated infeasible inspection schedules).

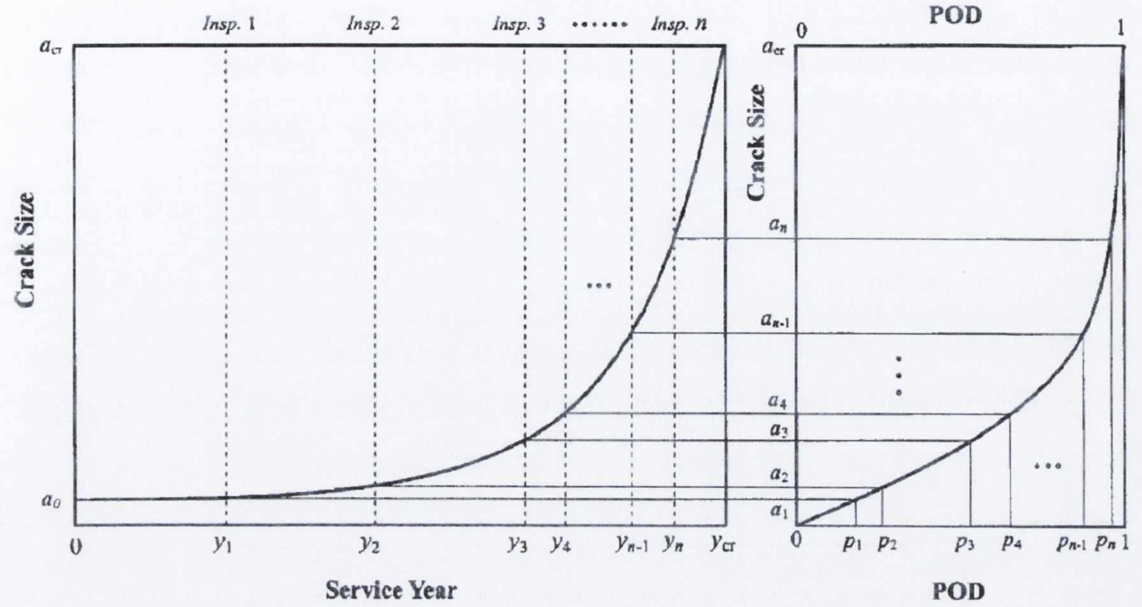


Figure 2.22. Relates the size of the crack to the PoD for each inspection time (Chung et al., 2006)

The above studies highlight the current research in the area of inspection modelling within a BMS, which focus on updating using inspection results (Rafiq et al., 2005a; Rafiq et al., 2005b; Rafiq et al., 2004; Zhang and Mahadevan, 2000) and probabilistic modelling of NDTs and inspection results (Chung et al., 2007; Chung et al., 2006; Straub and Faber, 2005; Straub and Faber, 2004; Rouhan and Schoefs, 2003; Faber and Sorensen, 2002; Straub and Faber, 2002; Rouhan and Schoefs, 2000; Mori and Ellingwood, 1994b; Mori and Ellingwood, 1994a). They provide rational approaches for (i) updating deterioration models based on results of visual and NDT inspections, (ii) the probabilistic modelling of inspection quality, and (iii) establishing the reliability of inspection results.

However, there are areas which require further research. The survey which was presented in Rens and Transue (1998) highlighted the importance of incorporating the ability to process results from NDTs when developing a BMS, yet none of the studies reviewed as part of this literature considered inspections for both detection and sizing. It was recognised in the literature that there are two aspects to an inspection (Rouhan and Schoefs, 2003; Rouhan and Schoefs, 2000), and that there are two types of uncertainties where inspections are concerned, detection and sizing (Zhang and Mahadevan, 2000). Therefore, as part of this thesis a maintenance model is developed which incorporates probabilistic inspection modelling for both the detection stage and sizing stage of an inspection. This also allows owners/managers to select the optimum combination of inspection techniques for a particular structure or group of structures. In addition, it was highlighted that deterioration data from laboratory and on site tests is essential to develop and update deterioration models (Chryssanthopoulos and Sterritt, 2002; Sterritt et al., 2002). To address this issue and to demonstrate the capabilities of the developed maintenance management model, an experimental study is also carried out as part of this thesis to compare the efficiency of three repair materials.

2.3. CORROSION OF REINFORCING STEEL IN CONCRETE

The corrosion of steel in reinforced concrete can be caused by carbonation of the concrete cover or by the ingress of chloride ions, which can break down the protective passive layer around the steel. This passive layer is created due to the alkalinity of the cement in the concrete mix (Ampadu et al., 1999; Thomas, 1991). The pH of the pore solution can be greater than 13. When corrosion of the steel occurs, corrosion products or oxides are formed on the outer surface of the steel. The chemical balance equations describing the corrosion of the reinforcing steel are presented in Equation 2.1 – Equation 2.3 (Advanced Concrete Technology Lecture F1, 1999). This process is also illustrated in Figure 2.23.

Anodic reaction (oxidation)

$$Fe \rightarrow Fe^{2+} + 2e^{-} \qquad \text{Equation 2.1}$$

$$Fe^{2+} + 2H_2O \rightarrow Fe(OH)_2 + 2H^{+} \qquad \text{Equation 2.2}$$

Cathodic reaction (reduction)

$$O_2 + 2H_2O + 4e^{-} \rightarrow 4OH^{-} \qquad \text{Equation 2.3}$$

An electrochemical cell develops in the concrete due to the difference in potential once the passive layer has been broken down by the chlorides (Neville, 2005). The cell is made up of an anode (where oxidation occurs and electrons are freed) and a cathode (where reduction occurs and free electrons are consumed), which are connected by an electrolyte which is pore water in this case (Neville, 2005; Bertolini et al., 2004). During oxidation, positively charged ferrous ions (Fe^{2+}) are formed at the anode which react with water to form ferrous hydroxide (rust). At the cathode, the free electrons react with water and oxygen to produce hydroxyl ions (OH^-), which maintain a high pH at the cathode (Neville, 2005).

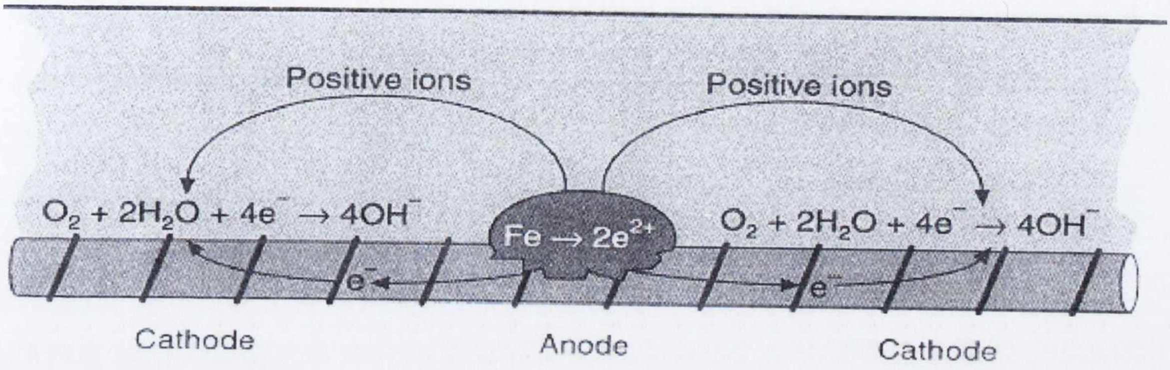


Figure 2.23. Electrochemical cell describing the corrosion of the reinforcing steel (Newman and Choo, 2003)

These oxides which form at the anode can have a volume 2-6 times that of the original base steel (Bertolini et al., 2004). The oxides fill the pore volume in the concrete surrounding the steel. When there is no longer any free pore space, and as the corrosion of the steel continues, this expansive reaction induces tensile stresses in the concrete around the steel. This causes cracking of the concrete surrounding the steel, which then propagates through the concrete cover to the outer surface of the concrete, which can subsequently lead to spalling of the concrete cover (Bertolini et al., 2004). This process is illustrated in Figure 2.24.

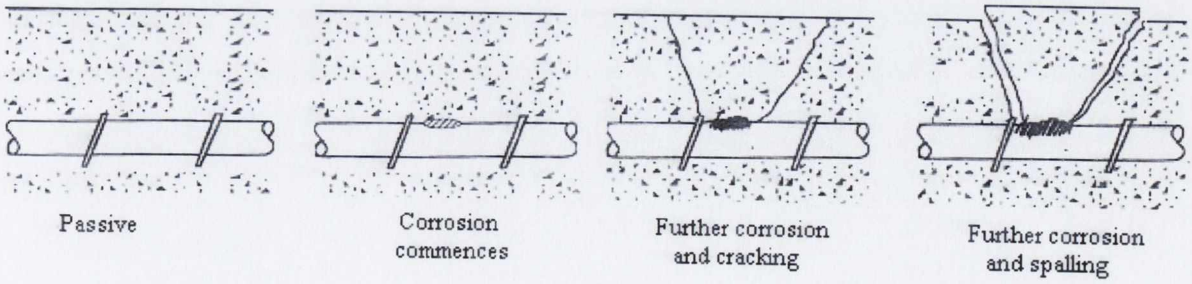


Figure 2.24. Cracking and spalling of reinforced concrete due to corrosion (Advanced Concrete Technology Lecture F4, 1999)

Factors which can influence the initiation phase of the corrosion (up until the depassivation of the steel) are the permeability of the concrete to chloride ions and oxygen, the temperature and relative humidity, the chloride binding properties, the cover depth of the concrete and the threshold concentration needed to depassivate the steel (Bertolini et al., 2004; Chryssanthopoulos and Sterritt, 2002). During the initiation phase, Fick's second law of diffusion is commonly used to model the ingress of chlorides into concrete (Bertolini et al., 2004). The factors influencing the propagation phase (once depassivation has occurred and corrosion of the steel begins) are the diffusion of moisture and oxygen into the steel, the electrical resistivity of the concrete, the chloride ion concentration of the pore solution and the temperature and humidity (Bertolini et al., 2004; Hussain and Rasheeduzzafar, 1994). The addition of cementitious replacement materials, such as PFA and GGBS can be used to improve the properties of concrete to slow down the initiation of corrosion of the reinforcing steel. This can lead to a more durable concrete, due to the slowing down of the ingress of chloride ions into the concrete due to a higher impermeability, and the increased chloride binding ability of the concrete. However, the effect of the addition of blended cements on the propagation phase of deterioration has not been previously studied. It has therefore been investigated in this thesis to allow the effect of these repair materials (i.e. OPC with the addition of PFA and GGBS) to be studied in relation to the initiation phase and the propagation phase of deterioration.

2.3.1. Chloride Binding

The addition of blended cements can affect the chemical processes which take place within the concrete in the presence of chlorides. One of these processes is chloride binding, which may have an influence on the corrosion rate of the reinforcing steel within the concrete. Therefore, the influence of blended cements such as PFA and GGBS on chloride binding is discussed in this section. Chlorides can be present in a concrete due to the addition of admixtures containing chloride ions (such as $CaCl_2$) during the mixing of fresh concrete, or due to the ingress of chloride ions into the hardened concrete from an external source (e.g. from de-icing salts or from a marine environment). These chloride ions can be present in the concrete matrix as free or bound ions. The free ions can diffuse further into the concrete towards the reinforcing steel which can lead to corrosion of the steel once the chloride concentration exceeds the threshold concentration. In addition, when chloride ions diffuse into the concrete and become bound, it is possible for these ions to subsequently detach, increasing the concentration of free chlorides that can diffuse towards the reinforcing bars (Neville, 2005). The proportion of free chloride ions in the pore fluid of the concrete mix is related to the corrosion rate of the reinforcing bars (Mangat and Molloy, 1995). In the case of this experimental study, chlorides are added to the concrete mix (as $CaCl_2$) and also ingress into the hardened concrete from the NaCl solution in the basin, where the concrete slabs are immersed, Figure 5.5.

The extent of chloride binding can be affected by the various constituents of the cement. In OPC it has been shown that the percentage of C_3A (which is shorthand for $Ca_3.Al_2O_6$, tricalcium aluminate) is a contributing factor to the binding capacity of the concrete mix. The chloride ions react with the C_3A in the mix to form Friedel's salt ($Ca_3.Al_2O_6.CaCl_2.10H_2O$), Equation 2.4 (Justnes, 1998). Therefore, the binding capacity of the concrete is related to the content of C_3A in the cement and the content of cement in the concrete mix (Neville, 2005). Hussain and Rasheeduzzafar (1994) investigated the pore solution composition of concrete with two types of OPC. One had a C_3A content of 2.43% and the other had a C_3A content of 14%. The results showed that the cement paste with 2.43% C_3A content had a chloride ion concentration 4.7 times that of the cement paste with 14% C_3A content. However, Byfors et al. (1986) indicated that chloride binding in OPC is not only related to the C_3A content in the cement, but also to the fineness of the cement and the original alkalinity of the cement. Therefore, the proportion of free chloride ions can be influenced by the cement type and proportion of cement used in the concrete mix. However, the effect of PFA and GGBS on the binding capacity of the concrete mix is also of interest in this case.

$$CaCl_2 + Ca_3.Al_2O_6 + 10H_2O \rightarrow Ca_3.Al_2O_6.CaCl_2.10H_2O \qquad \text{Equation 2.4}$$

The addition of PFA or GGBS can alter the binding capacity of the concrete. For a corresponding concrete of OPC alone, the addition of PFA or GGBS will lead to a reduction in the amount of cement in the mix which results in a lower percentage of C_3A. This would have the effect of reducing the chloride binding capacity of the mix, although overall, the addition of PFA or GGBS has the effect of increasing the percentage of bound chloride ions in the concrete cover (Bertolini et al., 2004; Dhir and Jones, 1999; Hussain and Rasheeduzzafar, 1994), therefore, reducing the rate of corrosion. However, it is not only the chloride ion concentration that dictates the corrosion rate of the reinforcing steel in the concrete. It has been suggested that the Cl^-/OH^- ratio in the pore solution is a more influencing parameter when considering the corrosion of steel in concrete (Bertolini et al., 2004). The addition of PFA has been shown to reduce the chloride ion concentration, but increase the Cl^-/OH^- concentration (Hussain and Rasheeduzzafar, 1994). However, the refinement of the pore structure and the reduction in the permeability due to the addition of the PFA outweighs the negative effects of the moderate increase in Cl^-/OH^- concentration (Hussain and Rasheeduzzafar, 1994).

When using the impressed current method of accelerated corrosion, the effects of binding are less evident, since the chloride ions are forced through the concrete by means of an external current. Therefore the effect of chloride binding has not been considered as part of this thesis. In addition, in this thesis some of the results are presented in terms of crack width against mass loss of reinforcing steel (Section 5.5.3 and Section 5.5.4), which are independent of the effects of chloride binding. In a natural environment, the addition of PFA or GGBS may lead to an increase or reduction in the corrosion rate due to an increased level of chloride binding, however, it is beyond the scope of this thesis to investigate the effect of chloride binding on the corrosion rate of reinforced concrete.

2.3.2. Carbonation

When considering the corrosion of the reinforcing bars in concrete, the effect of the ingress of carbon dioxide into the concrete cover must also be considered. Carbon dioxide can diffuse from the atmosphere into the concrete cover and react primarily with the calcium hydroxide, $Ca(OH)_2$. This can move through the concrete cover towards the reinforcing steel. Due to the reduction in the alkaline compound, calcium hydroxide, Equation 2.5 (Bertolini et al., 2004), the effect of carbonation is to reduce the pH of the pore solution, which can lead to the breakdown of the passive layer surrounding the concrete (Neville, 2005; Bertolini et al., 2004).

If present there can be an increase in the concentration of free chlorides in the pore solution, which can, with carbonation, lead to corrosion of the reinforcing steel. When the pH drops below a certain value, this can cause bound chlorides which have ingressed into the concrete to become released into the pore solution as free chloride ions. Therefore, in relation to the corrosion of reinforced concrete, carbonation can lead to a reduction in the initiation time and an increased corrosion rate. Therefore, the effect of the addition of PFA and GGBS on carbonation must also be considered. The main factors affecting carbonation are the diffusivity of the hardened cement paste (Neville, 2005) and the alkalinity of the cement paste (Bertolini et al., 2004). The rate of carbonation can also be influenced by environmental factors such as temperature, humidity and the concentration of CO_2 in the atmosphere (Bertolini et al., 2004).

$$CO_2 + Ca(OH)_2 \rightarrow CaCO_3 + H_2O \qquad \text{Equation 2.5}$$

PFA reacts with calcium hydroxide which leads to the formation of cementitious products. As a result, there is less calcium hydroxide available in the concrete to react with carbon dioxide which leads to a faster movement of the carbonation front through the concrete towards the reinforcing bars (Sulapha et al., 2003). However, due to the refined pore structure of PFA and GGBS, the diffusivity is reduced and carbonation can therefore be slowed down (Neville, 2005; Bertolini et al., 2004). The curing conditions can significantly affect the ability of PFA and GGBS concrete to resist carbonation. Inadequate curing speeds up the rate of carbonation (Neville, 2005; Bertolini et al., 2004). The carbonation rate depends on the fineness of the particles of PFA and GGBS. For PFA and GGBS with lower fineness, a higher rate of carbonation can be seen in comparison with OPC (Sulapha et al., 2003).

However, in the case of the experimental study carried out as part of this thesis, due to the short period of testing, carbonation was not an issue that needed to be considered when analyzing the results, although it is recognised that this may have an impact on the results over a longer period of testing. If adequate curing is carried out, the addition of PFA or GGBS will lead to a reduced carbonation rate in relation to OPC, which will in turn lead to a longer initiation time and a reduction in the corrosion rate. However, in the case of inadequate curing, this effect may be reversed.

2.3.3. Porosity and Permeability

The addition of blended cements also affects the porosity and permeability of concrete, which can in turn affect the corrosion rate. Therefore, the effect of PFA and GGBS on the porosity and permeability of concrete is discussed in this section. The porosity of a concrete is the ratio of the total volume of voids to the total volume of the concrete sample. The permeability, however, describes the ability of a liquid or gas to flow through the sample. Even if the porosity is high, the permeability can be low if the voids are not interconnected. The difference between porosity and permeability is illustrated in Figure 2.25. It has been widely recognised, that the addition of blended cements (e.g. PFA or GGBS) can reduce the permeability significantly due to the refined pore structure (McPolin et al., 2005; Neville, 2005; Bertolini et al., 2004; McCarthy et al., 2001; Hussain and Rasheeduzzafar, 1994; Dhir and Byars, 1993; Thomas, 1991), leading to a higher resistance to the diffusion of aggressive agents such as chlorides and CO_2 into the concrete.

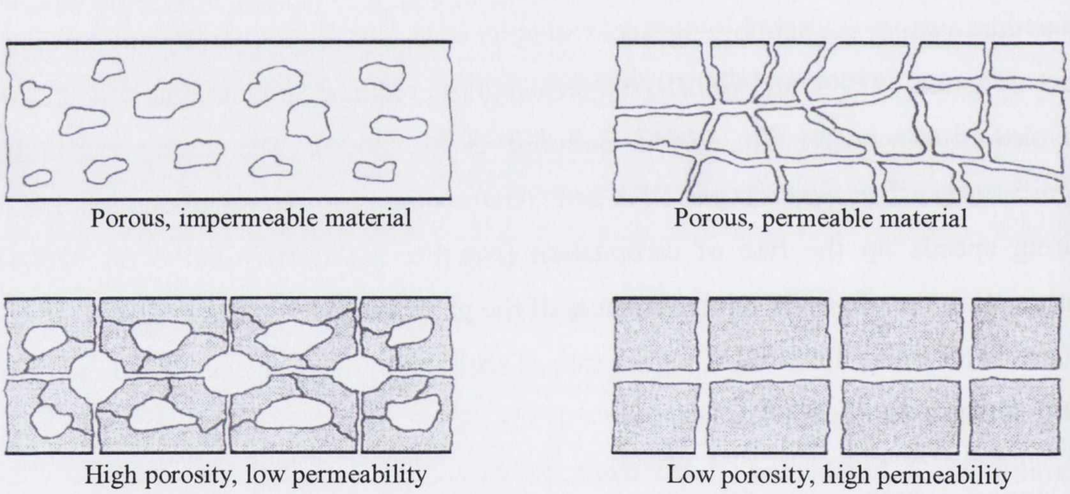


Figure 2.25. Distinction between porosity and permeability (Advanced Concrete Technology Lecture F3, 1999)

However, according to Sulapha et al. (2003), the cumulative pore volume of a OPC+GGBS concrete mix is less than a corresponding OPC mix and the cumulative pore volume of a OPC+PFA concrete mix is greater than a corresponding OPC mix. Therefore, although OPC+PFA has a higher porosity than OPC, it has a lower permeability. This is attributed to the fact that a PFA concrete has a finer pore structure, with more discontinuous pores (Hussain and Rasheeduzzafar, 1994; Dhir and Byars, 1993). Poon et al. (1999) also confirmed that the addition of PFA leads to an increase in the porosity of a cement paste, but leads to a smaller average pore size. Also, increasing the percentage of fly ash in the paste leads to an increase in the porosity and a reduction in the average pore diameter. These factors may affect the development of cracks in the concrete cover due to corrosion of the reinforcing bars. This will be discussed further in Section 5.5.

2.4. CONCLUSIONS

Based on the findings of the literature review, the areas of maintenance management which required further research were highlighted. Although a lot of research has been carried out in the area of BMS, none of these studies provide a method for a detailed complete bridge management system, including deterioration modelling, probabilistic modelling of NDTs and inspection results for the detection **and** sizing of defects, and costing models considering the inspection, repair and failure of structures or structural components. Therefore, it was decided to further develop an inspection based maintenance management model using a Markov process similar to existing BMS to simulate the deterioration and repair of components over time, considering inspection, repair and failure costs, thus providing the owner/manger of a structure with an optimum maintenance management strategy (i.e. optimum inspection interval, optimum combination of inspection techniques etc.) based on the specific constraints/requirements of the structure being considered. Similar to Higuchi and Macke (2007), and Macke and Higuchi (2007), this method is independent of material. By developing Markov transition matrices which can be tailored to laboratory or on site data it is possible to simulate many forms of deterioration.

The literature discussed above has demonstrated that a Markov based maintenance management system provides a rational framework for the prediction of deterioration and maintenance of a network of structures over time. In practice, many BMS use Markov chains to predict future deterioration and repair (Bakht and Mutsuyoshi, 2005; Rens et al., 2005; Adey et al., 2003; Frangopol et al., 2001; Czepiel, 1995). The reviewed studies have demonstrated the recent advances that have been made in the area of Markovian based maintenance management. However, there are areas which require further research and are within the scope of this thesis. Unlike Cesare et al. (1992) and Estes and Frangopol (2001), the possibility of a defect deteriorating more than one condition state over a one year period is considered as part of this thesis, to allow for the simulation of aggressive defect growth depending on the environment/deterioration mechanism under consideration. In addition, it was recognised by Roelfstra et al. (2004) that one of the limitations of using a Markov process to simulate deterioration is the inability to include an initiation period. On this basis, addressing this issue is one of the aims of this thesis. Since Kong and Frangopol (2005) considered the discretisation of deterioration into only four or five condition states to be a limitation of a Markov process, the range of possible deterioration is divided into 10 groups in this thesis to reduce the error due to discretisation. The sensitivity of the results to the chosen number of groups will also be studied. As in Corotis et al. (2005), it is assumed that the Markov process is stationary. Furthermore, the idea of partial inspections was highlighted in Mori and Ellingwood (1994a), where only a part of a structure or component was inspected. In this thesis, it is assumed that only hotspots or critical/specific locations on the structure are inspected, rather than the entire structure. However, it is recognised that the inspection strategy will be a function of the deterioration mechanism being considered. For example, considering uniform corrosion, using spatial variability the extent of deterioration at one point on a structure may be estimated based on the inspection results from another location on the structure (Rafiq et al., 2005a; Straub and Faber, 2002). In Mori and Ellingwood (1994b) it was concluded that the optimum inspection/repair intervals were almost uniform in all examples considered. Therefore, similar to Chung et al. (2006) and Chung et al. (2007), uniform/fixed inspection intervals (i.e. every ΔT years) are assumed in this thesis for practicality.

The studies which focused on reliability based maintenance management demonstrated the progress that has been made in the area in the last two decades, including the incorporation of multiple limit states, uncertainties, whole life costing, multiple failure modes etc. However, one area which requires further research is the incorporation of probabilistic inspection modelling into a maintenance management framework (Kong and Frangopol, 2004a; Kong and Frangopol, 2003), to provide an owner/manager of a structure with a rational and efficient method for selecting optimum inspection tools, based on the deterioration mechanism, limit state, environment etc. being considered. This issue is addressed as part of this thesis. The probability of repairs being carried out due to correct/incorrect inspection results is also considered. In addition, it was highlighted by Estes and Frangopol (2001) that many deterioration models which are being developed are expressed as a linear reduction in condition state over time. A linear deterioration rate was also assumed in Mori and Ellingwood (1994b). This limits the ability to consider many different materials and deterioration mechanisms. Therefore, in this thesis, a method is proposed to allow non-linear deterioration (e.g. defect growth rate varies with defect size) from inspections or on site data to be inputted into the maintenance management model.

Considering cost-benefit based maintenance management, the studies which were presented as part of this literature review highlighted the benefits of using a cost benefit analysis for the optimisation of maintenance interventions over the service life of a structure, based on the minimisation of expected costs. However, it was noted that in Higuchi and Macke (2007) and Macke and Higuchi (2007), when a structure was repaired, the deterioration characteristics remained the same, which implied that the repair material was the same as the construction material. This is considered to be a limitation of this methodology, since repairs of structures are often carried out using a material which is different to the construction material. Therefore, as part of this thesis, the ability to simulate the repair of a structure using a different material (i.e. with different deterioration characteristics) will be incorporated into the methodology. However, similar to other studies (Straub and Faber, 2005; Straub and Faber, 2004; Adey et al., 2003; Kong and Frangopol, 2003; Faber and Sorensen, 2002; Cesare et al., 1992), it was decided to assume repairs are perfect (i.e. component returns to the 'as new' condition following a repair) to reduce the number of possible outcomes.

In relation to inspection modelling within a BMS, there were many studies which focused on updating of inspection results and probabilistic inspection modelling. The importance of incorporating the ability to model the characteristics of an NDT was outlined in the literature since it is information from inspections (visual and NDT) that allows managers to estimate the condition of a structure, and predict future deterioration (Rens and Kim, 2007; Faber and Sorensen, 2002). Therefore, when developing a BMS it is important to incorporate the ability to process results from NDTs, and optimise the inspection effort (Rens and Kim, 2007; Rens et al., 2005). On this basis, probabilistic inspection modelling is incorporated into the maintenance management model which is developed as part of this thesis. It was recognised in the literature that there are two aspects to an inspection (Rouhan and Schoefs, 2003; Rouhan and Schoefs, 2000), and that there are two types of uncertainties where inspections are concerned, detection and sizing (Zhang and Mahadevan, 2000). Therefore, in this thesis the developed maintenance methodology includes a two stage inspection process to take into account the two aspects of an inspection, which are detection and sizing of defects, and also incorporates probabilistic inspection modelling of inspection quality and inspection results. This also allows owners/managers to tailor an optimum combination of inspection techniques for a particular structure or group of structures. In addition, it was highlighted that deterioration data from laboratory and on-site testing is essential to develop and update deterioration models (Chryssanthopoulos and Sterritt, 2002; Sterritt et al., 2002). To address this issue, and to demonstrate the capabilities of the developed maintenance management model, an experimental study is also carried out as part of this thesis to compare the efficiency of three repair materials. In this regard, some of the factors affecting the rate of deterioration in reinforced concrete were highlighted as part of the literature review. Using the results of this study, a practical example is carried out as part of this thesis to demonstrate the capabilities of the developed methodology.

In summary, following on from the literature review which was discussed in this chapter the objectives of this thesis are identified as:

1. The development of a maintenance management methodology which optimises inspection planning by taking into account the two aspects of an inspection, detection of a defect and sizing of a defect.
2. To incorporate the ability to consider many different forms of defect growth and different deterioration kinetics (i.e. abrupt and gradual growth, linear and non-linear).
3. To carry out an experimental study to investigate the rate of deterioration in different repair materials (OPC, OPC+PFA and OPC+GGBS) in relation to the rate of crack growth in reinforced concrete.
4. To incorporate the ability to simulate both the initiation phase and the propagation phase of deterioration using a Markov decision process.
5. To incorporate the ability to simulate the repair of a structure using a different material to the original construction material.

6. To demonstrate the capabilities of the developed maintenance management model using a practical example (i.e. using the results of the experimental study).
7. To perform sensitivity studies in the context of the derived methodology to investigate the effect of changes in the input parameters on the resulting optimal maintenance strategies.

CHAPTER 3 - THEORETICAL DEVELOPMENT

CHAPTER 3 - THEORETICAL DEVELOPMENT

3.1. INTRODUCTION

The main focus of this chapter is on the development of an inspection based decision scheme for the maintenance management of structures or groups of structures using Matlab. Originally, the two aspects of an inspection, namely detection and sizing are considered here to accurately simulate the inspection process carried out as part of a maintenance management strategy. Thus the task here is not to compare existing strategies but to suggest a new systematic approach that facilitates the owners/managers of structures or networks of structures in quantifying the cost of inspection and maintenance and in the optimisation of budgets. There have been many studies which focus only on the detection stage of an inspection, using various sets of parameters such as Probability of Detection and Probability of False Alarm (Schoefs and Clement, 2004; Rouhan and Schoefs, 2003), Probability of Detection and Probability of False Indications (Straub and Faber, 2003) or Probability of Detection and False Call Probability (Chung et al., 2006; Zhang and Mahadevan, 2001) to assess the quality of a particular inspection method.

In this thesis, it is considered important to make a distinction between an inspection carried out to detect a defect and an inspection carried out to size a defect. Since each stage of an inspection is carried out for a distinct purpose, different parameters are used to represent each procedure and both have been incorporated into the development of the maintenance management model. By separating these two procedures, an optimal maintenance management plan can be developed by choosing the most suitable inspection technique for each stage of the inspection, whether it is for detection or sizing, rather than using the same inspection technique for both procedures. This allows the owner/manager of a structure to choose the optimum inspection technique for each stage of the inspection, which results in a lower expected mean annual cost as well as a more efficient inspection process.

As part of the proposed methodology the first part of an inspection is concerned with the detection of existing defects (i.e. a screening process). The Probability of Detection (PoD) and the Probability of False Alarm (PFA) are used in this thesis in characterising a particular NDT tool used in the assessment to indicate the quality of the inspection method for detection. The second part of an inspection is taken to represent the assessment of the size/severity of the defect knowing that it has already been detected. For this part of the analysis, two new parameters are introduced, Probability of Good Assessment (PGA) and Probability of Wrong Assessment (PWA). In this context it has been necessary to introduce a distinction between good and wrong sizing assessments that lead to repair (PGA_R, PWA_R), and those which lead to no repair (PGA_{NR}, PWA_{NR}).

Using the methodology developed in Rouhan and Schoefs (2003), an events based decision theory is subsequently introduced to look at the effects of an individual good/bad inspection performance. Based on the inspection results, for detection or sizing, a decision is made on whether to carry out further inspection, or to repair. For evaluating the cost of the system, and to find the optimum costs, it is useful to investigate whether the decision to carry out a sizing assessment or a repair is correct/incorrect. On this basis, a decision scheme is introduced which considers four inspection events for each of the two stages of an inspection. The probability of these events evaluated using Bayes Theorem are subsequently introduced as parameters into cost functions which can be used to investigate the effect of cost overrun due to inaccurate inspection results.

3.2. PROBABILISTIC MODELLING OF INSPECTION RESULTS

When carrying out an inspection, the information obtained is just an estimation of what is present in reality. Therefore, due to this inherent uncertainty associated with inspections, many of the variables involved are modelled stochastically, and the simulation of inspection results is usually done in a probabilistic sense. Given the size/severity of the defect, and the inspection method being used, there is a certain probability of detection (Faber and Sorensen, 2002; Onoufriou and Frangopol, 2002; Madsen et al., 1987). On this basis, probabilistic methods are developed below which are used to model inspection results for detection and sizing assessment taking this uncertainty into account.

It is noted that the quantification of the on-site performance of inspections is difficult. Specific inter-calibration campaigns are needed, as initiated in the offshore field during the ICON project (Barnouin, 1993). Other recent works provide data for the probability of detection of the corrosion initiation in concrete (Bonnet et al., 2008), the probability of detection and false alarm for uniform (Schoefs et al., 2009; Schoefs et al., 2008) or localised (Pakrashi et al., 2008) corrosion of steel structures. Also, expert judgment can be introduced in this regard (Boéro et al., 2008).

3.2.1. Stage 1 - Detection

In the model it is assumed that an inspection is carried out every ΔT years. The first part of an inspection is concerned with the detection of existing defects. For an individual defect, it is assumed that detection of a defect by the first inspection leads to a further inspection to assess the size of the defect and that no detection leads to no further action. In this study, the Probability of Detection (PoD) and the Probability of False Alarm (PFA) are the parameters chosen to indicate the quality of an inspection method for detection and are used to assess if a defect will be detected or not when an inspection is carried out, Figure 3.1. The PoD is the probability that a defect is detected by the inspection, given that a defect is present, Equation 3.1, and the PFA is the probability that a defect is detected by the inspection, given that no defect greater than the detection threshold (d_{min}) is present, Equation 3.2. The symbol $\hat{d}_1$ represents the size of the detected defect from the first inspection, and d represents the actual size of the defect.

$$\mathrm{PoD} = \mathrm{P}\left(\hat{d}_1 \geq d_{min} \mid d \geq d_{min}\right) \quad \text{Equation 3.1}$$

$$\mathrm{PFA} = \mathrm{P}\left(\hat{d}_1 \geq d_{min} \mid d < d_{min}\right) \quad \text{Equation 3.2}$$

The results of an inspection (and the ability of a method to detect a defect) depend on many different factors, such as the NDT method, the detection threshold (d_{min}), the environment and several conditions of the structure such as the location (Breysse et al., 2007), the skill/experience of the operator, the characteristics of the defect (e.g. the deterioration mechanism such as chloride-induced corrosion, fatigue etc.) and primarily on the size of the defect. For a given test, the PoD depends on the defect size (for example the average defect size), the detection threshold and noise. The PFA, however, is independent of the size of the defect and, therefore, depends only on the detection threshold and noise, as illustrated in Figure 3.1.

Therefore, the PoD is the probability that the "signal+noise" is greater than the detection threshold, d_{min}, and the PFA is the probability that the signal "noise" is greater than the detection threshold (Rouhan and Schoefs, 2003). For the general form of this problem, two probability distributions are needed to model the results of an inspection. The "noise" distribution represents the error due to the inspection method, environmental conditions, human interference and the nature of what is being measured. The distribution of signal represents the physical uncertainty of the inspection, and the distribution of the population of defects at the time of inspection, Figure 3.1.

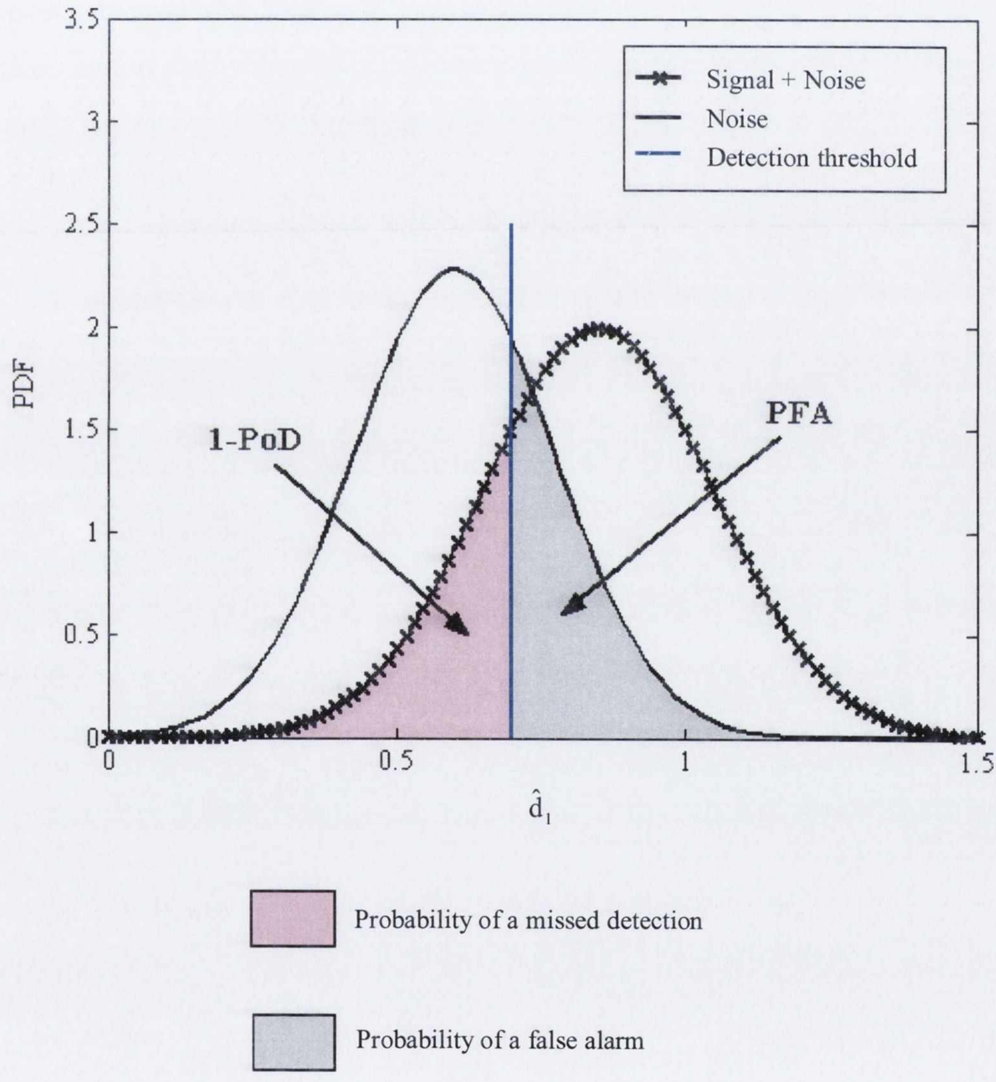


Figure 3.1. The effect of noise on inspection results (Rouhan and Schoefs, 2003)

3.2.2. Stage 2 - Sizing Assessment

The second part of an inspection, as considered in this work, deals with the assessment of the size/severity of a defect. This assessment is only carried out if the previous inspection has indicated that a defect exists. For this analysis, two new probabilities are defined in this thesis, the Probability of Good Assessment (PGA) and the Probability of Wrong Assessment (PWA). A repair of the defect is carried out if the inspection indicates that the size of the defect is greater than the critical defect size (d_c). The value of d_c will be fixed by the owner/manager, depending on the safety level he/she must/wants to ensure, and is inevitably limited by budgetary constraints. It can be, for instance, related to the annual probability of failure and would depend on the deterioration mechanism being considered. There is also a distinction made in this work between good and wrong assessments that lead to repair (subscript $_R$), and those which lead to no repair (subscript $_{NR}$), Equation 3.3-Equation 3.8. The symbol $\hat{d}_2$ represents the size of the detected defect from the second inspection and again d represents the actual size of the defect.

$$PGA_R = P\left(\hat{d}_2 \geq d_c \mid d \geq d_c \;\&\; \hat{d}_1 \geq d_{min}\right) \qquad \text{Equation 3.3}$$

$$PGA_{NR} = P\left(\hat{d}_2 < d_c \mid d < d_c \;\&\; \hat{d}_1 \geq d_{min}\right) \qquad \text{Equation 3.4}$$

$$PWA_R = P\left(\hat{d}_2 \geq d_c \mid d < d_c \;\&\; \hat{d}_1 \geq d_{min}\right) \qquad \text{Equation 3.5}$$

$$PWA_{NR} = P\left(\hat{d}_2 < d_c \mid d \geq d_c \;\&\; \hat{d}_1 \geq d_{min}\right) \qquad \text{Equation 3.6}$$

Note:

$$PGA_{NR} = 1 - PWA_R \qquad \text{Equation 3.7}$$

$$PWA_{NR} = 1 - PGA_R \qquad \text{Equation 3.8}$$

Again, for this inspection for sizing, the accuracy of the results can depend on many different factors and the noise can be due to effects of inspection quality, environmental conditions, human interference and the nature of what is being measured. In this case however, for a given inspection, both the PGA and the PWA depend on the defect size, the critical defect size and noise. Therefore, the inspection can be modelled using just one distribution, as shown in Figure 3.2, where $\overline{d}_i$ is the mean defect size within a group *i*.

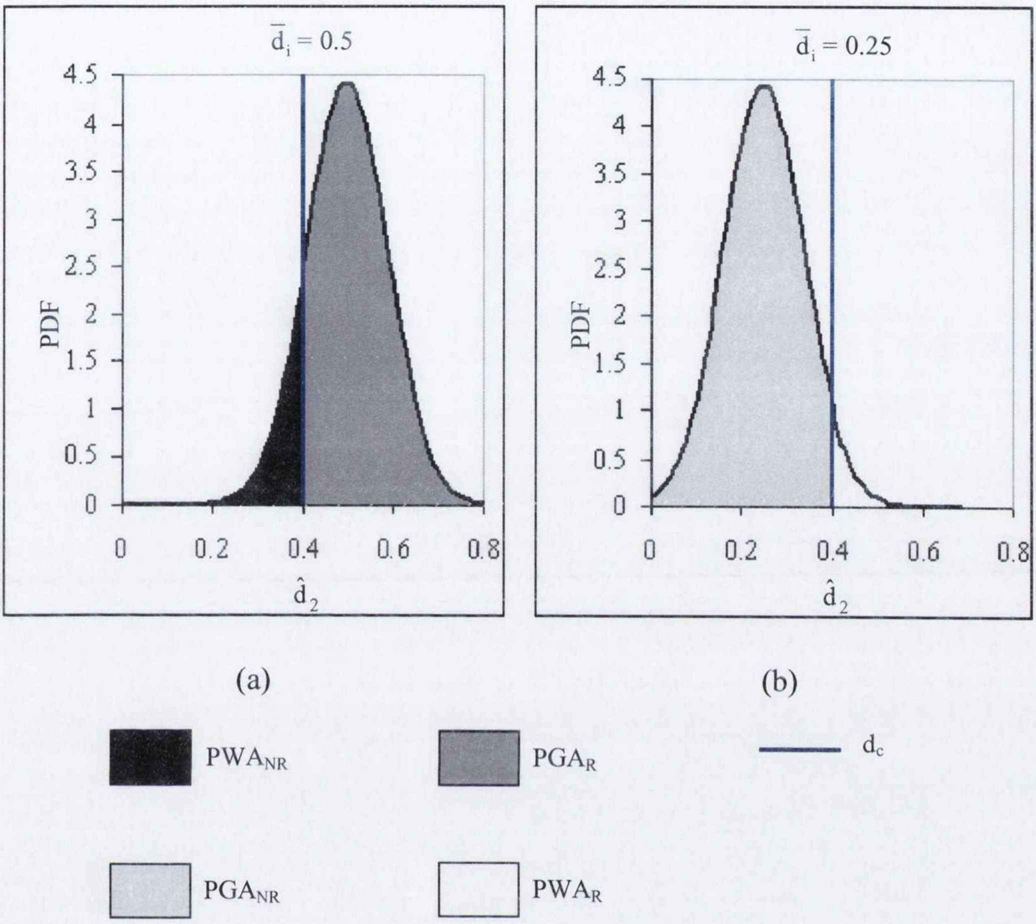


Figure 3.2. Example of the effect of noise on sizing inspection results ($d_c = 0.4$)

The PGA_R is the probability that the "signal+noise" is greater than the critical defect size (leading to repair), given that the actual defect is greater than the critical defect size, Figure 3.2(a) and the PGA_{NR} is the probability that the "signal+noise" is less than the critical defect size (leading to no repair), given that the defect is less than the critical defect size, Figure 3.2(b). Similarly, the PWA_R is the probability that the "signal+noise" is greater than the critical defect size (leading to repair), given that the actual defect is less than the critical defect size, Figure 3.2(b) and the PWA_{NR} is the probability that the "signal+noise" is less than the critical defect size (leading to no repair), given that the defect is greater than the critical defect size, Figure 3.2(a). The interaction between PGA_R, PWA_R, PGA_{NR}, PWA_{NR} and the critical defect size (d_c) is illustrated in Figure 3.3, where d represents the actual size of the defect.

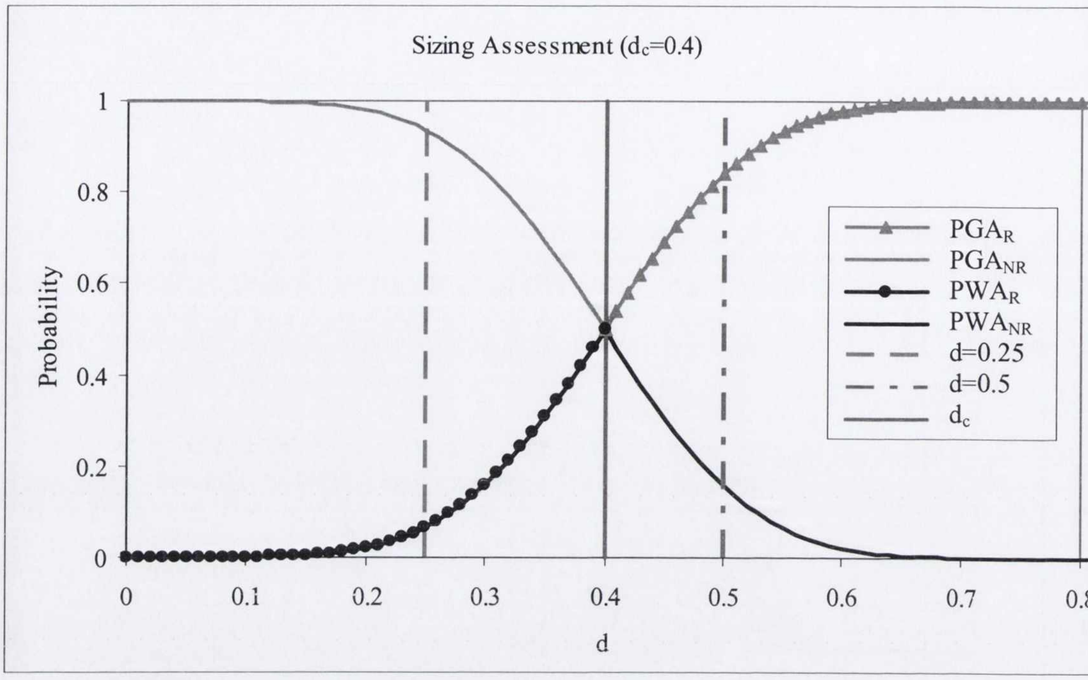


Figure 3.3. Example of the method used to model the sizing assessment of a defect

3.3. EVENTS BASED DECISION THEORY

As described in Rouhan and Schoefs (2003), an events based decision theory can be used to look at the effects of a good/bad inspection performance. Since there can be various sources of error when performing an inspection, it is useful to investigate the probability that each of the decisions taken (e.g. to carry out a further assessment for sizing or to repair) are correct/incorrect. In this thesis, this philosophy is incorporated into the overall methodology for detection and sizing, considering four inspection events for each of the two stages of an inspection. The probability of these events evaluated using Bayes Theorem are subsequently introduced as parameters into cost functions which are used to investigate the effect of cost overrun due to inaccurate inspection results.

3.3.1. Events at Stage 1 – Detection

Firstly, in the case of an inspection to detect a defect, a decision on whether to carry out a further assessment is made based on the inspection result $\hat{d}_1$. It is assumed that detection of a defect by the first inspection leads to a further inspection to assess the size of the defect, and that no detection leads to no further action. This decision on whether or not to carry out a further assessment can never be taken with certainty and the level of uncertainty depends on the quality of the inspection and the level of the other sources of noise associated with the inspection. To assess this risk, there are four events defined for the detection stage of an inspection, E_{1D}, E_{2D}, E_{3D} and E_{4D}. The question is, knowing that something is detected or not detected, what is the probability that there is a defect present or no defect present.

- E_{1D} is associated with a good decision, where the inspection indicates that there is no defect, when no defect greater than the detection threshold (d_{min}) actually exists, in which case no further sizing assessment is carried out, Equation 3.9.

$$P(E_{1D}) = P\left(d < d_{min} \middle| \hat{d}_1 < d_{min}\right) \quad \text{Equation 3.9}$$

- E_{2D} is associated with a bad decision, where the inspection indicates that there is a defect, when no defect greater than d_{min} actually exists, in which case an unnecessary sizing assessment is carried out, with an associated unnecessary inspection cost, Equation 3.10.

$$P(E_{2D}) = P\left(d < d_{min} \middle| \hat{d}_1 \geq d_{min}\right) \quad \text{Equation 3.10}$$

- E_{3D} is also associated with a bad decision, where the inspection indicates that there is no defect, when a defect greater than d_{min} actually exists, in which case no sizing assessment is carried out, but there is an associated failure risk cost, Equation 3.11.

$$P(E_{3D}) = P\left(d \geq d_{min} \middle| \hat{d}_1 < d_{min}\right) \quad \text{Equation 3.11}$$

- E_{4D} is associated with a good decision, where the inspection indicates that there is a defect, when a defect greater than d_{min} actually exists, in which case a necessary sizing assessment is carried out, resulting in an associated optimal resource allocation, Equation 3.12.

$$P(E_{4D}) = P\left(d \geq d_{min} \middle| \hat{d}_1 \geq d_{min}\right) \qquad \text{Equation 3.12}$$

The calculation of these probabilities is based on the PoD, PFA and a parameter γ, which is defined here as the probability that the actual defect size is greater than the detection threshold, Equation 3.13. The chosen values for γ can be based on prior knowledge regarding the deterioration of a similar group of structures. This process can be understood with reference to Figure 3.4.

$$\gamma = P(d \geq d_{min}) \qquad \text{Equation 3.13}$$

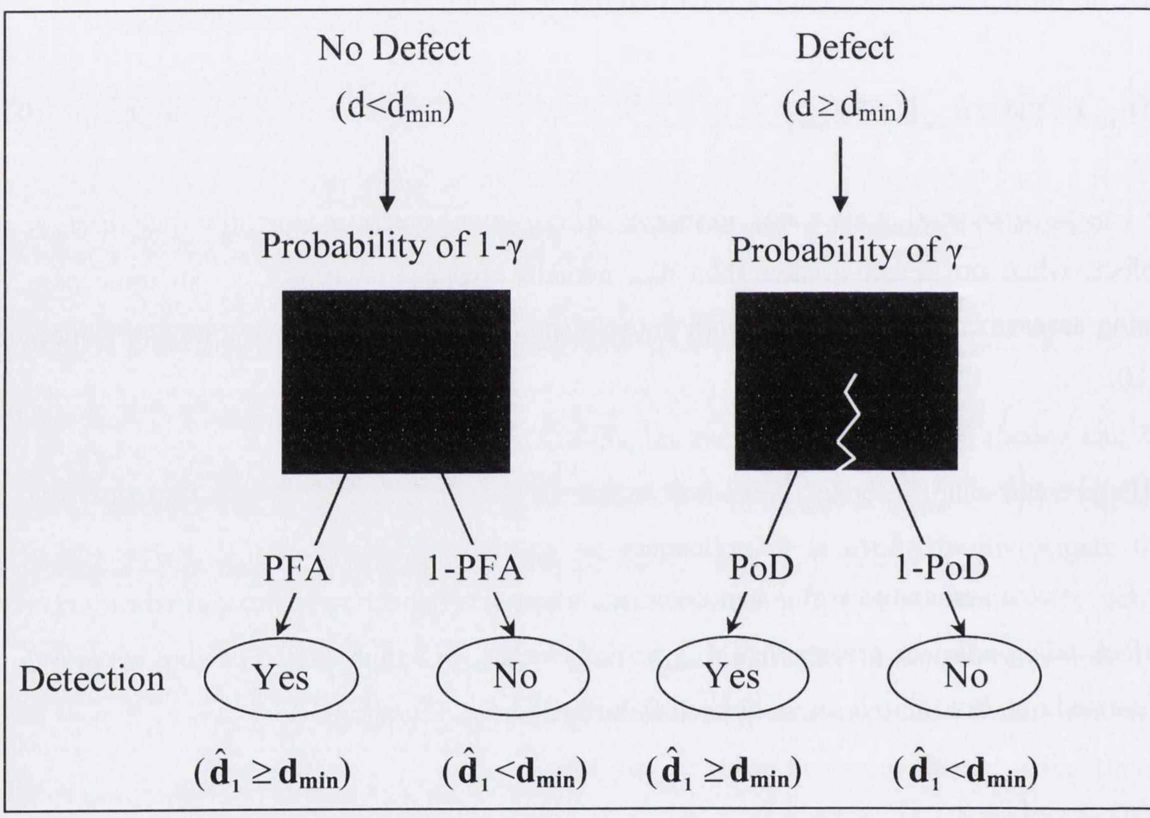


Figure 3.4. Inspection outcomes for detection

Calculation of the probabilities of these events is performed using Bayes Theorem, Equation 3.14-Equation 3.19.

$$P(E_{1D}) = \frac{(1 - PFA(d))(1-\gamma)}{(1 - PoD(d))\gamma + (1 - PFA(d))(1-\gamma)} \qquad \text{Equation 3.14}$$

$$P(E_{2D}) = \frac{PFA(d)(1-\gamma)}{PoD(d)\gamma + PFA(d)(1-\gamma)} \qquad \text{Equation 3.15}$$

$$P(E_{3D}) = \frac{(1-PoD(d))\gamma}{(1-PoD(d))\gamma + (1-PFA(d))(1-\gamma)} \qquad \text{Equation 3.16}$$

$$P(E_{4D}) = \frac{PoD(d)\gamma}{PoD(d)\gamma + PFA(d)(1-\gamma)} \qquad \text{Equation 3.17}$$

Note:

$$P(E_{1D}) + P(E_{3D}) = 1 \qquad \text{Equation 3.18}$$

$$P(E_{2D}) + P(E_{4D}) = 1 \qquad \text{Equation 3.19}$$

3.3.2. Events at Stage 2 – Sizing Assessment

For consistency in sizing assessment, the same methodology is employed. It is assumed that a repair is carried out if the size of the defect from the second inspection ($\hat{d}_2$) is larger than the critical defect size, d_c, and that no repair is carried out if the defect size is smaller than d_c. Again, this decision on whether or not to carry out a repair can never be taken with certainty. Therefore, four events are also defined for the sizing assessment stage of an inspection, E_{1A}, E_{2A}, E_{3A} and E_{4A}. Again, the question is, knowing that a defect has been sized greater than or less than the critical defect size and will be repaired or not repaired, what is the probability that it should have been repaired or not repaired.

- E_{1A} is associated with a good decision, where the inspection indicates that the defect is smaller in size than the critical defect size (d_c), when the size of the actual defect is less than d_c, in which case no repair is carried out, Equation 3.20.

$$P(E_{1A}) = P\left(d < d_c \middle| \hat{d}_2 < d_c\right) \qquad \text{Equation 3.20}$$

- E_{2A} is associated with a bad decision, where the inspection indicates that the defect is larger than d_c when the real defect is actually smaller than d_c, in which case an unnecessary repair is carried out, with an associated repair cost, Equation 3.21.

$$P(E_{2A}) = P\left(d < d_c \middle| \hat{d}_2 \geq d_c\right)$$ Equation 3.21

- E_{3A} is also associated with a bad decision, where the inspection indicates that the defect is smaller in size than d_c when the real defect is actually larger than d_c, in which case no repair is carried out, but there is an associated failure risk cost, Equation 3.22.

$$P(E_{3A}) = P\left(d \geq d_c \middle| \hat{d}_2 < d_c\right)$$ Equation 3.22

- E_{4A} is associated with a good decision, where the inspection indicates that the defect is larger in size than d_c when the real defect is actually larger than d_c, in which case a necessary repair is carried out, resulting in optimal repair resource allocation, Equation 3.23.

$$P(E_{4A}) = P\left(d \geq d_c \middle| \hat{d}_2 \geq d_c\right)$$ Equation 3.23

For the second stage of an inspection, the sizing assessment, the calculation of these probabilities is based on the PGA, PWA and a parameter λ, which is defined here as the probability that the size of the actual defect is greater than the specified critical defect size and as such requires repair, Equation 3.24. The chosen values for λ can be based on prior knowledge regarding the deterioration of a similar group of structures.

$$\lambda = P(d \geq d_c)$$ Equation 3.24

Again, the probabilities of these events are evaluated using Bayes Theorem, Equation 3.25-Equation 3.30.

$$P(E_{1A}) = \frac{PGA_{NR}(d)(1-\lambda)}{PWA_{NR}(d)\lambda + PGA_{NR}(d)(1-\lambda)}$$ Equation 3.25

$$P(E_{2A}) = \frac{PWA_R(d)(1-\lambda)}{PWA_R(d)(1-\lambda) + PGA_R(d)\lambda}$$ Equation 3.26

$$P(E_{3A}) = \frac{PWA_{NR}(d)\lambda}{PWA_{NR}(d)\lambda + PGA_{NR}(d)(1-\lambda)}$$ Equation 3.27

$$P(E_{4A}) = \frac{PGA_R(d)\lambda}{PWA_R(d)(1-\lambda) + PGA_R(d)\lambda} \quad \text{Equation 3.28}$$

Note:

$$P(E_{1A}) + P(E_{3A}) = 1 \quad \text{Equation 3.29}$$

$$P(E_{2A}) + P(E_{4A}) = 1 \quad \text{Equation 3.30}$$

3.4. DEVELOPMENT OF MAINTENANCE MANAGEMENT MODEL

Depending on the limit state being considered, the group of defects being inspected may be all on the same structure or at the same point on different structures. It is assumed, for this methodology, that the population of defects being inspected are all assessed under the same limit state. For example, when considering the serviceability limit state, there may be many points on one structure that require regular inspection and maintenance. In this case, for one structure alone there may be quite a large population of defects to be considered at each inspection interval (e.g. when crack width is the critical limit state). However, when considering the ultimate limit state, only the critical structural elements of the structure are of importance, meaning that only a few points on a structure need to be inspected and maintained on a regular basis (e.g. if moment capacity at mid span is the limit state being considered for a group of bridges, a zone around the mid span of each bridge will be inspected). Therefore, it is assumed that the law describing the probability of failure and the consequence of failure is the same for all defects within the population being considered in this model, where failure is defined as exceedance of a critical limit state. Similar to Scherer and Glagola (1994), it is assumed that all structures within a certain class have the same deterioration characteristics, and therefore, the same maintenance actions are also carried out on these structures.

When managing a structure or a group of structures it is important to be aware of and to have an accurate estimate of the growth of the population of defects present in the structure over time. Assuming that the state of the structure in each time period only depends on the state of the structure and the action applied to it in the preceding period (Ang and Tang, 1975), a Markov process may be employed to simulate the growth/evolving deterioration and repair of a population of defects over time (Roelfstra et al., 2004; Scherer and Glagola, 1994). A Markov decision process can be a useful tool for controlling and finding the optimal strategy when managing a large scale system (Orcesi and Crémona, 2006; Poinard et al., 2003; Micevski et al., 2002).

For the purpose of this assessment the total range of defect sizes is broken into defect groups, and a record is kept each year of the number of defects within each group, similar to Scherer and Glagola (1994). Based on the growth rate and the kinetics of the growth, the probability of moving from one defect group to a larger defect group is assessed. It is assumed that inspections are carried out every ΔT years, and when a repair is carried out the defect is repaired to the smallest defect group. Two Markov matrices are required (size NxN, where N is the number of defect groups), one to simulate the growth, repair and failure of the defects at an inspection year, and another to simulate the growth and failure of the defects between inspections (Corotis et al., 2005). It is assumed that defects return to the smallest group (i.e. to their initial size) after failure/repair. Therefore, at an inspection year, the first column in the matrix is controlled by the probability of repair and the probability of failure given that no repair is carried out, whereas, between inspections, this column is controlled by the probability of failure alone.

3.4.1. Simulation of the Growth of a Defect

The objective here is to develop the upper triangular part (growth part) of the Markov transition matrix of size NxN using the specified growth parameters. Therefore, given an initial population of defects in each group, the growth of the defects, and the movement of the defects into larger defect groups can be modelled over time.

When considering a Markov process, what is of interest is to calculate the probability that a defect moves from one group to a larger group. This probability is defined as P_{ij}, where P_{ij} is the probability that a defect will move from group i to group j in one time step, (Cesare et al., 1992; Ang and Tang, 1984). Also, in the transition matrix P_{ij} is the entry in the i^{th} row and the j^{th} column. Figure 3.5 illustrates an example (with five defect groups, N=5) of the process represented by the first row (i=1) of the growth transition matrix.

These entries therefore represent the probability that a defect will stay in group 1 (P_{11}) or the probability that a defect will move from group 1 to another group (P_{1j} for $j > 1$). For a finite number of groups the sum of these probabilities must equal 1 (Scherer and Glagola, 1994; Cesare et al., 1992). It is also possible for defects present in other groups to move to larger defect groups (e.g. the growth of a defect from group 2 to group 4), and this process is described by the other rows of the transition matrix.

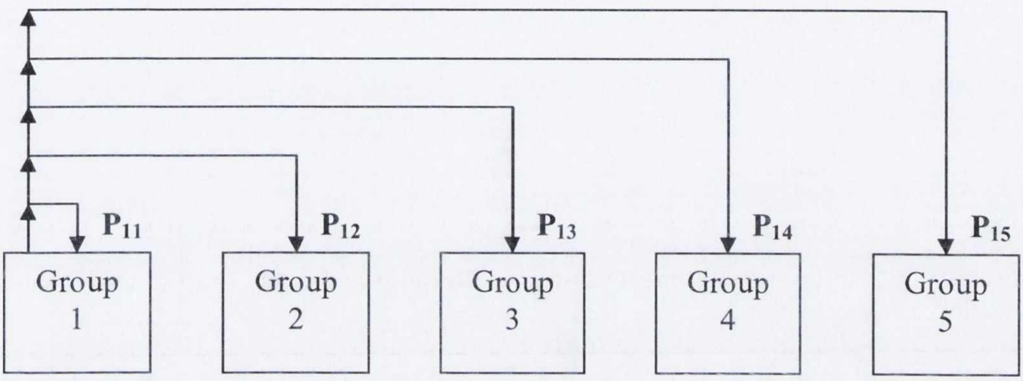


Figure 3.5. Schematic of growth process for Markov chain, for i=1

An example of a transition matrix describing the growth process followed by the defects (when considering ten defect groups, N=10) is illustrated in Figure 3.6. For the purpose of illustration all entries in the matrix are rounded to two decimal places. It is recognised that the sum of the entries in each row of the matrix must sum to 1.0, however, due to rounding errors, the sum of each row is not exactly 1.0 in the figures presented. Since a Markov process is employed, the upper triangular part of the matrix describes the growth of the defects, and the lower triangular part of the matrix describes repair or failure. At this stage, only the growth process is considered, so all entries in the lower diagonal part of the matrix (below the diagonal in bold in Figure 3.6) are 0. According to Roelfstra et al. (2004), one disadvantage of using a traditional Markov process to simulate the growth of a defect over time is the inability to incorporate an initiation phase into the deterioration. In this chapter of the thesis, only deterioration propagation is considered. Further development of this maintenance methodology is described in Chapter 6, where an initiation phase is incorporated into the deterioration process.

In this thesis, two new parameters are incorporated into the model which define the growth of a defect over time. The first is α, which describes the growth rate of a defect and therefore controls how quickly a defect moves from one defect group to another defect group. The other parameter is g, which determines how gradual or sudden the growth of an individual defect is. This parameter controls whether defects develop gradually and just move from one defect group to the next, or whether the growth of a defect is more abrupt, causing it to move from one group to a defect group a few sizes larger (rather than the one next to it). Therefore, this method is distinct from that described in Cesare et al. (1992), where a defect could move only from the group it was in to the group adjacent to it in one time step.

		To *j*									
		1	2	3	4	5	6	7	8	9	10
From *i*	1	**0.20**	0.28	0.14	0.09	0.07	0.06	0.05	0.04	0.04	0.03
	2		**0.20**	0.29	0.15	0.10	0.07	0.06	0.05	0.04	0.04
	3			**0.20**	0.31	0.15	0.10	0.08	0.06	0.05	0.04
	4				**0.20**	0.33	0.16	0.11	0.08	0.07	0.05
	5					**0.20**	0.35	0.18	0.12	0.09	0.07
	6						**0.20**	0.38	0.19	0.13	0.10
	7							**0.20**	0.44	0.22	0.15
	8								**0.20**	0.53	0.27
	9									**0.20**	0.80
	10										**1.00**

Figure 3.6. An example of a Markov matrix to simulate defect growth (α=0.8, g=1, N=10)

This allows many different forms of deterioration mechanism, which are associated with different environments and materials, to be simulated using this approach. Depending on the limit state being considered, the owner/manager of a structure will be concerned with different forms of deterioration (e.g. in relation to SLS, the owner/manage may be concerned with crack growth due to reinforced concrete corrosion in concrete structures), and based on field data or experimental results in the laboratory, these parameters can be estimated to predict the deterioration of a structure over time. Therefore, both parameters are used to calculate the entries in the Markov matrix which simulates growth only, P_{ij_GROWTH} (as subsequently described in Equation 3.32). For this thesis it was chosen to divide the defect growth into 10 groups. This value was chosen to correspond with the Federal Highways Agency inspection recommendations in the US, where each element in a structure is assigned one of 10 of the National Bridge Inventory Condition Ratings (Estes and Frangopol, 2001). The sensitivity of the number of groups chosen to the results of the model is addressed in Chapter 4.

A higher growth rate means that there is a lower probability that the defect will remain in the same defect group after each time step and a lower growth rate increases the likelihood that it will stay in the same group after the same period of time. For example, crack growth in a reinforced concrete structure in an aggressive marine environment is more likely to develop at a faster rate than a structure inland where the exposure to chlorides is minimal, and will therefore be more likely to move to a larger defect group within a certain time interval. To model this behaviour, the diagonal of the transition matrix is controlled by $(1-\alpha)$ alone, for $(0<\alpha<1)$. Each row in the transition matrix must sum to 1.0. Therefore, the remaining portion of the probabilities (α) must be distributed between the other groups (or the cells in that row of the matrix). The ratio by which they are divided is controlled by g, as g is used to describe the growth kinetics of the defects.

In Figure 3.6, α is equal to 0.8 (a fast growth rate), therefore the diagonal entries of the matrix are 0.2 ($P_{ii} = (1-\alpha)$). The remaining portion of probabilities (P_{ij}), which sum to α, must be divided between all other groups where $j > i$. This process is controlled by g. For a high value of g, (e.g. g=5), the growth of an individual defect is modelled as smooth and gradual, Figure 3.7. This figure represents the realisation of the deterioration and repair of a defect with gradual growth over time.

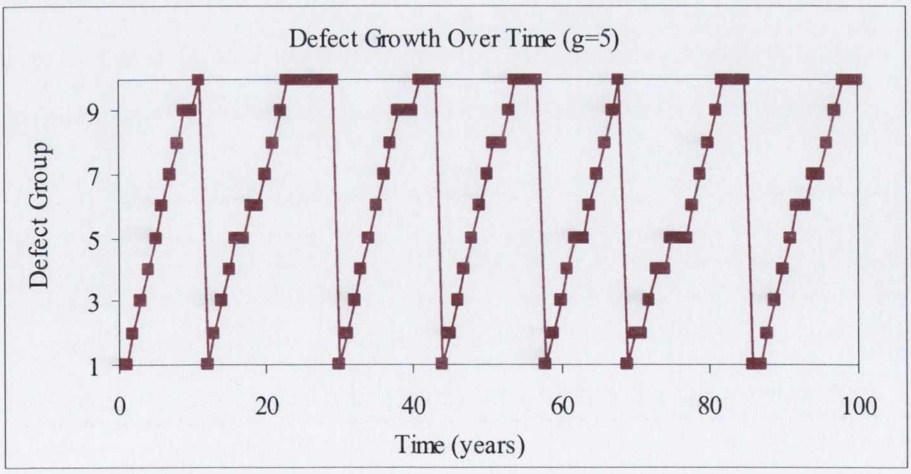


Figure 3.7. Gradual defect growth of an individual defect (g=5)

This form of growth would correspond to a deterioration process such as carbonation or chloride-induced corrosion in reinforced concrete as these processes develop gradually over time. In this case, the defect stays in the same group or moves to the next defect group, and does not readily skip groups by growing very suddenly in one time step (i.e. $P_{i,i+1} \approx \alpha$, and $P_{ij} \approx 0$ (for $j > i+1$)). An example of the transition matrix describing this behaviour is shown in Figure 3.8.

Lower values of g are used in the model describe more abrupt growth, with P_{ij} (for $j > i$) more evenly spread, increasing the likelihood that a defect jumps straight from the group it is in to a much larger defect group, skipping the groups adjacent to it. This allows many forms of deterioration mechanism to be simulated using the developed methodology.

From *i* \ **To *j***	1	2	3	4	5	6	7	8	9	10
1	**0.20**	0.78	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2		**0.20**	0.78	0.02	0.00	0.00	0.00	0.00	0.00	0.00
3			**0.20**	0.78	0.02	0.00	0.00	0.00	0.00	0.00
4				**0.20**	0.78	0.02	0.00	0.00	0.00	0.00
5					**0.20**	0.78	0.02	0.00	0.00	0.00
6						**0.20**	0.78	0.02	0.00	0.00
7							**0.20**	0.78	0.02	0.00
8								**0.20**	0.78	0.02
9									**0.20**	0.80
10										**1.00**

Figure 3.8. An example of a Markov matrix to simulate defect growth (α=0.8, g=5, N=10)

To calculate the values of P_{ij_GROWTH} in the transition matrix using α and g, another matrix G is created, Equation 3.31. This is an inverse power equation which is used to calculate the values in the upper triangular portion (growth part) of the matrix. This model was chosen as it allows values of g=0 and negative values of g to be modelled, resulting in an increased versatility to simulate different forms of deterioration mechanism. For example, gradual growth simulated by g=5 may represent deterioration due to steel corrosion or carbonation in reinforced concrete, whereas very abrupt growth simulated by g=-5 may represent deterioration (e.g. crack growth) due to the effect of overloading. For an intermediate form of growth kinetics (e.g. cracking due to SLS loading in reinforced concrete which subsequently leads to reinforcement corrosion), a value of g=0 may be used.

$$G_{ij} = \begin{cases} \dfrac{1}{(j-i)^{g}} & \text{for } j > i \\ 0 & \text{for } j \leq i \end{cases} \qquad \text{Equation 3.31}$$

From this matrix, the values of the transition matrix are calculated using Equation 3.32,

$$P_{ij\_GROWTH} = \frac{(\alpha)(G_{ij})}{\sum G_i} \qquad \text{for } j > i \qquad \text{Equation 3.32}$$

Using this method, it is therefore possible to input a value of 0, or a negative value for g. In the special case where g = 0, α is divided equally between all other groups, for $j > i$, Figure 3.9. Therefore, for a defect in group 1, for example, with a growth rate of 0.8 (a fast growth rate), the probability of the defect increasing in size and moving to any other group is 0.8, and this probability is divided equally between all other groups, meaning that the probability of the defect moving to each of the groups is equal, Figure 3.9. Figure 3.10 represents the realisation of the deterioration and repair of a defect with abrupt growth (g=0) over time.

From *i* \ **To *j***	1	2	3	4	5	6	7	8	9	10
1	**0.20**	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
2		**0.20**	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10
3			**0.20**	0.11	0.11	0.11	0.11	0.11	0.11	0.11
4				**0.20**	0.13	0.13	0.13	0.13	0.13	0.13
5					**0.20**	0.16	0.16	0.16	0.16	0.16
6						**0.20**	0.20	0.20	0.20	0.20
7							**0.20**	0.27	0.27	0.27
8								**0.20**	0.40	0.40
9									**0.20**	0.80
10										**1.00**

Figure 3.9. An example of a Markov matrix to simulate defect growth (α=0.8, g=0, N=10)

Similarly, when g is negative, the growth kinetics of a defect are different. For a negative value of g, when growth of a defect occurs, there is a higher probability that it will skip over the defect groups next to it and move to larger groups. This results in very abrupt growth, Figure 3.11. Figure 3.12 represents the realisation of the deterioration and repair of a defect with very abrupt growth (g=-5) over time.

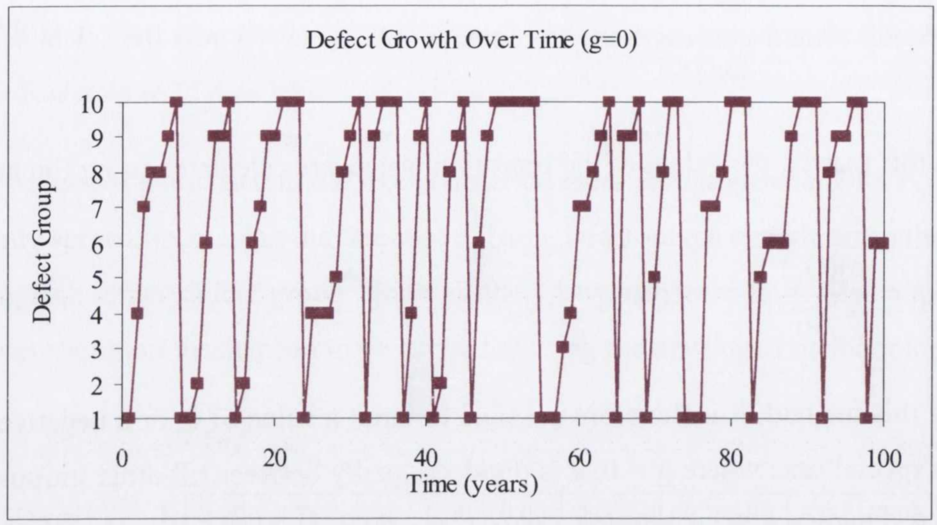


Figure 3.10. Abrupt defect growth of an individual defect (g=0)

Thus far, only deterioration has been considered. To consider maintenance management of a population of structural components, inspections, repair and failure events need to be incorporated into this model.

		To *j*									
		1	2	3	4	5	6	7	8	9	10
From *i*	1	**0.20**	0.00	0.00	0.00	0.01	0.02	0.05	0.11	0.22	0.39
	2		**0.20**	0.00	0.00	0.00	0.01	0.04	0.10	0.22	0.43
	3			**0.20**	0.00	0.00	0.01	0.03	0.09	0.21	0.46
	4				**0.20**	0.00	0.00	0.02	0.07	0.20	0.51
	5					**0.20**	0.00	0.01	0.04	0.19	0.56
	6						**0.20**	0.00	0.02	0.15	0.63
	7							**0.20**	0.00	0.09	0.71
	8								**0.20**	0.02	0.78
	9									**0.20**	0.80
	10										**1.00**

Figure 3.11. An example of a Markov matrix to simulate defect growth (α=0.8, g=-5, N=10)

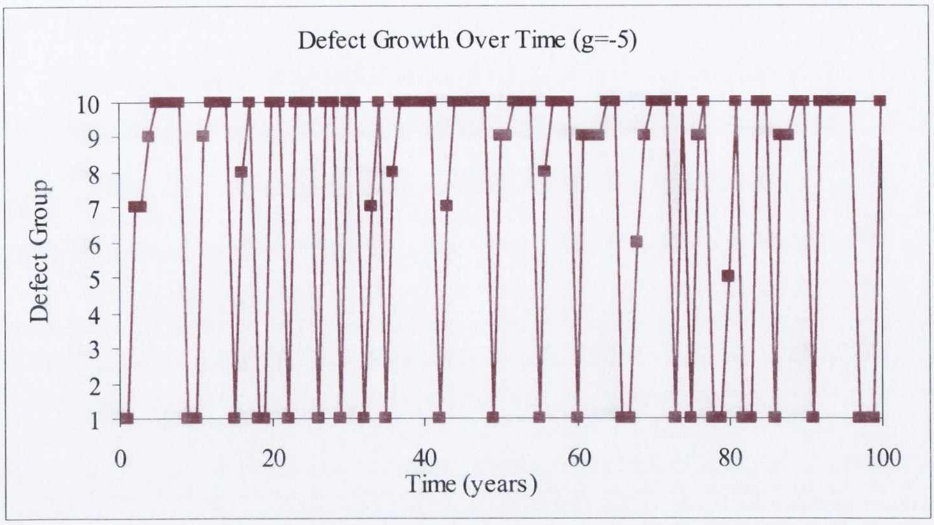


Figure 3.12. Very abrupt defect growth of an individual defect (g=-5)

3.4.2. Simulation of Failure between Inspections

If a defect continues to grow, without any repairs being carried out, failure will eventually occur. Therefore, failure must be simulated between inspections. For each defect group, the probability of failure is calculated to assess the probability that a defect will fail and subsequently be repaired, and will therefore return to the smallest defect group (i.e. P_{i1}). It was decided here to calculate the annual probability of failure, p_F ,using the Weibull cumulative distribution function (Ang and Tang, 1984; Weibull, 1951), based on the mean size of the defects in each group, $\overline{d}_i$, Equation 3.33. In Chapter 4, the sensitivity of the results of the maintenance management model to changes in the parameters of the Weibull distribution is discussed. The parameters chosen depend on the limit state and the mode of failure being considered by the owner/manager.

$$p_F(d) = 1 - \left[\exp - \left(\frac{\overline{d}_i - d_l}{d_{ref\_pf}} \right)^m \right] \qquad \text{Equation 3.33}$$

The purpose of the limit defect size, d_l, is described by the Equation 3.34 and Figure 3.13. When considering the time between inspections, there is no chance of repair being carried out before failure (where failure occurs between inspection events). Therefore, using Equation 3.33, the probability of failure for each group (with mean defect size $\overline{d}_i$) is calculated, and then used to determine the values for the first column in the Markov matrix which simulates the behaviour of a population of defects between inspections.

$$p_F = \begin{cases} 0 & d \leq d_l \\ p_F(d) & d > d_l \end{cases}$$ Equation 3.34

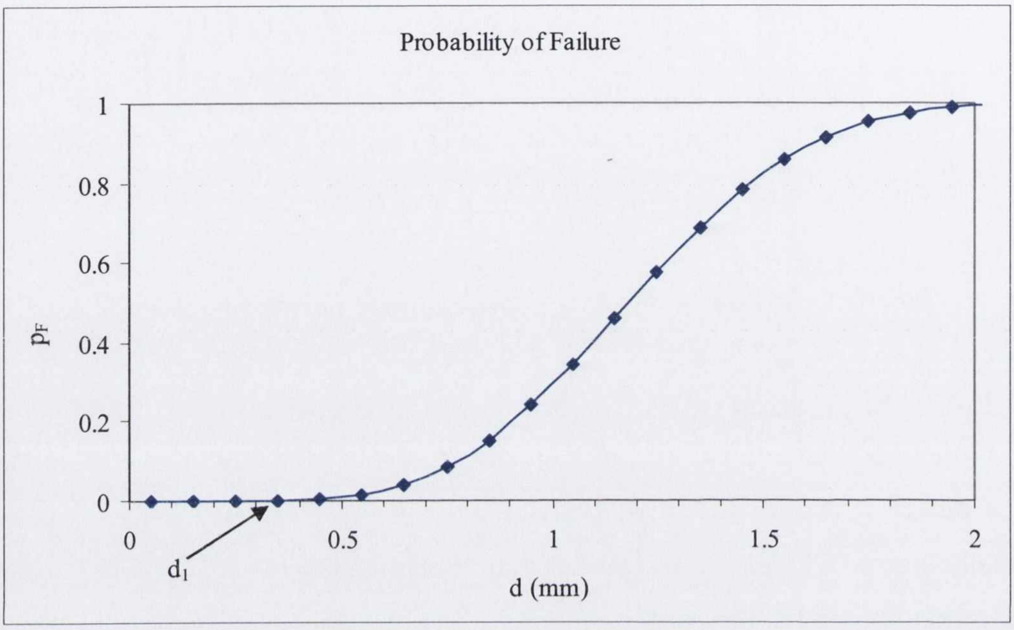


Figure 3.13. The increase in probability of failure with increasing defect size for $d>d_l$

3.4.3. Simulation of Repair and Failure at an Inspection Year

Over the lifetime of a structure, inspections and repairs are carried out and in some cases failure can occur, where failure is defined as the probability of exceeding a prescribed limit state. The limit state may be an ultimate limit state (e.g. structural collapse due to moment or shear) or serviceability limit state (e.g. failure due to excessive deflection). However, at an inspection year it is assumed that failure will only occur if a repair is not carried out or is postponed. This failure can be due to two outcomes. In the first case a defect is not detected, which can result in failure if the defect size is greater than the limit defect size, d_l. The second case is when a defect is detected but the sizing assessment indicates that the defect is less than the critical defect size and it is therefore not repaired. This may also result in failure.

In this case P_{i1} is calculated using a combination of the probability of repair and the probability of failure given that no assessment/repair has been carried out. Using the parameters associated with the inspection techniques and the failure of the components, the probability of repair and the probability of failure are calculated for each group. These values, along with the mean size and standard deviation of the defects in each group, are then used in the calculation of the P_{i1} column for the additional Markov matrix generated to simulate the behaviour of a population of defects at an inspection year.

3.4.3.1. Detection and Sizing Assessment

As outlined, it is assumed that a repair is only carried out if a defect is detected and is found to be greater than the critical defect size when the sizing assessment is carried out. Therefore, to calculate the probability of repair of defects in each group it is necessary to assess the PoD/PFA and PGA/PWA for each defect group.

The PoD and PFA are estimated for each defect group, as per Equation 3.36 and Equation 3.37, given the mean and standard deviation of the size of the defects in the group and the quality of the inspection method which is used, Q_1. It is assumed that the defect size and the noise are normally distributed and non-correlated and that for detection the quality of the inspection method (and hence cost) is inversely proportional to the distribution of the noise, σ_{ND}, Equation 3.35,

$$\frac{\sigma_{ND}}{d_{ref}} = \frac{1}{Q_1} \qquad \text{Equation 3.35}$$

The mean value of the noise is assumed to depend on environmental conditions (e.g. marine environment or inland) and human interference (e.g. experience of inspector), and is assumed to be the same for each defect group. Therefore, the PoD_i and PFA_i are estimated for each group, *i*, using Equation 3.36 and Equation 3.37.

$$PoD_i = \Phi\left(\frac{\bar{s}_i - d_{min}}{\sqrt{\sigma_d^2 + \sigma_{ND}^2}}\right) \qquad \text{Equation 3.36}$$

$$PFA_i = \Phi\left(\frac{\bar{n} - d_{min}}{\sigma_{ND}}\right) \qquad \text{Equation 3.37}$$

As discussed in Section 3.2.1, the PoD is assumed to depend on the mean of the "signal+noise" distribution ($\overline{s}_i$), the standard deviation of the distribution of defect sizes (σ_d), the standard deviation of the noise (σ_{ND}) and the detection threshold (d_{min}) of the inspection method, where $\overline{s}_i$ is the sum of the mean defect size for each group ($\overline{d}_i$) and the mean of the noise distribution ($\overline{n}$). The PFA is assumed to depend on the mean and standard deviation of the noise ($\overline{n}$ and σ_{ND}, respectively), and the detection threshold of the inspection method. Similar to Zhang and Mahadevan (2001), for the purpose of this thesis a cumulative normal distribution is used to model the PoD and PFA. Although it is recognised that other distributions may also be used. Figure 3.14 and Figure 3.15 illustrate the sensitivity of the PoD to $\overline{s}_i$ and the detection threshold of the inspection method (for assumed values of σ_{ND}=0.1, σ_d=0.05 and $\overline{n}$=0.1).

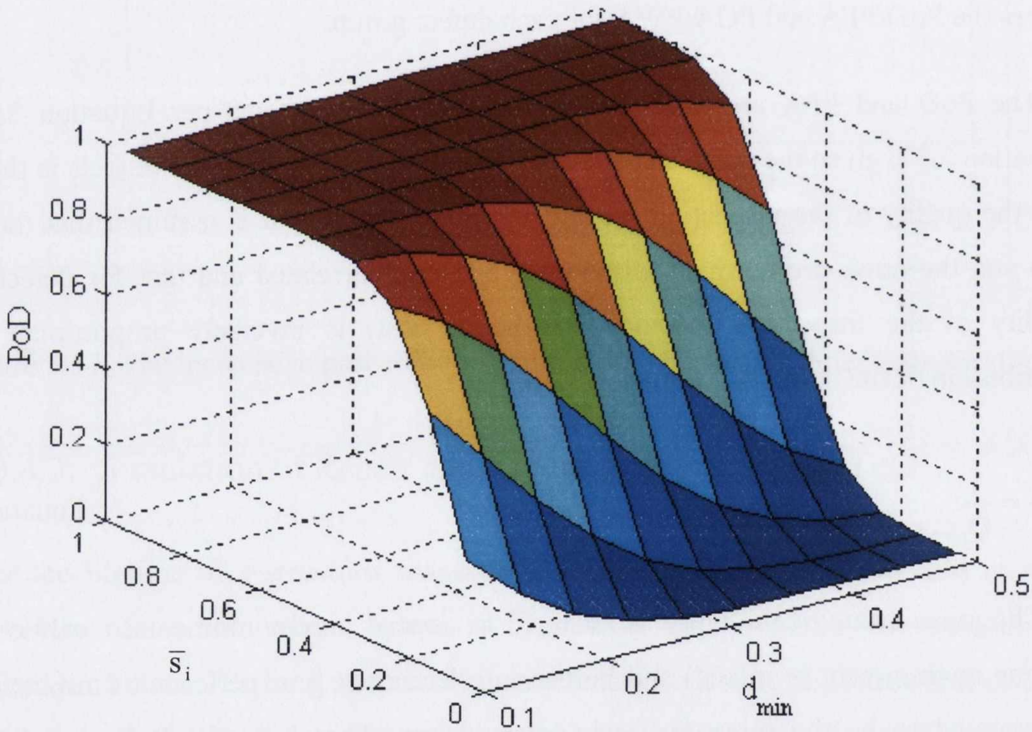


Figure 3.14. Sensitivity of PoD to variations in the signal mean and d_{min}

As previously described in Section 3.2.1, the PoD is the probability that the "signal+noise" is greater than the detection threshold. Figure 3.14 illustrates that as the mean value of the signal increases (e.g. a larger crack width), the PoD increases since the probability that the "signal+noise" is greater than the detection threshold increases.

Similarly, for a specific value of signal mean (e.g. for a given crack width), increasing the detection threshold of the inspection technique results in a lower PoD.

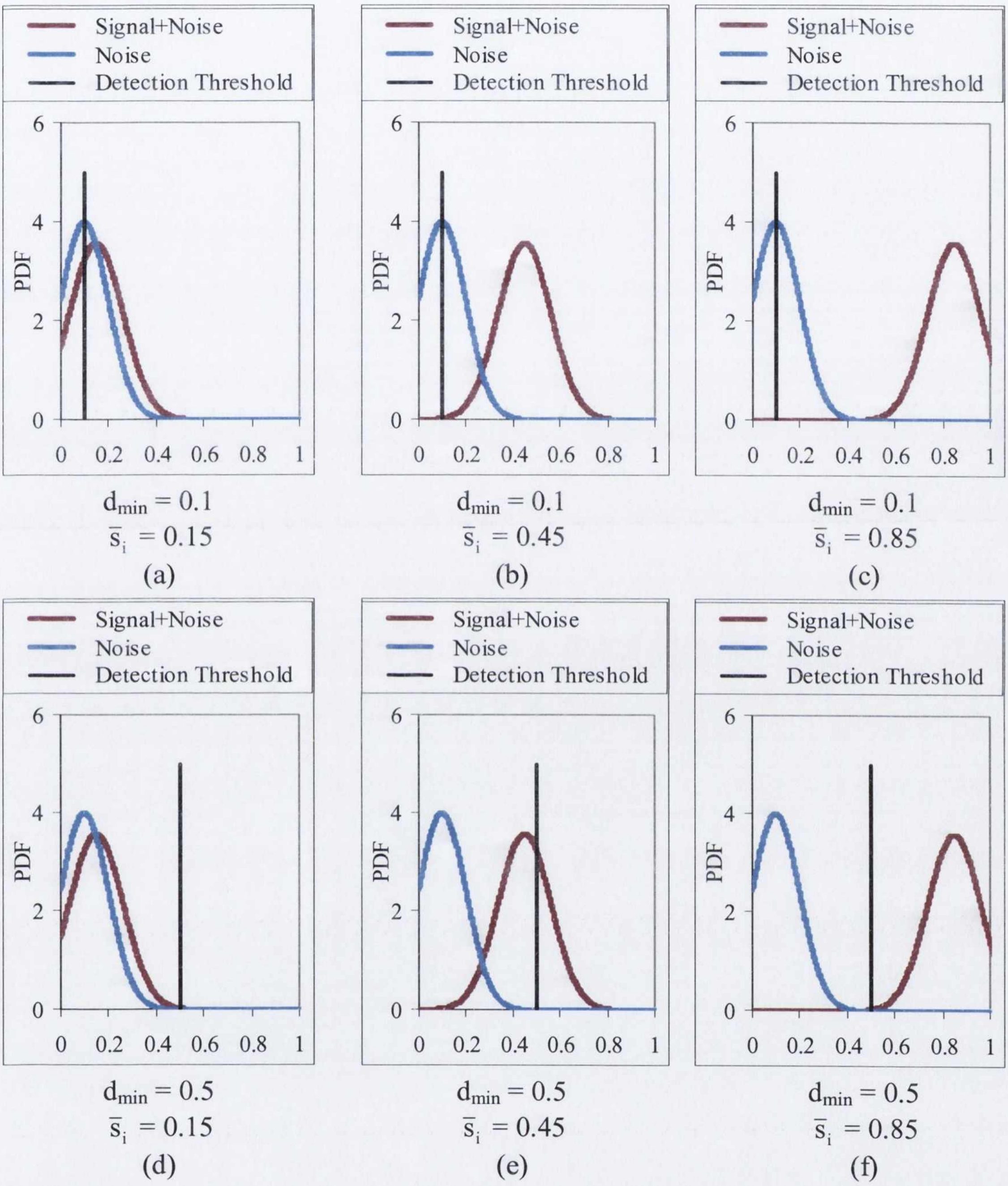


Figure 3.15. Illustration of variation in PoD for different values of signal mean and d_{min}

For the extreme values of d_{min} considered in Figure 3.14 (where d_{min} varies between 0.1 and 0.5), the variation in the distributions of "signal+noise" and "noise" for different values of signal mean is illustrated in Figure 3.15. The "noise" distribution does not change as it is a function of the mean value and the standard deviation of the noise distribution, which are both constant in this example. For a value of d_{min}=0.1, as the signal mean increases, the mean value of the "signal+noise" distribution increases and it moves to the right. For a mean "signal+noise" of 0.45 (case b) and 0.85 (case c), the PoD is 1.0, but for a mean "signal+noise" of 0.15 (case a) it is just 0.7. However, as the detection threshold increases the PoD decreases for a mean "signal+noise" of 0.15 (case d) and 0.45 (case e) to 0 and 0.3, respectively, while the PoD remains at 1.0 for a mean "signal+noise" of 0.85 (case f). These figures illustrate the sensitivity of the PoD to the size of the defect being inspected and the chosen detection threshold of the inspection technique being used. It is also important, however, to consider the effect of noise (e.g. environmental noise and skill of the operator) on the inspection results. On this basis, Figure 3.16 and Figure 3.17, illustrate the sensitivity of the PFA to the mean value of the noise distribution and the detection threshold of the inspection method (for assumed values of σ_{ND}=0.1, σ_d=0.05 and $\bar{d}_i = 0.25$).

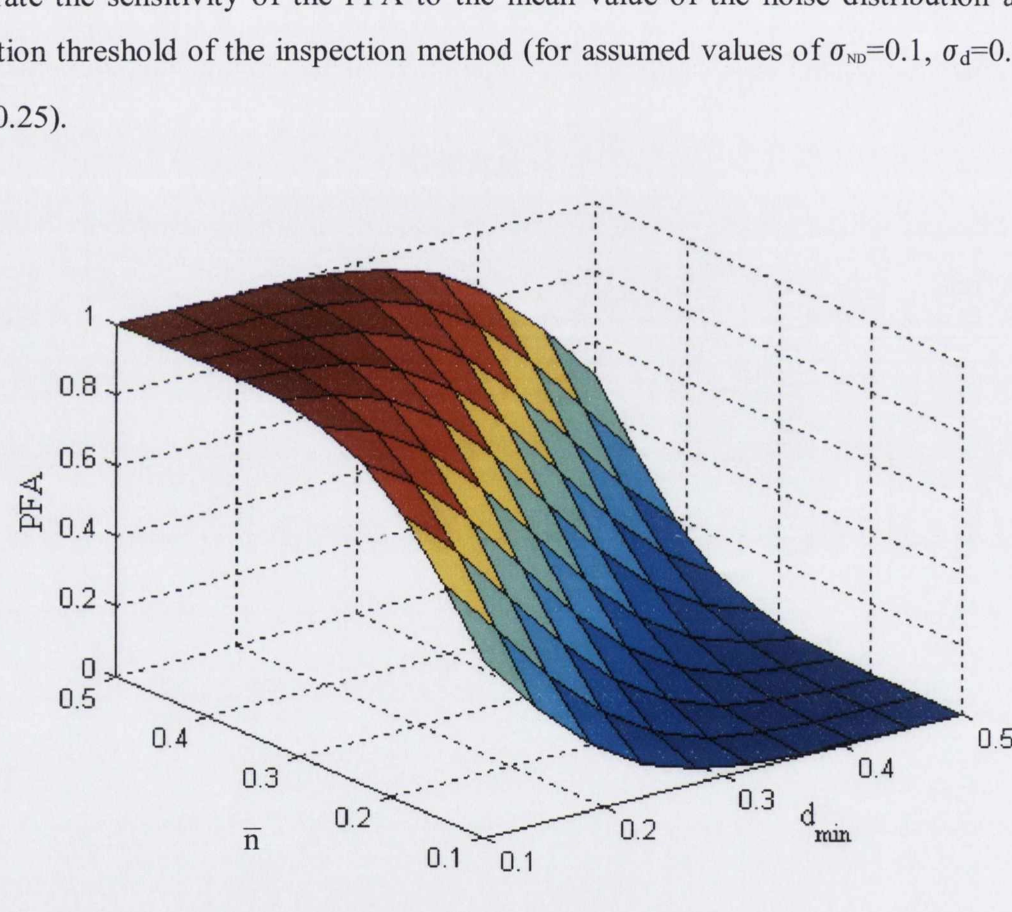
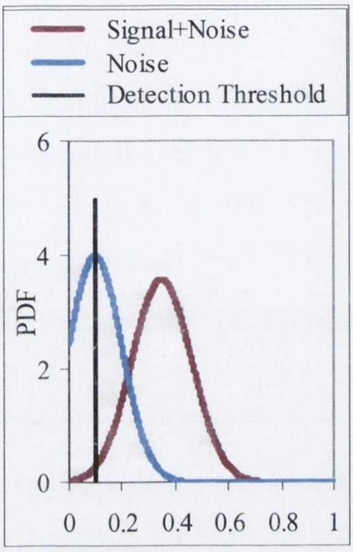
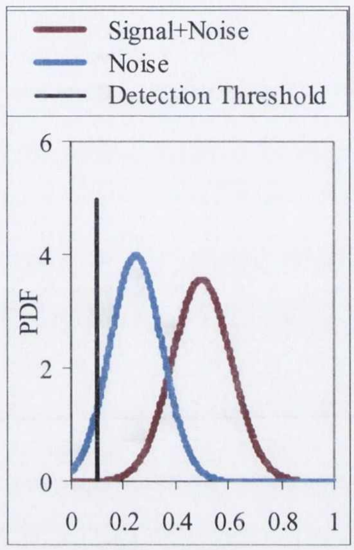


Figure 3.16. Sensitivity of PFA to variations in the mean value of the noise distribution and d_{min}

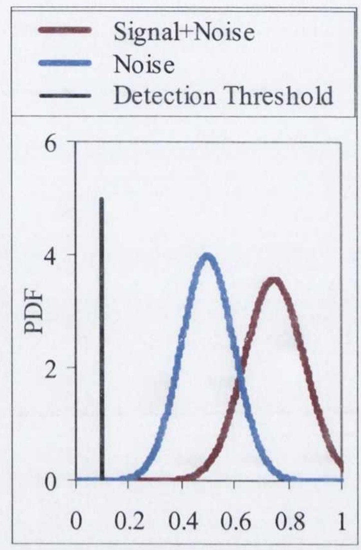
Figure 3.16 illustrates that as the mean value of the noise distribution increases (e.g. as the environmental noise becomes more severe, like an inspection under water), the PFA increases since the probability that the "noise" is greater than the detection threshold increases. In addition, for a specific value of the mean of the noise distribution, increasing the detection threshold of the inspection technique results in a lower PFA. Again, for the extreme values of d_{min} considered in Figure 3.16 (where d_{min} varies between 0.1 and 0.5), the variation in the distribution of "noise" for different values of the mean of the noise distribution is illustrated in Figure 3.17.



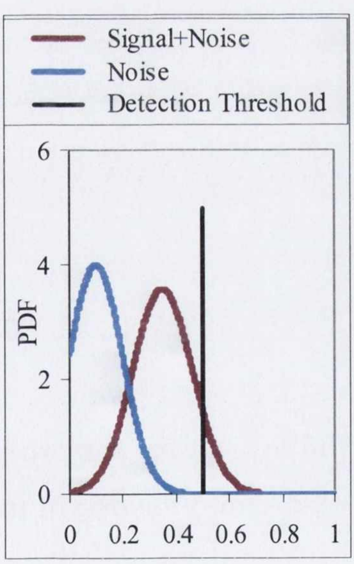
$d_{min} = 0.1$
$\bar{n} = 0.1$
(a)



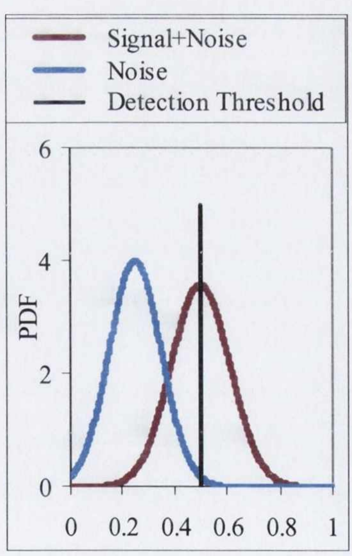
$d_{min} = 0.1$
$\bar{n} = 0.25$
(b)



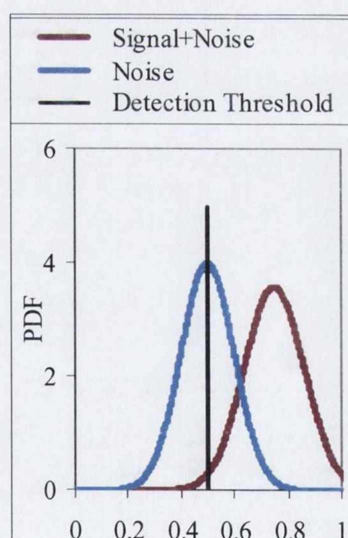
$d_{min} = 0.1$
$\bar{n} = 0.5$
(c)



$d_{min} = 0.5$
$\bar{n} = 0.1$
(d)



$d_{min} = 0.5$
$\bar{n} = 0.25$
(e)



$d_{min} = 0.5$
$\bar{n} = 0.5$
(f)

Figure 3.17. Illustration of variation in PFA for different values of mean of the noise distribution and d_{min}

For a value of d_{min}=0.1, as the mean value of the noise distribution ($\overline{n}$) increases, it moves to the right. For a $\overline{n}$ value of 0.1 (case a) and 0.25 (case b), the PFA is 0.5 and 0.9, respectively, but for a $\overline{n}$ value of 0.5 (case c) it is 1.0 (since the whole distribution is greater that the detection threshold). However, as the detection threshold increases the PFA decreases for a $\overline{n}$ value of 0.25 (case e) and 0.5 (case f) to 0.05 and 0.5, respectively, while the PFA is 0 for a $\overline{n}$ value of 0.1 (case d). These figures illustrate the sensitivity of the PFA to the mean value of the noise distribution (e.g. environmental noise) and the chosen detection threshold of the inspection technique being used. Similarly, the position of the "signal+noise" distribution also moves, since its mean value is a function of the mean value of the noise distribution. The mean of the "signal+noise" distribution increases with the mean of the noise distribution, leading to an increase in the PoD. Therefore, even though reducing the detection threshold increases the PoD, it can also increase the PFA, depending on the mean value and standard deviation of the noise distribution (i.e. the quality of the inspection technique). It is beyond the scope of this thesis to correlate these parameters with actual inspection techniques. However the sensitivity of the results of the maintenance management model to changes in the inspection quality will be discussed in Chapter 4.

Similar to the PoD and PFA, for each defect group the values of PGA and PWA are estimated, given the assumed mean and standard deviation of the defects in the group and the quality of the inspection method which is used for sizing. It is assumed for assessment also that the quality of the inspection method is inversely proportional to the distribution of the noise, σ_{NA}, Equation 3.38,

$$\frac{\sigma_{NA}}{d_{ref}} = \frac{1}{Q_2} \qquad \text{Equation 3.38}$$

While it is recognised that the form of distribution can be different, for the purpose of this development, a normal distribution is assumed to describe the error on sizing of the inspection technique. In this case the noise has the properties $N(0,\sigma_{NA})$. This is similar to the signal response method, as described by Chung et al. (2006), where the recorded signal response is related to the actual crack size using an error term which is modelled using a normal distribution, with some standard deviation and a mean value of 0. Therefore, the PGA_i and PWA_i are estimated for each group *i* using Equation 3.39-Equation 3.42. As discussed previously, there is also a distinction made in this work between good and wrong assessments that lead to repair (subscript $_R$) and those which lead to no repair (subscript $_{NR}$).

$$PGA_{R\_i} = \Phi\left(\frac{\bar{d}_i - d_c}{\sqrt{\sigma_d^2 + \sigma_{NA}^2}}\right) \qquad \left(\text{for } \bar{d}_i \geq d_c\right) \qquad \text{Equation 3.39}$$

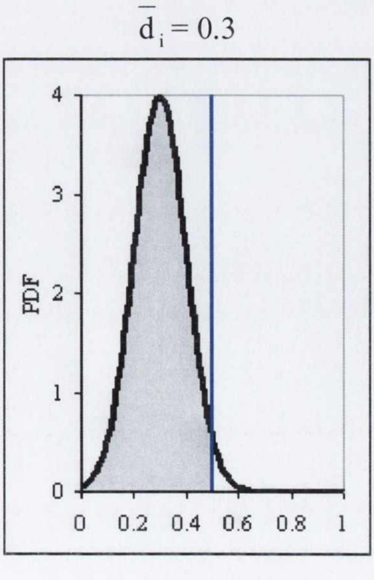
$$PGA_{NR\_i} = 1 - \Phi\left(\frac{\bar{d}_i - d_c}{\sqrt{\sigma_d^2 + \sigma_{NA}^2}}\right) \qquad \left(\text{for } \bar{d}_i < d_c\right) \qquad \text{Equation 3.40}$$

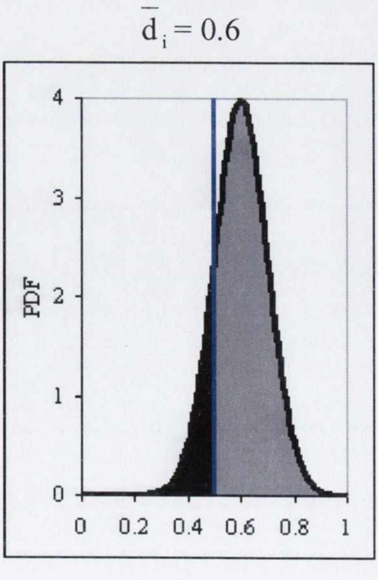
$$PWA_{R\_i} = \Phi\left(\frac{\bar{d}_i - d_c}{\sqrt{\sigma_d^2 + \sigma_{NA}^2}}\right) \qquad \left(\text{for } \bar{d}_i < d_c\right) \qquad \text{Equation 3.41}$$

$$PWA_{NR\_i} = 1 - \Phi\left(\frac{\bar{d}_i - d_c}{\sqrt{\sigma_d^2 + \sigma_{NA}^2}}\right) \qquad \left(\text{for } \bar{d}_i \geq d_c\right) \qquad \text{Equation 3.42}$$

Defects are repaired if the results of the inspection indicate that the size of the defect is greater than the critical defect size, d_c. The interaction between the PGA_R, PGA_{NR}, PWA_R and PWA_{NR} and the mean defect size and critical defect size was discussed previously in Section 3.2.2. It is assumed in this thesis that there is no bias with respect to noise when a sizing assessment is being carried out. The effect of either a positive or negative bias on the PGA_R, PFA_{NR}, PWA_R and PWA_{NR} is illustrated in Figure 3.18(a)-(c).

No Bias
N(0,0.1)





(a)

Bias = +1.0
N(1.0,0.1)

Bias = -1.0
N(-1.0,0.1)

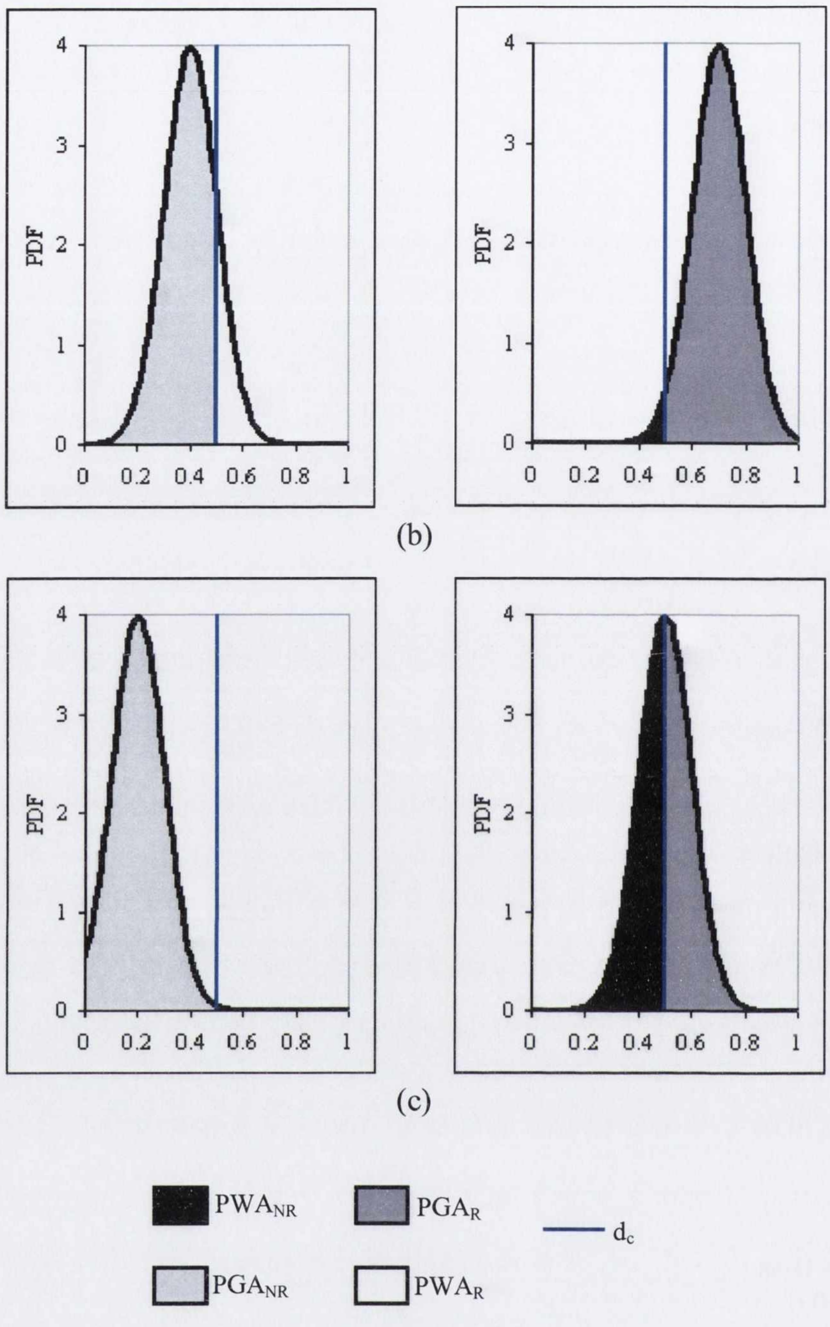


Figure 3.18. Illustration of the effect of bias on the PGA$_R$, PFA$_{NR}$, PWA$_R$ and PWA$_{NR}$

Introducing a positive bias (e.g. N(1.0,σ_{NA})) would have the effect of increasing the probability of repair (i.e. increasing PGA$_R$ for $\overline{d}_i \geq d_c$ and PWA$_R$ for $\overline{d}_i < d_c$). Introducing a negative bias (e.g. N(-1.0,σ_{NA})) would have the effect of reducing the probability of repair (i.e. reducing PWA$_{NR}$ for $\overline{d}_i \geq d_c$ and PGA$_{NR}$ for $\overline{d}_i < d_c$).

3.4.3.2. Simulation of Repair/Failure at Inspection Year

When the first inspection is carried out to detect a defect, there can be two decision outcomes. One is to carry out a further assessment and the other is to do nothing. Similarly, when a second inspection is carried out to assess the size of a defect, there can also be two decision outcomes. One is to repair, in which case the defect returns to the initial defect group and the other is to carry out no repair. If at the detection stage no further assessment is carried out, or if at the assessment stage no repair is carried out, there is still a remaining probability of failure (which will be larger in the second case as the defect is larger). Similar to the event of repair, if failure occurs the defect is modelled as returning to the initial defect group. These probabilities are calculated analytically and are combined to assess P_{i1} for each defect group, which is illustrated in Figure 3.19 (N=5).

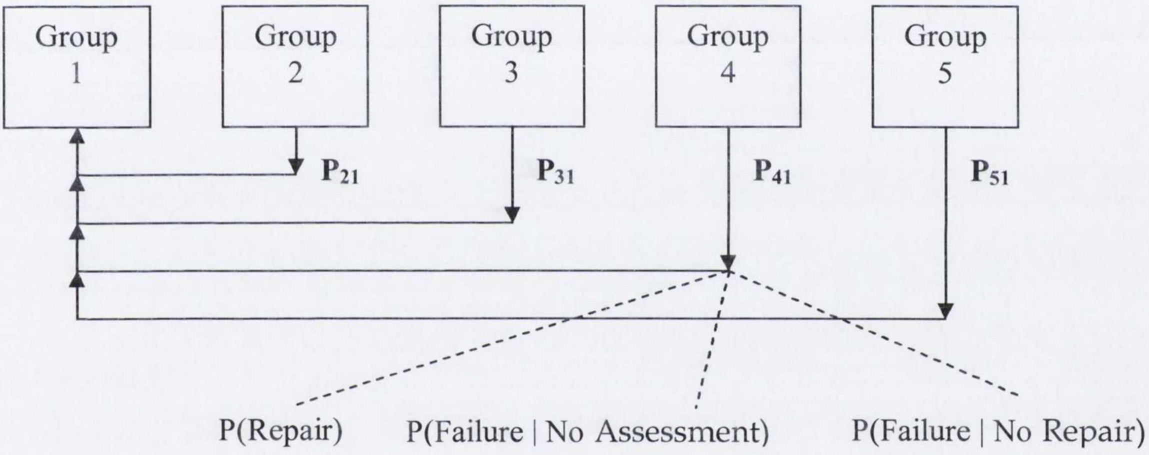


Figure 3.19. Schematic of repair process for Markov chain, for j=1 (N=5)

Therefore, at an inspection year, the probability of repair or failure (event combination $R \cup F$) for each group, $p_{R \cup F\_i}$, can be calculated using Equation 3.43,

$$p_{R \cup F\_i} = P_i(\text{Repair}) + P_i(\text{Failure} \mid \text{No Assessment}) + P_i(\text{Failure} \mid \text{No Repair}) \qquad \text{Equation 3.43}$$

Assuming that inspection results are independent, Equation 3.44-Equation 3.46 can be developed,

$$P(\text{Repair}) = \sum_{i=1}^{N} P_i(\text{Repair}) = \sum_{i=1}^{N} P\left(\hat{d}_{1\_i} \geq d_{min}\right) * P\left(\hat{d}_{2\_i} \geq d_c\right) \qquad \text{Equation 3.44}$$

$$\begin{aligned} P(\text{Failure} \mid \text{No Assessment}) &= \sum_{i=1}^{N} P_i(\text{Failure} \mid \text{No Assessment}) \\ &= \sum_{i=1}^{N} P\left(\hat{d}_{1\_i} < d_{min}\right) * p_{F\_i} \end{aligned} \qquad \text{Equation 3.45}$$

$$\begin{aligned} P(\text{Failure} \mid \text{No Repair}) &= \sum_{i=1}^{N} P_i(\text{Failure} \mid \text{No Repair}) \\ &= \sum_{i=1}^{N} P\left(\hat{d}_{1\_i} \geq d_{min}\right) * P\left(\hat{d}_{2\_i} < d_c\right) * p_{F\_i} \end{aligned} \qquad \text{Equation 3.46}$$

Two parameters, γ and λ were previously defined in Equation 3.13 and Equation 3.24, representing the probability that the actual defect size is greater than the detection threshold and the probability that the size of the actual defect is greater than the specified critical defect size, respectively. They are calculated analytically for each group using the cumulative normal distribution indicated in Equation 3.47-Equation 3.48, although they could also be assigned values based on expert judgement (i.e. based on the experience of the owner/manager or inspector of the structure).

$$\gamma_i = \Phi\left(\frac{\bar{d}_i - d_{min}}{\sigma_d}\right) \qquad \text{Equation 3.47}$$

$$\lambda_i = \Phi\left(\frac{\bar{d}_i - d_c}{\sigma_d}\right) \qquad \text{Equation 3.48}$$

Using these parameters, along with PoD/PFA and PGA/PWA which were previously defined for each of the groups, the P_i(Repair), P_i(Failure | No Assessment) and P_i(Failure | No Repair) can be written analytically, Equation 3.49-Equation 3.51.

$$\Rightarrow P_i(\text{Repair}) = \left[\text{PoD}_i \gamma_i + \text{PFA}_i (1-\gamma_i)\right] * \left[\text{PGA}_{R\_i} \lambda_i + \text{PWA}_{R\_i} (1-\lambda_i)\right] \qquad \text{Equation 3.49}$$

$$\Rightarrow P_i(\text{Failure} \mid \text{No Assessment}) = \left[(1-\text{PoD}_i)\gamma_i + (1-\text{PFA}_i)(1-\gamma_i)\right] * p_{F\_i} \qquad \text{Equation 3.50}$$

$$\Rightarrow P_i(\text{Failure} \mid \text{No Repair}) = \left[PoD_i \gamma_i + PFA_i(1-\gamma_i)\right]* \left[PWA_{NR\_i}\lambda_i + PGA_{NR\_i}(1-\lambda_i)\right]* p_{F\_i} \qquad \text{Equation 3.51}$$

The values of P_i(Repair), P_i(Failure | No Assessment) and P_i(Failure | No Repair) are calculated and summed together to find the total probability for repair or failure for each group, $p_{R \cup F\_i}$, Equation 3.43. These values are then used to calculate the values to be inputted into the first column of the Markov matrix, P_{i1}, simulating the behaviour of a population of defects at an inspection year, Figure 3.20. A similar matrix is developed for years between inspections, where the values to be inputted into the first column of the Markov matrix, P_{i1}, are calculated using the probability of failure between inspections, as discussed in Section 3.4.2.

		To *j*										
		1	2	3	4	5	6	7	8	9	10	**Sum**
From *i*	1	**0.21**	0.67	0.08	0.02	0.01	0.00	0.00	0.00	0.00	0.00	**1.00**
	2	0.01	**0.20**	0.67	0.08	0.02	0.01	0.00	0.00	0.00	0.00	**1.00**
	3	0.02		**0.20**	0.67	0.08	0.02	0.00	0.00	0.00	0.00	**1.00**
	4	0.04			**0.20**	0.67	0.08	0.01	0.00	0.00	0.00	**1.00**
	5	0.16				**0.20**	0.61	0.03	0.00	0.00	0.00	**1.00**
	6	0.59					**0.18**	0.23	0.00	0.00	0.00	**1.00**
	7	0.93						**0.07**	0.01	0.00	0.00	**1.00**
	8	1.00							**0.00**	0.00	0.00	**1.00**
	9	1.00								**0.00**	0.00	**1.00**
	10	1.00									**0.00**	**1.00**

P_{i1}

Figure 3.20. An example of a Markov matrix to simulate the growth and repair/failure of a defect at an inspection year (N=10)

This complete two stage inspection process is described in Figure 3.21. As discussed, when an inspection for detection is carried out, this can result in two inspection outcomes, detection or no detection. Detection leads to a sizing assessment and no detection leads to no further action. The effect of correct/incorrect inspection results can also be investigated using the events based decision theory which was outlined in Section 3.3.

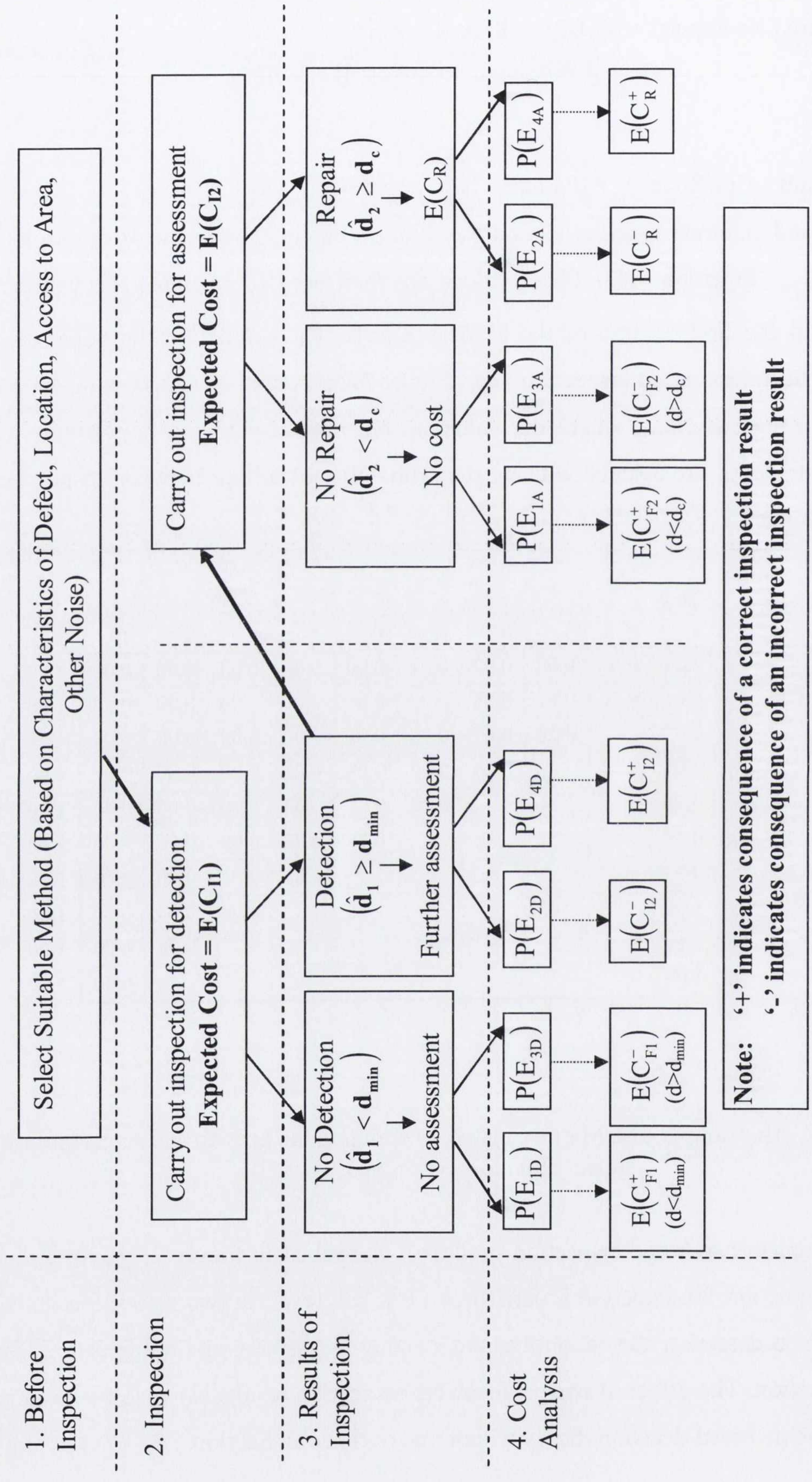


Figure 3.21. Inspection outcomes for a defect group

3.4.4. Merging Growth Matrix and Repair/Failure

At this stage the Markov growth matrix for deterioration has been developed and the values for the probability of repair/failure at an inspection year and the values for the probability of failure between inspections have been calculated for each group. These values need to be added to the first column of the growth matrix (whether it be for an inspection year, or a year between inspections) to develop the two complete Markov matrices which can then be used in the maintenance management model.

This Markov process is assumed to be stable, therefore, each row of the matrix must sum to 1.0, and since the entries in each of the rows in the deterioration matrix summed to 1.0 before the introduction of the P_{i1} values, this deterioration process must be modified to take repair and failure into account and maintain the stability of the Markov process. The methodology described assumes that in any year of simulation, the inspection, repair and failure occur at the end of the year in question, Figure 3.22. Therefore, when modifying the matrix it is assumed that defects may grow throughout the year before inspections, repair or failure occur.

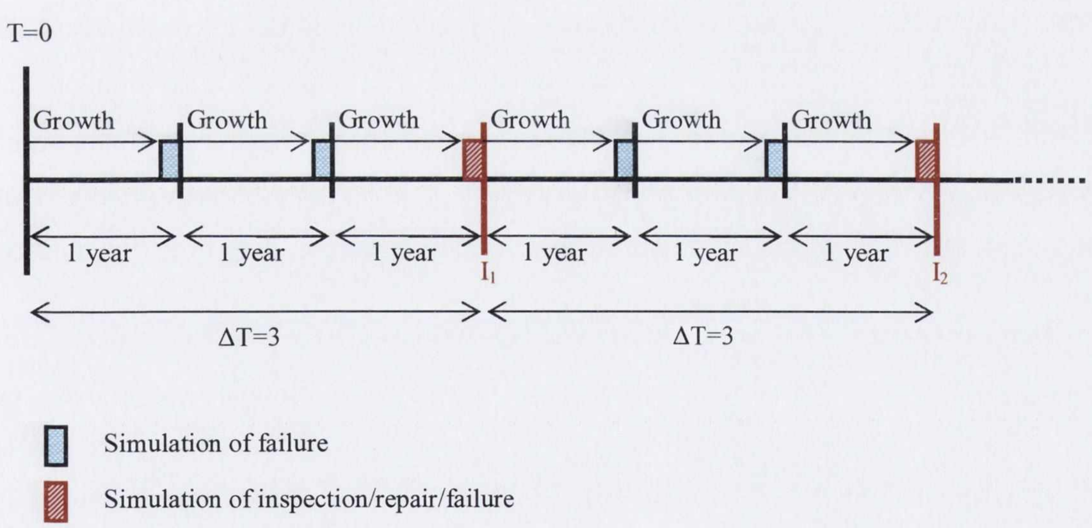


Figure 3.22. Illustration of the methodology for the development of the Markov matrix

The probability that a defect in any one group will be repaired (or will fail) in one time step is represented by P_{i1}. However, in one time step, before the inspection takes place the defects in each group may grow and move to larger groups, in which case the probability of repair or failure for the larger groups are also used to calculate the value of P_{i1} for one row in the matrix. The entries in the complete Markov matrix for an inspection year, $P_{ij\_MARKOV\_1}$, are calculated using Equation 3.52-Equation 3.53, and the entries in the complete Markov matrix for a year between inspections, $P_{ij\_MARKOV\_2}$, are calculated using Equation 3.54-Equation 3.55. Using this method is consistent as the entries in each row still sum to 1.0.

$$P_{i1\_MARKOV\_1} = P_{i1\_GROWTH} + \sum_{j=2}^{N} \left[P_{ij\_GROWTH} * p_{R \cup F\_j} \right] \qquad \text{Equation 3.52}$$

$$P_{ij\_MARKOV\_1} = P_{ij\_GROWTH} * \left(1 - p_{R \cup F\_j}\right), \qquad (\text{for } j > 1) \qquad \text{Equation 3.53}$$

$$P_{i1\_MARKOV\_2} = P_{i1\_GROWTH} + \sum_{j=2}^{N} \left[P_{ij\_GROWTH} * p_{F\_j} \right] \qquad \text{Equation 3.54}$$

$$P_{ij\_MARKOV\_2} = P_{ij\_GROWTH} * \left(1 - p_{F\_j}\right), \qquad (\text{for } j > 1) \qquad \text{Equation 3.55}$$

3.4.5. Stabilisation of Process over Time

In this thesis it is assumed that inspections are carried out at regular intervals, every ΔT years. For this reason, two Markov matrices have been developed, one to simulate the growth and failure of a defect between inspections and another to simulate growth, repair and failure of a defect at an inspection year. Once both matrices have been formulated they are used to simulate the growth and repair of a population of defects over time. Each defect group is assumed to have an initial population of defects.

Between inspections, the Markov matrix that is developed to simulate the growth and failure of the defects is used. At the time of an inspection, the Markov matrix that is developed to simulate the growth, inspection, repair and failure of the defects is used. In both cases, defects can return to the smallest defect group (i.e. fail or be repaired) as well as moving to large groups (i.e. defect growth). Using this methodology, the number of defects in each group is calculated on a yearly basis using the relevant Markov matrix, and the number of defects in each group from the previous year (k-1), Equation 3.56.

$$\#d_{j@Y_k} = \sum_{i=1}^{N} \left(\#d_{i@Y_{k-1}} * P_{ij}\right) \qquad \text{Equation 3.56}$$

For example, considering five defect groups, the number of defects in group 2 at the second year can be calculated using Equation 3.57,

$$\begin{aligned}\#d_{2@Y_2} = &\left(\#d_{1@Y_1} * P_{12}\right) + \left(\#d_{2@Y_1} * P_{22}\right) + \left(\#d_{3@Y_1} * P_{32}\right) + \left(\#d_{4@Y_1} * P_{42}\right) \\ &+ \left(\#d_{5@Y_1} * P_{52}\right)\end{aligned} \qquad \text{Equation 3.57}$$

Effectively, this is the number of defects in group 1 that grow and move to group 2, plus the number of defects in group 2 that stay in group 2, plus the number of defects in groups 3, 4 and 5 that are repaired and move back to group 2. The last three terms will be equal to zero for this model, as it is assumed that once a repair is carried out the defect moves back to the smallest defect group (i.e. group 1 in this case).

To find the optimum time between inspections or to carry out a costing analysis using this model for a structure or network of structures under the control of a management authority, the number of defects in each group is assumed to reach a steady state. Figure 3.23 illustrates the stabilisation of the coefficient of variation of defect size for all of the groups for an inspection period of 1 year, considering growth (gradual growth, g=5), repair and failure for different growth rate (α) parameters. In Figure 3.23, for each growth rate the process stabilises between 25 and 30 years. To illustrate the effect of the growth kinetics on the stabilisation process, another figure was developed for abrupt growth (g=0), Figure 3.24.

However, in Figure 3.24, unlike the previous case, the time taken for the coefficient of variation to stabilise varies depending on the growth rate. For abrupt growth (g=0) and a fast growth rate (α=0.8), stabilisation occurs after approximately 5 years. Since the growth rate is high, a high proportion of the defects which began in the first group are distributed rapidly, and since the defect growth is so abrupt, the defects are well distributed between the different groups. This leads to a very quick stabilisation process (approximately 5 years, Figure 3.24). As the growth rate decreases this stabilisation process is slowed down, with a stabilisation time of approximately 8 years, 18 years and 33 years for growth rates of α=0.4, α=0.2 and α=0.1, respectively.

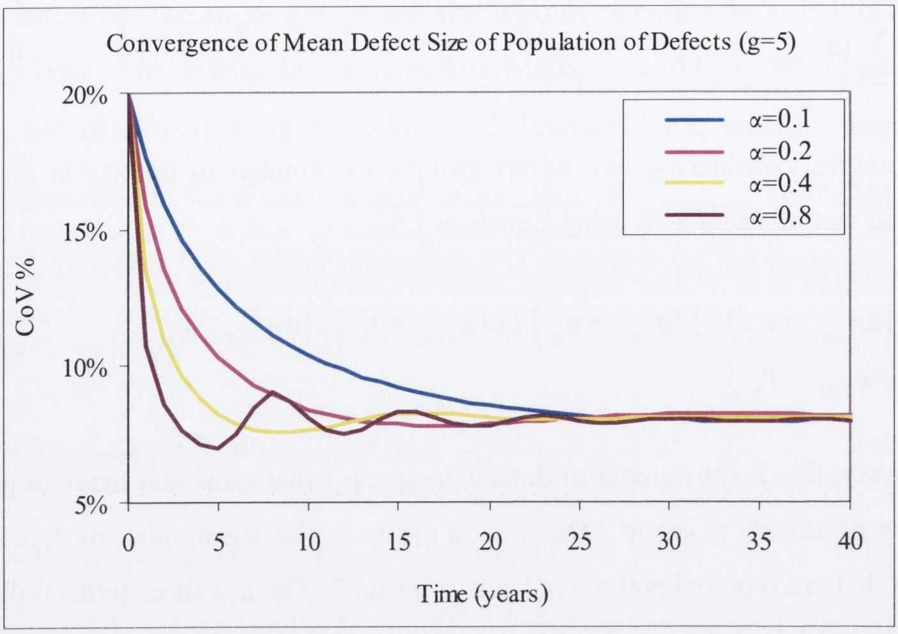


Figure 3.23. The stabilisation of the CoV for a population of defects (for ΔT=1, g=5)

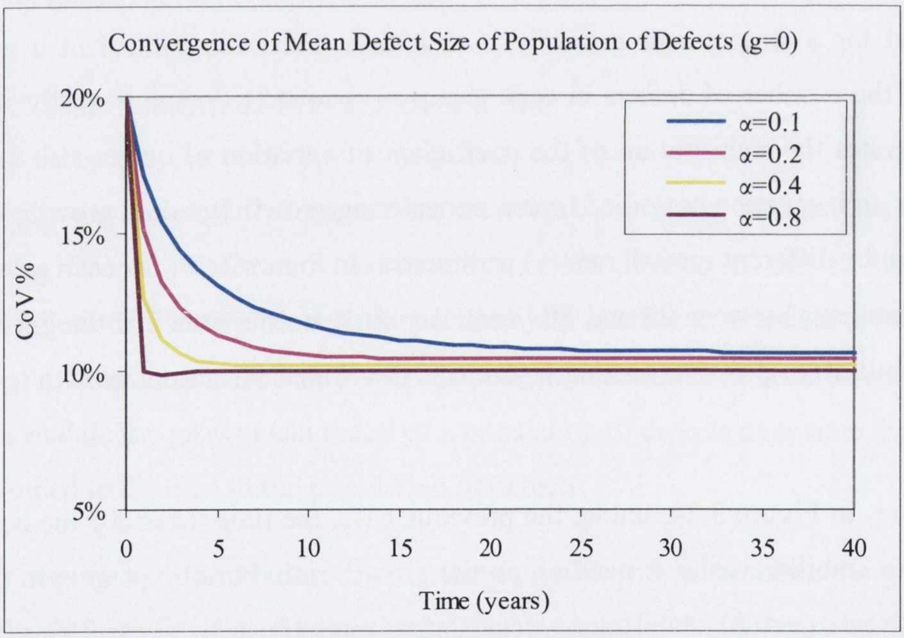


Figure 3.24. The stabilisation of the CoV for a population of defects (for ΔT=1, g=0)

Since two different matrices are used in this study simultaneously (for an inspection interval greater than 1 year), the number of defects in each group will not converge to one value over time, but will converge to a set of ΔT values, one value for each year in the ΔT cycle, Figure 3.25. This illustrates that for each year in the ΔT cycle there is an array which contains the stabilised number of defects in each group for that year in the cycle. The size of this array is (Nx1), where N is the number of defect groups being considered. For a one year inspection interval the size of the matrix containing the stabilised number of defects is just (Nx1), recording the number of defects at an inspection year. For a two year inspection interval there is a stabilised number of defects for the inspection year (assumed to be Year 1 in this case) and a stabilised number of defects for the year between inspections (assumed to be Year 2 in this case). Therefore the size of the matrix storing the stabilised number of defects in this case is (Nx2).

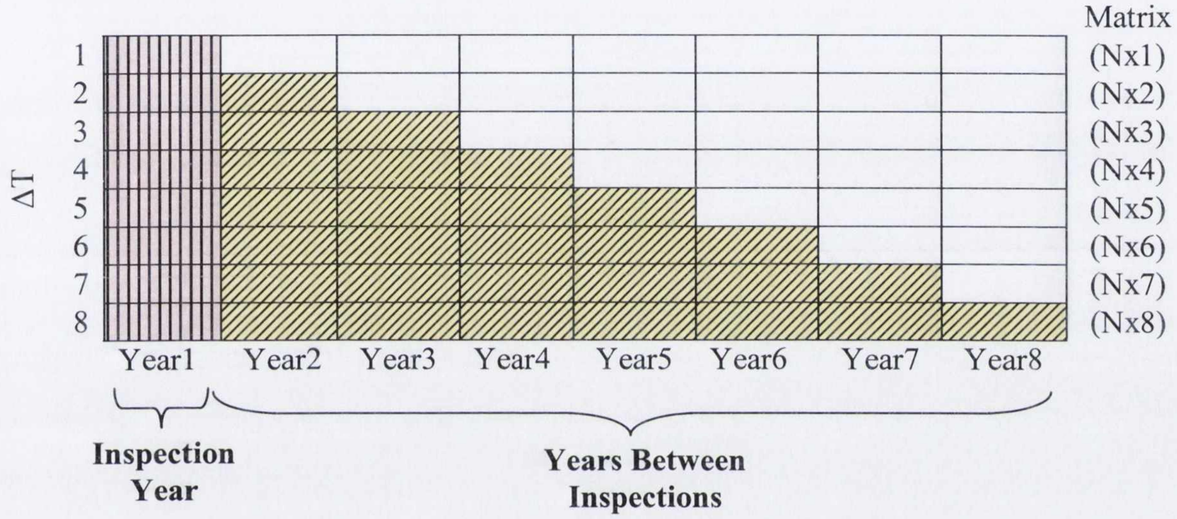


Figure 3.25. Convergence of stabilised number of defects in each group with one array (Nx1) for each year in the ΔT cycle

When the Markov matrices were developed it was assumed that within each time interval of one year the defects can grow from the groups they were in at the beginning of the year to other groups before the inspections, repairs or failures for that year occur. To calculate the number of inspections, repairs and failures based on the calculated probabilities in Section 3.4.2 and Section 3.4.3, it is necessary to have the stabilised number of defects in each group directly before the events of inspection or failure. Therefore, once the stabilised number of defects (at yearly intervals) is evaluated, it is necessary to compute the number of defects in each group (for each year) directly before inspection or failure. This is evaluated using the number of defects at the beginning of each year and the relevant Markov matrix to calculate the number of defects in each group after growth alone for that year (i.e. before inspection, repair or failure). This modified matrix of the stabilised number of defects in each group is necessary to facilitate the calculation of the expected cost of inspection, repair and failure, for each value of ΔT. On this basis, the optimum inspection interval can be determined in relation to the minimisation of the expected mean annual total cost. This is an important management tool for infrastructure owners/managers. The optimum inspection interval will depend on many factors, such as the environment, deterioration kinetics being considered, structure material, structural importance and costs.

3.4.6. Cost Functions

As discussed, the number of defects in each group stabilises after a certain number of simulations, depending on the deterioration process and repair strategy. These stabilised values are used to calculate the expected annual costs of the system. In the proposed methodology the expected cost of inspections ($E(C_{I\_TOTAL})$), repair ($E(C_{R\_TOTAL})$) and failure ($E(C_{F\_TOTAL})$) are considered, which are summed to find the expected annual total cost of the structure ($E(C_{TOTAL})$). These are the direct costs associated with maintenance management of a structure or network of structures. It is recognised that other indirect costs, such as user delay costs and penalty costs for reduced serviceability could also be included, but since no data is available to accurately quantify the parameters associated with the assumed cost models, it was decided to model only the direct costs. For the costing models associated with the direct costs, cost coefficients were assumed, and sensitivity studies were carried out to investigate the effect of some of these cost coefficients on the results of the developed maintenance management framework. These sensitivity studies will be discussed in Chapter 4. The development of accurate costing models for direct and indirect costs is beyond the scope of this thesis. The cost analysis developed in this thesis is used to compare the relative implications of different management decisions (e.g. different inspection quality, inspection intervals etc.) and it is recognised that these cost models are subjective and should only be used to provide an indication of the relative benefits of different management strategies.

Once the cost of inspection, repair and failure for an individual defect in a group is calculated (using these assumed cost coefficients), the expected total cost is calculated by multiplying by the number of defects in the group and the probability that inspection, repair or failure occurs with the associated cost implications. In relation to inspection and repair, the expected costs are calculated using the stabilised number of defects in each group at an inspection year. The expected failure cost at an inspection year must also be calculated each year using the stabilised number of defects in each group at an inspection year, the P(Failure | No Assessment) and the P(Failure | No Repair). The expected failure cost between inspections, however, must be calculated using the stabilised number of defects in each group, for each year between inspections. The various expected costs for each group are then divided into ratios (depending on the four inspection events for detection and assessment) to assess which costs are due to good/bad decisions, Figure 3.21. This is also an important management tool for assessing the effect of various maintenance strategies.

3.4.6.1. Inspection Cost

Once the number of defects in each group, and the expected number of inspections to be carried out at each inspection year are determined, from Equation 3.58-Equation 3.60, the expected annual inspection cost of the structure can be calculated.

$$E_i(\#1\text{st Inspections}) = \#\text{ defects in group } i \qquad \text{Equation 3.58}$$

$$E_i(\#2\text{nd Inspections}) = \#\text{ defects in group } i * P_i(2\text{nd Inspection}) \qquad \text{Equation 3.59}$$

where

$$P_i(2^{nd}\text{ Inspection}) = P\left(\hat{d}_{1\_i} \geq d_{min}\right) = PoD_i\gamma_i + PFA_i(1-\gamma_i) \qquad \text{Equation 3.60}$$

An initial inspection or first inspection of each defect is carried out every ΔT years, at an expected cost of $E_i(C_{I1})$ for each group *i*. If the first inspection results in detection (with a probability $P\left(\hat{d}_{1\_i} \geq d_{min}\right)$), then a second inspection takes place to size the detected defect, Equation 3.60. The expected cost of the second inspection is $E_i(C_{I2})$. The expected annual inspection cost for each group is evaluated using Equation 3.61-Equation 3.65.

$$\Rightarrow E_i(C_{I1}) = E_i(\#1^{st}\text{ Inspections}) * {CI1}/{\Delta T} \qquad \text{Equation 3.61}$$

$$\Rightarrow E_i(C_{I2}) = E_i(\#2^{nd}\ \text{Inspections}) * {CI2}/{\Delta T} \qquad \text{Equation 3.62}$$

where

$$CI1 = C_o k_{I1}\left(Q_1/Q_{ref}\right)^{0.5} \qquad \text{Equation 3.63}$$

$$CI2 = C_o k_{I2}\left(Q_2/Q_{ref}\right)^{0.5} \qquad \text{Equation 3.64}$$

$$k_{I2} = \eta k_{I1} \qquad \text{Equation 3.65}$$

The cost of an individual inspection (CI1 and CI2) is assumed to be proportional to the inspection quality. Using this non-linear cost model, for a specific increase in inspection quality (ΔQ), the cost increase for the inspection is less for higher values of inspection quality. For example, increasing Q_1 from 5 to 10 (thus reducing the standard deviation of the noise distribution by 50%) results in a higher cost than increasing Q_1 from 45 to 50 (thus reducing the standard deviation of the noise distribution by 10%). The reference inspection quality (Q_{ref}) is used as a normalising factor. It is also assumed that the cost of the second inspection is less than the cost of the first inspection, due to the discounted cost of multiple inspections (e.g. costs of transporting equipment, setup costs, erecting scaffolding etc.). For this reason the cost coefficient for the second inspection (k_{i2}) is assumed to be a fraction (η) of the cost coefficient of the first inspection (k_{i1}), Equation 3.65. The sensitivity of the overall results to the value chosen for η is discussed in Chapter 4.

The expected annual total inspection cost of the structure ($E(C_{I\_TOTAL})$) is then calculated by summing the expected cost of inspection for detection $E_i(C_{I1})$, and inspection for sizing assessment $E_i(C_{I2})$, for each group i, Equation 3.66.

$$\Rightarrow E(C_{I\_TOTAL}) = \sum_{i=1}^{N}\left[E_i(C_{I1}) + E_i(C_{I2})\right] \qquad \text{Equation 3.66}$$

Once the first inspection indicates detection, it is assumed that a second inspection takes place. This decision to carry out another inspection can be seen as a good decision (where in reality the defect size is greater than the detection threshold, event E_{4D}) or as a bad decision (where in reality the defect size is less than the detection threshold, event E_{2D}), Equation 3.67-Equation 3.69.

$$E_i(C_{I2}^{+}) = P_i(E_{4D})E_i(C_{I2}) \qquad \text{Equation 3.67}$$

$$E_i(C_{I2}^{-}) = P_i(E_{2D})E_i(C_{I2}) \qquad \text{Equation 3.68}$$

$$\text{Note}: P_i(E_{2D}) + P_i(E_{4D}) = 1 \qquad \text{Equation 3.69}$$

3.4.6.2. Repair Cost

With regards to repair, the sizing assessment takes place if the first inspection results in detection. If the second inspection indicates that the defect is larger than the critical defect size, then it is assumed that a repair is carried out. If the inspection indicates that the defect size is smaller than the critical defect size, then no further action is taken. Therefore, the number of repairs carried out depends on the critical defect size specified by the owner/manager of the structure, which depends on the limit state being considered. Again, the expected annual costs can be calculated, knowing the number of defects in each group *i*, Equation 3.70-Equation 3.72.

$$E_i(\#\text{Repairs}) = \#\text{ defects in group } i * P_i(\text{Repair}) \qquad \text{Equation 3.70}$$

$$E_i(C_R) = E_i(\#\text{Repairs}) * {CR}/{\Delta T} \qquad \text{Equation 3.71}$$

where

$$CR = C_0 k_R \qquad \text{Equation 3.72}$$

It is assumed that the cost of repair is constant, depending only on the initial construction cost (C_o) and the repair cost coefficient (k_R). In practice, the cost of repair may consist of a fixed and a variable component, where the variable component depends on the extent of the repair being carried out. The type of costing model would depend on the deterioration mechanism and defect type being considered and various other costs associated with repair (e.g. labour costs, material costs, site setup costs etc.). However, since this would be specific to the material deterioration mechanism, it is assumed for the purpose of this thesis that the repair cost is constant. The expected total annual repair cost of the structure or component ($E(C_{R\_TOTAL})$) is then calculated by summing the expected cost of repair ($E_i(C_R)$), for each group, Equation 3.73.

$$\Rightarrow E(C_{R\_TOTAL}) = \sum_{i=1}^{N}[E_i(C_R)] \quad \text{Equation 3.73}$$

When a repair is carried out, this can be due to a good assessment (i.e. the real defect size is greater than d_c, event E_{4A}) or due to a wrong assessment (i.e. the real size of the defect is actually less than d_c, event E_{2A}), Figure 3.21. The expected cost of repair due to a good assessment is denoted $E_i(C_R^+)$, and the expected cost due to a bad assessment (or an expected cost overrun) for each group is denoted $E_i(C_R^-)$, Equation 3.74-Equation 3.76.

$$E_i(C_R^+) = P_i(E_{4A})E_i(C_R) \quad \text{Equation 3.74}$$

$$E_i(C_R^-) = P_i(E_{2A})E_i(C_R) \quad \text{Equation 3.75}$$

$$\text{Note}: P_i(E_{2A}) + P_i(E_{4A}) = 1 \quad \text{Equation 3.76}$$

The expected cost for each group can be summed to find the total expected cost of repair due to a good assessment, $E(C_{R\_TOTAL}^+)$, and the expected total cost due to a bad assessment, $E(C_{R\_TOTAL}^-)$, Equation 3.77-Equation 3.78.

$$\Rightarrow E(C_{R\_TOTAL}^+) = \sum_{i=1}^{N}[E_i(C_R^+)] \quad \text{Equation 3.77}$$

$$\Rightarrow E(C_{R\_TOTAL}^-) = \sum_{i=1}^{N}[E_i(C_R^-)] \quad \text{Equation 3.78}$$

3.4.6.3. Failure Cost at Inspection Year

In the case of an inspection year when it is possible for repairs to be carried out, P_i(Failure | No Assessment) and P_i(Failure | No Repair) are used to calculate the probability of failure for each group *i*, depending on the results of the two stages of the inspection. There is a probability of failure at the detection stage (P(Failure | No Assessment), Equation 3.50) if no further assessment is carried out as there may be a defect present which is not being considered for sizing assessment and therefore will not be repaired. Similarly, after a second inspection if no repair is carried out, there is also a remaining probability of failure (P(Failure | No Repair), Equation 3.51). In this case the probability of failure is likely to be higher as the un-repaired defect would be larger in size.

By calculating the expected number of failures in each group at detection stage, Equation 3.79 and at the sizing assessment stage, Equation 3.80, the corresponding expected cost of failure can be calculated for each group, Equation 3.81-Equation 3.83.

$E_i(\#(\text{Failures@Detection}))= \#\text{ defects in group } i * P_i(\text{Failure}|\text{No Assessment})$ Equation 3.79

$E_i(\#(\text{Failures@Assessment})) = \#\text{ defects in group } i * P_i(\text{Failure}|\text{No Repair})$ Equation 3.80

$$\Rightarrow E_i(C_{F1}) = E_i(\#(\text{Failures@Detection})) * {}^{CF}\!/_{\Delta T}$$ Equation 3.81

$$\Rightarrow E_i(C_{F2}) = E_i(\#(\text{Failures@Assessment})) * {}^{CF}\!/_{\Delta T}$$ Equation 3.82

where

$$CF = C_0 k_F$$ Equation 3.83

The cost of failure is assumed to be a multiple of the initial cost of construction (C_o). The failure impact coefficient (k_F) is decided upon based on the impact of failure, which depends on factors such as the importance of the structure or structural component and the limit state being considered (e.g. SLS or ULS). In this thesis, failure is defined as the exceedance of a specified limit state.

In the case of the first inspection (i.e. for detection of defects present), if there is no detection one scenario is that the real defect is less than d_{min} (with a probability of $P_i(E_{1D})$), in which case the probability of failure and associated expected cost of failure will be small ($E_i(C_{F1}^{+})$). However, if the inspection results are incorrect, then there is a defect present with a size greater than d_{min} (with a probability of $P_i(E_{3D})$), and the probability of failure and expected annual cost of failure $E_i(C_{F1}^-)$ will be larger than for a correct inspection. The breakdown of the expected failure costs at detection stage is given in Equation 3.84-Equation 3.86.

$$E_i(C_{F1}^+) = P_i(E_{1D})E_i(C_{F1}) \quad \text{Equation 3.84}$$

$$E_i(C_{F1}^-) = P_i(E_{3D})E_i(C_{F1}) \quad \text{Equation 3.85}$$

$$\text{Note: } P_i(E_{1D}) + P_i(E_{3D}) = 1 \quad \text{Equation 3.86}$$

Similarly, the expected annual cost of failure is calculated on this basis if a second assessment takes place and no repair is carried out. The expected cost of failure due to a good assessment is $E_i(C_{F2}^+)$ (with a probability $P_i(E_{1A})$) and the expected cost of failure due to a wrong assessment is $E_i(C_{F2}^-)$ (with a probability $P_i(E_{3A})$), Equation 3.87-Equation 3.89.

$$E_i(C_{F2}^+) = P_i(E_{1A})E_i(C_{F2}) \quad \text{Equation 3.87}$$

$$E_i(C_{F2}^-) = P_i(E_{3A})E_i(C_{F2}) \quad \text{Equation 3.88}$$

$$\text{Note: } P_i(E_{1A}) + P_i(E_{3A}) = 1 \quad \text{Equation 3.89}$$

3.4.6.4. Failure Cost between Inspections

For any defect greater than the limit defect size, d_l, there is some probability of failure, depending on the size of the defect. As described previously, for years between inspections the probability of failure for each defect group is calculated based on the mean size of the defects in the group *i*, Equation 3.33. Using this methodology, failure between inspections must also be simulated when finding the optimum time between inspections or the expected annual costs of the structure. At each year between inspections, there will be a different number of defects in each group, and hence, a different number of failures each year. Therefore, the expected number of failures in each group each year must be calculated and summed together (assuming that they are non-correlated), Equation 3.90, to find the total expected cost of failure between inspections, $E_i(C_{F3})$, for each group *i*, Equation 3.91.

$$E_i(\#\text{Failures}) = \sum_{t=1}^{\Delta T-1} (\#\text{defects in group i})_t * p_{F\_i} \qquad \text{Equation 3.90}$$

$$\Rightarrow E_i(C_{F3}) = E_i(\#\text{Failures}) * {CF}/{\Delta T} \qquad \text{Equation 3.91}$$

The expected annual total failure cost of the structure ($E(C_{F\_TOTAL})$) is then calculated by summing the expected cost of failure due to no sizing assessment $E_i(C_{F1})$, the expected cost of failure due to no repair $E_i(C_{F2})$, and the expected cost of failure between inspections $E_i(C_{F3})$, for each group, Equation 3.92.

$$\Rightarrow E(C_{F\_TOTAL}) = \sum_{i=1}^{N} [E_i(C_{F1}) + E_i(C_{F2}) + E_i(C_{F3})] \qquad \text{Equation 3.92}$$

3.5. CONCLUSIONS

Using the framework which was developed and presented in this chapter, the optimal inspection interval for an infrastructural element/network can be assessed, based on the minimisation of the numbers of failures and of the service life costs for a specified limit state. The effect of inspection quality of the two techniques employed can also be assessed. However, when carrying out repairs on the basis of inspection results, it can be useful for owners/managers to have the ability to investigate the probability of correct/incorrect inspection results, to look at the effect of inspection quality on cost overrun due to unnecessary repairs. This study can be carried out using the events based decision theory developed in Section 3.3. The effect of the inspection interval and the inspection quality will be studied in Chapter 4.

The separation of the inspection process of a structure into two stages enables the investigation to study the effect of both stages of the inspection on the expected annual costs of the structure, and the optimal maintenance plan for the structure. Budgets are allocated to infrastructure managers each year for the implementation of inspection programmes. This developed methodology enables the optimisation of this process by the selection of specific methods at each stage of an inspection as a function of many different factors (such as environment, deterioration mechanism, severity of deterioration, allowable probability of failure) to deliver maximum benefits. The separation of these procedures and the interaction of the two inspection techniques have not previously been considered. By modelling the two stages of an inspection as separate procedures, using different parameters, the effect of different combinations of techniques can be investigated and the optimal combination can be selected. The effect of inspection quality will be studied and the optimum combination of techniques for this two stage process will be determined in Chapter 4. Depending on the requirements of the owner/manager, it may be more convenient to use a low quality screening technique for detection and a higher quality inspection technique for sizing to assess which defects should be repaired. The developed methodology allows for the first time the effect of such decisions to be evaluated. Therefore, the first objective of this thesis, which was the development of a maintenance management methodology which takes into account the two aspects of an inspection (detection and sizing), has been achieved.

As well as separating the inspection process into two stages, this methodology has the capability to simulate many forms of deterioration, so that different deterioration mechanisms can be considered depending on factors such as the material, environment and limit state. Therefore, in relation to the second objective of this thesis, which was to allow many different forms of deterioration to be simulated (i.e. abrupt and gradual, linear and non-linear), this chapter achieves part of this objective, where different deterioration kinetics can be simulated. The ability to realistically simulate non-linear deterioration propagation will be discussed in Chapter 6. Different modes of failure (e.g. sudden or progressive failure) can also be considered by choosing suitable parameters for the Weibull distribution. In addition, the effect of choosing different limit states can be studied by varying the repair threshold value or critical defect size (d_c). On this basis, the capabilities and flexibility of the developed methodology will be demonstrated and discussed in the following chapter. An important comparison between this developed two stage inspection process and the traditional one stage process will also be made in Chapter 4.

It is important to study the sensitivity of the assumed parameters chosen (e.g. for deterioration parameters, inspection qualities, parameters for probability of failure, allowable probability of failure) so that the owner/manager of a structure can study the implications of changes in these parameters. For example, if inspections indicate that the deterioration kinetics are more abrupt than initially anticipated, the owner/manager must be aware of how this will effect the optimal maintenance strategy and the associated expected costs and number of failures, and tailor the maintenance strategy to suit as appropriate. Therefore, the sensitivity of the results of the maintenance management model to the parameters introduced in this chapter will be studied and discussed in Chapter 4.

CHAPTER 4 - APPLICATIONS OF MODEL

CHAPTER 4 - APPLICATIONS OF MODEL

4.1. INTRODUCTION

The theoretical development of a maintenance model is presented in Chapter 3. In this chapter the model is employed to study the implications and the benefits of this new maintenance management methodology to owners/managers of infrastructure systems. The methodology outlined was used to determine the optimum time between inspections for a structure or network of structures. Subsequently, the effect of the quality of the inspection techniques for detection and sizing on the optimum time between inspections and on the expected annual total costs of the structure was analysed to optimise the budget available for maintenance.

Using the events based decision scheme introduced in Section 3.3, the effect of the quality of inspections on cost overrun, such as unnecessary repairs, was also investigated. This is a useful tool for decision makers when planning maintenance strategies. It is also demonstrated how the optimum combination of techniques can be chosen for the two stages of an inspection process and how beneficial this can be for decision makers when selecting suitable inspection procedures. An important comparison is then made between the two stage inspection methodology developed in this thesis and the traditional single stage approach in relation to expected mean annual total cost and expected number of failures. This two stage inspection approach allows the optimal inspection techniques for each stage of an inspection to be selected, therefore, minimising costs and optimising available budgets. This is not possible with a traditional one stage process.

Studies were carried out to demonstrate the sensitivity of the results of the model to changes in parameters used to describe the deterioration process, inspection quality, the mode of failure, and the limit state. These studies are necessary to allow the owner/manager to understand the implications of changes to the assumed deterioration and maintenance procedures. For example, it is useful to investigate the effect of changing the limit state being considered (e.g. from SLS to ULS) on the expected costs and the expected number of failures and tailor the maintenance strategy to suit as appropriate.

Alternatively, it is of interest to determine how a change of environment could affect the optimal maintenance management strategy. Considering for example a reinforced concrete bridge, the effect of using de-icing salts on the structure (i.e. altering deterioration mechanism/rate) could be assessed. In addition, the sensitivity of the optimum inspection interval to the number of groups chosen for the discretisation of the possible range of defect growth has been investigated. These issues are addressed in the following chapter for an assumed set of input parameters.

4.2. RESULTS

For the purpose of this example, the possible range of defect sizes was subdivided into 10 groups (similar to the National Bridge Inventory Condition Rating system in the US), each modelled statistically with an assumed mean and standard deviation, as outlined in Table 4.1. The defect size is assumed to vary from 0-1.0mm, with a 0.1mm defect range for each group. However, it is recognised in this case that the extent of noise has a direct influence on the minimum measurable defect size and the ability to size a defect within a particular group. In Chapter 5 and Chapter 6, the form of deterioration considered is cracking of concrete due to chloride-induced corrosion of the reinforcing bars. For this deterioration process, the limit crack width is considered to be 1.0mm (Vu et al., 2005; Vu, 2003). For this reason, the defect size is assumed to vary from 0-1.0mm in this chapter.

	Range					**Initial**
Defect Group	**From**	**To**	$\bar{d}$	σ_d	**CoV**	**Population**
1	0	0.1	0.05	0.02	40.0%	100
2	0.1	0.2	0.15	0.02	13.3%	0
3	0.2	0.3	0.25	0.02	8.0%	0
4	0.3	0.4	0.35	0.02	5.7%	0
5	0.4	0.5	0.45	0.02	4.4%	0
6	0.5	0.6	0.55	0.02	3.6%	0
7	0.6	0.7	0.65	0.02	3.1%	0
8	0.7	0.8	0.75	0.02	2.7%	0
9	0.8	0.9	0.85	0.02	2.4%	0
10	0.9	1.0	0.95	0.02	2.1%	0

Table 4.1. Defect group data used in the model

Since no data could be obtained for the CoV for these defect groups, the CoV is assumed to decrease as the defect group increases. The sensitivity of the results to the number of defect groups chosen will be studied in Section 4.3.1. The standard deviation of the defect size for each group represents the scatter in the range of actual size of defects in the group and is not related to the error in sizing of a defect. It was assumed that the standard deviation of the range of defect sizes in a group is independent of the mean defect size and a constant value was assumed for each group. In addition, it was assumed initially that there were 100 defects in the smallest defect group, and no defects in all other groups (i.e. taken to represent a new structure), Table 4.1.

Using this methodology (i.e. that developed in Chapter 3), the first inspection for detection is carried out as a screening exercise to detect any defects that are present. A second inspection is then carried out to assess the size of the defect. Any defects greater than the critical defect size are repaired. This process is illustrated in Figure 3.21. In this chapter, the repair threshold (d_c) is calculated based on an allowable probability of failure ($P_{f\_allowable}$), Equation 4.1, which can be specified by the owner/manager of a structure depending on the limit state being considered. The variation in the critical defect size with the limit state (allowable probability of failure) is illustrated in Figure 4.1.

$$d_c = d_1 + d_{ref\_P_f}\left(-\log\left(1 - P_{f\_allowable}\right)\right)^{1/m} \qquad \text{Equation 4.1}$$

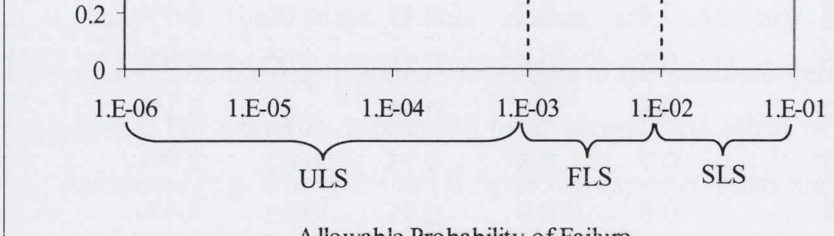


Figure 4.1. Variation in critical defect size for different limit states

Using this formulation, the critical crack width is greater for SLS than for ULS. It is recognised that generally, considering a single defect within an individual structure, that the critical defect size for SLS is expected to be smaller than for ULS. For example, as the reinforcing bars in the concrete corrode, cracks develop in the structure, which is an SLS condition. However, as corrosion progresses further, the reduction in the reinforcing bar diameter becomes critical to the ULS due to a reduction in the structural capacity of the section. However, in this thesis, considering ULS, only the critical structural capacity locations are considered to be of importance and must therefore be repaired before reaching a critical defect size (corresponding to a particular reduction in bar diameter) which is determined based on the allowable probability of failure. On a different part of the structure, which is less critical structurally, the critical defect size can be larger where the governing limit state is SLS and the allowable probability of failure is therefore higher.

The values for the allowable probability of failure are grouped according to the serviceability limit state (SLS), the fatigue limit state (FLS), and the ultimate limit state (ULS), based on suggestions in the ISO standards (ISO 2394:1998(E)). The assumed values for the Weibull distribution parameters (d_1, d_{ref_pf}, m) which are also included in Equation 4.1 are presented in Table 4.2. The sensitivity of the results to these parameters will be discussed in Section 4.3.4.

Table 4.3 also presents the allowable probability of failure, considering the consequence of failure and the relative cost of safety measures (i.e. maintenance and repair), based on the recommendations from the ISO standards (ISO 2394:1998(E)). SLS is concerned with the serviceability of a structure. An unserviceable structure can lead to user delays, but will not lead to collapse of a structure; therefore, the allowable probability of failure is relatively high (between 0.5 and 10^{-2}). In this case, for the SLS the critical defect size varies from about 0.6 mm to 0.87 mm (i.e. defects can grow relatively large before they need to be repaired). When considering the FLS or ULS, the growth of a defect may be an indication of a more serious form of deterioration (i.e. fatigue or cracking due to corrosion of reinforcing bars in concrete), which could ultimately lead to reduced strength or even structural collapse. Due to a higher consequence of failure, the allowable probability of failure reduces (between 10^{-2} and 10^{-3} for FLS and between 10^{-3} and 10^{-6} for ULS). In this case the critical defect size reduces (from 0.48 to 0.6 for FLS and from 0.32 to 0.48 for ULS), resulting in smaller defect sizes being repaired compared to SLS. It is recognised that the parameters used to define the Weibull distribution (i.e. the parameters used in Equation 4.1), and the probability of failure also depend on the limit state being considered. As previously mentioned, this will be discussed in Section 4.3.4.

Model Properties	**Assumed Value**
Growth of defect	
Growth rate, α	0.5
Deterioration kinetics parameter, g	3
Reference defect size, d_{ref}	1.0
Probability of failure	
Probability of failure exponent, m	4
Limit defect size, d_l	0.3
Reference defect size, d_{ref_pf}	1.0
Allowable probability of failure, $P_{f\_allowable}$	0.01
Detection	
Detection threshold, d_{min}	0.2
Quality of inspection for detection, Q_1	10
Mean of noise distribution, $\bar{n}$	0.1
Sizing Assessment	
Critical defect size, d_c	0.62
Quality of inspection for sizing assessment, Q_2	20
Cost Analysis	
Initial construction cost, C_o	1000
Inspection coefficient, k_{I1}	0.01
Ratio of inspection cost coefficients, η	0.2
Reference quality, Q_{ref}	20
Repair coefficient, k_R	0.1
Failure impact coefficient, k_F	1.0

Table 4.2. Parameter values used in Markov maintenance model

Relative costs of safety measures	**Consequence of failure**			
	Small	Some	Moderate	Great
High	0.5	10^{-1}	10^{-2}	10^{-3}
Moderate	10^{-1}	10^{-2}	10^{-3}	10^{-4}
Low	10^{-2}	10^{-3}	10^{-4}	10^{-5}

Table 4.3. Values for the allowable probability of failure based on suggestions in the ISO standards (ISO 2394:1998(E))

4.2.1. Assumed Parameters

Table 4.2 presents the assumed set of parameters that were used as inputs to the model for the purpose of the results presented in this chapter. Since the objective of this thesis is the development of a maintenance management methodology and no data is readily available for the parameters describing the deterioration, inspection, repair and failure of defects over time, a set of parameters were assumed in this chapter to illustrate the capabilities of the methodology and to carry out sensitivity studies. Future work is recommended to calibrate the parameters used in this model to actual data, as it is beyond the scope of this thesis.

The first section of Table 4.2 presents the parameters which are used to simulate the deterioration of a structure or structural component over time. The formulation of the Markov matrix using these parameters was described in Section 3.4.1, where α represents the growth rate of a defect and g represents the growth kinetics of the structure. The combination of these two parameters enables many forms of deterioration mechanism to be simulated. Moderate values were assumed for both parameters (i.e. α=0.5, g=3). The sensitivity of the results to the growth kinetics (g) will be presented in Section 4.3.1, while the effect of different growth rates on the results will be presented in Chapter 6, using the results from the experimental study (Chapter 5) which was carried out to compare the rate of crack growth in different repair materials. An average growth rate of α=0.5 was used in this study. The value of g=3 was chosen based on the growth kinetics of crack growth in reinforced concrete due to corrosion, which was studied as part of this thesis and will be discussed further in Chapter 5. The reference defect size is a normalising parameter which is assumed to be 1.0.

As discussed in Section 3.4.2, the Weibull cumulative distribution is used to calculate the probability of failure, where the position, shape and spread of this curve is determined by the parameters d_1, m and d_{ref_pf}. The assumed values for these parameters are presented in the second section of Table 4.2. In this chapter, a progressive failure rather than a sudden failure is assumed, resulting in parameter values of d_1=0.3, m=4 and d_{ref_pf} =1.0. These values can be altered depending on the mode of failure (e.g. progressive or sudden failure) and the limit state (e.g. SLS, FLS, ULS) being considered. As discussed in the previous section, the allowable probability of failure is specified according to the limit state and the consequence of failure. For this chapter (unless otherwise stated), a serviceability limit state is assumed with an allowable probability of failure of $P_{f\_allowable}$=0.01. On this basis, the effect of these parameters (i.e. mode of failure and limit state) on the results will be studied in Section 4.3.4 and Section 4.3.5.

The next two sections of Table 4.2 present the assumed parameters for the two stage inspection process developed in this thesis. The parameters describing the detection stage of the inspection are d_{min}, Q_1 and $\overline{n}$. A low level of noise ($\overline{n}$=0.1) and a detection threshold of d_{min}=0.1 were assumed. These parameters are used to calculate the PoD and PFA for the inspection technique being considered for detection. The sensitivity of PoD and PFA to the values of d_{min} and $\overline{n}$ was studied in Section 3.4.3.1. The parameters describing the sizing assessment stage of the inspection are d_c and Q_2. These parameters are used to calculate the PGA and PWA for the inspection technique being considered for assessment. The interaction between the PGA, PWA and the critical defect size (d_c) was discussed previously in Section 3.2.2. The quality of the inspection technique for each stage of the inspection (i.e. Q_1 for detection and Q_2 for the sizing assessment) is assumed to be inversely proportional to the standard deviation of the noise distribution (i.e. the higher the inspection quality the lower the noise associated with the inspection technique). It is assumed for the studies carried out in this chapter (unless otherwise stated), that a low/moderate quality inspection technique is used for detection (Q_1=10) and a moderate/high quality inspection technique is used for the sizing assessment (Q_2=20). However, using this two stage methodology it is possible to determine the optimum inspection quality for each stage of the inspection process. This will be discussed further in Section 4.2.3.

The last section in Table 4.2 presents the assumed parameters for the cost analysis. As discussed in Section 3.4.6, the assumed costing models and cost coefficients presented in this thesis are limited and are used only to compare the relative implications of different management decisions. Values of C_o=1000, k_{I1}=0.01, k_R=0.1 and k_F=1.0 are used in this chapter unless otherwise stated. The relative values of the cost coefficients correspond to a serviceability limit state condition. The effect of the cost coefficients (k_{I1}, k_R and k_F) on the results will be discussed as part of Section 4.3.4. The sensitivity of the results to the ratio of inspection cost coefficients (η) will also be studied in Section 4.3.2. According to Straub and Faber (2005), the fixed component of an inspection forms the major part of the costs.

The fixed component arises from the costs such as accessing the hotspot to carry out the inspections and the possible temporary closure of the structure. Therefore, a value of η=0.2 was chosen. Q_{ref} is a normalising parameter as part of the costing models for inspection quality, and is assumed to have a value of Q_{ref} =20 (moderate/high quality inspection technique).

4.2.2. Calculation of Optimal Time between Inspections

Using these parameters presented in Table 4.2, the optimum time between inspections was determined on the basis of the minimum expected mean annual total cost of the structure, $E(C_{TOTAL})$, which was assessed according to the cost functions outlined in Section 3.4.6. These cost functions were used to estimate the expected mean annual cost of inspections, repair and failure for a network of structures. Using these cost estimations it is possible to study the effect of the inspection interval on the expected mean annual costs. Based on this information, the owner/manager can decide on the optimal inspection interval, which in this case is based on the minimisation of the expected mean annual total cost. Figure 4.2 shows the results of the analysis, illustrating that for the case considered a period of 3 years represents the optimum inspection interval.

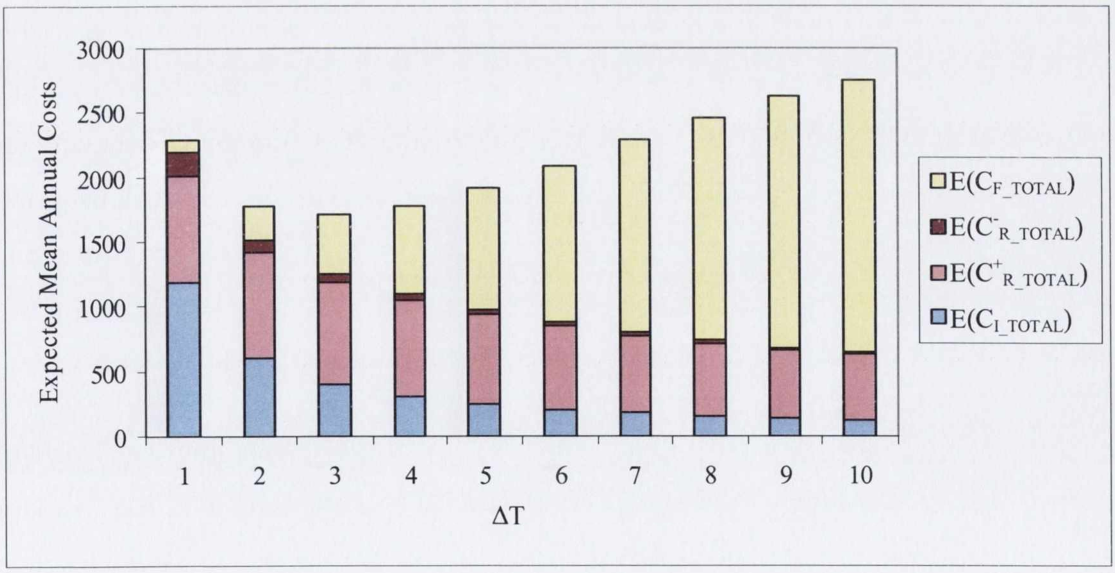


Figure 4.2. The effect of the inspection interval on expected mean annual costs

The sensitivity of this result to the number of defect groups selected will be studied in Section 4.3.1. As illustrated in Figure 4.2, the inspection interval has a significant effect on the expected mean annual inspection cost, $E(C_{I\_TOTAL})$, and the expected mean annual failure cost, $E(C_{F\_TOTAL})$. The expected inspection cost ranges from 52% of the total cost for a 1 year inspection interval to just 5% of the total cost for a 10 year inspection interval. As expected, an inverse trend emerges for the failure cost, with the expected failure cost ranging from just 4% of the total cost at a 1 year inspection interval to 77% of the total cost for a 10 year inspection interval. For example, considering a service life of 80 years, a one year inspection interval results in 80 inspections over the service life, resulting in a high inspection cost but a low failure cost since all defects are detected and subsequently repaired when they are found to be greater than the critical defect size. For an inspection interval of 10 years, however, there are only 8 inspections over the service life, resulting in a lower annual inspection cost but a higher failure cost since defects are undetected and unrepaired for a period of 10 years between inspections, resulting in failures.

Figure 4.2 also illustrates the effect of the inspection interval on the expected cost of repair. Using the developed events based decision theory, outlined in Section 3.3, the proportion of repairs which are carried out due to correct and incorrect inspection results can also be investigated. As expected, the proportion of unnecessary repairs (due to incorrect inspection results) is inversely proportional to the inspection interval. For a longer inspection interval the defects have a longer time to grow, and therefore a higher proportion the defects are in the larger groups, which require repair. There are fewer defects in the groups where the defect size is less than d_c. As discussed previously, for an allowable probability of failure of 0.01 the critical defect size is 0.62, which is in group 7. Taking inspection intervals of 1 year and 10 years as an example, the distribution of the defects within the defect groups before an inspection is illustrated in Figure 4.3.

For a 1 year inspection interval, there are only 11% of the defects in group 7 or larger, whereas for a 10 year inspection interval there are 55% of the defects in group 7 or larger. Therefore, even though the probability of a correct/incorrect inspection for each group is independent of inspection interval, there are fewer repairs due to incorrect inspection results for greater inspection intervals since there are more defects in larger groups which require repair. Referring to Figure 4.2, for an inspection interval of 1 year, 19% of all repairs carried out ($E(C^{+}_{R\_TOTAL})+E(C^{-}_{R\_TOTAL})$) are due to incorrect inspection results ($E(C^{-}_{R\_TOTAL})$), whereas, for an inspection interval of 10 years, only 3% of all repairs carried out ($E(C^{+}_{R\_TOTAL})+E(C^{-}_{R\_TOTAL})$) are due to incorrect inspection results ($E(C^{-}_{R\_TOTAL})$). Therefore, as well as determining the optimum inspection interval, this work allows the owner/manager of a structure to study the variation in different costs with inspection interval. Of particular interest is the breakdown of repair cost into necessary and unnecessary repairs, due to correct and incorrect inspection results. This provides the decision maker with more information, which can be of interest when planning maintenance strategies. The effect of different parameters (Q, g and $P_{f\_allowable}$) on the percentage of necessary and unnecessary repairs will be studied later in this chapter (Section 4.2.3, Section 4.3.1 and Section 4.3.5, respectively).

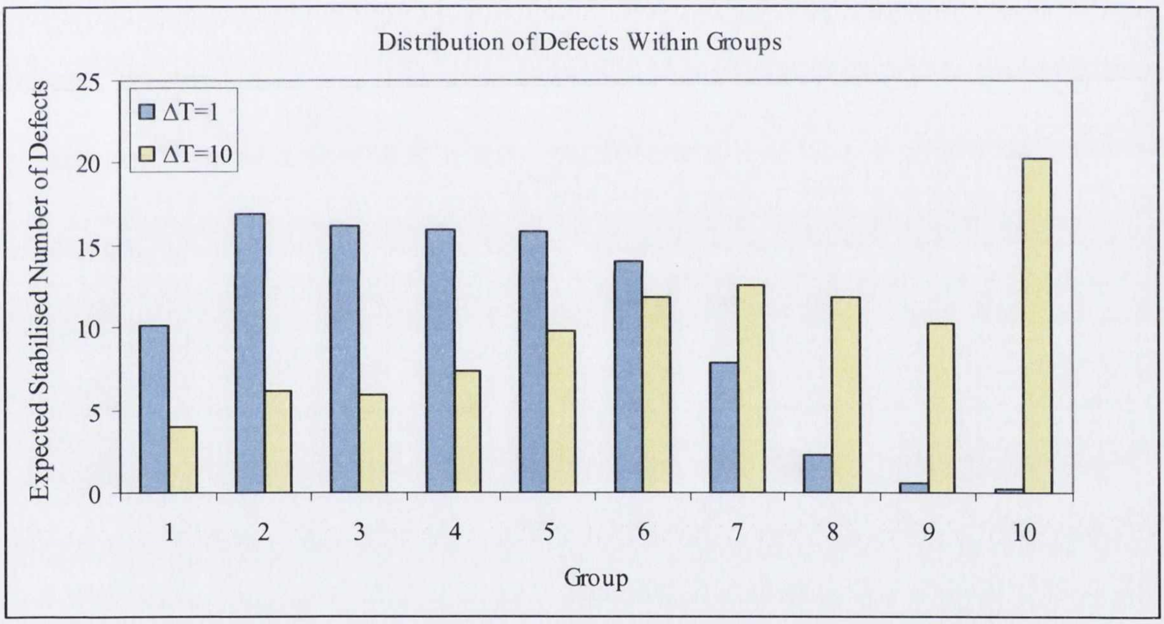


Figure 4.3. Distribution of defects within defect groups for an inspection interval of 1 year and 10 years

4.2.3. Inspection Quality

Using the parameters given in Table 4.2, the effect of the inspection quality was investigated for an inspection interval of 3 years (the optimum inspection interval, Figure 4.2). An inspection interval of 6 years was also studied, as principal inspections are recommended every 6 years in the UK (Vassie and Arya, 2006), and from 1 year to 6 years in Ireland, depending on the condition of the structure and the environment (Duffy, 2004). Initially, the effect of changing both inspection qualities simultaneously was studied (i.e. Q=10 implies that Q_1=10 and Q_2=10).

For an inspection interval of 3 years, the results indicate that the optimum inspection quality for both inspection methods is Q=8, as illustrated in Figure 4.4. An inspection quality of Q=8 represents a low/moderate inspection quality for both detection and sizing assessment. This results in the lowest expected mean annual total cost. As illustrated in Figure 4.4, the inspection cost increases with inspection quality, while the failure cost decreases. The expected failure cost falls from 50% of the total cost to just 20% of the total cost by increasing the inspection quality from Q=2 to Q=50. However, it is recognised that when increasing the inspection quality from Q=2 to Q=50, depending on the techniques available, a range of inspection qualities may correspond to the same technique (e.g. Q=2 and Q=4 may correspond to the same technique).

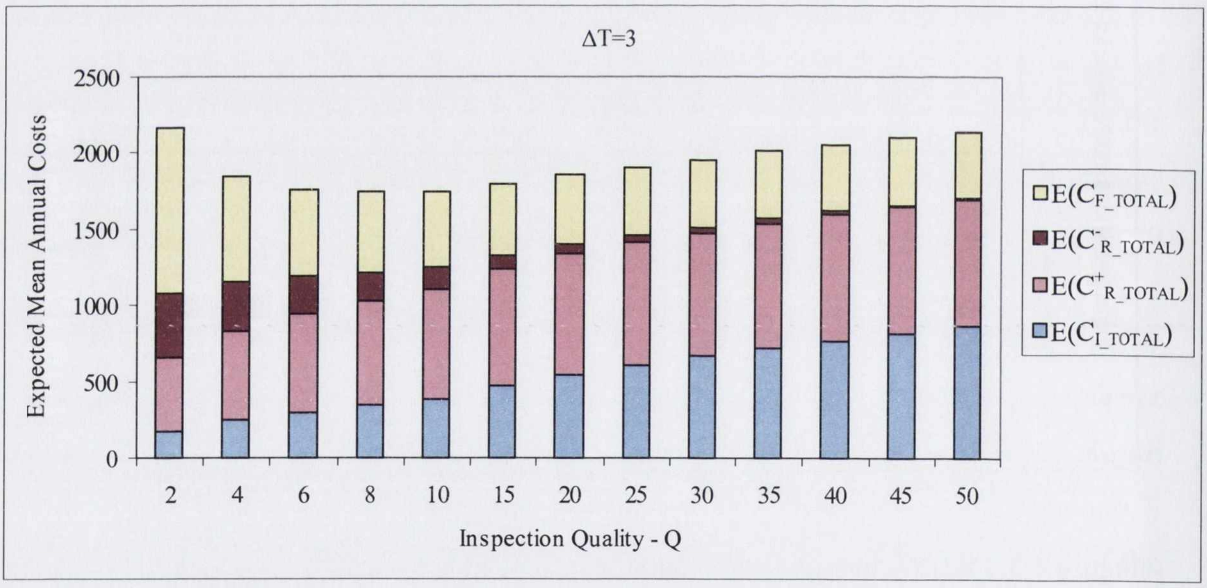


Figure 4.4. The effect of inspection quality on expected mean annual costs (ΔT=3)

Although the total cost of repair appears to be relatively insensitive to inspection quality (varies from 42% to 39% of the total cost), the cost of repair due to incorrect inspection results ($E(C^-_{R\_TOTAL})$) reduces from 46% to 2% of the total repair cost ($E(C^+_{R\_TOTAL})+E(C^-_{R\_TOTAL})$) by increasing the inspection quality from Q=2 to Q=50. This can be explained by studying the distribution of the number of defects, the probability of repair and the resulting number of repairs for each group. The expected mean annual number of repairs (E(MeanAnnualRepairs)) for each group is illustrated in Figure 4.5 for an increasing inspection quality.

As the inspection quality increases, the number of repairs in groups 1-5 (where the defects are less than the critical defect size) decreases. This is due to the greater accuracy of the higher quality inspection techniques (i.e. less noise associated with inspections), and therefore, fewer defects that are less than the critical defect size are inaccurately sized and unnecessarily repaired. Similarly for group 6, as the inspection quality increases the probability of repair decreases. However the number of defects in group 6 increases with Q, resulting in an increase in the number of repairs initially (for Q=2→4), and a subsequent drop in the number of repairs with a further increase in Q (i.e. Q>4). This initial increase in the number of repairs is due to the differential in slope between the increase in the number of defects and the decrease in the probability of repair.

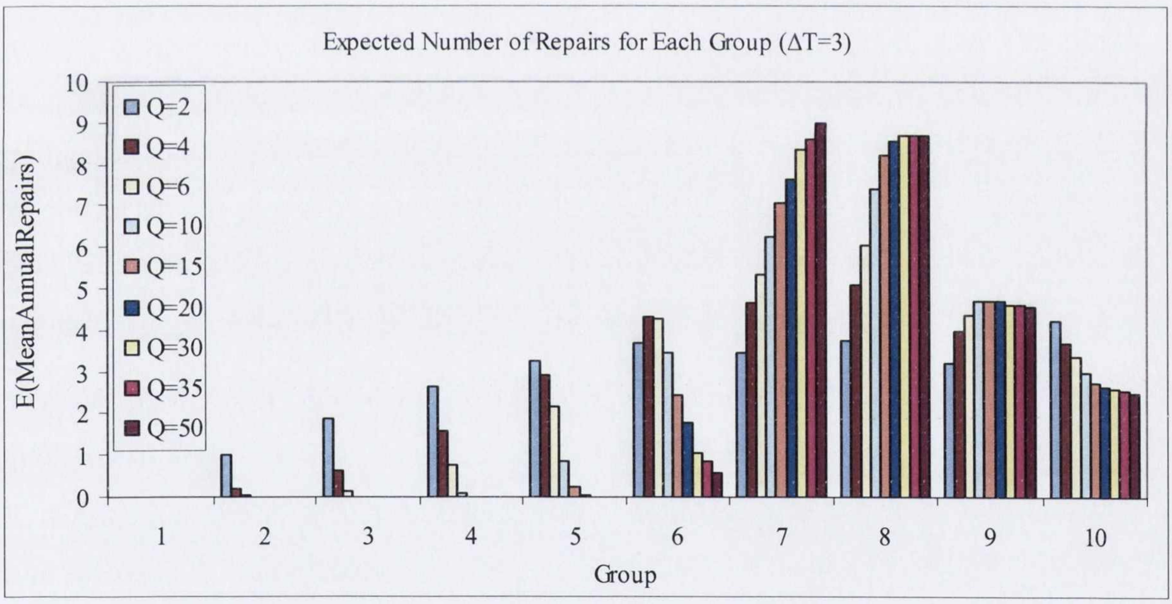


Figure 4.5. Variation in the number of repairs as the inspection quality increases ($\Delta T=3$)

In groups 7-8 (where the mean defect size is greater than d_c) the probability of repair increases with Q. There are also a higher proportion of defects in groups 7-8 as Q increases, which along with a higher probability of repair, leads to more repairs. In group 9, although the probability of repair increases with Q, the number of defects in the group decreases, leading to an increase in the number of repairs initially as Q is increased, but then a decrease in the number of repairs as Q increases further (i.e. for Q>10). Following this trend, for group 10, the number of defects in the group decreases as Q increases (as all defects are correctly repaired before reaching the largest group). This leads to the number of repairs in group 10 being reduced although the probability of repair does increase. This demonstrates that the number of repairs depends on the value for the critical defect size (and hence $P_{f\_allowable}$ or limit state considered). This analysis allows owners/managers to study how the quality of the inspection technique can affect the variation in costs and number of failures, and to choose the optimum inspection techniques for inspections (when considering the same inspection technique for both stages of the inspection process). The selection of the optimum combination of techniques will be discussed later in this section.

Considering an inspection interval of 6 years, the optimum inspection quality is Q=15, representing a moderate inspection technique, as illustrated in Figure 4.6. This is higher than the optimum quality inspection technique for an inspection interval of 3 years (i.e. Q=8). This demonstrates that if the owner/manager of a structure or network of structures decides to delay the inspection (e.g. due to current budgetary constraints), a higher quality inspection technique is required to detect and correctly size defects present and minimise the number of failures. Similar to an inspection interval of 3 years, the inspection cost increases with inspection quality, while the failure cost decreases. The expected failure cost falls from 76% of the total cost to just 51% of the total cost by increasing the inspection quality from Q=2 to Q=50.

In relation to repairs due to correct and incorrect inspection results, a similar trend that is observed for an inspection interval of 3 years can be seen for an inspection interval of 6 years, with the cost of unnecessary repairs ($E(C^-_{R\_TOTAL})$) reducing from 32% to 1% of the total repair cost ($E(C^+_{R\_TOTAL})+E(C^-_{R\_TOTAL})$) by increasing the inspection quality from Q=2 to Q=50.

However, for an inspection interval of 6 years the total cost of repair appears to be more sensitive to inspection quality (varies from 20% to 30% of the total cost) compared to an inspection interval of 3 years (where the repair cost varies from 42% to 39% of the total cost). Again, this can be explained by studying the distribution of the number of defects, the probability of repair and the resulting number of repairs for each group. For an inspection interval of 6 years, the expected mean annual number of repairs for each group is illustrated in Figure 4.7 for an increasing inspection quality.

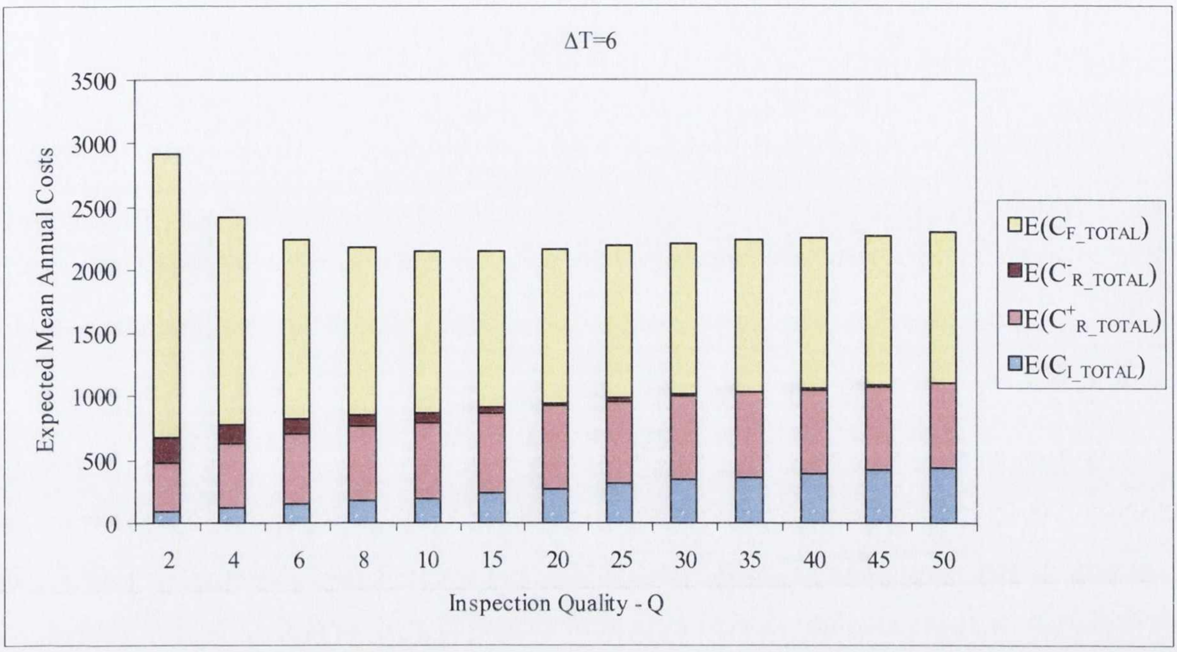


Figure 4.6. The effect of inspection quality on expected mean annual costs (ΔT=6)

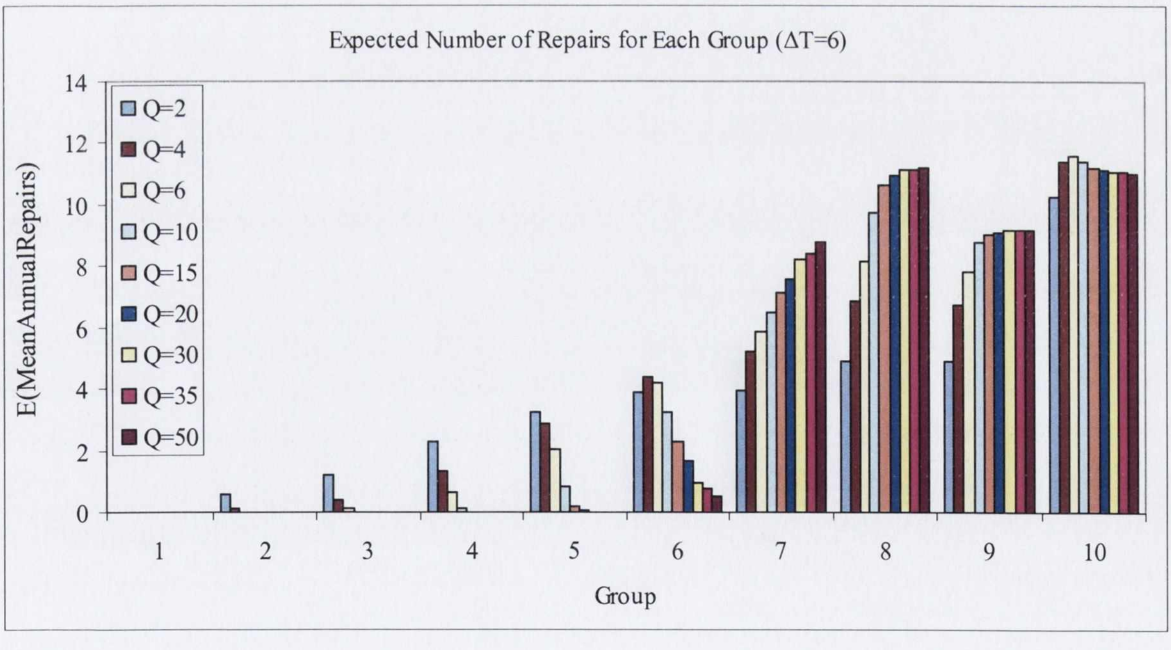


Figure 4.7. Variation in the number of repairs in each group as the inspection quality increases (ΔT=6)

Comparing this to an inspection interval of 3 years (Figure 4.5), a similar trend emerges for groups 1-8. Although for group 9, in this case, the number of failures increases with inspection quality (unlike for $\Delta T=3$). This is due to an increase in the number of defects in group 9 as the inspection quality increases for $\Delta T=6$, where there is a decrease for $\Delta T=3$. This is because defects that are less than the critical defect size are correctly sized, and therefore not repaired. These defects then have time to grow to group 9 between inspections, whereas, for an inspection interval of 3 years these defects would be repaired before growing to group 9. Therefore, as Q increases, the number of repairs increases for $\Delta T=6$, whereas for $\Delta T=3$, there is an increase initially, followed by a decrease as Q is increased further.

Also, considering group 10, for an inspection interval of 3 years an inverse relationship exists between the number of repairs and the inspection quality (Figure 4.5). However, for an inspection interval of 6 years the number of repairs increases initially with Q, but subsequently reduces as Q is increased further. This leads to a higher relative difference in the number of repairs for an inspection interval of 3 years and 6 years. This analysis allows owners/managers to study how the inspection interval and delaying inspections can affect variation in costs and the optimum inspection techniques (when considering the same technique for both stages of the inspection process).

Given that the cost coefficients can vary from case to case and can have a significant influence on the results, it can be useful to look at results such as the expected number of failures and the expected number of unnecessary repairs to compare maintenance strategies, since they are independent of assigned cost coefficients. It is recognised that the number of failures is related to the allowable probability of failure (as specified by the owner/manager). The sensitivity of the results to the limit state (i.e. $P_{f\_allowable}$) considered will be discussed in Section 4.3.5.

Again, taking an inspection interval of 3 years and 6 years, the effect of the quality of the inspections on the number of failures was investigated. Using the events based decision theory the proportion of repairs due to correct/incorrect inspection results was also studied. This will, again, allow the owner/manager to study the effect of the inspection quality and inspection interval on the expected number of failures and the proportion of necessary/unnecessary repairs. Depending on the limit state being considered, the number of failures may be of more interest to the owner/manager than the expected costs (since indirect costs such as user delay are not considered as part of the cost analysis in this thesis). The effect of inspection quality on the expected mean annual number of failures (E(MeanAnnualFailures)) is illustrated in Figure 4.8.

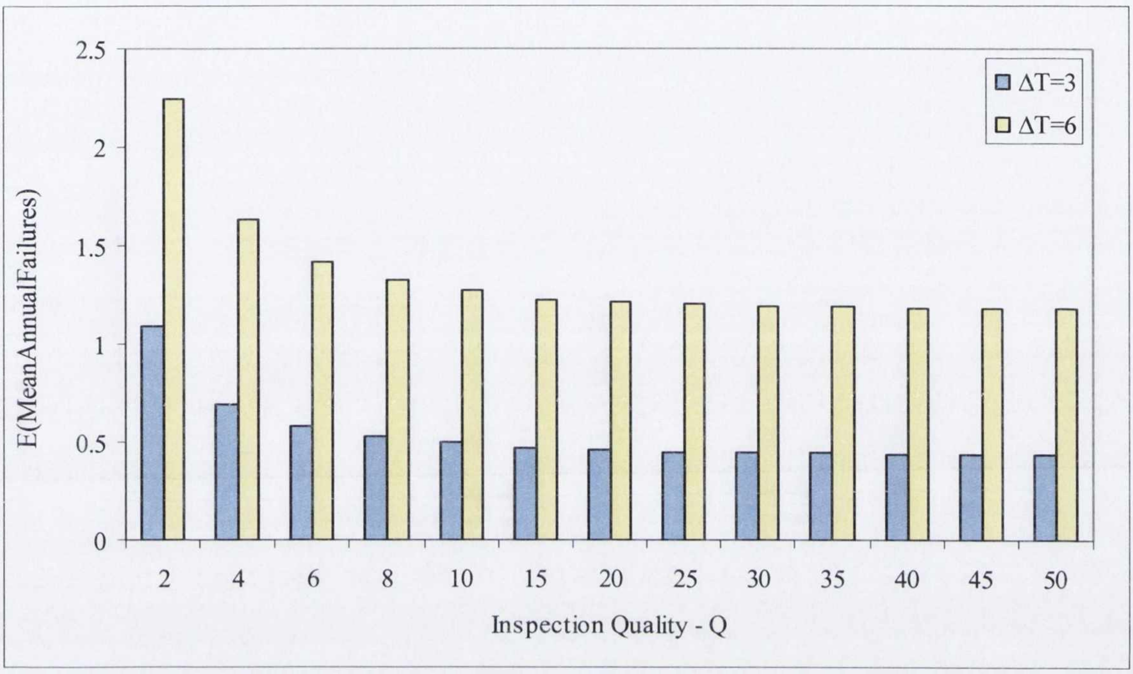


Figure 4.8. The effect of inspection quality on the mean annual number of failures (for $\Delta T=3$ and $\Delta T=6$)

As the inspection quality is increased from Q=2 to Q=20, there is a significant decrease in the mean annual number of failures for an inspection interval of 3 years and 6 years, Figure 4.8. Further increasing the inspection quality doesn't have much of an effect on the number of failures for either inspection interval. This point is significant for the development of maintenance strategies. For example, increasing the inspection quality greater than Q=20 in this case would result in an unnecessary increase in the inspection cost, as this has very little effect on the number of failures. This would result in the inefficient allocation of restricted maintenance budgets.

It is recognised that the exact point at which the number of failures plateaus (about Q=20 in this case) depends on the specific values of the parameters being considered. Reducing the number of failures within the structure has the effect of increasing the optimal inspection interval, as can be seen by studying Figure 4.2. This may be more convenient for an owner/manager of a structure as inspections can be carried out less often, reducing user delay costs and inconvenience to road users.

Figure 4.9 and Figure 4.10 illustrate that the inspection quality affects the breakdown of the expected repair cost into necessary and unnecessary repairs, for an inspection interval of 3 years and 6 years, respectively. As discussed previously, the proportion of unnecessary repairs (due to incorrect inspection results) is inversely proportional to the inspection interval. Similar to the number of failures, the most significant decrease in the number of unnecessary repairs can be seen by increasing the inspection quality from Q=2 to Q=20. The number of repairs carried out depends on the number of defects that are sized and are found to be greater than the critical defect size (which is a function of the allowable probability of failure), therefore, the cost overrun of unnecessary repairs reduces as more accurate inspections for sizing are carried out. This suggests that the manager of a structure should focus more resources on phase 2 (sizing assessment) of the inspection than phase 1 (detection). The interaction of the two phases of an inspection will be discussed later in this section.

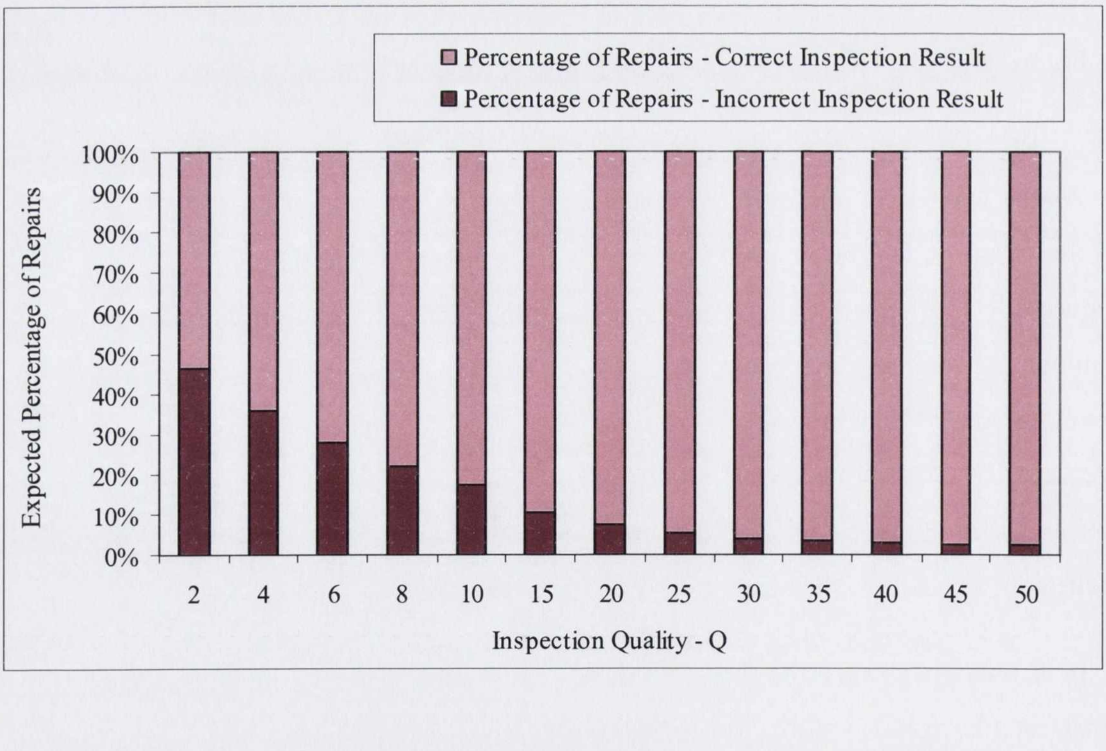


Figure 4.9. The effect of inspection quality on the proportion of repairs due to correct/incorrect inspection results ($\Delta T=3$)

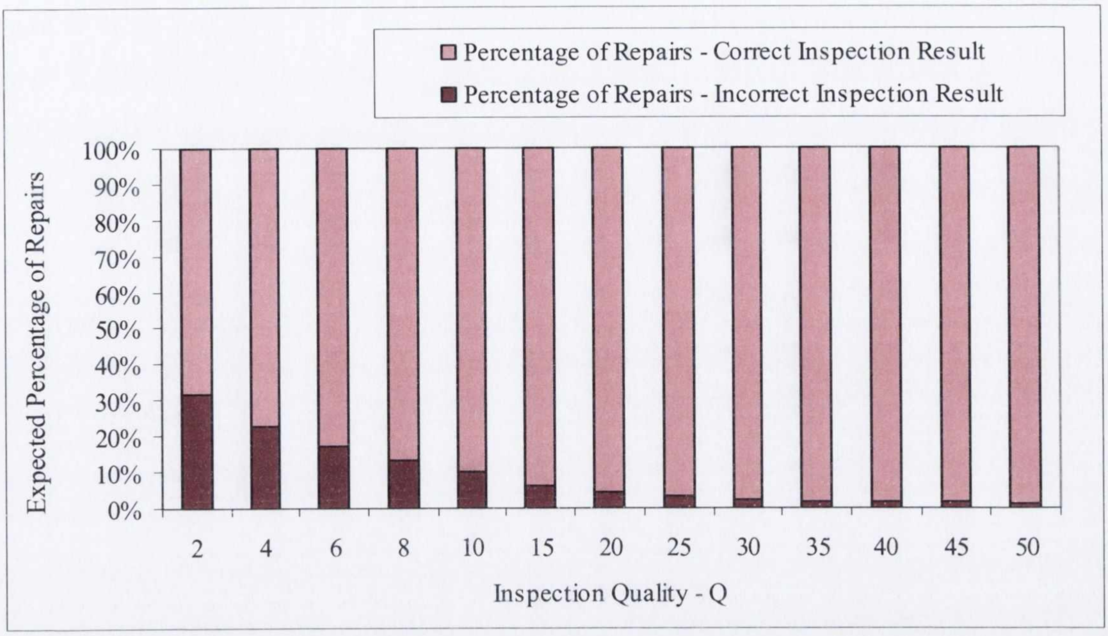


Figure 4.10. The effect of inspection quality on the proportion of repairs due to correct/incorrect inspection results ($\Delta T=6$)

By using a better quality inspection technique (e.g. Q=20, which is associated with a low level of noise), defects that could lead to failure of a component are repaired, rather than defects that are incorrectly sized (where repair is not yet required). Again, the exact point at which the percentage of repairs due to correct/incorrect inspection results plateaus (about Q=20 in this case) depends on the specific values of the parameters being considered. Therefore, the sensitivity of the number of repairs due to correct/incorrect inspection results to the growth kinetics and the allowable probability of failure will be studied in Section 4.3.1 and Section 4.3.5, respectively.

Using the developed methodology, it is also possible to look at the interaction of the quality of the inspection techniques for detection and sizing, and see how this affects the optimum inspection interval and the expected mean annual total cost. By varying the inspection quality of both techniques independently, the optimum combination of techniques can be found, which results in the minimum expected mean annual total cost. This provides the owner/manager of a structure with a useful decision tool when selecting a combination of inspection techniques to be used as part of a maintenance management plan, rather than using the same quality inspection technique for both stages of an inspection. Figure 4.11(a)-(f) illustrates for an inspection interval of 1 year to 6 years, how a different combination of inspection techniques can affect the expected mean annual cost of the structure. An inspection interval of 1 year to 6 years was studied, as principal inspections are recommended every 6 years in the UK (Vassie and Arya, 2006), and from 1 year to 6 years in Ireland (Duffy, 2004).

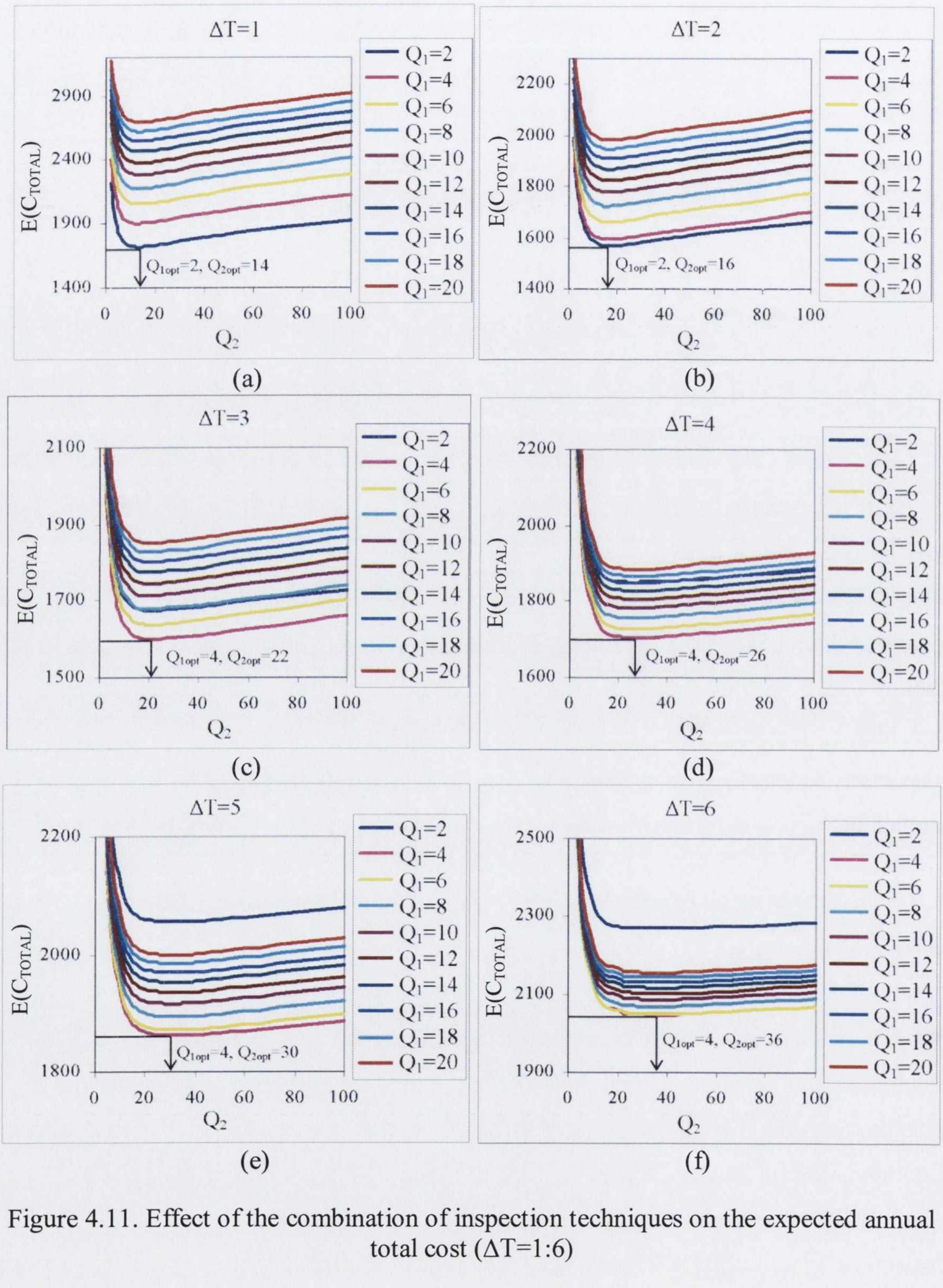


Figure 4.11. Effect of the combination of inspection techniques on the expected annual total cost (ΔT=1:6)

In relation to the first inspection, a higher quality technique, Q_1, reduces the noise associated with the inspection procedure (Equation 3.35), and therefore, more accurately determines which defects should be further assessed, which consequently reduces the number of failures due to undetected defects. Similarly, in relation to the quality of the second technique, Q_2, a better technique reduces the number of failures, as a higher proportion of defects are sized correctly and repaired when necessary. The parameters and cost coefficients used for the purpose of this example are those presented in Table 4.2. For each inspection interval, by varying the inspection quality of both techniques independently, the optimum combination of techniques and the corresponding expected mean annual total cost were found. The sensitivity of these results to the limit state considered (i.e. the allowable probability of failure) will be discussed in Section 4.3.5.

The results of the analysis are presented in Table 4.4. Based on the minimum expected mean annual total cost, the optimum inspection interval was 2 years, as presented in Table 4.4. Also presented is the percentage increase in the expected cost for each inspection interval, with respect to the minimum total cost (i.e. @$\Delta T=2$). This allows the owner/manager of a structure to see the effect of increasing the inspection interval when the optimum combination of inspection techniques is used to carry out the inspection for detection and sizing assessment.

ΔT	**Q_1**	**Q_2**	**$E(C_{TOTAL})$**	**Increase wrt Minimum**
1	2	14	1724	10%
2	2	16	1571	0%
3	4	22	1600	2%
4	4	26	1706	9%
5	4	30	1863	19%
6	4	36	2048	30%

Table 4.4. Optimum combination of inspection techniques for each inspection interval

For example, the expected increase in total annual cost by increasing the inspection interval from 2 years (the optimum inspection interval) to 6 years is 30%. The optimum quality of both inspections increases with inspection interval. The optimum combination of techniques, which results in the minimum expected mean annual total costs, are illustrated in Figure 4.11. The optimum inspection quality for detection is very low (with an associated high level of noise), varying from Q_1=2 to Q_1=4 for the limit state considered. The effect of the limit state on these results will be studied in Section 4.3.5. This first inspection represents a screening exercise to determine which defects require further inspection. Inaccurate inspection results in a higher number of second inspections or a small possibility of failure.

The optimum inspection quality for sizing is much higher, ranging from Q_2=14 to Q_2=36, representing moderate to high quality inspection techniques, which are associated with a lower level of noise. These values are highlighted by arrows in Figure 4.11(a)-(f), for an inspection interval ranging from 1 year to 6 years. In relation to the second inspection, the consequence of inaccurate sizing is more serious than an inaccurate inspection for detection, as defects greater than d_c that are not repaired are likely to result in failure (i.e. exceedance of the limit state). This suggests that the manager of a structure should focus more resources on phase 2 (sizing assessment) of the inspection than phase 1 (detection), as indicated by the results of this analysis.

When carrying out a traditional one stage inspection, the detection threshold (d_{min}) is essentially the same as the repair threshold, as all defects that are detected are automatically repaired. This two stage methodology allows the owner/manager of a structure to specify a detection threshold, so that defects that are present in the structure can firstly be detected and also specify a repair threshold, where defects sized and found to be greater than this threshold can be repaired. To illustrate the advantages of this approach, the expected mean annual total cost ($E(C_{TOTAL})$) and the expected number of failures (E(MeanAnnualFailures)) will be compared for structures which are maintained according to the traditional single stage inspection process and the developed two stage inspection process. This study will be discussed in the next section.

4.2.4. Single Stage Vs Two Stage Inspection Process

The aim of this section of the thesis is to demonstrate that a two stage inspection process is more optimal than a single stage inspection process in relation to minimisation of expected mean annual total cost and expected mean annual number of failures. The purpose of this is to quantitatively illustrate to an owner/manager the advantages of using this two stage process to select the optimum combination of inspection techniques and optimise the maintenance strategy of a structure or network of structures.

For the purpose of this example, a single stage inspection process is modelled through consideration of detection only, as in the literature (Higuchi and Macke, 2007; Macke and Higuchi, 2007; Schoefs and Clement, 2004; Rouhan and Schoefs, 2003; Rouhan and Schoefs, 2000; Frangopol et al., 1997; Mori and Ellingwood, 1994a). As such, it is assumed that any defect which is detected is automatically repaired, i.e. returned to the smallest group (defect size 0–0.1mm). By comparison, in the two stage process the first inspection is concerned with detecting defects present in the structure and the second inspection, which is only performed where defects are detected in stage 1, is carried out to size the detected defects (as illustrated in Figure 3.21).

The same input parameters (as in Table 4.2) were used for the single stage inspection and the two stage inspection process so that a direct comparison could be made of the two methods. For the single stage process, however, the detection threshold (d_{min}) was calculated based on the allowable probability of failure ($P_{f\_allowable}$) similar to the critical defect size for the two stage process. The relation between d_c (or d_{min} in this case) and $P_{f\_allowable}$ (i.e. limit state) was discussed in Section 4.2 and illustrated in Figure 4.1. In relation to the single stage inspection process, by varying Q_1 for each inspection interval the optimum technique which results in the lowest expected mean annual total cost was determined, as illustrated in Figure 4.12.

As expected, the optimum inspection quality for detection is directly proportional to the inspection interval. As the inspection interval is increased the defects have longer to grow to the larger groups where failure may occur. Therefore, a higher quality inspection technique (i.e. one which is associated with a lower level of noise) is needed to accurately detect defects as the inspection interval is increased. The optimum technique for each of the inspection intervals can be seen in Table 4.5. Similar to the two stage inspection, the optimum inspection interval is 2 years and the expected increase in total annual cost by increasing the inspection interval from 2 years (the optimum inspection interval) to 6 years is 29%. Since the inspection interval is so short (2 years) the optimum inspection quality for detection is a low quality technique, Q_1=6. The quantification of these parameters as a function of the actual inspection techniques available is beyond the scope of this thesis, but it is recognised that this is important for future work.

Using these results, the expected mean annual total cost ($E(C_{TOTAL})$) and the expected mean annual number of failures (E(MeanAnnualFailures)) were compared for a single stage inspection and a two stage inspection. When comparing the two processes, the optimum inspection technique (single stage process) or combination of techniques (two stage process) were used. For this example the optimum combination of techniques for the two stage process was determined in Section 4.2.3, and presented in Table 4.4.

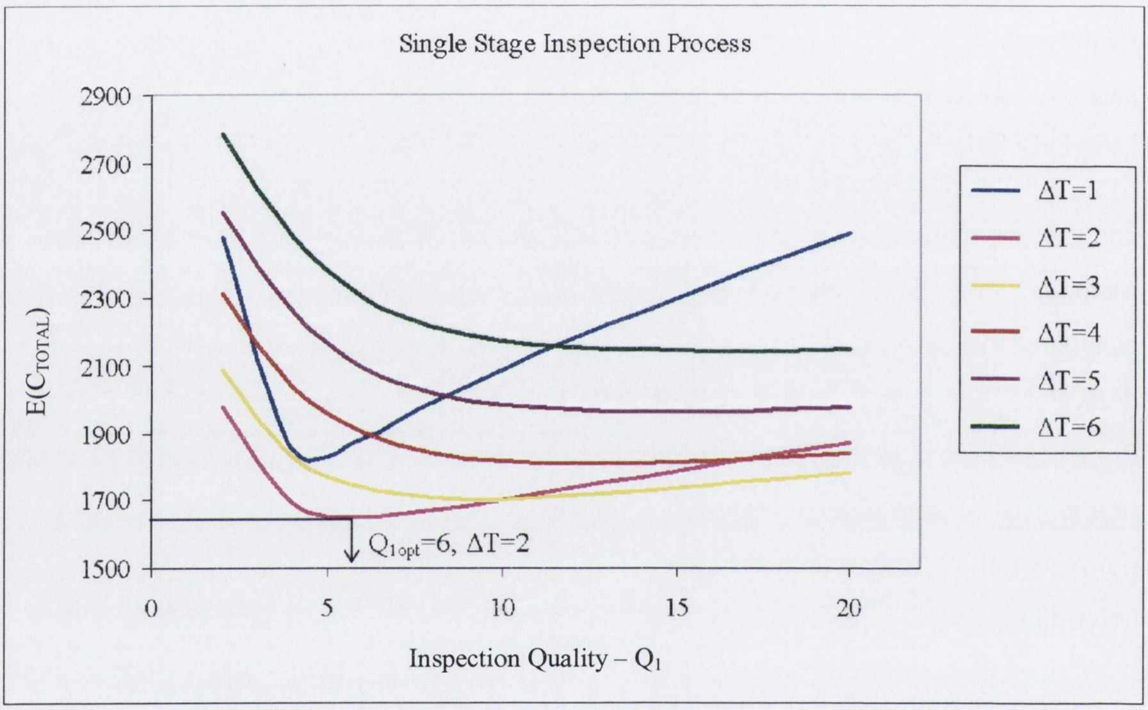


Figure 4.12. Effect of the inspection technique on the expected annual total cost (ΔT=1:6)

ΔT	Q_1	$E(C_{TOTAL})$	Increase wrt Minimum
1	4	1853	12%
2	6	1656	0%
3	10	1704	3%
4	12	1812	9%
5	14	1964	19%
6	16	2143	29%

Table 4.5. Optimum inspection technique for each inspection interval

Figure 4.13 and Figure 4.14 facilitate the direct comparison of the implication of the choice of a single stage or two stage methodology. For each inspection interval, a comparison was made between the costs and number of failures associated with each process, using the optimum techniques associated with each method, which were determined above. Figure 4.13 shows that the two stage inspection process results in lower expected mean annual total costs than the single stage inspection process, for the optimum combination of techniques. Although the single stage inspection process results in lower inspection costs (since only 1 inspection is carried out), this is outweighed by the lower cost of failure associated with the two stage process.

This result is confirmed by studying the expected number of failures, considering both the failures between inspections and the failures in an inspection year, as discussed in Section 3.4.2 and Section 3.4.3. The number of failures for the two stage process is less than the number of failures for the single stage inspection process for each value of inspection interval. The high quality sizing assessment results in accurate sizing of defects, thus defects that are greater than the critical defect size are repaired, resulting in fewer failures.

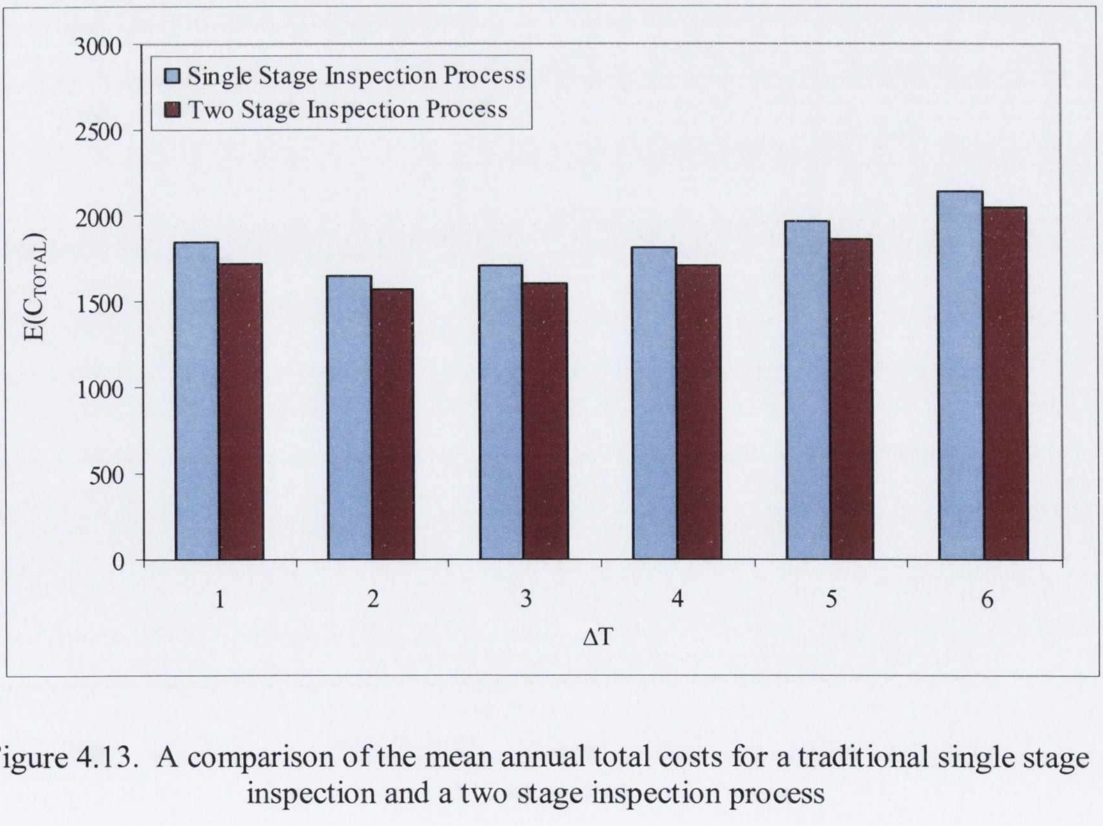


Figure 4.13. A comparison of the mean annual total costs for a traditional single stage inspection and a two stage inspection process

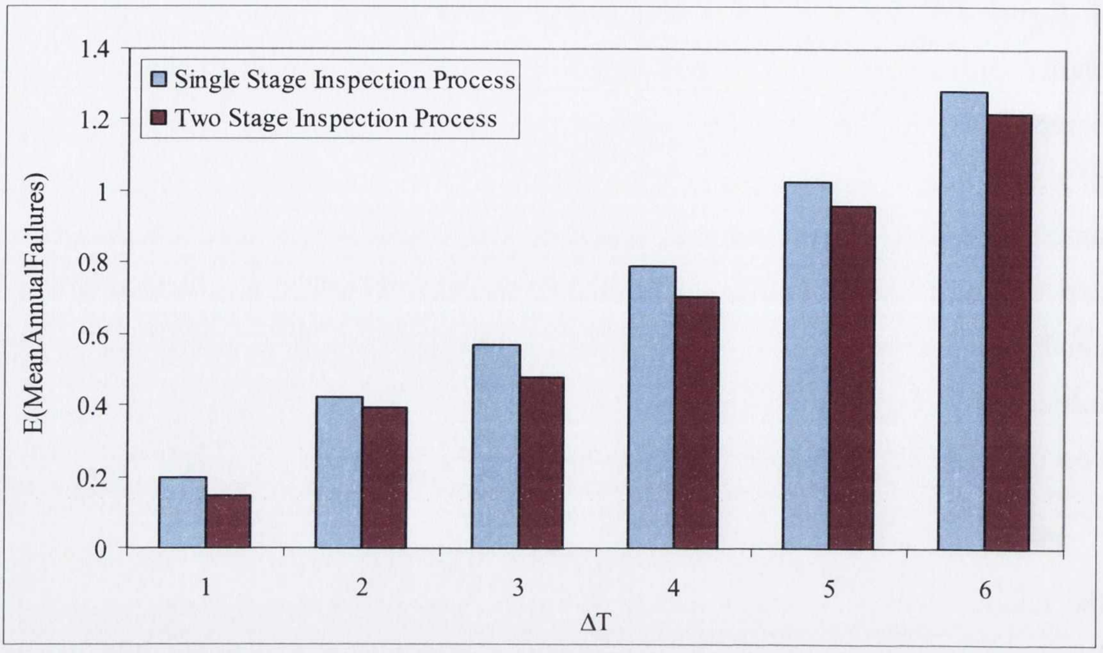


Figure 4.14. A comparison of the mean annual number of failures for a traditional single stage inspection and a two stage inspection process

This demonstrates that a two stage inspection process is more beneficial to an owner/manager of a group of structures in terms of cost and possible number of failures. The screening exercise, followed by an optimum quality inspection for sizing is effective at ensuring that necessary repairs are carried out when defects are greater than the critical defect size, resulting in fewer failures and lower costs. It is recognised that the results presented are for one specific example, considering the parameters in Table 4.2. For this reason, the sensitivity of these results to the growth kinetics of the defect and the allowable probability of failure (i.e. limit state considered) will be studied in Section 4.3.1 and Section 4.3.5, respectively.

4.3. SENSITIVITY STUDIES

By performing a sensitivity analysis, it is possible to see how the growth parameters, inspection qualities, modes of failure or limit states influence the overall results of the maintenance management strategy. This is useful for the owner/manager to gain insight into the sensitivity of the model and the implications of changing parameters or assumptions. Considering for example a change in the environmental conditions (i.e. affecting deterioration mechanism) or limit state (e.g. from SLS to FLS), the owner/manager would have to change the maintenance strategy accordingly. It is important, therefore, to have an indication of how the optimal maintenance strategy will be affected if such changes do arise. In the case of a reinforced concrete bridge for example, the effect of using de-icing salts (which might alter the deterioration mechanism) on the structure could be assessed. On this basis, the sensitivity of the results (such as the optimum inspection interval, the optimum inspection quality and the number of failures) to changes in the values of parameters such as the number of groups, cost parameters, growth parameters, probability of failure parameters and the allowable probability of failure are studied in this section of the thesis.

4.3.1. Effect of the Number of Groups

In this thesis, the range of defect sizes is broken into 10 groups (similar to the National Bridge Inventory Condition Rating system in the US). Although Pontis (the most commonly used BMS in the US) divides condition states into 5 groups, 10 groups was chosen to reduce the modelling error due to discretisation, which can be considered a limitation of a Markov process (Kong and Frangopol, 2005). It is interesting, however, to study the effect of the number of defect groups on the results of the analysis. On this basis, the inspection, repair and failure cost were determined using 5 groups, and the optimum inspection interval was determined using the values of the parameters presented in Table 4.2.

As in Section 4.2, the defect size is assumed to vary from 0-1.0mm. Again, it was assumed that the standard deviation of the range of defect sizes in a group is independent of the mean defect size, and a constant value was assumed for each group, with the CoV decreasing as the defect group increases. It is noted that the standard deviation of the range of defect sizes (i.e. σ_d) in a group is unknown and may be larger for a larger range within a group. For the purpose of calculation, a value of 0.04 is assumed in this section of the thesis. It is recognised that σ_d may depend on the number of groups (or range of defect sizes within a group) and the deterioration mechanism being considered. Therefore, further investigation of this parameter behaviour is recommended for future work. It was also assumed initially that there were 100 defects in the smallest defect group, and no defects in all other groups (i.e. taken to represent a new structure), Table 4.6.

Defect Group	Range From	Range To	$\overline{d}$	σ_d	CoV	Initial Population
1	0	0.2	0.1	0.04	40.0%	100
2	0.2	0.4	0.3	0.04	13.3%	0
3	0.4	0.6	0.5	0.04	8.0%	0
4	0.6	0.8	0.7	0.04	5.7%	0
5	0.8	1.0	0.9	0.04	4.4%	0

Table 4.6. Defect group data used in the model (for 5 groups)

Using this method, the variation of expected costs with the inspection interval was determined, and is presented in Figure 4.15. For comparison purposes, the corresponding variation of expected costs using 10 groups is presented in Figure 4.16. The results indicate that reducing the number of groups leads to an increase in the expected repair cost (for small inspection intervals in particular) and the expected failure cost (for large inspection intervals in particular). This can be attributed to the increased modelling error due to discretisation of the defect size range into only 5 groups.

Using only 5 groups results in an increase in the repair cost with respect to 10 groups ranging from 89% ($\Delta T=1$) to 34% ($\Delta T=10$). With an inspection interval of 1 year the inspection/repair process has a higher influence on the overall results (since there are more inspections over the lifetime of the structure) which leads to a higher difference in repair cost for $\Delta T=1$ than $\Delta T=10$. However, considering the failure cost, using only 5 groups results in an increase in failure cost ranging from 8% ($\Delta T=1$) to 88% ($\Delta T=10$). The difference in failure cost is higher for $\Delta T=10$ because there is more time between inspections for failure to occur. Since the values for PoD/PFA and PGA/PWA are calculated for each group based on the inspection quality, noise and the mean defect size, a different value is calculated for each group which results in a value for the probability of repair for each group. Using medium/high quality inspection techniques, the probability of repair will increase dramatically for the groups where the mean defect size is greater than the critical defect size. However, since one value is calculated for each group, the group containing the critical defect size will have a high probability of repair (e.g. 0.8), even though there is a portion of the defects in the group that are less than the critical defect size. The greater the range of defects within a group, the higher the proportion of defects within that group which are repaired when they are less than the critical defect size. This leads to an increase in the number of repairs using 5 defect groups rather than 10.

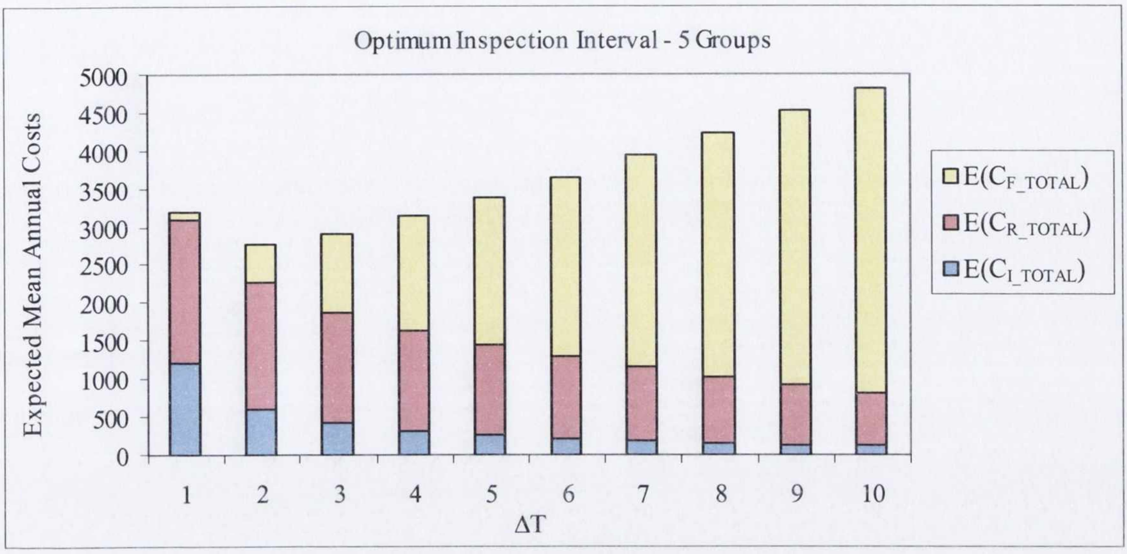


Figure 4.15. The effect of the inspection interval on expected mean annual costs using 5 groups

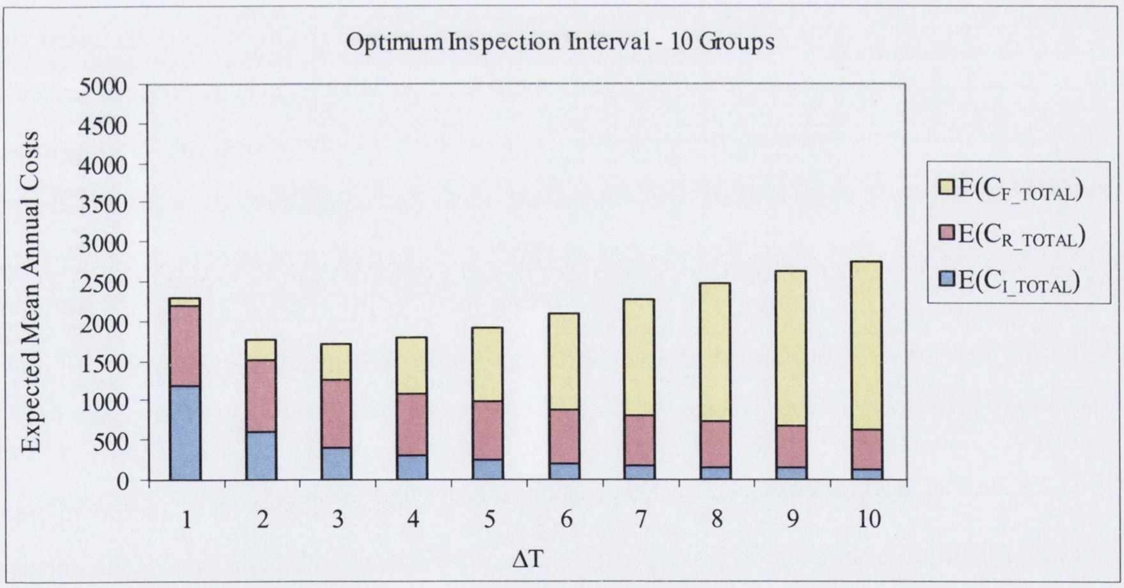


Figure 4.16. The effect of the inspection interval on expected mean annual costs using 5 groups

On the other hand, since there is a certain probability of repair for this group (e.g. 0.8), only a certain percentage of the defects that are actually greater than the critical defect size will be repaired. Again, the greater the range of defects within a group (i.e. the fewer the number of groups), the higher the proportion of defects within that group which are greater than the critical defect size and are not repaired, which can ultimately lead to failure. Using a smaller number of groups to define the range of defect sizes results in the prediction of a higher number of failures and a higher corresponding failure cost.

Therefore, using a larger number of groups results in a reduction in the modelling errors and allows owners/managers to make more accurate and informative decisions when developing a maintenance management strategy for a structure or group of structures. As discussed, further work is required to identify the behaviour of these parameters (number of groups, standard deviation of defect sizes) as a function of factors such as the deterioration mechanism etc., and as such is not considered further here.

4.3.2. Effect of Ratio of inspection cost coefficients, η

It is assumed in this thesis that the inspection cost coefficient of a second inspection (k_{I2}) is only a fraction of the inspection cost coefficient of a first inspection (k_{I1}) since many of the fixed costs associated with an inspection process have been included during the setup of the first inspection. The fixed component of an inspection process arises from the costs such as accessing the area of the structure to carry out the inspections, the possible temporary closure of the structure and transportation of equipment to the structure. According to Straub and Faber (2005), the fixed component of an inspection forms the major part of the overall costs. Therefore, in this thesis it is assumed that the ratio of the cost coefficient of the second inspection to the cost coefficient of the first inspection is 0.2.

However, it is recognised that this ratio may be different depending on the inspection processes being considered, and the structure being inspected. On this basis, the sensitivity of the variation in costs and the optimum inspection interval to the value of this ratio, η, was studied (varying from η=0.1 to η=0.6). All other parameters were assigned the values which were presented in Table 4.2 and were kept constant for each of the cases considered in this section to allow a direct comparison to be made. The results of the sensitivity study are illustrated in Figure 4.17.

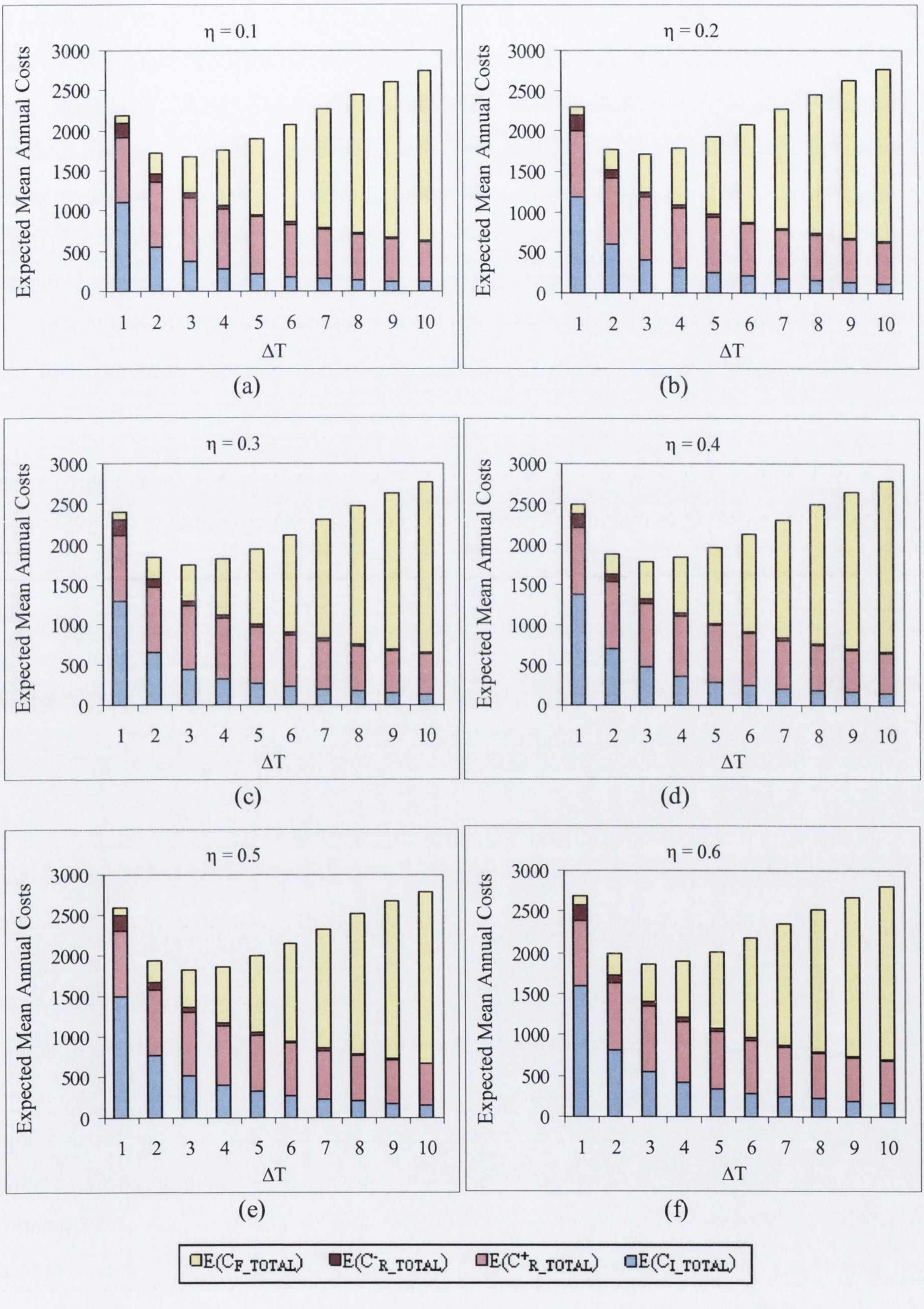


Figure 4.17. Effect the ratio of cost coefficients on the optimum inspection interval

Figure 4.17(a)-(f) illustrate that increasing this ratio (η) has the effect of increasing the inspection cost, while all other costs remain the same. For an inspection interval of 1 year, varying η from 0.1 to 0.6, results in a 45% increase in the inspection cost, and a 22% increase in the expected total cost. However, this percentage increase in the expected total cost is inversely proportional to the inspection interval. Although the percentage increase in the inspection cost is 56% for an inspection interval of 10 years, there is only a 2% increase in the expected total cost. Therefore, although the inspection cost does increase with η, this is not considered significant enough to increase the expected total costs of the structure for short inspection intervals such that it would increase the optimum inspection interval.

For each of the cases considered in Figure 4.17(a)-(f), the optimum inspection interval is 3 years. The results illustrate that the optimum inspection interval of 3 years is not sensitive to changes in the value of the ratio of the inspection cost coefficient for the second inspection (k_{I2}) to the inspection cost coefficient for the first inspection (k_{I1}). This demonstrates to owners/managers of a structure that if there is change in the ratio of cost coefficients for inspections (e.g. if the fixed cost is less significant), this will not affect the optimum inspection interval significantly.

4.3.3. Effect of Growth Kinetics (g parameter)

The aim of this study is to look at the effect of the defect growth kinetics (abrupt/gradual) on the expected mean annual total cost of a structure, for a range of inspection intervals (ΔT=1:6, as discussed previously). Also, the effect of the growth kinetics on the mean number of defects in each group and on the expected mean annual number of failures was studied for a range of inspection intervals. As discussed in Section, 3.4.1, many forms of deterioration kinetics can be simulated using the methodology developed, ranging from gradual growth to very abrupt growth. For example, gradual growth simulated by g=5 may represent deterioration due to steel corrosion or carbonation in reinforced concrete, whereas very abrupt growth simulated by g=-5 may represent deterioration (e.g. crack growth) due to the effect of loading. For an intermediate form of growth kinetics (e.g. cracking due to loading in reinforced concrete which subsequently leads to reinforcement corrosion), a value of g=0 can be used.

It is of interest to determine how the growth kinetics (deterioration mechanism) of the defect affects the distribution of the defects within the groups, which in turn affects the number of failures. An interesting investigation of the effect of inspection quality on the number of failures for abrupt and gradual growth was also carried out. To illustrate the relative effect of inspection quality for different forms of defect growth, initially the quality of both inspection techniques were changed simultaneously (i.e. Q=20 implies that Q_1=20 and Q_2=20). Subsequently, the effect of the deterioration kinetics on the optimum combination of techniques for detection and sizing was investigated. These results were then used to compare the performance of the single stage and two stage inspection processes for different forms of deterioration.

Figure 4.18 shows the effect of the defect growth on the expected mean annual total cost using the values of the parameters presented in Table 4.2. Table 4.7 presents the correlation between the g parameter in the Markov model and the type of defect growth. There is a drastic reduction (ranging from 60% for ΔT=1 to 82% for ΔT=6) in the costs as the growth becomes more gradual. For an inspection interval of 3 years, the cost for gradual growth (g=5) is only 20% of the costs for very abrupt growth (g=-5). The optimum inspection interval also changes, increasing from 1 year to 4 years as the growth moves from very abrupt to gradual.

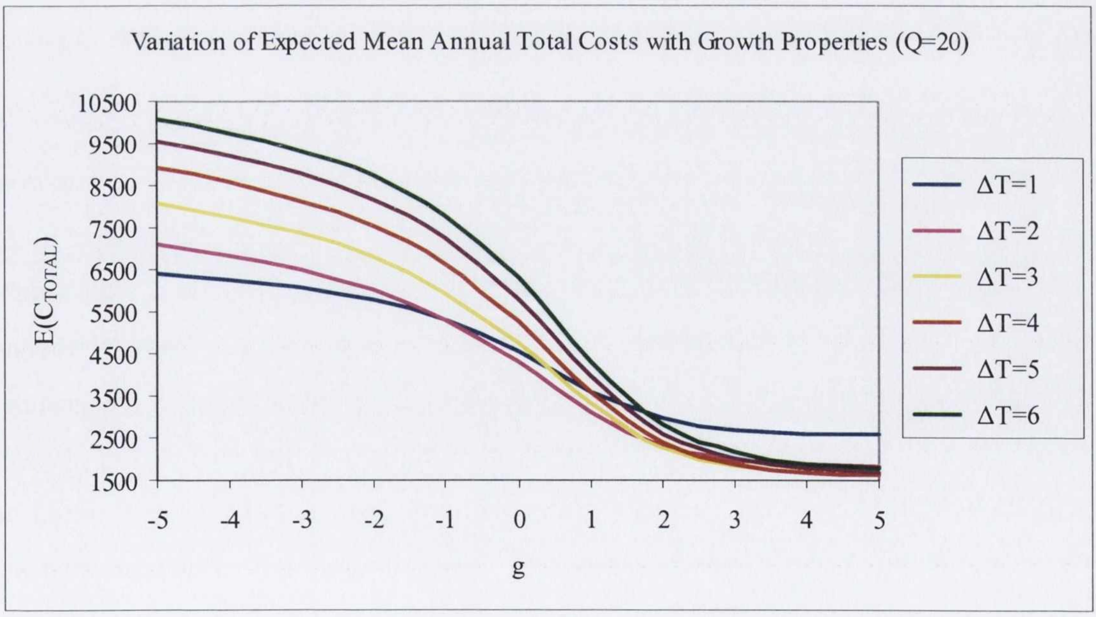


Figure 4.18. Effect of g on expected mean annual total costs (ΔT=1:6)

g	Defect Growth
-5	Very abrupt
0	Abrupt
5	Gradual

Table 4.7. Correlation between g parameter and defect growth

For the case considered the inspection quality is Q=20. Reducing the inspection quality would affect the optimum inspection interval (i.e. reducing it), but similar to the case considered here, the costs would reduce as the growth becomes more gradual. The effect of inspection quality on the results of the expected costs and number of failures will be studied later in this section. Based on these initial results, the inspection strategy should be selected as a function of the deterioration mechanism being considered. In this case the growth rate is constant (α=0.5). However, the results of an experimental study which investigates the growth rates of different repair materials will be presented in Chapter 5. The effect of the growth rate on the results of the optimal maintenance strategy will then be discussed in Chapter 6.

The number of repairs and failures carried out, and hence the cost, depends on the distribution of the defects within the defect groups once stabilisation of the process has occurred since each group has a different probability of repair and failure, as discussed in Section 3.4. On this basis, Figure 4.19 illustrates the distribution of defects between the different groups for abrupt and gradual growth, for an inspection interval of 3 years.

As introduced in Section 3.4.1, for very abrupt growth (g=-5) defects can grow from the first group directly to larger groups, and in this case there are 67% of the defects in groups 7-10. These defects may then results in repair or failure, which leads to a high number of defects in group 1 (31%), leaving just 2% of the defects in groups 2-6. For g=0, the growth is still quite abrupt, but the defects are more evenly distributed between the groups. For gradual growth (g=5), most of the defects are distributed between groups 1-6 (82%). Due to the gradual growth, even for a large inspection interval most defects are repaired before they grow to groups 8-10, where failure is likely. Figure 4.20 illustrates that for an inspection interval of 6 years, only 15% of the defects lie in groups 8-10. This demonstrates to owners/managers how the deterioration mechanism can affect the distribution of defect sizes, and consequently, the number of repairs and failures which are expected to occur.

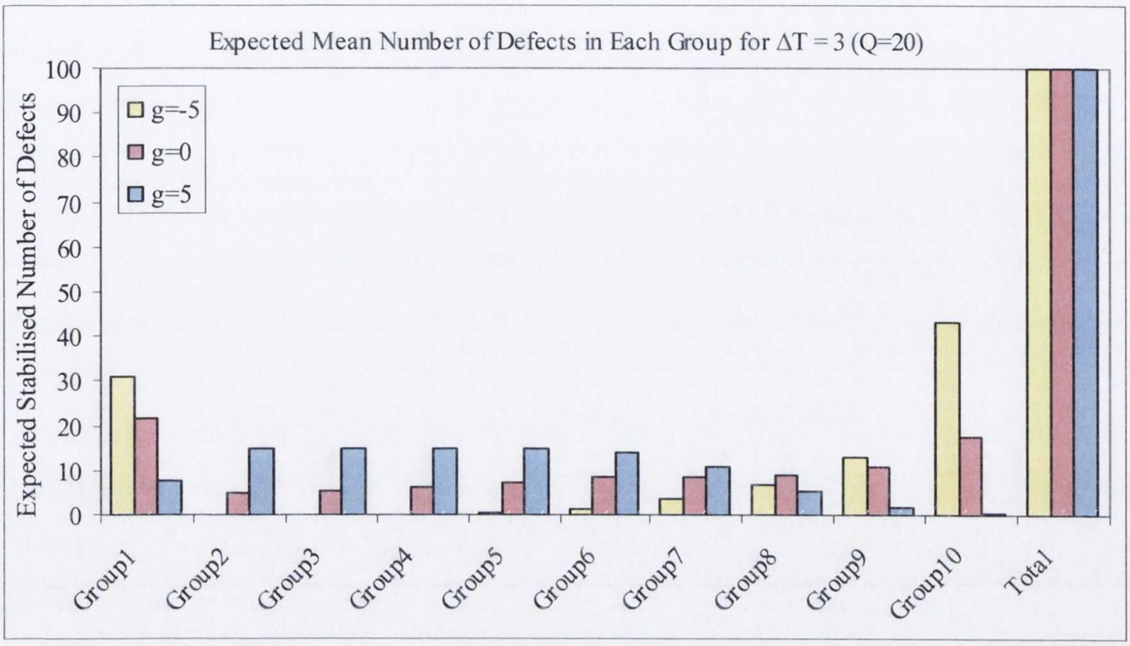


Figure 4.19. Distribution of defects between groups for gradual and abrupt growth (ΔT=3)

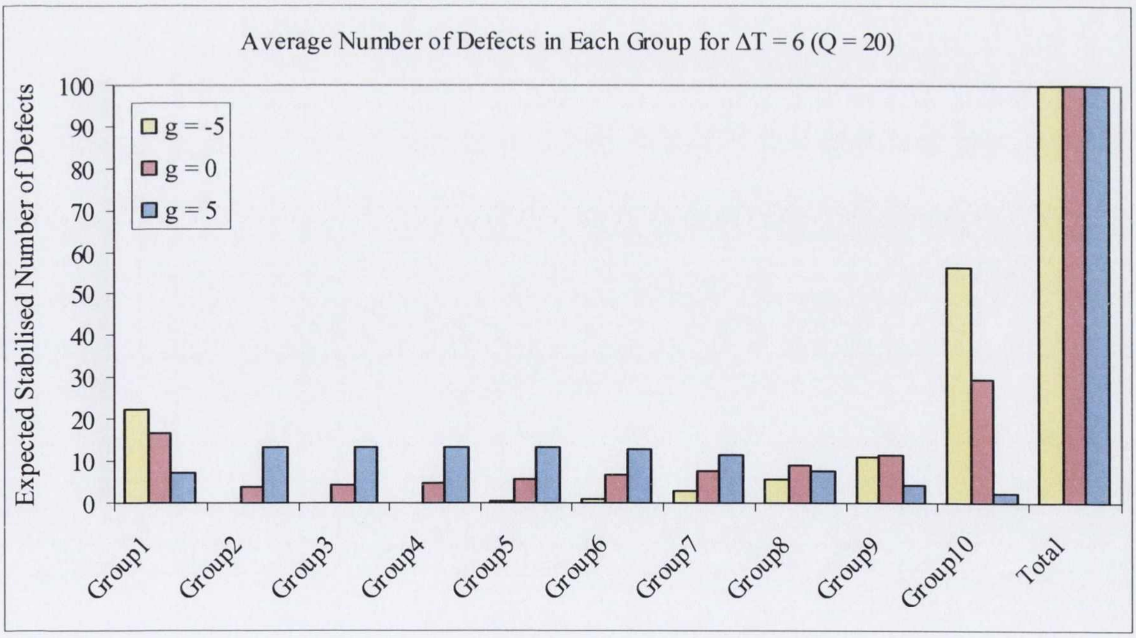


Figure 4.20. Distribution of defects between groups for gradual and abrupt growth (ΔT=6)

Since a more abrupt form of deterioration results in a higher number of defects in the larger groups, this in turn leads to a higher number of failures. On this basis, the expected mean annual number of failures (E(MeanAnnualFailures)) for each inspection interval was studied for different growth parameters, using a moderate/high quality inspection technique (Q=20), Figure 4.21. As expected, the very abrupt growth resulted in a higher proportion of failures than for gradual growth. For an inspection interval of 6 years for example, the number of failures for gradual growth (g=5) was only 11% of the number of failures for very abrupt growth. For abrupt growth, where g=0, the number of failures was 57% of the number of failures for g=-5. The relative difference in the number of failures increases as the inspection interval decreases, where, for an inspection interval of 2 years, the number of failures for gradual growth (g=5) was only 7% of the number of failures for very abrupt growth. For abrupt growth, where g=0, the number of failures was 42% of the number of failures for g=-5. In this case the number of failures for ΔT=1 is negligible for all forms of defect growth, as illustrated in Figure 4.21. However, using a poor inspection quality (e.g. Q=2) more failures occur for abrupt and very abrupt growth for an inspection interval of 1 year, as illustrated in Figure 4.22. Although, a similar trend remains for an inspection interval of 6 years, with the number of failures for gradual growth (g=5) only 19% of the number of failures for very abrupt growth (g=-5).

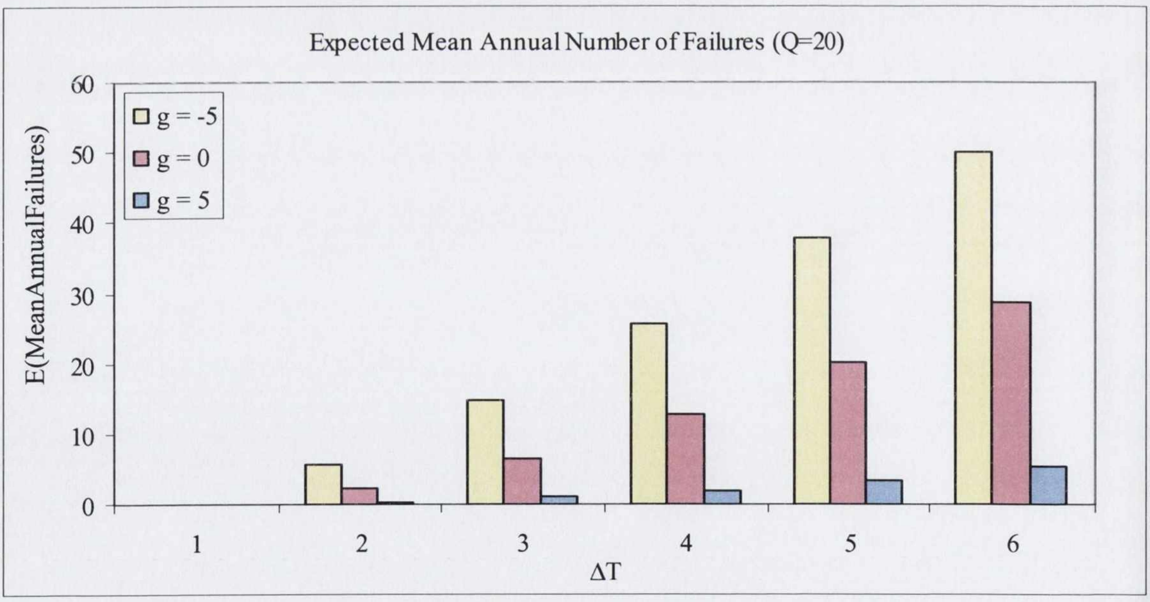


Figure 4.21. Expected mean annual number of failures for gradual and abrupt growth (Q=20)

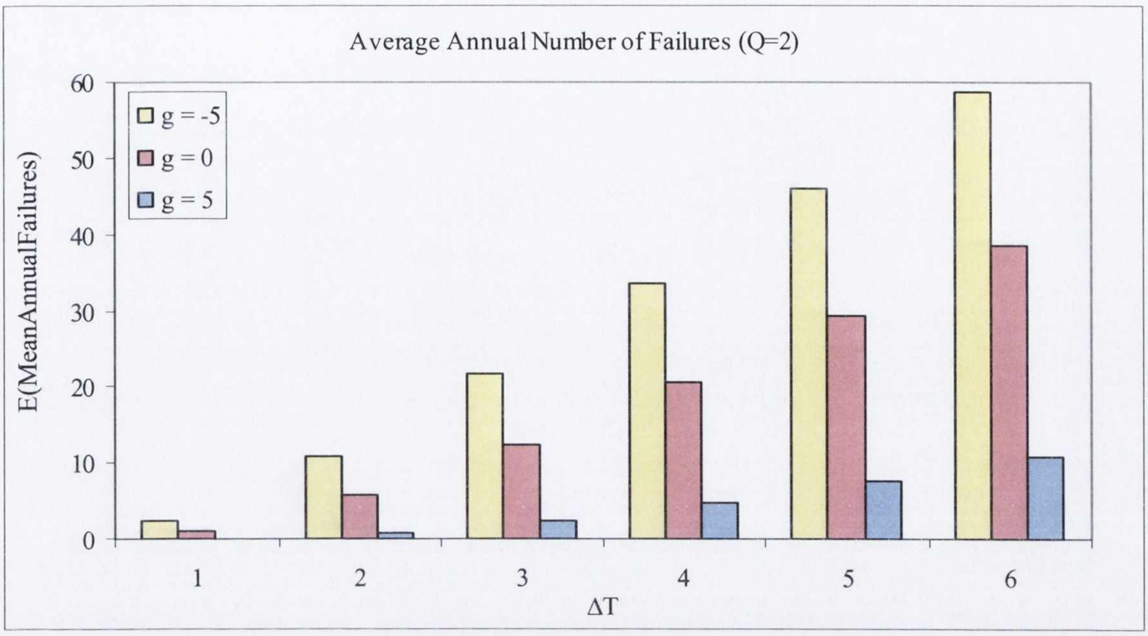


Figure 4.22. Expected mean annual number of failures for gradual and abrupt growth (Q=2)

This illustrates that the inspection quality affects the number of failures for different forms of growth. This will be investigated further to give owners/managers an understanding of how the inspection quality affects the maintenance strategy for different deterioration kinetics, and whether it is worth investing more capital using high quality inspection techniques to reduce the number of failures. On this basis, Figure 4.23 to Figure 4.25 illustrate the effect of inspection quality on the number of failures for the various forms of defect growth. For $\Delta T=1$, each year is an inspection year so all of the failures are due to the probability of failure given that no sizing assessment has been carried out (P(Failure | No Assessment)) and the probability of failure given that no repair has been carried out (P(Failure | No Repair)). There are no years between inspections for failures to take place. Using high quality inspection techniques, the defects in groups less than the critical defect size are not repaired, but there is still a small chance of failure before an inspection is carried out the following year. The magnitude of the failure probability for these groups is controlled by the specified allowable probability of failure (i.e. the limit state being considered, SLS, FLS or ULS). These failures occur in groups 1-6 (where the defect size is less than d_c). The distribution of the defects within the groups was illustrated in Figure 4.19 ($\Delta T=3$), for different forms of deterioration kinetics.

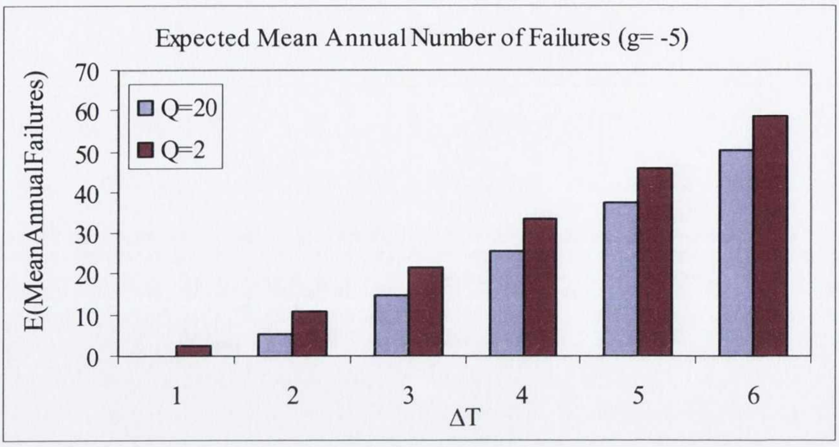


Figure 4.23. Expected mean annual number of failures for very abrupt growth

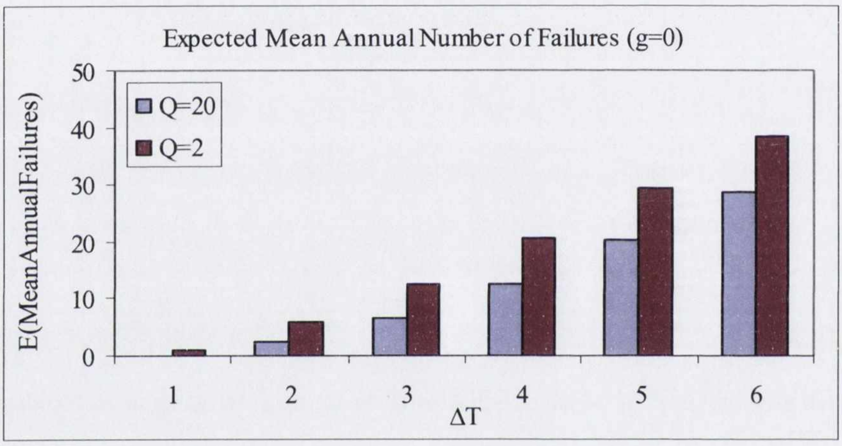


Figure 4.24. Expected mean annual number of failures for abrupt growth

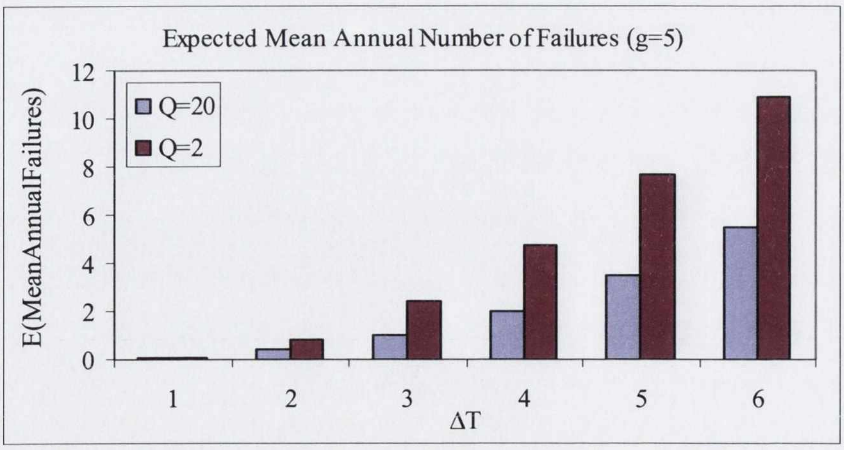


Figure 4.25. Expected mean annual number of failures for gradual growth

For very abrupt growth (g=-5), there are very few defects in these groups (1-6). Therefore, for an inspection interval of 1 year there are very few failures, as all defects in the larger groups are repaired after inspection. Figure 4.23 indicates that for an inspection interval of 1 year, the inspection quality has an effect on the number of failures for abrupt growth. There are more failures for lower quality inspection techniques, Q=2. The number of failures is negligible (0.03) using a moderate/high quality inspection technique (Q=20).

This is because there are so many defects in the larger groups, and the defects are larger than the critical defect size so should be repaired. Higher quality inspection techniques (i.e. Q=20) lead to more defects being sized correctly and repaired. This in turn leads to fewer failures, although this does result in a higher inspection cost for owners/managers of structures. As discussed in Section 4.2.3, the optimum combination of techniques can be determined for the two stage inspection process to minimise expected total costs. This will be studied later in this section. Similar results can be seen for abrupt growth where g=0, Figure 4.24. Although for an inspection interval of 1 year the number of failures is also negligible using a moderate/high quality inspection technique (Q=20), there is an increase in the number of failures to 0.09.

However, comparing very abrupt growth (g=-5) and abrupt growth (g=0), the percentage difference in the number of failures by reducing the inspection quality from Q=20 to Q=2 is higher for abrupt growth (g=0) for all other inspection intervals (i.e. $\Delta T=2$-$\Delta T=6$). Reducing the inspection quality from Q=20 to Q=2 leads to an increase in failures ranging from 91% ($\Delta T=2$) to 17% ($\Delta T=6$) for very abrupt growth, whereas, this leads to an increase in failures ranging from 148% ($\Delta T=2$) to 34% ($\Delta T=6$) for abrupt growth. This means that it is more beneficial to use a higher quality inspection technique for abrupt growth than for very abrupt growth.

When defect growth is gradual (g=5), a significant portion of the defects are in groups 1-6, Figure 4.19. Defects in these groups can lead to a small number of failures. Figure 4.25 indicates that for gradual growth the number of failures is independent of inspection technique for a 1 year inspection interval. Therefore, it would be an inefficient use of resources in this case to use a high quality inspection technique, where a low quality inspection technique (with a lower inspection cost) results in the same number of failures. This is because the defects that are failing are less than the critical defect size and are therefore not repaired. There are actually less failures for the lower quality inspection techniques (Q=2) due to incorrect inspection results which indicate that the defects are actually greater than the critical defect size (when they are not) and are therefore repaired.

However, for an inspection interval of 6 years, the effect of the inspection quality on the number of failures is quite different. Between inspections, defects in the larger groups which are greater than the critical defect size (groups 7-10) have a high probability of failure. Therefore, for very abrupt growth (g=-5) a lot of failures occur due to a high proportion of defects in groups 7-10. In this case, the quality of the inspection has little effect on the number of failures. The growth is so abrupt between inspections that defects grow to the larger defect groups regardless of whether repairs had been carried out at the previous inspection year. There is only a 16% increase in the number of failures by reducing the quality of the inspection techniques from Q=20 to Q=2. For less abrupt growth (g=0), the inspection quality has more of an effect, with the number of failures increasing by 34% when the inspection quality is reduced from Q=20 to Q=2.

For gradual growth however, the quality of inspection has a much larger influence on the expected mean annual number of failures for an inspection interval of 6 years. Using a high quality inspection technique, defects greater than the critical defect size are repaired and return to the smallest group. These defects subsequently grow gradually from one group to the next in the interval between inspections. Using a low quality inspection technique, defects that should be detected and repaired are not. These defects then grow larger in the interval between inspections and subsequently fail. For an inspection interval of 6 years there is a 99% increase in the number of failures by reducing the inspection quality from Q=20 to Q=2.

Therefore, the effect of the quality of techniques on the number of failures depends on the type of deterioration being considered and on the inspection interval chosen. The growth kinetics not only affects the optimum inspection interval, but also the optimum quality of inspection techniques. Depending on the inspection interval, the effectiveness of increasing the quality of the inspection technique can vary for different forms of growth kinetics. It is important for the owner/manager of a structure to be aware of the growth properties and choose inspection methods accordingly. Increasing the inspection quality doesn't always result in a lower number of failures (or a lower cost). As demonstrated in

Figure 4.25 ($\Delta T=1$), in some cases increasing the inspection quality may be an inefficient use of limited financial resources.

As well as investigating the effect of the growth kinetics on the number of failures, it is also interesting to look at the percentage of repairs which are carried out due to correct/incorrect inspection results for different forms of deterioration, to determine the benefits of using higher quality techniques in relation to reducing the number of unnecessary repairs. This was discussed in Section 4.2.3 for the specific case were g=3 (i.e. moderate to gradual defect growth). Figure 4.26 presents the results of this analysis for $\Delta T=3$ and $\Delta T=6$. For very abrupt growth (g=-5), most repairs (99-100%) that are carried out are due to correct inspection results, regardless of the quality of the inspections and the inspection interval, Figure 4.26(a)-(b). Since the growth is so abrupt, most defects (67%) are in groups 7-10, and only 2% of the defects are in groups 2-6 (Figure 4.19) where defects are more likely to be incorrectly sized, leading to unnecessary repairs. Therefore, a very high percentage of repairs that are carried out are necessary. This demonstrates to the owner/manager of a structure that, in relation to repairs due to very abrupt growth, increasing the inspection quality or inspection interval has a negligible effect on the percentage of unnecessary repairs and would be an inefficient use of resources.

For abrupt growth (g=0), increasing the inspection quality does result in a reduction in the number of unnecessary repairs. In this case, increasing the inspection quality from Q=2 (poor quality) to Q=50 (very high quality) results in a reduction in the percentage of repairs carried out due to incorrect inspection results from 11% to 1% for $\Delta T=3$, and from 7% to 0% for $\Delta T=6$. This is due to defects being well distributed between groups (Figure 4.19), where defects less than the critical defect size can be incorrectly sized leading to unnecessary repairs. As discussed in Section 4.2.3, the proportion of unnecessary repairs (due to incorrect inspection results) is inversely proportional to the inspection interval. This study demonstrates that this is true for all forms of defect growth kinetics. Similarly, for gradual growth (g=5), increasing the inspection quality results in a reduction in the number of unnecessary repairs, ranging from 53% to 3% for $\Delta T=3$ and from 37% to 1% for $\Delta T=6$. Again, referring back to Figure 4.19, most of the defects are distributed between groups 1-6 (82%) for gradual growth. Poor quality inspection techniques result in inaccurate sizing in these groups and hence unnecessary repairs.

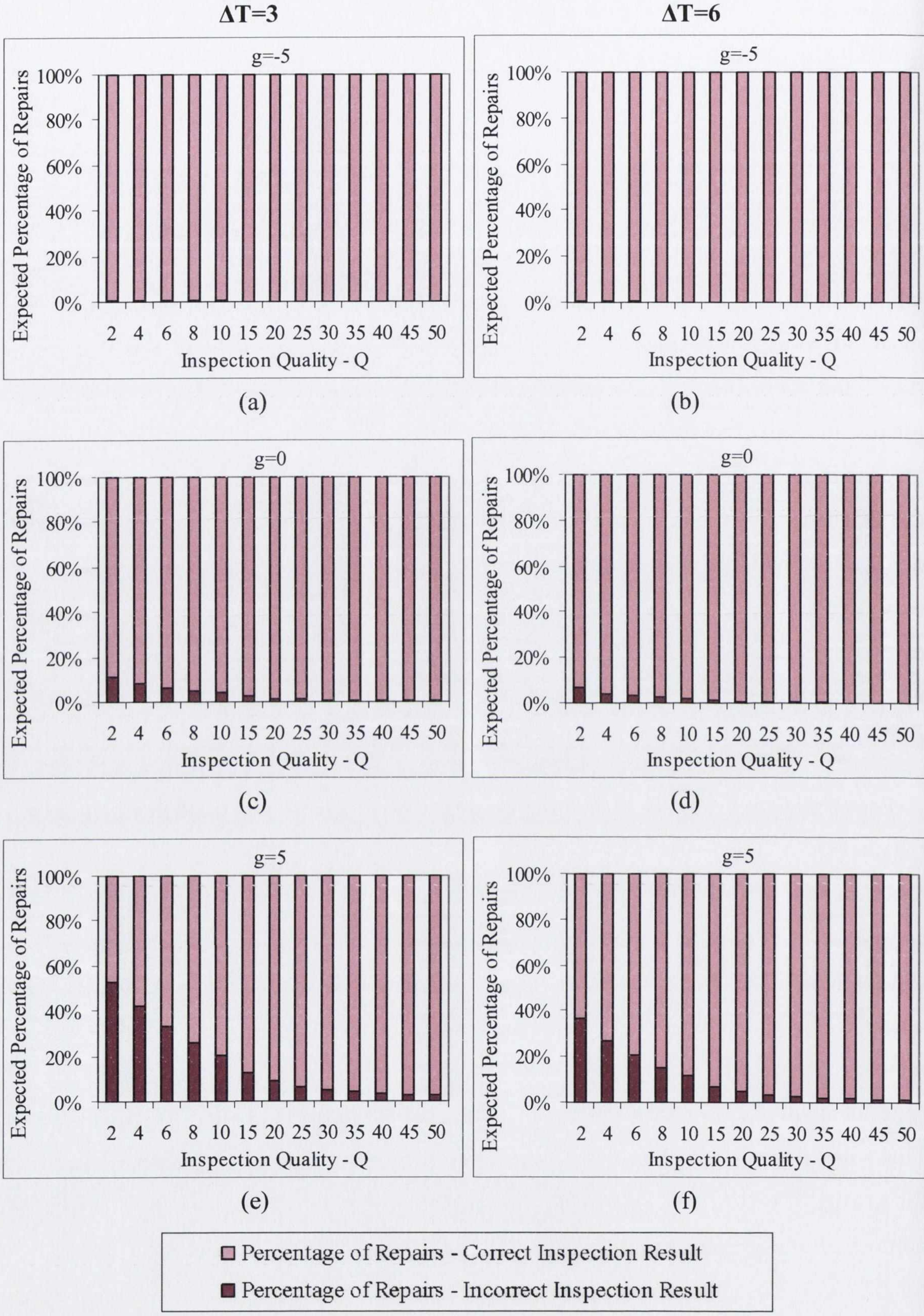


Figure 4.26. Effect of growth kinetics on the percentage of repairs carried out due to correct/incorrect inspection results

In summary, these results indicate that the more gradual the defect growth the more sensitive the percentage of repairs due to correct/incorrect inspection results is to the inspection quality. Therefore, in relation to the percentage of unnecessary repairs of structures subject to a very abrupt form of deterioration kinetics, increasing the inspection quality would have a negligible effect on the percentage of unnecessary repairs for all inspection intervals and to do so would be an be an inefficient use of resources, whereas, for structures subject to gradual deterioration, increasing the inspection quality can be beneficial in reducing the percentage of unnecessary repairs.

Since the sensitivity of the results of the maintenance model (such as the number of failures and the percentage of necessary/unnecessary repairs) to the inspection quality depends on the deterioration kinetics, it is of interest to compare the performance of the developed two stage inspection process and the traditional single stage inspection process for different forms of deterioration (i.e. very abrupt, abrupt or gradual). It was demonstrated in Section 4.2.4 that the two stage inspection process results in a lower expected total cost and expected number of failures than the traditional one stage process for the specific case considered (g=3).

Using the same method, the optimum combination of inspection techniques (Q_1 and Q_2) was determined for the two stage inspection process, and the optimum inspection technique for detection (Q_1) was determined for the single stage inspection process, based on the minimisation of the expected mean annual total cost. Three forms of deterioration were considered, very abrupt growth (g=-5), abrupt growth (g=0) and gradual growth (g=5). For the single stage and two stage inspection processes, the optimum inspection techniques and the expected total costs are presented in Table 4.8. The percentage increase with respect to the minimum expected cost (i.e. optimum inspection interval) is also presented for each form of defect growth.

As discussed earlier in this section, as the defect growth becomes more abrupt, the effect of reducing the inspection quality becomes less apparent. A similar trend can be seen here in relation to the change in the inspection quality with the inspection interval for both the single stage and the two stage inspection process. Considering an inspection interval of 1 year, the optimum inspection quality for detection is reduced as the defect growth becomes more gradual (i.e. from Q_1=6 for very abrupt to Q_1=4 for gradual growth for a single stage inspection, and from Q_1=4 for very abrupt to Q_1=2 for gradual growth for a two stage inspection). This is because it is more important to detect defects where growth is abrupt so repair can be carried out, avoiding failure before the next inspection the following year. For gradual growth it is less likely that defects will fail before the next inspection, so a lower quality technique for detection is optimal. However, as the inspection interval is increased, the change in optimal inspection techniques is different for all forms of defect growth.

		Single Stage Inspection			Two Stage Inspection			
Growth	ΔT	Q_1	$E(C_{TOTAL})$	Increase wrt Min Cost	Q_1	Q_2	$E(C_{TOTAL})$	Increase wrt Min Cost
g=-5	1	6	5637	0%	4	10	5588	0%
	2	8	6757	20%	4	10	6703	20%
	3	8	7861	39%	4	12	7818	40%
	4	8	8751	55%	4	12	8717	56%
	5	8	9436	67%	4	12	9408	68%
	6	8	9957	77%	4	12	9933	78%
g=0	1	6	3703	0%	2	16	3643	0%
	2	8	3994	8%	4	18	3939	8%
	3	10	4551	23%	4	18	4484	23%
	4	10	5143	39%	4	20	5082	40%
	5	10	5709	54%	4	20	5655	55%
	6	12	6219	68%	4	20	6173	69%
g=5	1	4	1705	15%	2	12	1581	14%
	2	6	1476	0%	2	16	1383	0%
	3	10	1485	1%	4	22	1382	0%
	4	12	1543	5%	4	28	1436	4%
	5	16	1645	11%	4	34	1542	12%
	6	16	1780	21%	4	40	1684	22%

Table 4.8. Optimum inspection techniques and corresponding expected total cost for single stage and two stage inspection process

Considering very abrupt growth (g=-5), increasing the inspection interval has a minimal effect on the optimal inspection quality (ΔQ_1=2 for single stage, ΔQ_1=0 and ΔQ_2=2 for two stage). For this form of growth, the defects are so abrupt that failure occurs between inspections even if repair is carried out after the previous inspection (for all inspection intervals), so increasing the inspection quality has very little effect on the number of failures. For abrupt growth (g=0), the optimum inspection quality does increase with the inspection interval (ΔQ_1=6 for single stage, ΔQ_1=2 and ΔQ_2=4 for two stage). In this case, when defects are repaired, it takes longer for these defects to grow to the larger groups where failure can occur (compared to very abrupt growth), so it is worth using a higher quality inspection technique to detect (or size) these defects as the inspection interval increases.

For gradual growth (g=5), as the inspection interval increases it becomes more important to correctly detect and size defects so that they are repaired and the number of failures between inspections is reduced. Defects that are repaired and are returned to the original defect group are less likely to grow to the larger defect groups between inspections (compared to abrupt growth and very abrupt growth), where there is a higher probability of failure. Therefore, for gradual growth increasing the inspection interval has a significant effect on the optimal inspection quality (ΔQ_1=12 for single stage, ΔQ_1=2 and ΔQ_2=28 for two stage). Therefore, when a manager is planning maintenance of a structure with very abrupt or abrupt growth, it is optimal to carry out inspections every year to detect (or detect and size) and repair defects that are present, resulting in fewer failures. Unlike gradual growth, increasing the inspection quality does not lead to an increase in the optimal inspection interval. In relation to gradual growth, the owner/manager can decide to increase the inspection quality which can lead to an increase in the optimal inspection interval (which is ΔT=3 in this case). The results of the single stage process and the two stage process are compared in Figure 4.27, to determine which inspection process is optimal for different forms of defect growth kinetics.

Figure 4.27 (a)-(b) indicate that for very abrupt growth (g=-5), the difference in the expected costs and expected number of failures for the single stage process and two stage process is minimal. In relation to the expected costs, there is less than 1% difference, with the higher costs associated with the single stage inspection process. For the number of failures, the only significant difference is for an inspection interval of 1 year where the two stage inspection process results in 0.2 fewer failures (69%). This is due to the difference in the distribution of defects within the defect groups where the mean defect size is less than the detection threshold/critical defect size. For an inspection interval of 1 year, using a single stage inspection process there are more defects in group 6 (less than the detection threshold) which are not detected, and therefore fail before the next inspection. Using the two stage inspection process these defects are detected and have a higher probability of repair due to sizing error.

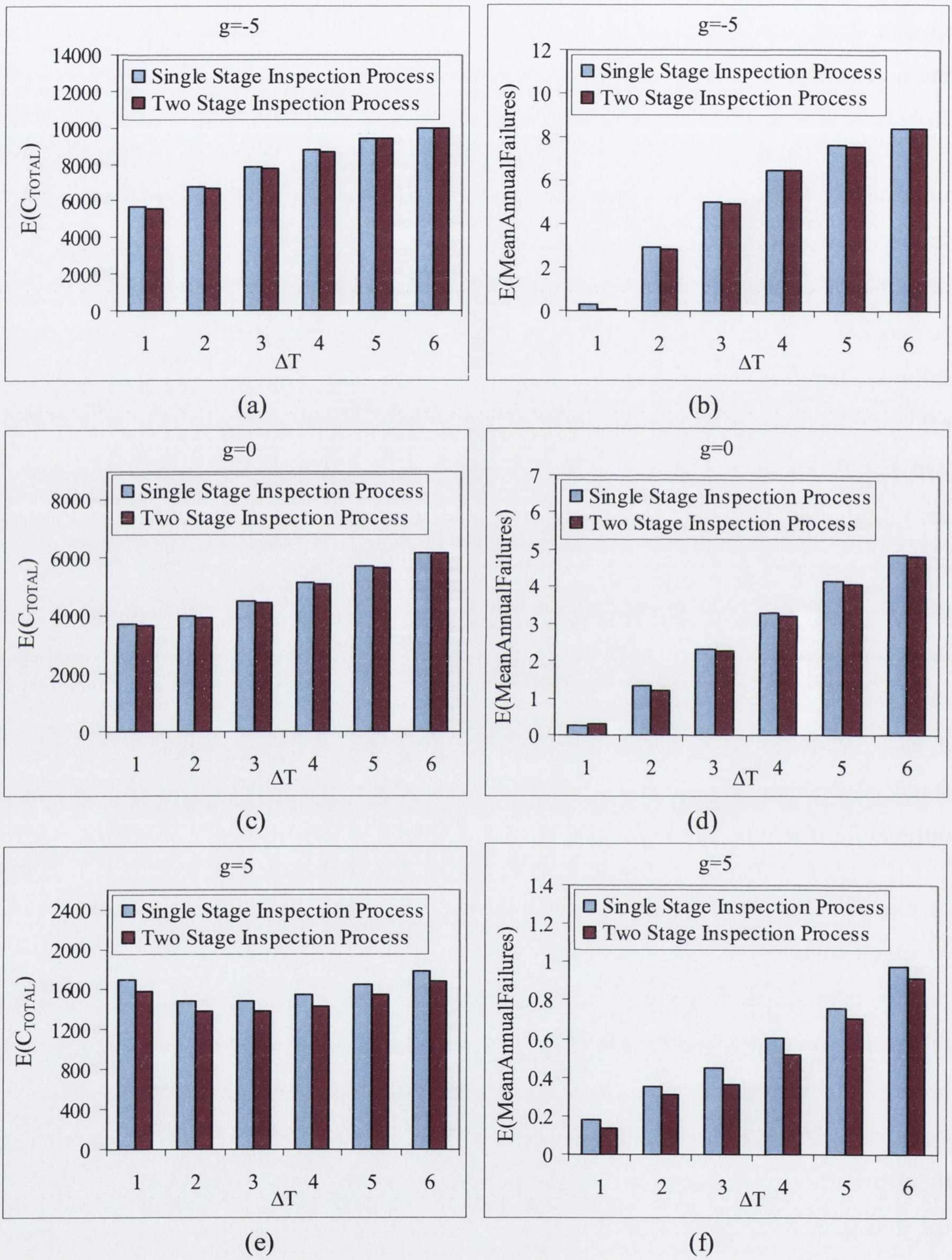


Figure 4.27. Comparison of single stage and two stage inspection processes for very abrupt, abrupt and gradual growth

For all other inspection intervals ($\Delta T=2$ to $\Delta T=6$), defects have more time to grow between inspections and the difference in the distribution of defects below the detection threshold/critical defect size is negligible. As a result, for $\Delta T>1$ the difference in the number of failures is minimal, ranging from 0.05 (1.6%) to 0.03 (0.3%), with the higher number of failures associated with the single stage inspection process.

For abrupt growth (g=0) there is slightly higher difference in the expected cost (up to 1.6%) and the expected number of failures (up to 10%, excluding $\Delta T=1$). In relation to the costs, the two stage inspection process results in lower expected costs than the single stage process, although, for an inspection interval of 1 year, the two stage process results in a higher number of failures (0.04, 15%). Again, this is due to the distribution of defects within the groups. For the two stage inspection process there are a higher number of defects in groups 9-10, which has a lower probability of repair than for the single stage process. This is due to a low quality inspection technique for detection ($Q_1=2$). However, for larger inspection intervals, defects are more evenly distributed, and a higher probability of repair (due to higher quality inspection techniques) leads to a lower number of failures using the two stage inspection process.

The results for gradual growth (g=5) are similar to those presented in Section 4.2.4. For all inspection intervals, the two stage process results in lower expected costs (up to 7%) and failures (up to 24%). The high quality sizing assessment results in accurate sizing of defects, thus defects that are greater than the critical defect size are repaired. This results in fewer failures between inspections. For abrupt and very abrupt growth the number of failures will be high between inspections regardless of the inspection process which is used. Although the two stage process does result in lower costs and fewer failures (except for $\Delta T=1$, g=0) for very abrupt and abrupt growth, it may be more convenient for the manager of a structure to carry out a single stage inspection process since the difference between the two processes is minimal. For gradual growth, however, the difference is more significant. Due to the reduction in costs and number of failures for gradual growth, the two stage inspection process is optimal.

The results presented in this section demonstrate the sensitivity of the maintenance management model to the deterioration kinetics. These must be carefully estimated when used as inputs into a maintenance model. It was demonstrated that the more gradual the defect growth, the more sensitive the percentage of repairs due to correct/incorrect inspection results is to the inspection quality. Therefore, considering the maintenance of structures subject to a very abrupt form of deterioration kinetics, increasing the inspection quality would have a negligible effect on the percentage of unnecessary repairs and to do so would be an inefficient use of resources, whereas, for structures subject to gradual deterioration, increasing the inspection quality can be beneficial in reducing the percentage of unnecessary repairs.

Similarly, considering the optimum inspection quality for the single stage and two stage inspection processes, increasing the inspection quality has little effect on the optimal inspection interval for abrupt and very abrupt growth. An inspection interval of 1 year is optimal and increasing the inspection quality would be an inefficient use of resources, although for gradual growth, a larger inspection is optimal (3 years in the case considered) using higher quality inspection techniques. A larger inspection interval is more convenient for owners/managers as it reduces the annual intangible costs associated with inspections (e.g. user delay costs). In relation to single stage and two stage inspection processes, for very abrupt and abrupt growth the benefits of carrying out a two stage process are minimal, whereas for gradual growth the benefits are significant. Therefore, it is recommended that managers of structures carry out a two stage inspection process where gradual deterioration is present in the structure (e.g. corrosion, carbonation). In conclusion, the results of this section indicate that the growth kinetics must be carefully estimated before deciding on a maintenance strategy, as the optimal maintenance strategy can vary significantly depending on the deterioration characteristics of the structure.

4.3.4. Effect of p_F Curve

The shape and spread of the curve defining the probability of failure (according to the Weibull distribution) are controlled by the d_{ref_pf} and m parameters. A study was carried out to determine the effect of the shape of this curve on the expected mean annual number of failures for a group of structures. The shape of the curve corresponds to different modes of failure, or different limit states. It may be specified by the owner/manager of a structure, along with the allowable probability of failure (i.e. depending on the limit state being considered). Four different cases were considered and the number of failures for the different cases were compared. The list of parameters for each case is given in Table 4.9. Different values of m and d_{ref_pf} were considered in each case to study the effect of varying the shape of the p_F curve. Since d_l just dictates the starting point of the curve and increasing it would indicatively lead to less failures, it was chosen to use a constant value for each case. Since the effect of the chosen limit state will be studied in Section 4.3.5, constant values for the allowable probability of failure were also used. Using Equation 4.1, the critical defect size for each case was calculated and is also presented in Table 4.9.

	Case1	**Case2**	**Case3**	**Case4**
m	4	1	2	10
d_l	0.3	0.3	0.3	0.3
d_{ref_pf}	1	3	0.4	0.5
$P_{f\_allowable}$	0.01	0.01	0.01	0.01
d_c	0.62	0.33	0.33	0.62

Table 4.9. Parameters for different cases considered for p_F curves

For the purpose of this study, an inspection interval of 3 years (optimum inspection interval from Section 4.2.2) was chosen. To compare the effect of the mode of failure and limit state being considered, it was decided to use high quality techniques for both inspections (i.e. Q_1=20 and Q_2=20) so that the number of failures is due to the shape of the curve defining the probability of failure and not due to inaccurate inspection results. All other parameters were assigned the values stated in Table 4.2. The curves representing the variation in probability of failure with mean defect size, for each of the four cases, are illustrated in Figure 4.28 and the corresponding expected mean annual number of failures for each group are given in Figure 4.29.

These figures illustrate that for a specified allowable probability of failure (which relates to SLS in this case), the number of failures can vary significantly depending on the other parameters which dictate the shape of the p_F curve. The different shape curves relate to different modes of failure. For example, cases 1 and 2 could represent a progressive failure mode, while cases 3 and 4 could represent a sudden failure mode. For cases 2 and 3 the lower critical defect size may indicate the importance of the component being considered to the overall condition of the structure, whereas for cases 1 and 4 a higher critical defect size may indicate that these components are less critical.

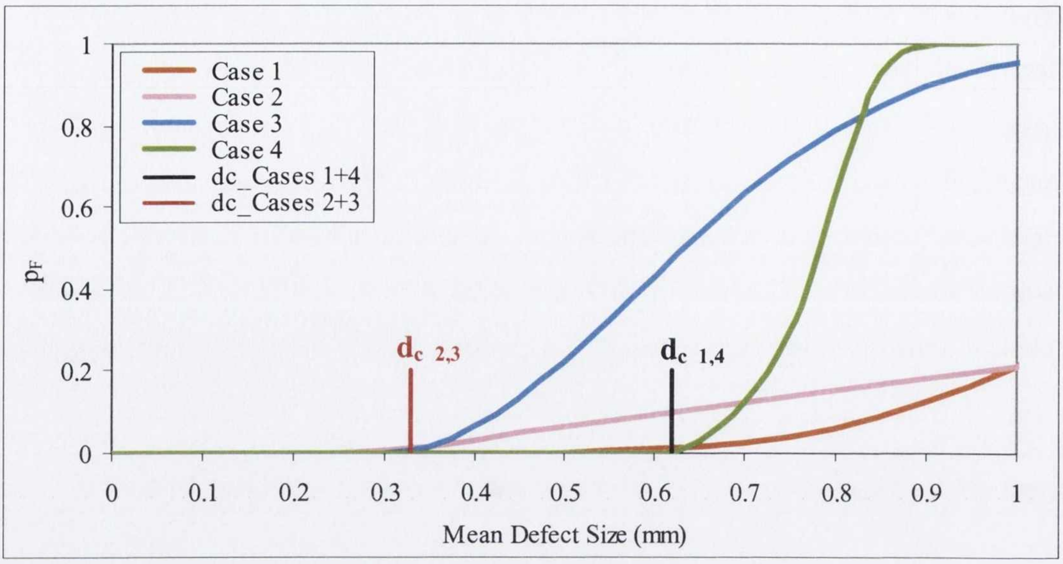


Figure 4.28. The probability of failure curve for the 4 cases considered

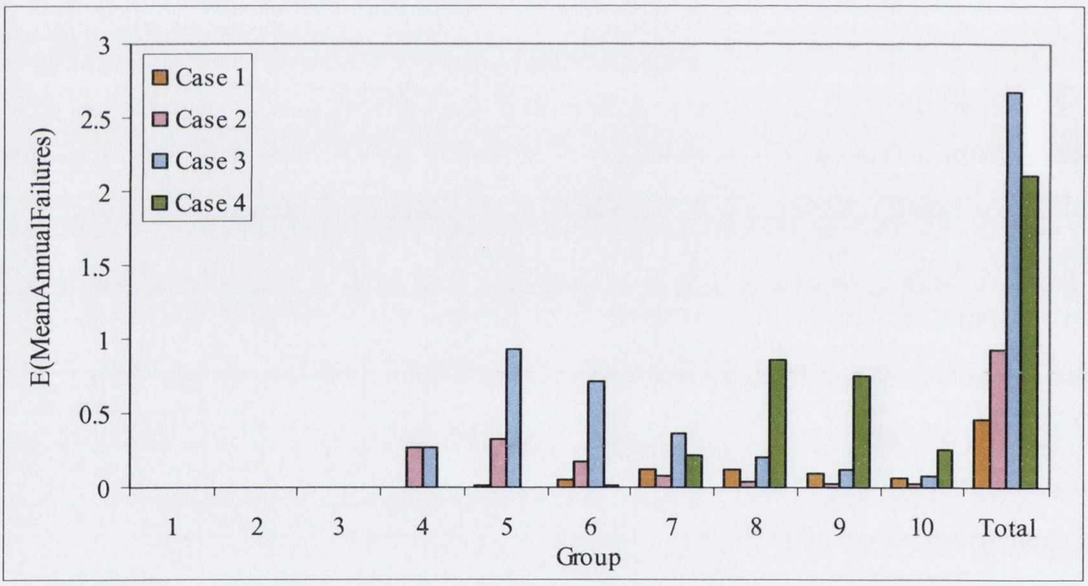


Figure 4.29. The expected mean annual number of failures for the 4 cases considered

Comparing case 1 and case 2, the probability of failure for defects that are 1.0mm in size is about 0.2 for each, yet the number of failures for case 2 is over twice the number of failures for case 1. The critical defect size (d_c) for case 2 is smaller, and due to the shape of the curve, this leads to a lot of failures as defects can grow larger than d_c between inspections and therefore lead to failure. Most failures occur in groups 4-5. Even though the probability of failure is quite small, there are a lot of defects present in these groups.

Similarly, cases 3 and 4 have a probability of failure of 0.95 and 1, respectively, for a defect size of 1.0mm, yet the number of failures for case 3 is higher. As with the first two cases, the case with the lower d_c value (case 3) results in a higher number of failures.

Case 1 and case 4 have the same d_c values, however, the modes of failure are very different and the number of failures for case 4 is 4.6 times higher than the number of failures for case 1. For case 4, once the defect size grows greater than the critical defect size, the probability of failure increases dramatically, resulting in a high number of failures in groups 8-9. Cases 2 and 3 also have the same values for d_c, with case 3 resulting in 2.8 times more failures than case 2, due to a higher proportion of failures of defects greater than the critical defect size. The effect of the growth rate on the expected number of failures for each of the cases was also considered.

For each case, the number of failures is proportional to the growth rate of the defects. However, increasing the growth rate (i.e. considering defects in a more aggressive environment) has a different effect depending on the mode of failure being considered, as illustrated in Figure 4.30. As discussed previously, considering both sudden (cases 3 and 4) and progressive (cases 1 and 2) failures, the component with the lower critical defect size (cases 2 and 3) results in a higher number of failures. There is a larger increase in the number of failures for cases 3 and 4 (sudden failures) since more defects grow to larger groups between inspections and there is a dramatic increase in the probability of failure with defect size once the defect grows greater than the critical defect size.

For cases 1 and 2 (progressive failures), the increase in the number of failures is less significant since there is a gradual increase in the probability of failure once the defect grows larger than the critical defect size. This demonstrates that the sensitivity of the number of failures to the growth rate depends on the mode of failure being considered. For a progressive failure mode, the effect of an inaccurate estimation of the growth rate of a defect will be significantly less than for a defect with a sudden failure mode. Therefore, it is important for the owner/manager to take this into account when developing a maintenance management strategy. In addition, a similar increase in the number of failures emerges with an increase in the abruptness in growth (i.e. decrease in g) or the inspection interval (ΔT).

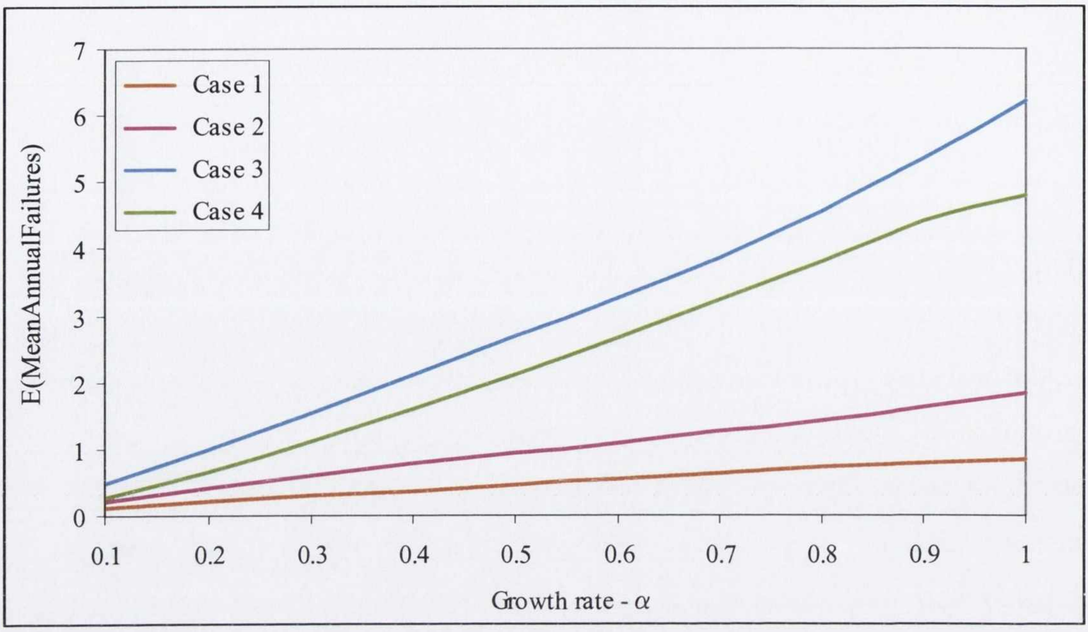


Figure 4.30. Effect of defect growth rate on expected total number of failure for 4 cases considered

These results demonstrate that there are many parameters that define the probability of failure and a variety of modes of failure can be inputted into the model using these different parameters. The sensitivity of the results of the maintenance management model (i.e. expected number of failures) to changes in these parameters has also been demonstrated, emphasising the importance of having accurate and realistic input data. On this basis, an experimental study which was carried out to determine the growth rate of cracks in reinforced concrete in different repair materials will be described in Chapter 5. In addition, for accurate management of structures it is recommended that future work should focus on the calibration of the parameters of the Weibull probability of failure model.

4.3.5. Effect of Allowable Probability of Failure ($P_{f\_allowable}$)

The allowable probability of failure can be specified by the owner/manager of a structure, depending on the limit state being considered and based on recommendations such as those provided by the ISO standards, as discussed in Section 4.2. The effect of the specified allowable probability of failure on the expected mean annual total costs and the expected mean annual number of failures is investigated. Two cases at serviceability limit state (SLS) and two cases at ultimate limit state (ULS) were considered. The four cases are presented in Table 4.10. The cost coefficients were chosen to reflect the allowable probability of failure and the limit state being considered. Since the consequence of failure is more serious for ULS, a higher failure impact cost coefficient (k_F, Equation 3.83) is chosen.

In addition, it is assumed that more extensive inspections and repairs would be carried out for ULS, with associated higher cost coefficients (k_{I1}, Equation 3.63 and k_R, Equation 3.72 respectively). For SLS and an allowable probability of failure of 0.1, the consequence of failure is minimal, hence a failure impact coefficient (k_F) of 0.5. For an allowable probability of failure of 0.01, the impact of failure is more significant, so the failure impact coefficient is doubled to 1.0. For the ULS cases, the cost coefficients for inspection and repair are increased to take into account the cost of more sophisticated inspection and repair methods. Again, a more serious consequence of failure is reflected in the magnitude of the failure impact coefficient.

	SLS1	**SLS2**	**ULS1**	**ULS2**
C_o	1000	1000	1000	1000
k_{I1}	0.01	0.01	0.05	0.05
k_R	0.1	0.1	0.5	0.5
k_F	0.5	1	100	200
$P_{f\_allowable}$	0.1	0.01	10^{-5}	10^{-6}
d_c	0.87	0.62	0.36	0.33

Table 4.10. Parameter values used for sensitivity study of allowable probability of failure

Considering the four cases presented in Table 4.10, Figure 4.31 illustrates the effect of the allowable probability of failure on the expected mean annual costs. These figures illustrate that as the allowable probability of failure is reduced, the optimum time between inspections also reduces (from $\Delta T=4$ to $\Delta T=1$), based on the minimisation of expected total cost. It is recognised that cost coefficients assigned for the different cases can be quite subjective and can have a major influence the optimal inspection interval. The results for the expected mean annual costs are therefore accompanied by the expected mean annual number of failures for each inspection interval.

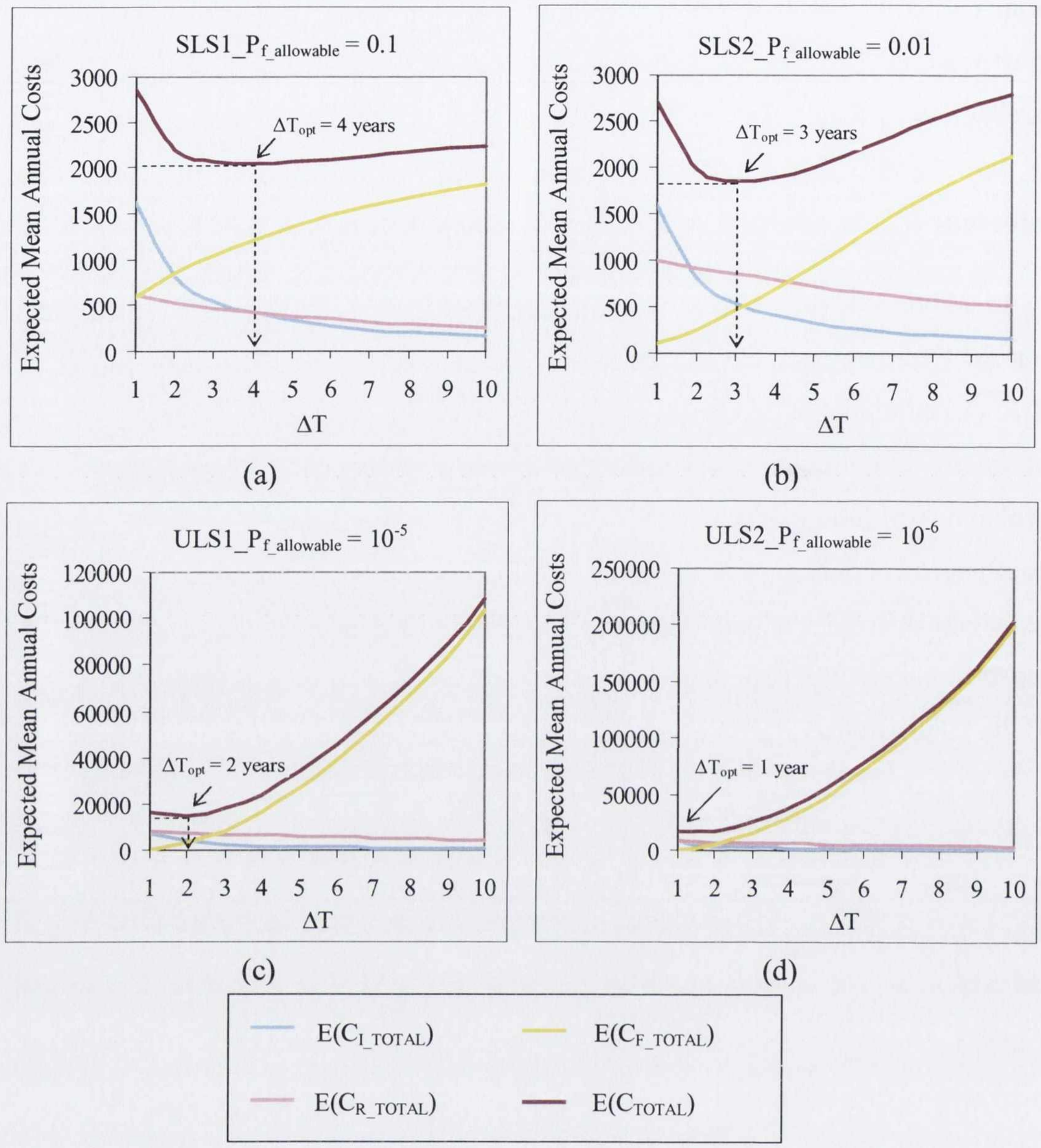


Figure 4.31. Optimal inspection interval for the four cases in Table 4.10

Figure 4.32 compares the number of failures for each of the cases given in Table 4.10. Similar to Section 4.3.4, it was decided to use high quality techniques for both inspections (i.e. Q_1=20 and Q_2=20) so that the number of failures is due to the allowable probability of failure and not due to inaccurate inspection results. Unless specified otherwise, all other parameters are assigned the values presented in Table 4.2. The effect of the allowable probability of failure (limit state) on the percentage of necessary/unnecessary repairs and on the optimum combination of techniques for a two stage and single stage inspection process will be discussed later in this section.

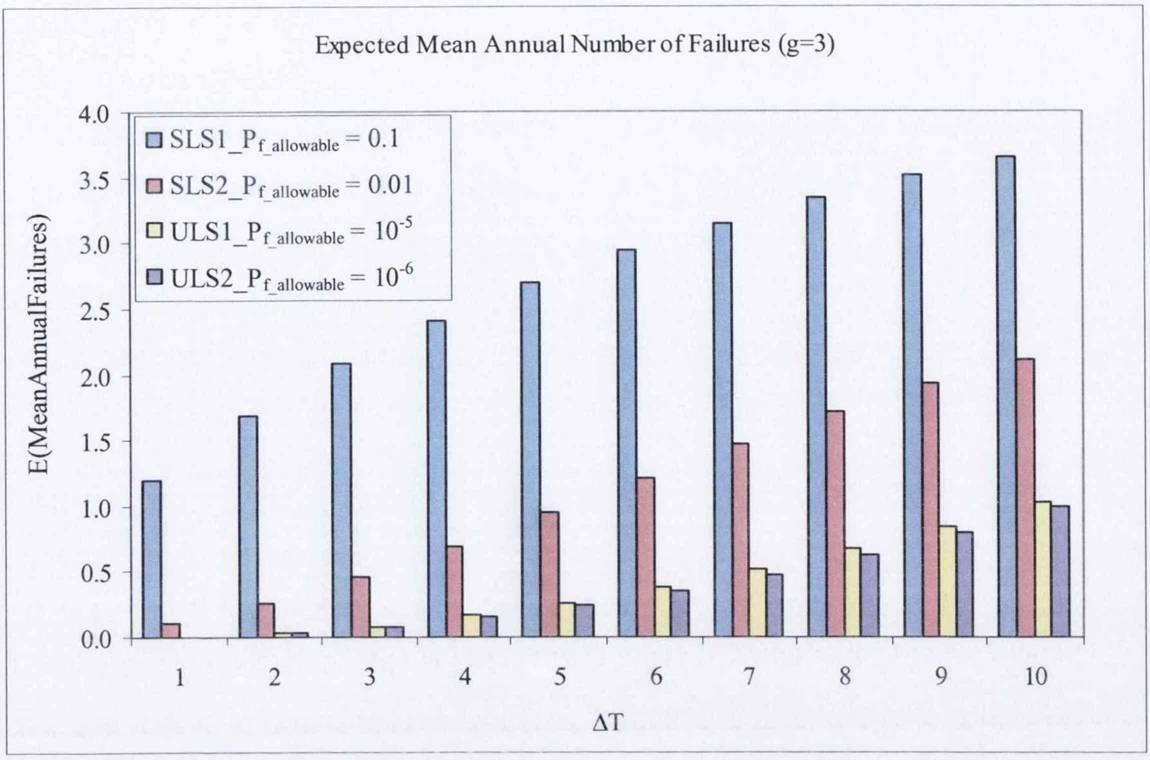


Figure 4.32. Expected number of failures for four cases given in Table 4.10

Using the set of parameters in Table 4.10, for the SLS cases the optimal inspection interval is reduced from 4 years to 3 years by reducing $P_{f\_allowable}$ from 0.1 to 0.01. The lower critical defect size for SLS2 results in a higher number of repairs, and therefore, a higher repair cost. The number of failures is lower due to the higher number of repairs which are carried out, but the cost of failure is higher due to the higher failure impact coefficient. In relation to the costs for ULS, changing $P_{f\_allowable}$ from 10^{-5} to 10^{-6} has a similar effect to the SLS cases, reducing the optimal inspection interval from 2 years to 1 year, for the cost coefficients used in this study. However, due to the large failure impact coefficient (and associated high failure cost) for the ULS cases, the shapes of the graphs in Figure 4.31(c)-(d) are different to Figure 4.31(a)-(b). The inspection cost and the repair cost are very similar for the two ULS cases. Reducing $P_{f\_allowable}$ from 10^{-5} to 10^{-6} does not have much of an effect on the critical defect size and, therefore, does not have much of an effect on the number of repairs or failures. The higher failure cost for ULS2 is primarily due to the larger failure impact coefficient. These results indicate that the number of repairs and failures are more sensitive to changes in $P_{f\_allowable}$ for the SLS, as this results in a bigger change in the critical defect size.

In relation to the number of failures, the effect of an increase in the abruptness of the growth may be of interest to the owner/manager of a structure considering different limit states. On this basis, considering abrupt growth (g=0), the expected number of failures for each of the cases outlined in Table 4.10 was determined for each inspection interval, as illustrated in Figure 4.33. An increase in the abruptness of the growth results in an increase in the number of failures (as discussed in Section 4.3.1) for all limit states considered. As well as increasing the number of failures, it also reduces the relative difference in the number of failures for each of the limit states, with the exception of ΔT=1.

In relation to the SLS, considering gradual growth (g=3), the number of failures for SLS1 varies from 6.5 times (ΔT=2) to 1.7 times (ΔT=10) the number of failures for SLS2 (Figure 4.32), whereas for abrupt growth (g=0) this only varies from 2.6 times (ΔT=2) to 1.1 times (ΔT=10) the number of failures for SLS2 (Figure 4.33). As discussed previously, the number of failures for ULS is similar for both cases, since the difference in the critical defect size is minimal. This is true regardless of the deterioration mechanism, as illustrated in Figure 4.32 and Figure 4.33. However, comparing SLS1 and ULS1, the difference in the number of failures decreases as the defect growth becomes more abrupt. For gradual growth (g=3), the number of failures for SLS1 varies from 53 times (ΔT=2) to 3.5 times (ΔT=10) the number of failures for ULS1 (Figure 4.32), whereas for abrupt growth (g=0) this only varies from 3.3 times (ΔT=2) to 1.1 times (ΔT=10) the number of failures for ULS1 (Figure 4.33).

This difference in the relative number of failures is due to the distribution of defects within the groups. For abrupt growth, more defects are present in the larger groups where failure is likely to occur, Figure 4.19. Even if the allowable probability of failure is low (i.e. the critical defect size is small resulting in more repairs), the defects can still grow to large defect groups between inspections (for ΔT>1) after repairs have been carried out, unlike for gradual growth. However, for an inspection interval of 1 year the relative difference is minimal since the defects are inspected and repaired each year (with high quality inspection techniques in this case) and the defects have less time to grow between inspections, so the effect of the deterioration mechanism is less obvious. The effect of the inspection quality on the number of failures for each of the limit states considered will be studied later in this section.

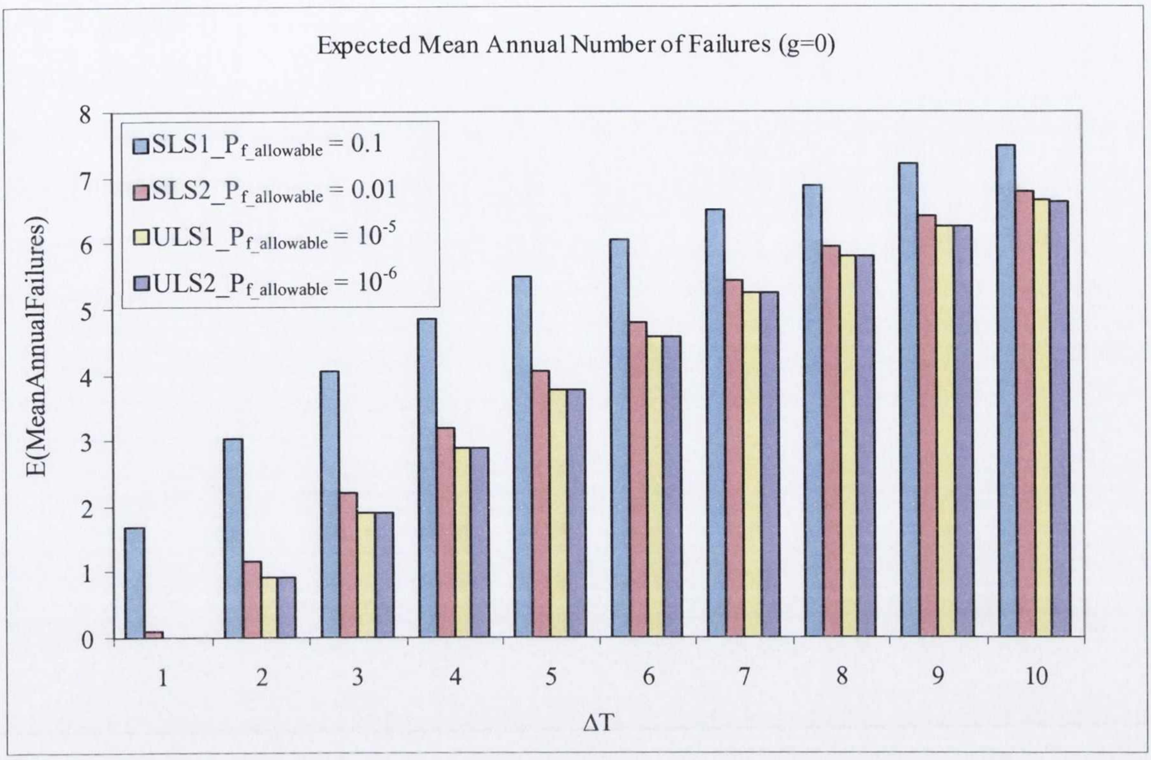


Figure 4.33. Expected number of failures for four cases given in Table 4.10, for abrupt growth (g=0)

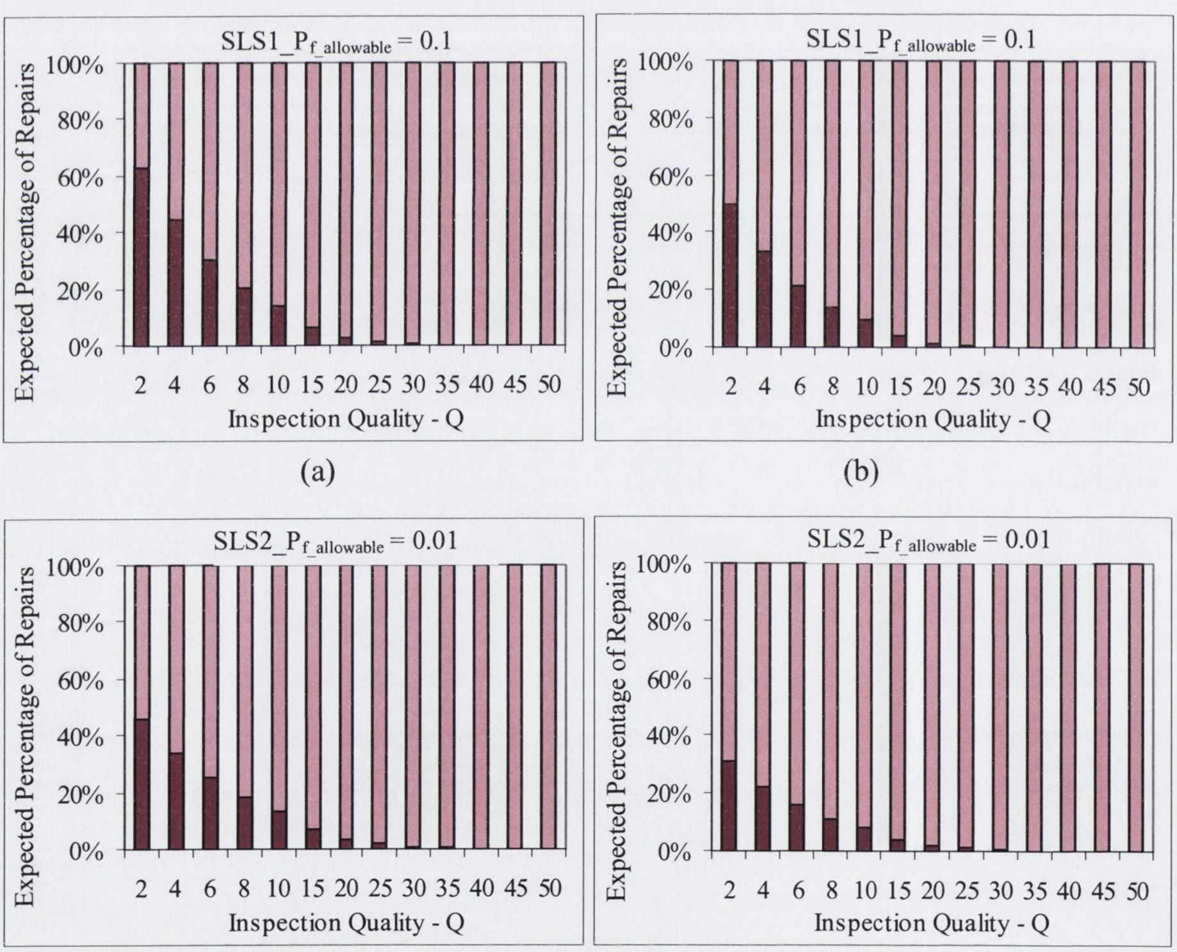
Based on these results, it is more important for the owner/manager of a structure to carefully consider the allowable probability of failure for the serviceability limit state, based on the consequences of failure and the relative cost of safety measures (i.e. maintenance and repair), as this will significantly affect the expected number of failures especially for gradual growth. For the ULS, the cost coefficients dictate the change in the results by increasing the failure impact coefficient to take into account more serious failure consequences.

The difference in the expected number of failures is not significant in comparison to the change in SLS. However, the difference in the number of failures between SLS and ULS decreases as the growth becomes more abrupt, indicating that the choice of limit state has less of an effect on the number of failures for abrupt growth. Therefore, it is interesting to note that if the owner/manager of a structure is concerned with different limit states within a particular structure, a change in the deterioration kinetics of the structure will not result in a proportionate change in the number of failures for each of the limit states. This must be taken into consideration when altering the maintenance strategy.

As discussed in Section 4.2.3, it is also interesting to look at the percentage of repairs which are carried out due to correct/incorrect inspection results. In this section, these results will be studied in terms of different limit states to determine the benefits of using higher quality techniques in relation to reducing the number of unnecessary repairs. Figure 4.34 presents the results of this analysis for an inspection interval of 3 years and 6 years, where the parameters have been assigned the values presented in Table 4.2, unless stated otherwise.

For an allowable probability of failure of 0.1 (SLS1), increasing the inspection quality from Q=2 (poor quality) to Q=50 (very high quality) results in a reduction in the percentage of repairs carried out due to incorrect inspection results from 63% to 0% for ΔT=3 and from 50% to 0% for ΔT=6.

ΔT=3 **ΔT=6**



(a) (b)

(c) (d)

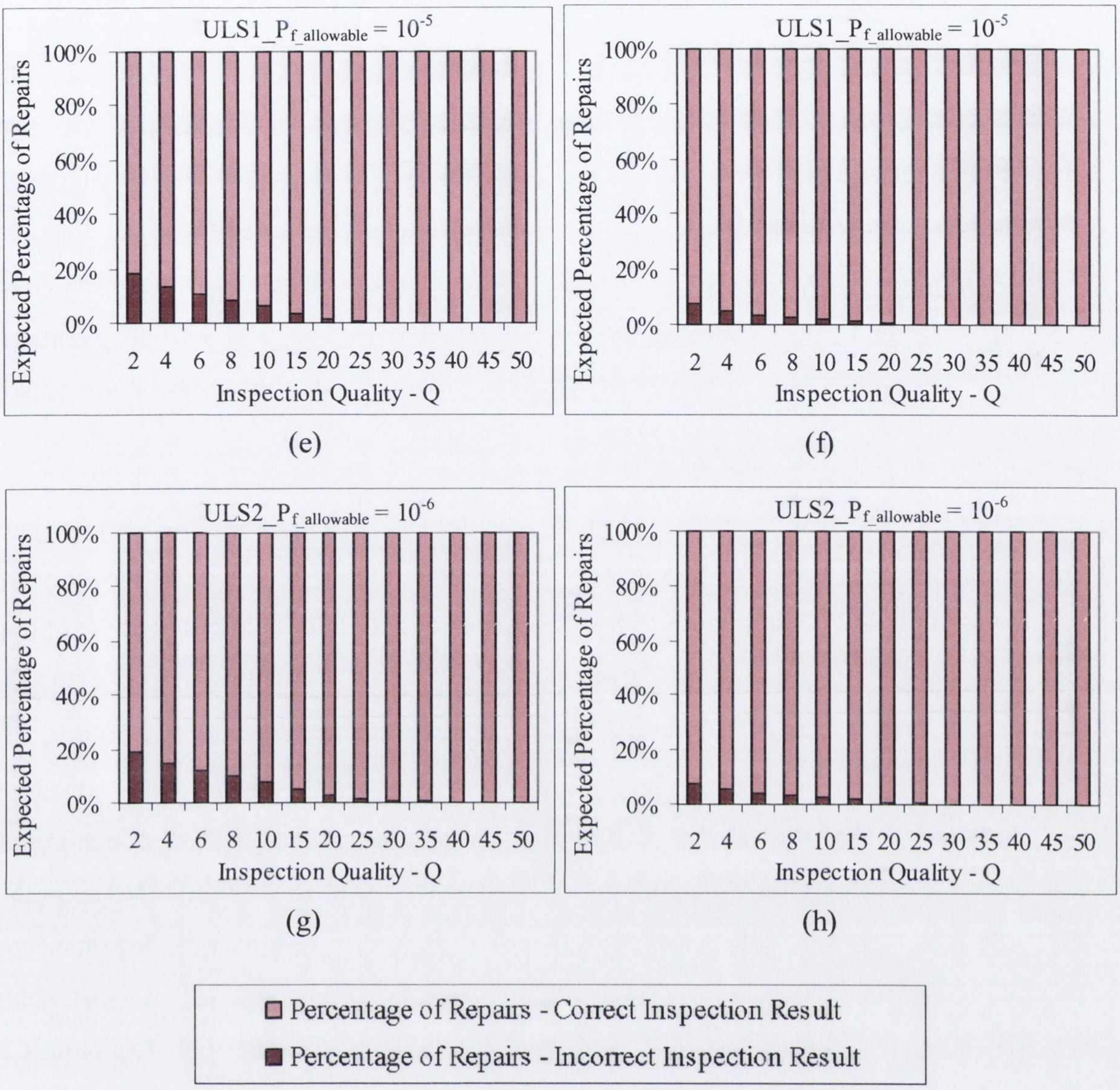


Figure 4.34. Effect limit state on the percentage of repairs carried out due to correct/incorrect inspection results

As discussed in Section 4.2.3, the proportion of unnecessary repairs (due to incorrect inspection results) is inversely proportional to the inspection interval. This study demonstrates that this is true for all limit states considered. As Figure 4.34(a)-(h) illustrates, the percentage of unnecessary repairs decreases with the allowable probability of failure (and hence critical defect size). Considering an allowable probability of failure of 10^{-6} (ULS2), increasing the inspection quality from Q=2 (poor quality) to Q=50 (very high quality), results in a less significant reduction in the percentage of repairs carried out due to incorrect inspection results, ranging from 19% to 0% for $\Delta T=3$, and from 8% to 0% for $\Delta T=6$.

Unnecessary repairs are carried out if defects that are less than the critical defect size are repaired. For a large critical defect size there are many groups less than the critical defect size where defects can be incorrectly sized and repaired. In this case, increasing the inspection quality will reduce the noise associated with the inspection for all groups, resulting in accurate sizing and less unnecessary repairs in the groups where the defect size is less than the critical defect size. However, as $P_{f\_allowable}$ is reduced, this also reduces the critical defect size. Therefore, for the same inspection interval and growth parameters, there are fewer defects less than the critical defect size, resulting in fewer unnecessary repairs.

In summary, these results indicate that for the ultimate limit state, a higher proportion of the repairs that are carried out are necessary repairs compared to serviceability limit state, regardless of the inspection quality. This is due to a smaller critical defect size for ultimate limit state conditions. For all limit states, however, there is a plateau in the percentage of unnecessary repairs between Q=25 and Q=30, whereby increasing the inspection quality results in no further reduction in the percentage of unnecessary repairs. This indicates to owners/managers of structures that, for this case considered in relation to unnecessary repairs, it would be a waste of resources to increase the inspection quality above Q=25 or Q=30 for each of the limit states considered. However, for this study the inspection qualities of both techniques were altered simultaneously. It is also interesting to study how the limit state considered affects the optimum combination of inspection techniques for detection and sizing.

It was demonstrated in Section 4.2.4 that the two stage inspection process results in a lower expected total cost and expected number of failures than the traditional one stage process for an allowable probability of failure of 0.01 (SLS). Using the same method, the optimum combination of inspection techniques (Q_1 and Q_2) was determined for the two stage inspection process and the optimum inspection technique for detection (Q_1) was determined for the single stage inspection process, based on the minimisation of the expected mean annual total cost. The four cases presented in Table 4.10 were considered (i.e. two SLS cases and two ULS cases). For the single stage and two stage inspection process, the optimum inspection techniques and the expected total costs are presented in Table 4.11. The percentage increase with respect to the minimum expected cost (i.e. optimum inspection interval) is also presented for each of the limit states.

		Single Stage Inspection			Two Stage Inspection			
Limit State	ΔT	Q_1	$E(C_{TOTAL})$	Increase wrt Min Cost	Q_1	Q_2	$E(C_{TOTAL})$	Increase wrt Min Cost
$P_{f\_allowable} = 0.1$ (SLS1)	1	4	1894	5%	2	6	1713	4%
	2	4	1802	0%	2	6	1653	0%
	3	8	1857	3%	2	18	1704	3%
	4	12	1895	5%	4	20	1771	7%
	5	14	1940	8%	4	22	1829	11%
	6	16	1987	10%	4	24	1889	14%
$P_{f\_allowable} = 0.01$ (SLS2)	1	4	1853	12%	2	14	1724	10%
	2	6	1656	0%	2	16	1571	0%
	3	10	1704	3%	4	22	1600	2%
	4	12	1812	9%	4	26	1706	9%
	5	14	1964	19%	4	30	1863	19%
	6	16	2143	29%	4	36	2048	30%
$P_{f\_allowable} = 10^{-5}$ (ULS1)	1	8	13059	0%	4	18	12272	0%
	2	10	13400	3%	6	26	13149	7%
	3	18	17519	34%	8	38	17058	39%
	4	26	24268	86%	12	50	23613	92%
	5	34	33607	157%	12	62	32777	167%
	6	40	45023	245%	14	74	44013	259%
$P_{f\_allowable} = 10^{-6}$ (ULS2)	1	10	13887	0%	4	20	12948	0%
	2	12	16530	19%	8	28	16299	26%
	3	20	24630	77%	12	40	24251	87%
	4	32	37406	169%	14	54	36944	185%
	5	38	54851	295%	16	68	54352	320%
	6	44	76240	449%	20	82	75681	484%

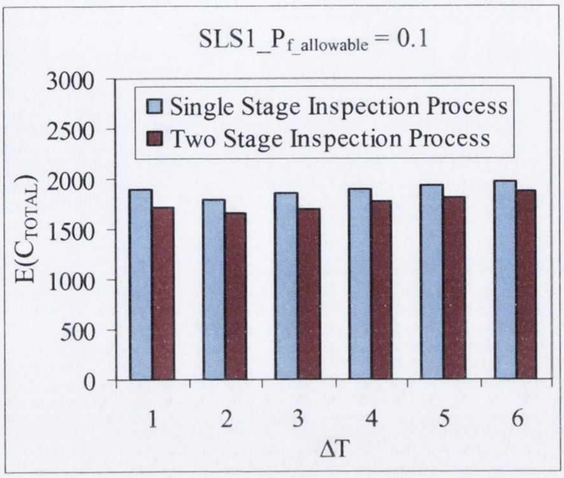
Table 4.11. Optimum inspection techniques and corresponding expected total cost for single stage and two stage inspection process for limit states presented in Table 4.10

In relation to the optimum inspection techniques, these results demonstrate that as the allowable probability of failure is reduced, higher quality techniques (which are associated with less noise) become optimum. A higher quality inspection technique reduces the noise associated with the inspection process, allowing for a more accurate detection/sizing procedure. Therefore, if an inspector decides to use high quality inspection techniques, there are fewer missed detections and inaccurate sizing procedures which lead to no repair. This in turn results in fewer failures.

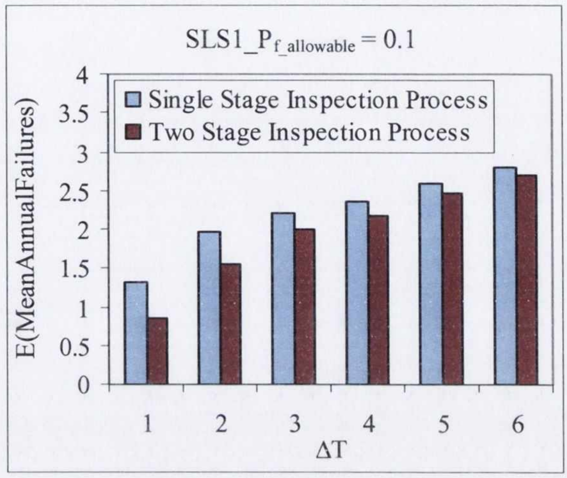
For example, considering the two SLS cases (with an allowable probability of failure of 0.1 and 0.01) and an inspection interval varying from $\Delta T=1$ to $\Delta T=6$, the optimum inspection quality varies from $Q_1=4$ to $Q_1=16$ for a single stage inspection and from $Q_1=2$ to $Q_1=4$ and $Q_2=6$ to $Q_2=36$ for a two stage inspection process, as presented in Table 4.11. Whereas, for the corresponding two ULS cases higher quality techniques are optimum, with the optimum inspection techniques for the single stage inspection varying from $Q_1=8$ to $Q_1=44$, and for the two stage inspection varying from $Q_1=4$ to $Q_1=20$ and $Q_2=18$ to $Q_2=82$. It is recognised that the inspection qualities presented here do not relate to specific inspection techniques, and it is beyond the scope of this thesis to correlate the specific inspection techniques and inspection quality. Once a correlation has been developed, this method can also be used to select the optimum inspection techniques for a given range of techniques that are available to the owner/manager of a structure.

This study demonstrates that a higher inspection quality is needed for ULS than SLS for both the single stage and the two stage inspection process. For a two stage process, a higher quality inspection for detection is necessary for ULS, since undetected defects may lead to an unacceptable failure rate. For SLS, the critical defect size is larger and the allowable probability of failure is higher, therefore, undetected defects which are greater than the detection threshold but less than the critical defect size which may lead to failure between inspections are acceptable, resulting in a low quality optimum inspection technique for detection. Failure due to defects that are larger than the critical defect size would lead to an unacceptable level of failure, therefore, higher quality inspection techniques are optimum for the sizing assessment to correctly size and repair defects that are larger than the critical defect size. This is for the specific case of gradual growth. The effect of the growth kinetics on the optimum inspection quality was discussed in Section 4.3.1.

A comparison of the results for the single stage and two stage inspection process is presented in Figure 4.35. The purpose of this section of the thesis is to determine if it is optimum for the owner/manager of a structure to carry out a two stage inspection for all limit states considered. Figure 4.35(a)-(d) illustrates the advantage of a two stage inspection process over a single stage inspection process in relation to the expected mean annual total costs and the expected mean annual number of failures. However, Figure 4.35(e)-(h) illustrate that the two ULS cases have very similar results, with the percentage difference for costs and number of failures for the single and two stage inspection process differing by a maximum of 1.5% (ΔT=6, Figure 4.35(e),(g)) and 2% (ΔT=2, Figure 4.35(f),(h)), respectively.



(a)



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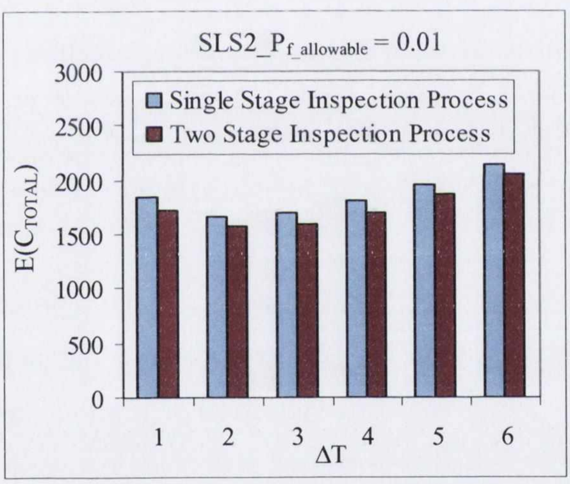

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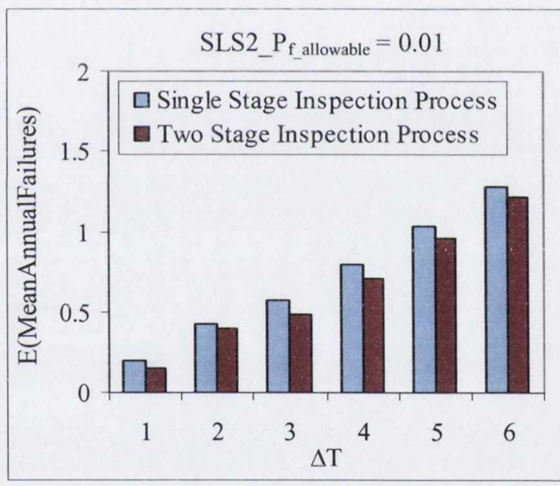

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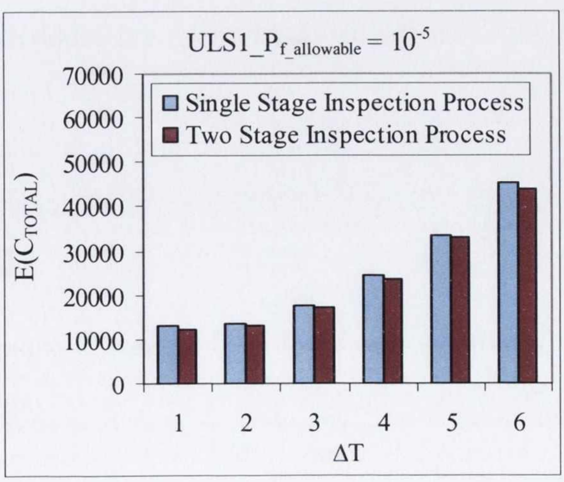
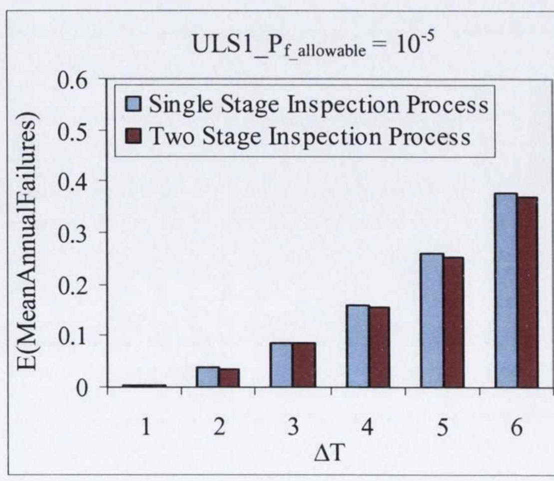
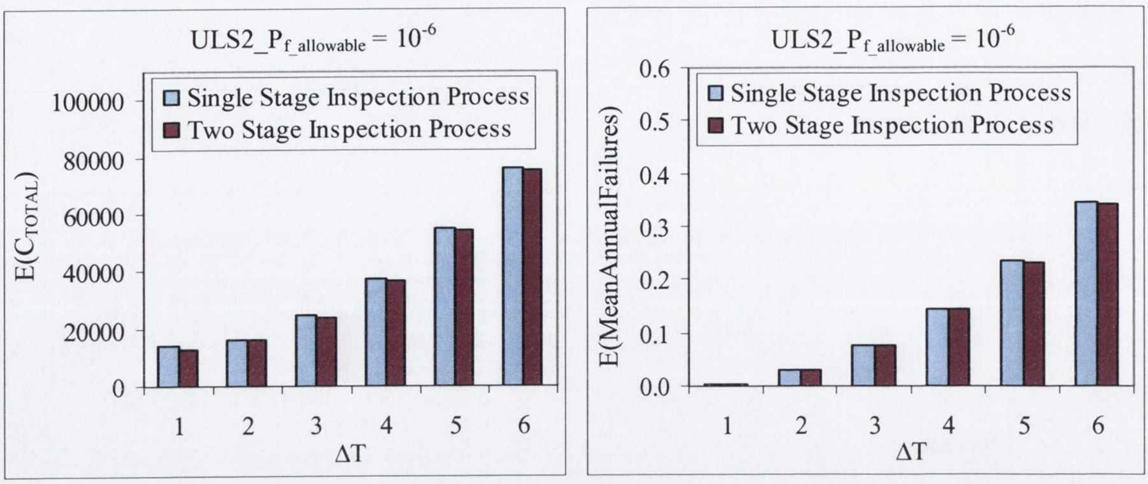


(e)



(f)



(g) (h)

Figure 4.35. Comparison of single stage and two stage inspection processes for limit states presented in Table 4.10

The results also demonstrate that the difference in performance between the single stage and two stage inspection process continues to decrease with the allowable probability of failure, with only up to a 6% ($\Delta T=1$, Figure 4.35(e)) and 8% ($\Delta T=2$, Figure 4.35(f)) reduction in the expected costs and expected number of failures, respectively, using the two stage inspection process for the ULS cases.

The reduction in the advantages of the two stage process over the single stage process with a decrease in the allowable probability of failure is due to the reduction in the difference between the detection threshold and the critical defect size. For the four limit state cases considered (SLS1, SLS2, ULS1, ULS2), the respective difference between d_c (Table 4.10) and d_{min} (0.2, Table 4.2) for the two stage process is 0.67, 0.42, 0.16, 0.13. This also explains why there is a larger difference in the expected costs and number of failures for the SLS cases than for the ULS cases. When the critical defect size is close to the detection threshold the advantages of using the sizing assessment are reduced and a detection procedure using a high quality inspection gives similar results, where all detected defects are repaired.

These results indicate that for the cases considered in this section of the thesis, it is more beneficial for owners/managers to choose a two stage inspection process over a single stage inspection process for serviceability limit state conditions rather than for ultimate limit state conditions. Although the two stage inspection process does result in lower costs and number of failures for ULS for all inspection intervals (ΔT=1:6) considered, the difference is less significant than for SLS, and owners/managers may consider it more convenient to carry out a single stage inspection process. However, as stated previously, the inspection quality of the techniques used in this thesis do not relate to any specific inspection techniques, and considering a specific set of inspection techniques (for detection and sizing) available to the owner/manager of a structure, a lower quality inspection for detection followed by a high quality sizing assessment may result in a significantly higher performance than a single inspection for detection for ULS.

Considering the SLS, the results demonstrate that a two stage inspection process is more beneficial to an owner/manager of a structure in terms of cost and possible number of failures. The screening exercise for detection (with a low detection threshold), followed by a high quality inspection for sizing is effective at ensuring that necessary repairs are carried out when defects are greater than the critical defect size, resulting in fewer failures and lower costs.

4.4. CONCLUSIONS

This chapter has demonstrated the capabilities and diversity of the developed Markov maintenance model. Firstly, it was shown how the methodology makes operable the determination of the optimal inspection interval for a particular set of input parameters. The variation in the different costs, such as inspection cost, repair cost and failure cost was illustrated for a range of inspection intervals, which in this case resulted in an optimum inspection interval of 3 years. An application of the event based decision scheme was demonstrated by studying the repairs that were carried out due to correct/incorrect inspection results. The results of this study indicate that the proportion of unnecessary repairs (due to incorrect inspection results) is inversely proportional to the inspection interval. The effect of inspection quality on the number of failures and on the proportion of repairs carried out due to correct/incorrect inspection results was also investigated. The results illustrate that the most significant decrease in the number of unnecessary repairs can be seen by increasing the inspection quality from Q=2 to Q=20, whereby increasing the inspection quality further results in no further reduction in the percentage of unnecessary repairs. This indicates to owners/managers of structures that it would be a waste of resources to increase the inspection quality above Q=20, for the specific case considered.

The sensitivity of these results to the growth kinetics and allowable probability of failure was also studied. The results indicate that the more gradual the defect growth the more sensitive the percentage of repairs due to correct/incorrect inspection results are to the inspection quality. Therefore, in relation to the percentage of unnecessary repairs, for structures subject to gradual deterioration, increasing the inspection quality can be beneficial in reducing the percentage of unnecessary repairs, whereas for very abrupt growth this would be an inefficient use of limited resources, as increasing the inspection quality would have a negligible effect on the percentage of unnecessary repairs. In relation to the allowable probability of failure, the results indicate that for the ultimate limit state, a higher proportion of the repairs that are carried out are necessary repairs compared to the serviceability limit state, regardless of the inspection quality. In addition, there is a plateau in the percentage of unnecessary repairs between Q=25 and Q=30,

An important study was also carried out to investigate the effect of varying the quality of the two inspections independently. When carrying out an inspection there are two points of interest, the presence of a defect, and the size of a defect present. Since each stage of the inspection has a different purpose, it is necessary to separate these procedures to accurately model an inspection process which is to be incorporated into a maintenance management plan. The separation of the inspection process of a structure into two stages enables the investigation to study the effect of both stages of the inspection on the expected annual costs of the structure and the maintenance plan for the structure. By finding the optimum combination of techniques, the two stage process could be compared to a corresponding single stage inspection process with the same input parameters. The optimum inspection technique for each inspection interval was also determined for a traditional single stage process and compared to the two stage inspection. The results indicated that the two stage inspection process was superior to the single stage in terms of the expected costs and expected number of failures. In addition, a survey carried out in the US indicated that 91% of state agencies (39 out of the 43) consider it necessary to carry out NDT tests to assess the condition of a concrete structure beyond a visual inspection (Rens and Transue, 1998). Using this methodology, the optimum combination of techniques can be determined for each stage of an inspection, to optimise budget spend. The inspection for the detection of defects can be a screening exercise such as a visual inspection which is then followed by an NDT to determine the size/extent of the defect. Whilst for the cases considered in this chapter the two stage process was deemed superior to the single stage inspection process, it is recognised that this will not always be the case. Depending on factors such as the deterioration mechanism, location and limit state, there will be a trade off between the two stage and single stage inspection processes.

Following on from this, the importance of the ability to carry out sensitivity studies within the Markov maintenance model was demonstrated. By varying the number of defect groups it was illustrated that the accuracy of the results of the model is sensitive to the discretisation of the range of defect sizes. Also, the results illustrate that the optimum inspection interval is not sensitive to changes in the value of the ratio of the inspection cost coefficient for the second inspection (k_{I2}) to the inspection cost coefficient for the first inspection (k_{I1}). In addition, it was demonstrated how the deterioration kinetics of defects from very abrupt growth to gradual growth can be inputted into the model, allowing many forms of defect growth to be considered when optimising maintenance strategies. The effect of the inspection quality was investigated for different forms of defect growth. Depending on the inspection interval, the effectiveness of increasing the quality of the inspection technique can vary for different forms of growth kinetics. It is important for the owner/manager of a structure to be aware of the growth properties and to choose inspection methods accordingly. Increasing the inspection quality doesn't always result in a lower number of failures (or a lower cost) and in some cases increasing the inspection quality may be an inefficient use of limited financial resources (i.e. for very abrupt growth in this case). In relation to single stage and two stage inspection processes, for very abrupt and abrupt growth, the benefits of carrying out a two stage process are minimal, whereas for gradual growth the benefits are significant. Therefore, the growth parameters of the deterioration mechanism must be carefully estimated before deciding on a maintenance strategy, as the optimal maintenance strategy can vary significantly depending on the deterioration characteristics of the structure. On this basis, an experimental study was carried out, which will be discussed in detail in Chapter 5, which investigates the rate of defect growth in different materials to study the effectiveness of different repair materials.

In relation to the sensitivity studies, the effect of the shape parameter (m) and the spread parameter (d_{ref_pf}) of the Weibull distribution was investigated. The shape of the curve corresponds to different modes of failure, or different limit states. It may be specified by the owner manager of a structure, along with the allowable probability of failure (i.e. depending on the limit state being considered). As expected, the results illustrate that for a particular allowable probability of failure sudden failure modes result in more failures than progressive failure modes, but the number of failures also depends on the critical defect size (e.g. depending on importance of the structural component). This indicates that the results are sensitive to the parameters used to model the failure probability and should be carefully chosen depending on the mode of failure and limit state being considered.

Also, the allowable probability of failure and cost coefficients related to a structure can vary depending on the limit state being considered and the consequence of failure. On this basis, different cases of serviceability limit state and ultimate limit state were also studied to demonstrate the ability of the developed model to simulate many different situations and limit states. The results illustrate that the allowable probability of failure is directly proportional to the optimum inspection interval. The sensitivity studies carried out demonstrate that a higher inspection quality is needed for ULS than SLS for both the single stage and the two stage inspection process. In addition, these results indicate that for the cases considered in this thesis, it is more beneficial for owners/managers to choose a two stage inspection process over a single stage inspection process for serviceability limit state conditions rather than for ultimate limit state conditions. Considering the SLS, the results demonstrate that a two stage inspection process is more beneficial to an owner/manager of a structure in terms of cost and possible number of failures. The screening exercise for detection followed by a high quality inspection for sizing is effective at ensuring that necessary repairs are carried out when defects are greater than the critical defect size, resulting in fewer failures and lower costs.

As well as demonstrating the capabilities of the developed maintenance management model, this chapter also demonstrates the sensitivity of the model to the input parameters, in particular, the parameters defining the inspection quality, probability of failure and the deterioration kinetics, therefore achieving part of objective 4 of this thesis. Further sensitivity studies will be carried out in Chapter 6. The results and sensitivities presented in this chapter emphasise that these parameters need to be estimated as accurately as possible to give meaningful results from the maintenance model. The calibration of the parameters for inspection quality and probability of failure are beyond the scope of this thesis, although the calibration of the parameters defining the growth of the defects for different repair materials will be discussed in Chapter 5.

CHAPTER 5 – EXPERIMENTAL SETUP AND RESULTS

CHAPTER 5 - EXPERIMENTAL SETUP AND RESULTS

5.1. INTRODUCTION

As discussed in Chapter 4, the results of the maintenance management model are sensitive to the growth parameters of the deterioration mechanism being considered. On this basis, to accurately model the rate of deterioration of ageing structures and the effectiveness of different repair strategies, an experimental programme was carried out to investigate the rate of crack growth in concretes of different mix constitution. According to Tilly (2007), results indicate that corrosion is the most common form of deterioration in reinforced concrete and is the cause of over 55 percent of the repairs carried out. Therefore, the form of deterioration studied in this thesis is chloride-induced corrosion of reinforced concrete, which can lead to cracking and spalling of the concrete cover. Research has been carried out in this area which studies the rate of crack growth in concrete slabs with different w/c ratios, strength, concrete cover, reinforcing bar diameters, concrete quality, cover/diameter ratio and proportions of cement (Al-Harthy et al., 2007; Vu et al., 2005; Alonso et al., 1998). However, the crack opening effect in blended cements such as pulverised fuel ash (PFA) and ground granulated blast-furnace slag (GGBS) has not previously been considered in the literature. Blended cements are used in new structures to enhance the durability of the structure and also in patch repairs of existing structures. It is therefore important to have information on the deterioration characteristics of concrete structures containing these blended cements.

For the purpose of this study three typical concrete repair mixes were designed, comparing OPC, OPC+PFA and OPC+GGBS. Adding PFA or GGBS to a concrete mix is an alternative to using OPC alone and can result in an increase in the durability of a new concrete structure or of a repair of an existing structure. An accelerated corrosion method was used to induce corrosion of the steel reinforcing bars and hence cracking of the concrete cover. The rate of crack growth in the different slabs was continuously monitored over a five month period to investigate the development of the cracks over time, with the aim of comparing the rate of crack propagation in the different repair mixes. This data can then be incorporated into the developed maintenance management methodology by fitting the growth parameters (α and g, described in Chapter 3) to the experimental data obtained, to study relative benefits of a number of repair materials (i.e. OPC, OPC+PFA and OPC+GGBS) and assist in maintenance planning. When considering the durability of concrete structures and repairs, both the initiation phase and propagation phase of deterioration must be considered. The effect of blended cements on the initiation phase has been previously studied in the literature (McPolin et al., 2005; Neville, 2005; Bertolini et al., 2004; McCarthy et al., 2001; Hussain and Rasheeduzzafar, 1994; Dhir and Byars, 1993; Thomas, 1991), and therefore, using the results of this experimental study the effectiveness of different blended cements on the durability of a structure or repair can be investigated in terms of both the initiation phase and the propagation phase of deterioration.

5.2. CONCRETE SAMPLES

Three mixes were designed to investigate crack propagation, comparing OPC, OPC+PFA (30% PFA) and OPC+GGBS (50% GGBS), each with the same w/c ratio of 0.5. The same coarse uncrushed aggregate (20mm and 10mm) and fine aggregate was used for all mixes. Each mix was designed for a slump of 60-180mm, with a target mean strength of 42N/mm^2. The relative density of the saturated surface dry aggregate was estimated at 2.6 and the fine aggregate had approximately 60% passing through a 600μm sieve. The quantities for each mix were designed based on saturated surface dry aggregate and modified based on a 2% water absorption of fine aggregate and a 1% water absorption of coarse aggregate (which were assumed based on characteristics of the materials in the laboratory), Table 5.1.

Three percent of Calcium Chloride dihydrate ($CaCl_2 - H_2O$) by weight of binder was added to each of the mixes to accelerate the corrosion of the reinforcing bars (Vu et al., 2005; Alonso et al., 1998; Andrade et al., 1993). Three slabs, cylinders and cubes were made from each mix. All samples were moist cured in a curing tank at 20°C ± 2°C for 28 days (according to BS EN 12390-2:2000).

The cubes and cylinders were tested after 28 days to determine the compressive strength (according to BS EN 12390-3:2002) and the tensile splitting strength (according to BS EN 12390-6:2000) of the concrete mixes. The rate of loading for the compressive strength test was 0.3 MPa/s (according to BS EN 12390-3:2002) and 0.06 MPa/s for the splitting test (according to BS EN 12390-6:2000). A testing jig was manufactured to position the cylindrical specimen for the splitting test. The cylinder was positioned in the jig between two hardboard packing strips as outlined in BS EN 12390-6:2000, and is illustrated in Figure 5.1.

	OPC	**OPC+PFA**	**OPC+GGBS**
Slab	3	3	3
Cylinder	3	3	3
Cube	3	3	3
Percent to add	15	15	15
Total Volume (m^3)	**0.06**	**0.06**	**0.06**
Cement (kg)	23.0	20.9	11.2
PFA (kg)	-	8.9	-
GGBS (kg)	-	-	11.2
Water (kg)	12.9	13.1	12.6
Fine Aggregate (kg)	35.8	33.3	36.4
10mm Aggregate (kg)	22.5	21.3	22.8
20mm Aggregate (kg)	44.7	42.4	45.3
$CaCl_2$ (3%) (kg)	0.7	0.9	0.7
Density of mix (kg/m^3)	**2355**	**2370**	**2365**

Table 5.1. Mix quantities

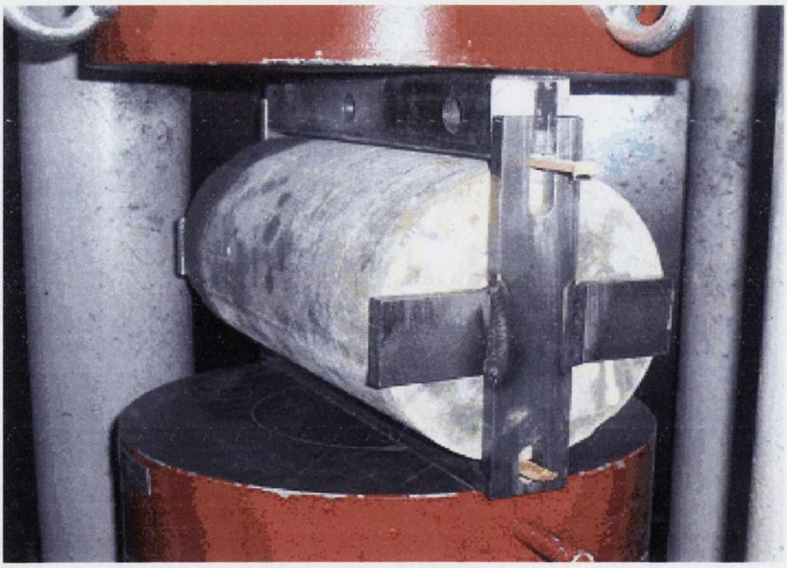


Figure 5.1. Apparatus for splitting test

The results of the compressive tests and tensile splitting tests are presented in Figure 5.2 and Figure 5.3, respectively. It is thought that the larger scatter in the results for the OPC is due to the accelerating properties of $CaCl_2$ in the concrete mix. When mixing, the slabs were poured first, followed by the cubes and cylinders. Therefore, when the cubes and cylinders were being poured, it became more difficult to vibrate the mix, which is considered to be the cause of the reduced strength and increased scatter of results.

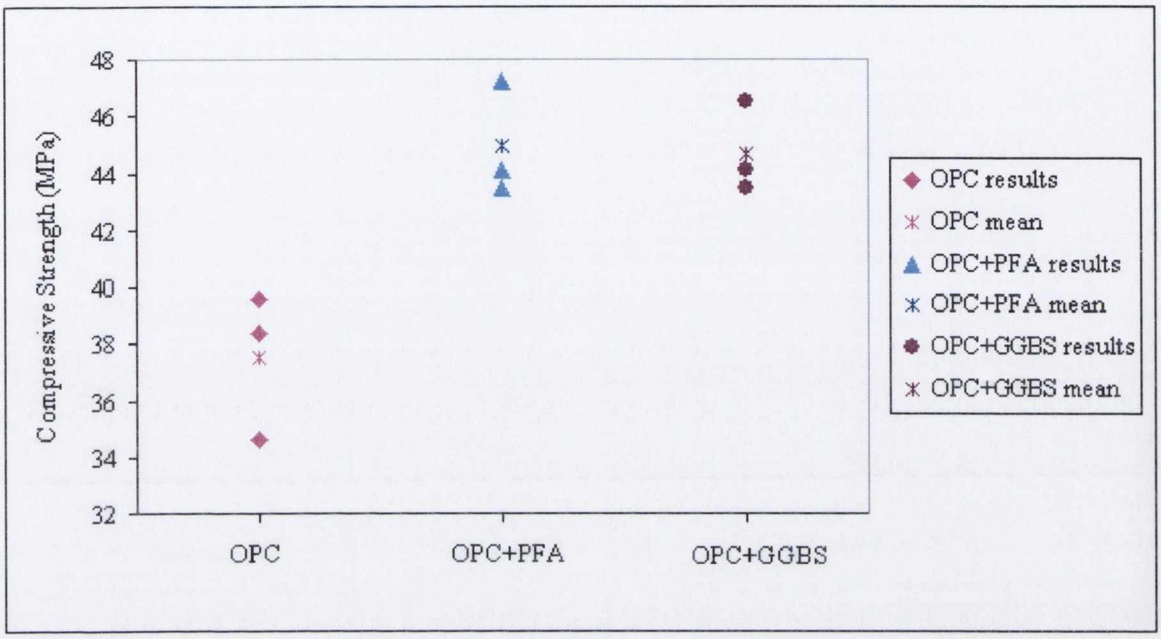


Figure 5.2. Results and mean for compressive strength of concrete specimens

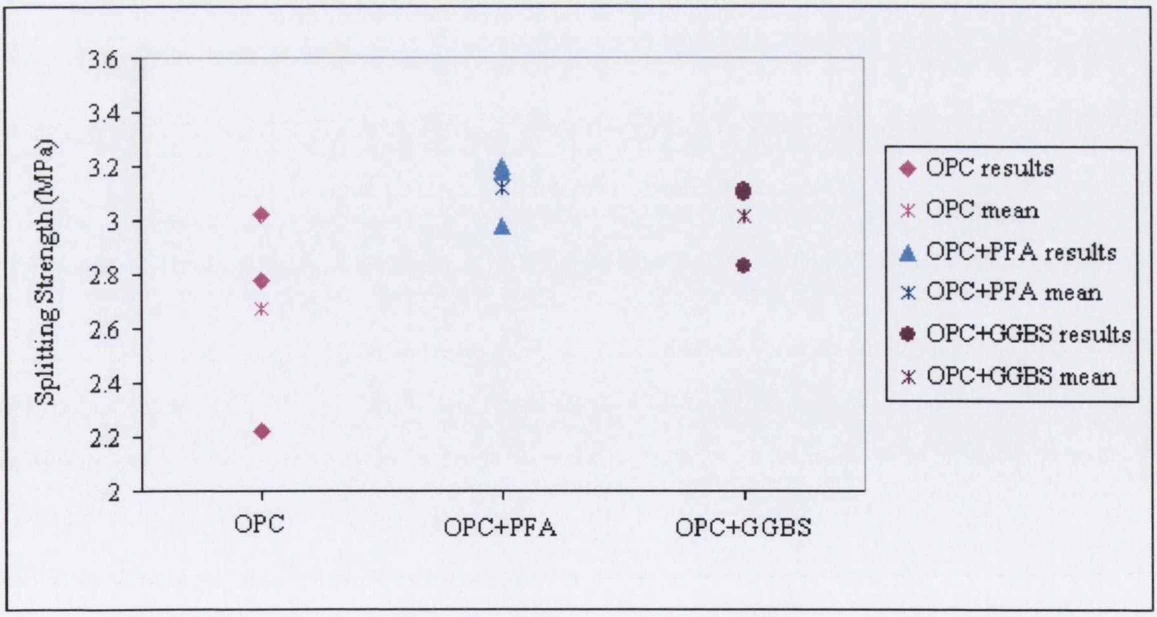


Figure 5.3. Results and mean for splitting strength of concrete specimens

This effect was negligible for the OPC+PFA and OPC+GGBS mixes since the blended cements lead to a higher workability of the concrete mix. The results of the experimental study will indicate that the splitting strength and compressive strength of the concrete are not the dominating factors when considering the rate of crack growth due to chloride-induced corrosion (this will be discussed further in the results section of this chapter), therefore, there will be no further discussion of the scatter in results for the OPC cubes and cylinders. However, it is recommended for future work that there is sufficient assistance present during pouring to ensure all cubes and cylinders are poured before the concrete starts to stiffen.

For the purpose of the experiment, three slabs were poured from each mix. All slabs were 300mm x 300mm square with a depth of 120mm and two 16mm diameter mild steel reinforcing bars in each, with a cover of 25mm. Figure 5.4 illustrates the positions of the steel bars in the slabs.

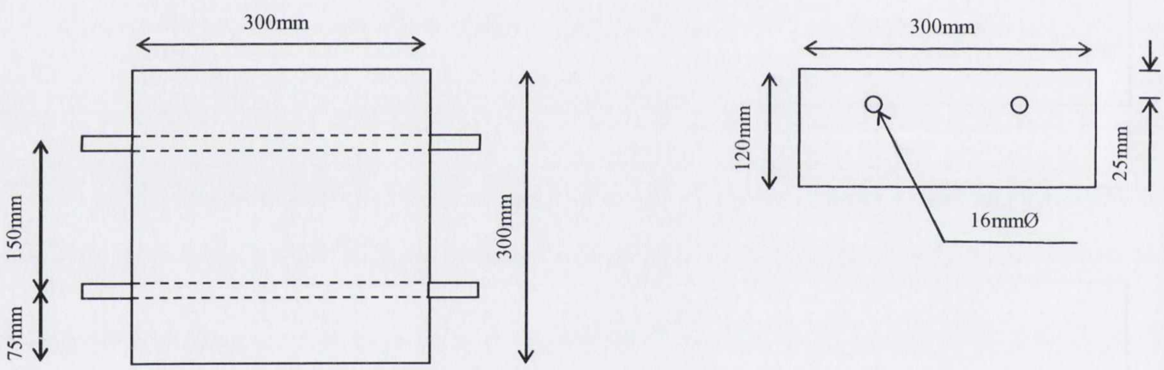


Figure 5.4. Position of steel reinforcing bars in slabs

5.3. ACCELERATED CORROSION

The purpose of the accelerated corrosion test is to simulate, over a short period of time, the corrosion of the steel reinforcement bars in reinforced concrete which occurs in real structures. The corrosion of the steel leads to the formation of corrosion products (rust) or oxides. These corrosion products have a higher volume than the original steel and, therefore, fill the voids in the concrete surrounding the reinforcing bar. This process induces stresses in the concrete, which in turn leads to cracking of the concrete cover (Vu and Stewart, 2005; Alonso et al., 1998). This process will be discussed in more detail in

Section 2.3. For this thesis the impressed current accelerated corrosion method was used. The experimental procedure is similar to Vu et al. (2005). The slabs were placed in a basin of 5% NaCl solution (using deionised water) immersed to a depth of about 30mm. A stainless steel electrode (150mm x 20mm) was placed in the basin to a depth of about 50mm and a constant electrical current was applied to this stainless steel electrode and the reinforcing bars, Figure 5.5. A cell was set up causing the chloride ions from the NaCl solution to diffuse towards the reinforcing bars. The steel bar acted as the anode electrode, the stainless steel plate acted as the cathode electrode and the pore fluid in the concrete was the electrolyte (Vu et al., 2005; Alonso et al., 1998). A corrosion rate of $100\mu A/cm^2$ was used in this study, which is the maximum found in real conditions in chloride contaminated concrete (Andrade et al., 1993). This corrosion rate corresponds to an electrical current of 15mA per bar. From a 12V power supply, the current was controlled using a LM317 current regulator for each bar.

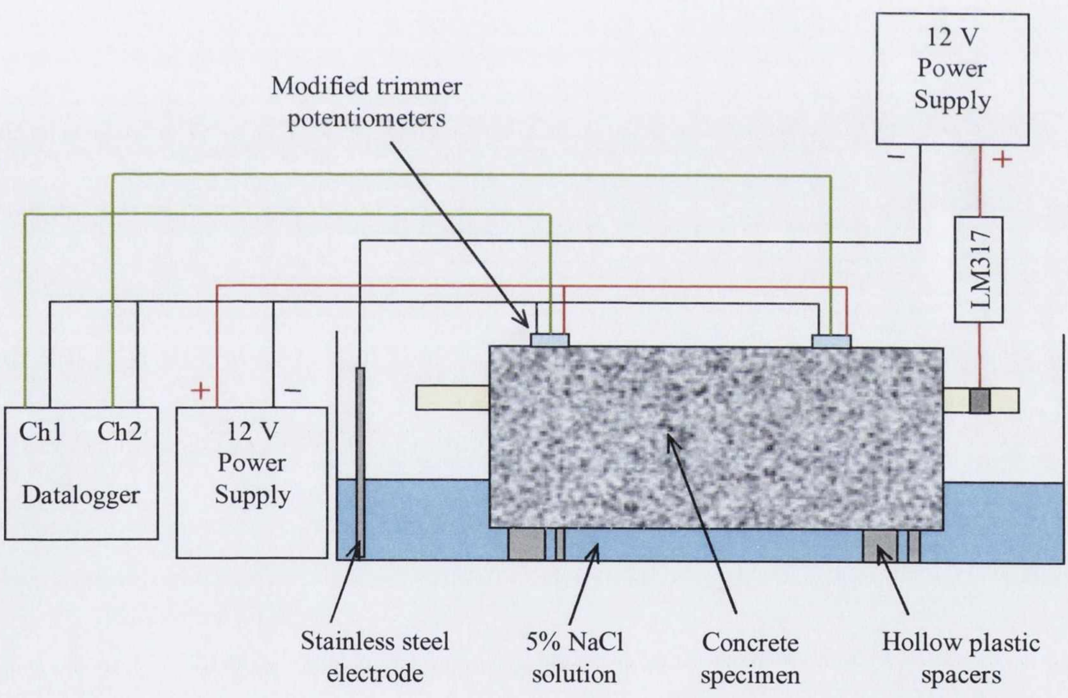


Figure 5.5. Experimental setup of slabs

5.4. CRACK MEASUREMENT

After four days of constant current, hairline cracks could be seen above the reinforcing bars on all slabs, running parallel to the bars. Two crack measurement devices were then glued onto the concrete along each crack to monitor the growth of the cracks over time, Figure 5.6. To measure the growth of the cracks over time, 10kOhm trimmer potentiometers (15 turns) were modified to perform similar to a Linear Variable Differential Transformer (LVDT). One of the original components can be seen in Figure 5.7(a), and the modified version is shown in Figure 5.7(b).

The performance of these modified trimmer potentiometers (POTs) was assessed using the StrainSmart system and a conventional LVDT. They were calibrated using the same system at 12V excitation. The body of an LVDT and a POT were fixed, while the arms of both were bonded together. Therefore, as the arms were moved, this allowed a direct comparison of the linear displacement (and hence voltage) which was recorded by both devices. The results of the comparison are illustrated in Figure 5.8. The readings recorded by the LVDT are tracked closely by the results recorded by the POTs, with a coefficient of determination value (R^2) of 0.998. Therefore, to reduce the experimental cost significantly, the POTs were used to monitor the rate of crack displacement over time.

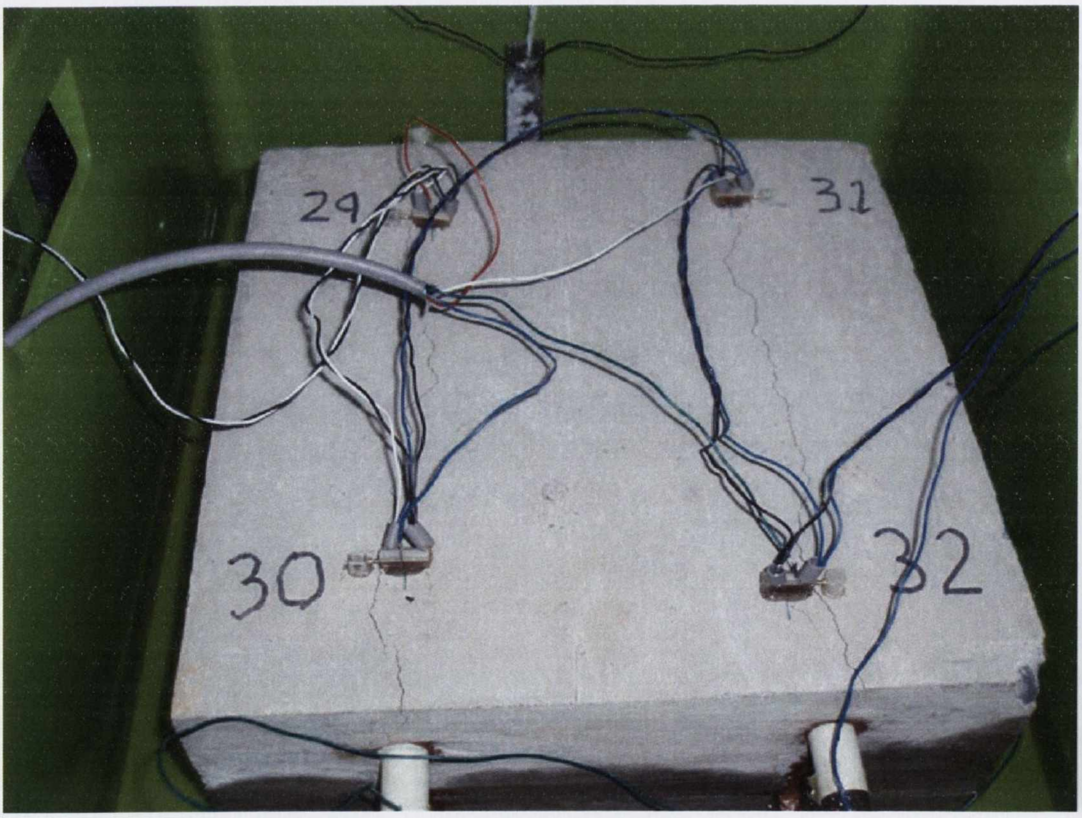
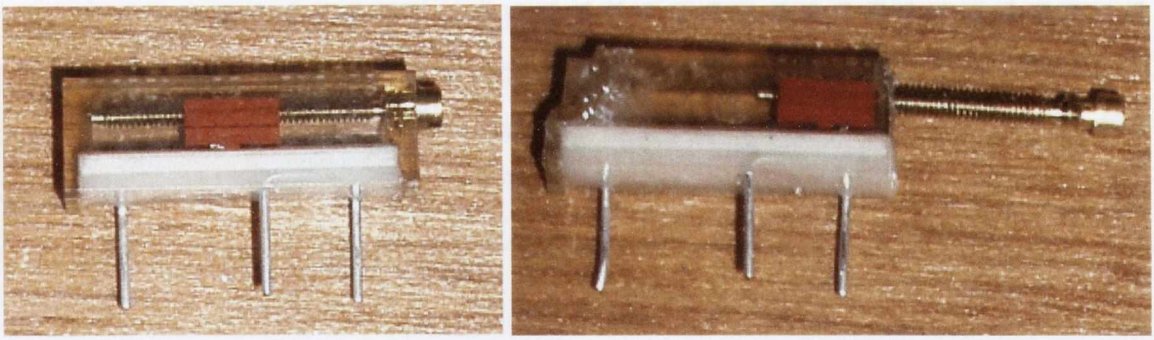


Figure 5.6. Arrangement of crack measurement devices on slab



(a) (b)

Figure 5.7. Original and modified potentiometer components

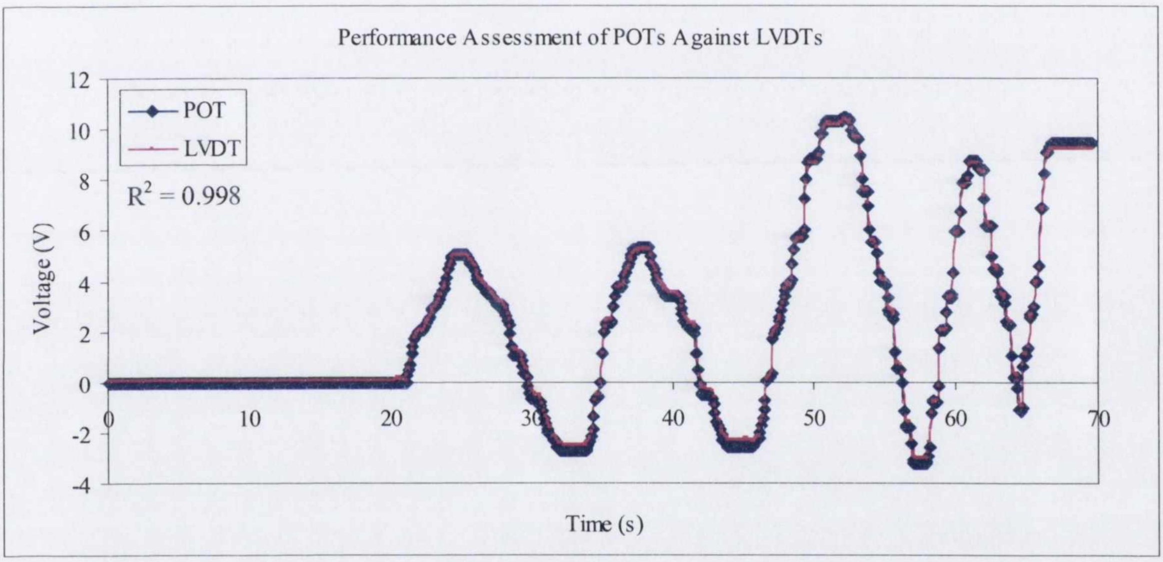


Figure 5.8. Performance Assessment of POTs against LVDT

To measure the crack width, the POTs were glued on to one side of the crack and a plastic fixing block was glued on the other side, perpendicular to the crack to get an accurate measure of the crack opening. Figure 5.9 illustrates the arrangement of the POTs on the slab. A 12V power supply was used for the POTs. Before the power supply to the POTs was turned on (two days later), an approximate measurement of the crack widths was made using a 10X microscope and a crack width measurement card (with an accuracy of 0.05mm). The average width of the cracks for OPC, OPC+PFA and OPC+GGBS slabs were estimated at 0.1mm, 0.05mm, 0.15mm, respectively. Figure 5.10 shows the hairline crack in one of the OPC+GGBS slabs.

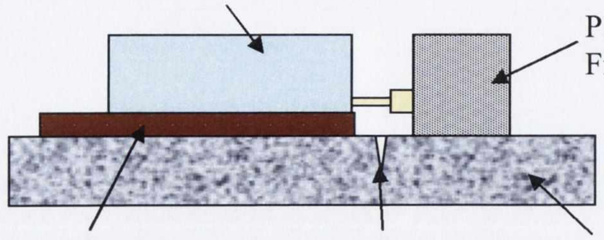


Figure 5.9. Schematic of modified trimmer potentiometers

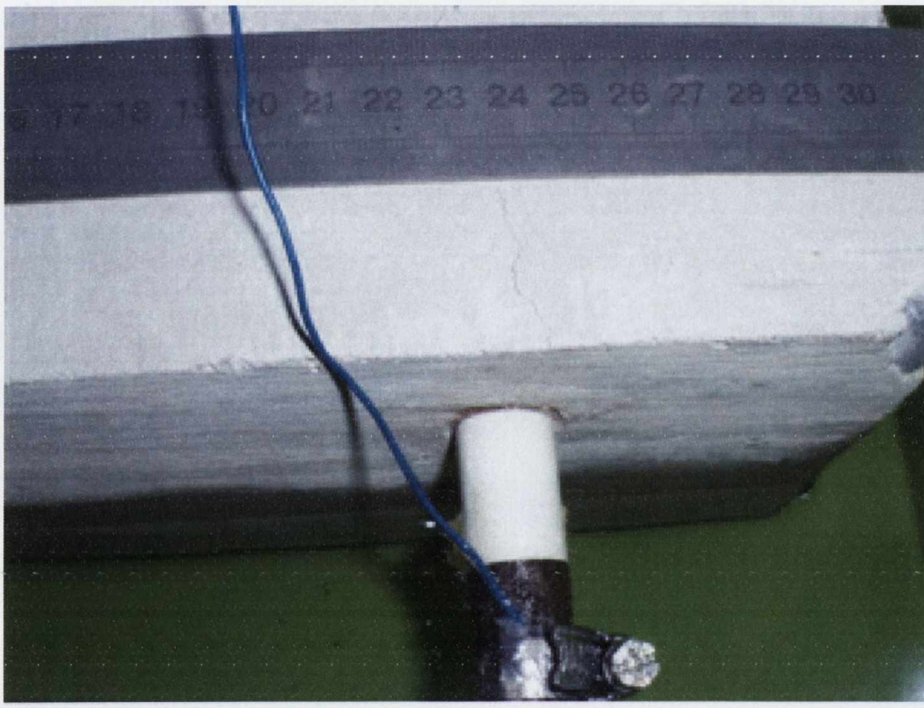


Figure 5.10. Initial crack in OPC+GGBS slab before the POTs were glued on

Three dummy POTs were also glued onto the slabs (one was an OPC slab, one was an OPC+PFA slab and one was an OPC+GGBS slab) at the centre of the slabs where no cracks had developed. The purpose of this was to monitor fluctuations in the excitation power supply. A forty channel Datascan datalogging system was set up to log the data from the thirty nine POTs every hour. The nine slabs were set up on a shelving unit, Figure 5.11, and monitored over the course of the experiment. The NaCl solution was topped up regularly to account for evaporation and keep a consistent level in all basins.

Figure 5.11. Experimental setup

5.5. RESULTS

In the previous section, the effect of blended cements on various concrete properties was investigated by considering previous studies carried out in the literature. In relation to the initiation phase of deterioration, the effect of the addition of blended cements has been considered (McPolin et al., 2005; Neville, 2005; Bertolini et al., 2004; McCarthy et al., 2001; Hussain and Rasheeduzzafar, 1994; Dhir and Byars, 1993; Thomas, 1991), although none of these studies consider the effect of the addition of PFA or GGBS on the rate of crack propagation in reinforced concrete. When considering the durability of concrete structures and repairs, both the initiation phase and propagation phase of deterioration must be considered. On this basis, an experimental study has been carried out as part of this thesis to investigate the rate of crack growth in different repair materials. It is recognised that the effect of blended cements on the different processes outlined in the previous section may influence the results of this experimental study. For the purpose of this thesis, three mixes have been considered, as outlined in Section 5.2 (i.e. OPC, OPC+PFA and OPC+GGBS). Adding PFA or GGBS to a concrete mix is an alternative to using OPC alone, and can result in an increase in the durability of a new concrete structure or of a repair of an existing structure (Neville, 2005; Bertolini et al., 2004; McCarthy et al., 2001;

Hussain and Rasheeduzzafar, 1994; Dhir and Byars, 1993; Thomas, 1991). The objective of this experiment is to compare the post initiation behaviour of concretes of different mix constitution (i.e. compare an OPC mix with a mix of OPC and blended cements such as PFA and GGBS). The relative growth of the cracks due to corrosion of the reinforcing bars over time is the main area of interest. These results will be incorporated into the developed maintenance management methodology. This will be achieved by fitting the growth parameters (α and g, described in Chapter 3) to the experimental data obtained, to study the relative benefits of a number of repair materials and assist in maintenance planning. In addition, the crack width and the corresponding mass loss of the steel bars is investigated at different stages in the experiment for each of the mixes to directly compare the performance of the different mixes.

The experiment was run for a total of five months until the crack width in each of the slabs had reached at least 1.0mm approximately (this is considered the limit crack width, as discussed in Section 4.2), and the information from the POTs was downloaded on a weekly basis. The slabs were also monitored regularly and an approximate measurement of the cracks was taken using a crack width measurement card to check the performance of the POTs. The electrical drift of the system was monitored using the dummy POTs and was found to have a negligible effect of the results. The readings from the dummy POTs was also converted to millimetres using the conversion factors for the POTs, and the results are illustrated in Figure 5.12.

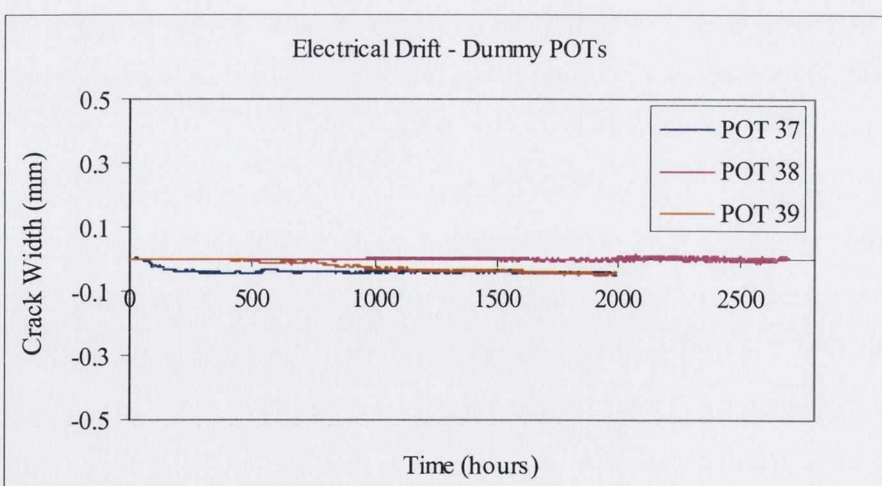


Figure 5.12. Electrical drift of the experimental setup, converted to mm

5.5.1. Average rate of crack growth for different mixes

After running the experiment for four days, hairline cracks could be seen on all concrete slabs, similar to Figure 5.10. Since crack propagation is of interest in this experiment and not crack initiation (as explained in Section 5.5), the crack measurement devices (POTs) were not glued on until cracks had appeared in all the slabs and monitoring of all the cracks began at the same time. The POTs were glued to the slabs on either side of the cracks and used to measure the progressive growth of the cracks over time using dataloggers. An approximate measure of each of the cracks was taken using a crack width measurement card before the POTs were glued on. The cracks in the slabs with OPC+GGBS were noticeably bigger than the slabs with OPC only and OPC+PFA. The average approximate crack width for each of the slabs is given in Table 5.2 (approximated using a crack width measurement card with an accuracy of 0.05mm).

Mix	Name	Steel Number	Approx. Crack Width (mm)
OPC	Slab 1	1.1	0.1
		1.2	0.1
	Slab 2	2.1	0.1
		2.2	0.1
	Slab 3	3.1	0.1
		3.2	0.1
OPC+PFA	Slab 4	4.1	0.05
		4.2	0.05
	Slab 5	5.1	0.05
		5.2	0.05
	Slab 6	6.1	0.05
		6.2	0.06
OPC+GGBS	Slab 7	7.1	0.15
		7.2	0.15
	Slab 8	8.1	0.15
		8.2	0.15
	Slab 9	9.1	0.15
		9.2	0.15

Table 5.2. Average approximate crack widths of each crack before POTs were glued on

All of the cracks developed on the surface of the slabs directly above the reinforcing bars as one long crack or as a series of two or three cracks. In some of the slabs (Slab 1, OPC and Slab 9, OPC+GGBS) after the initial cracks developed and the POTs were glued on, during the course of the experiment these cracks appeared to be closing. This was due to the development of new cracks parallel and close to the cracks in question. An example of this can be seen in Figure 5.13. In this case the POTs were moved to a new location on the crack. As the cracks on the top surface of the slabs grew larger, cracks could be seen growing on the side of the slab from the reinforcing bar across the slab towards the other bar and towards the end of the slab, as well as up through the concrete cover towards the top of the slab. This cracking pattern can be seen in Figure 5.14.

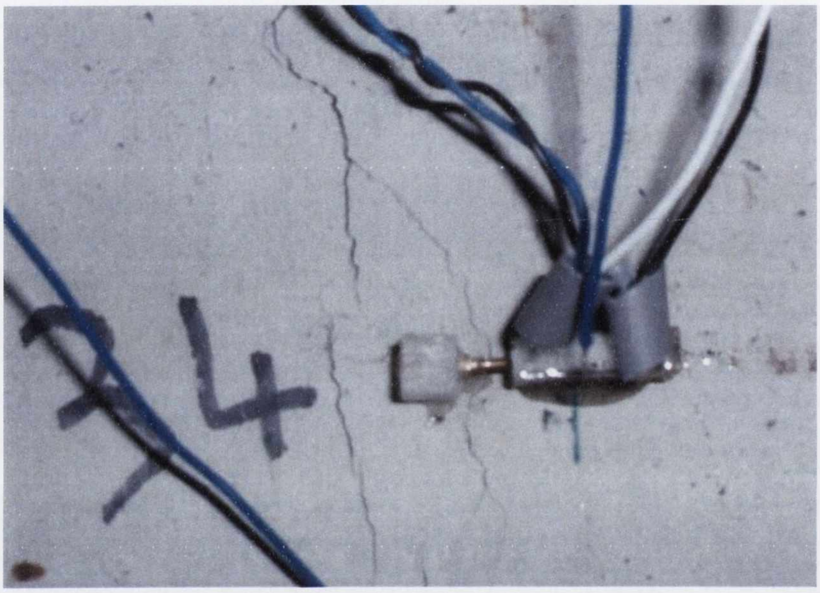


Figure 5.13. Opening of new crack beside existing crack where POT was located

Figure 5.14. Propagation of cracks in three directions away from the reinforcing bar

The growth of the cracks was monitored regularly. It could be seen visually that the cracks in the slabs containing OPC+GGBS were progressing at a higher rate than in the slabs containing OPC only and OPC+PFA. Also the OPC+PFA slabs continued to grow at a remarkably slow rate. The results from the POTs for each group of slabs are illustrated in Figure 5.15 – Figure 5.17.

The mean value, standard deviation (SD) and coefficient of variation (CoV) of the crack width for each mix is also presented at regular intervals in these figures. In Figure 5.15 (OPC slabs), there is only a small variation in the SD over time (max difference of 0.03mm), resulting in a reduction in the CoV for each time step considered. In relation to OPC+PFA (Figure 5.16), since the SD increases with the mean displacement, there is only a small difference in the CoV (max difference of 0.03) for all of the time steps considered. However, for OPC+GGBS (Figure 5.17), there is no obvious trend, with the SD reducing and then increasing in relation to the time steps considered.

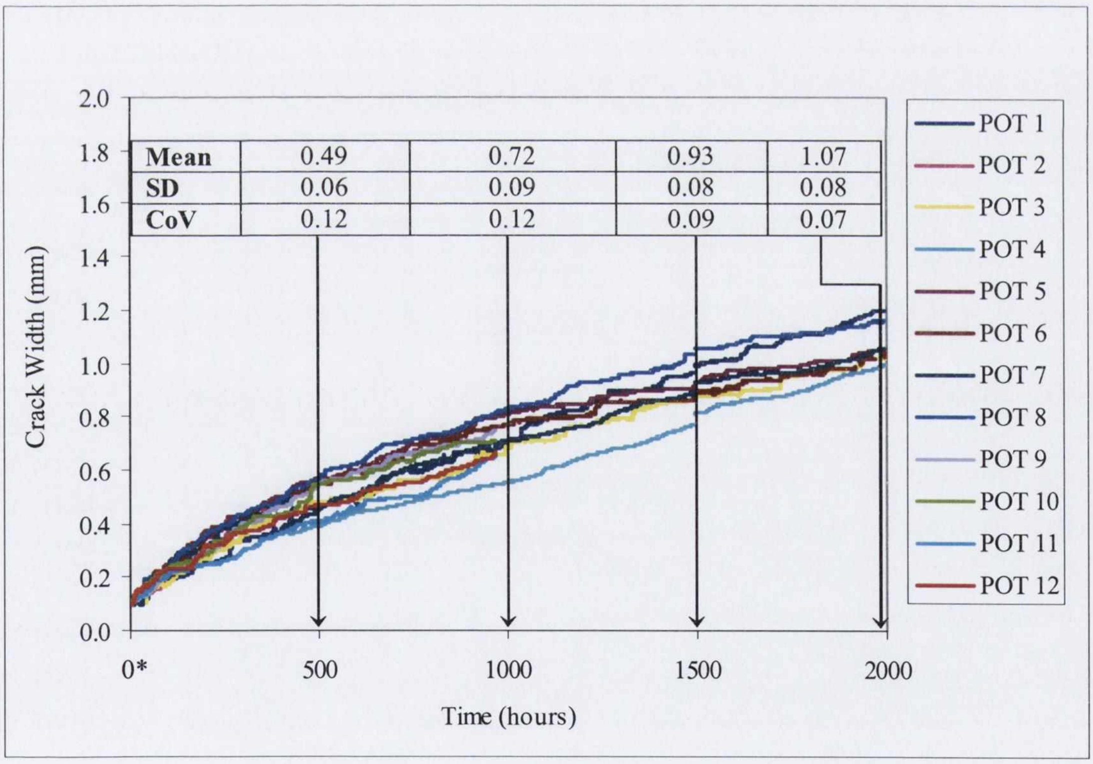


Figure 5.15. POT data for growth of cracks in OPC slabs over time (Note: 0* see nomenclature)

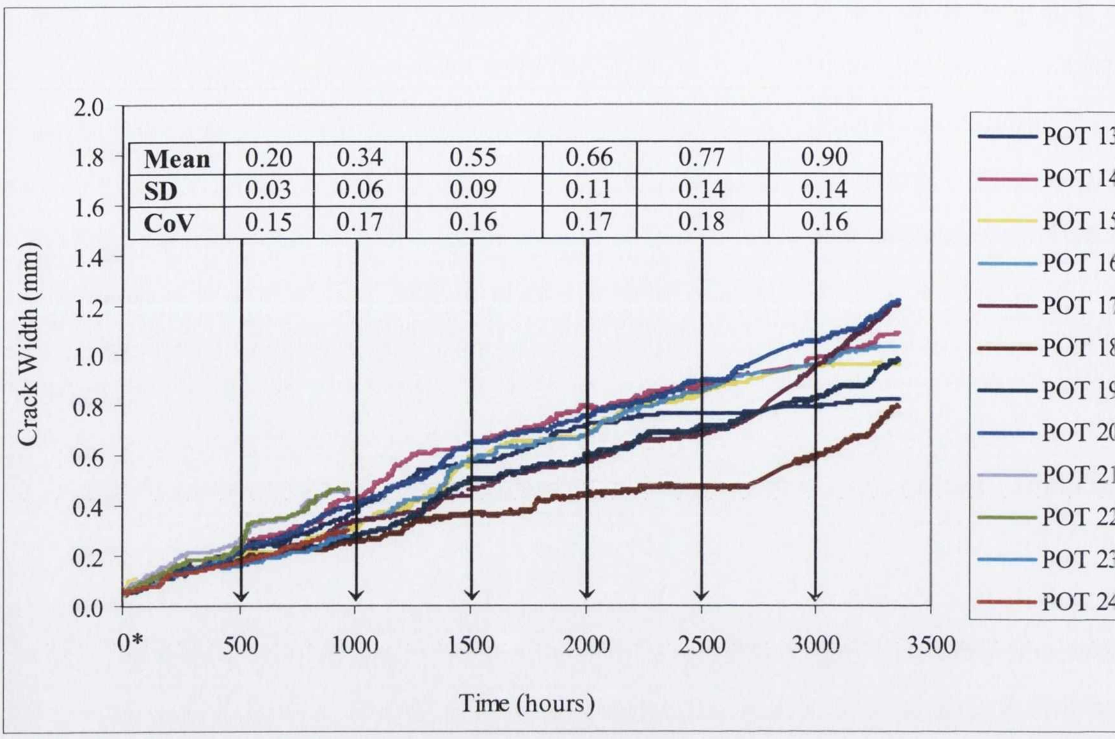


Figure 5.16. POT data for growth of cracks in OPC+PFA slabs over time

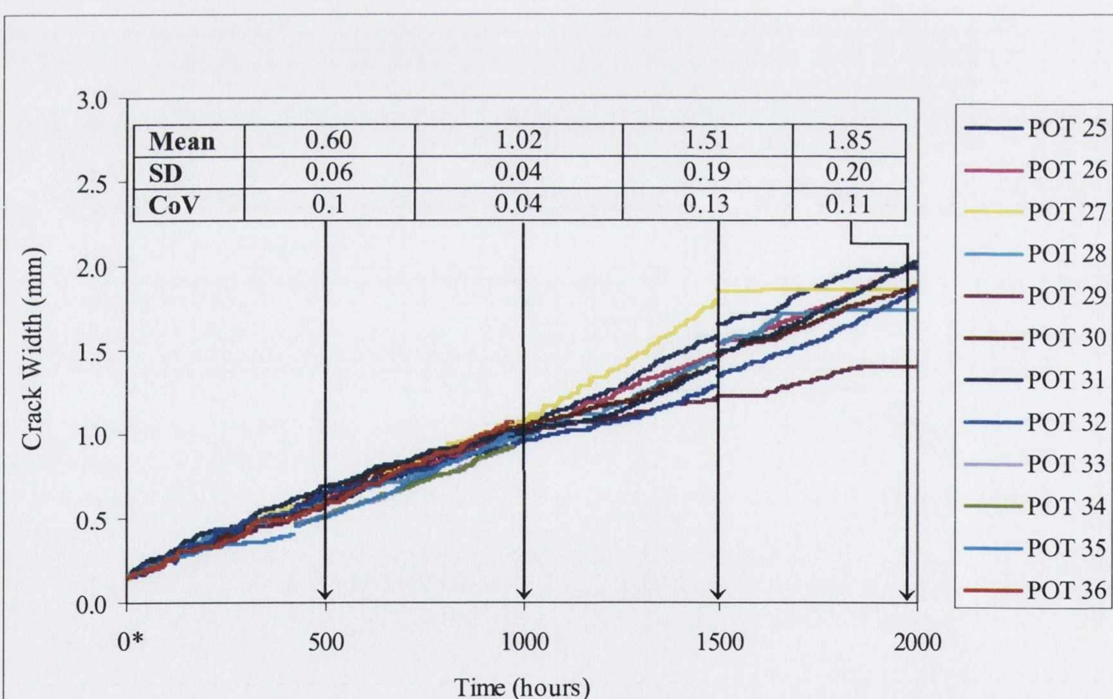


Figure 5.17. POT data for growth of cracks in OPC+GGBS slabs over time

It is thought that the noticeable reduction in the SD after about 400 hrs in OPC+GGBS is due to the relocation of some POTs (noticeably POT 35, Figure 5.17) following the development of parallel cracks close to the POTs (as discussed previously, Figure 5.13). The variation in the standard deviation values is due to a combination of the position of the POTs on the cracks, the behaviour of different cracks, and the performance of the POTs (particularly in relation to crack widths greater than 1.5mm).

The average rate of crack growth for each of the mixes can be seen in Figure 5.18. For the OPC slabs, there is a change in slope in the average rate of crack growth. Initially the slope was similar to the OPC+GGBS slabs, however, at about 350 hours the slope changed and the average rate of crack growth became more like that of the OPC+PFA, with only a small difference in the final slope of the two mixes. This is thought to be due to the build up of oxides on the reinforcing bar, inhibiting the diffusion of oxygen and moisture into the steel.

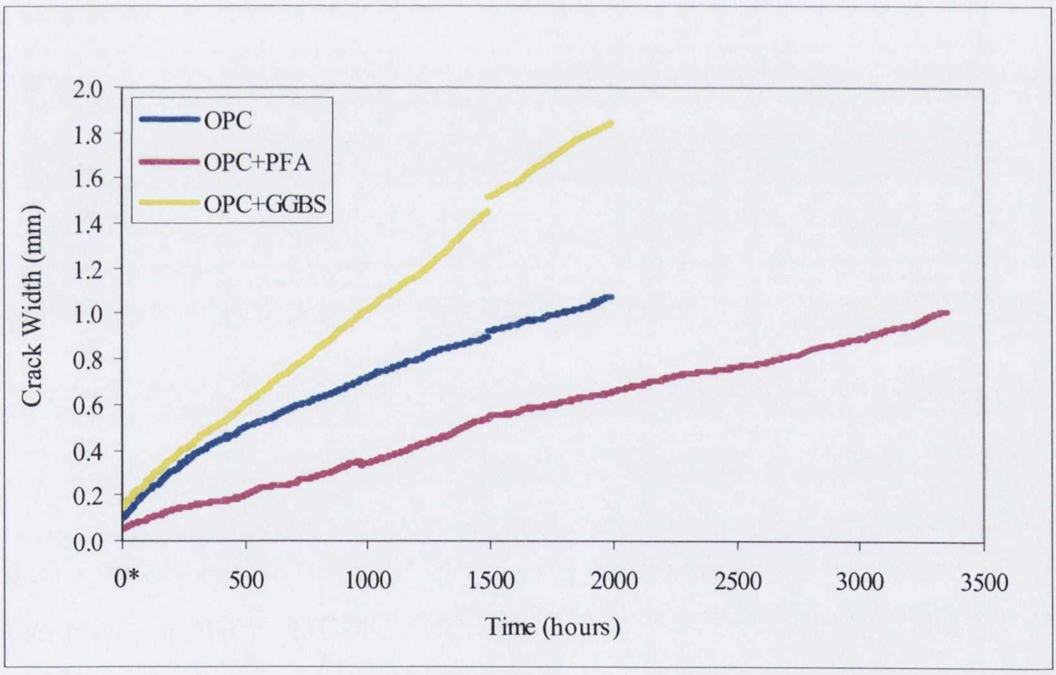


Figure 5.18. Average rate of crack growth over time for 3 mixes

The OPC+PFA slabs undoubtedly performed better than the other two mixes in terms of cracking due to corrosion of reinforcing bars, however, the appearance of the OPC+PFA slabs may be considered a problem. Throughout the experiment, pore water from the

OPC+PFA concrete came to the surface of the slab, possibly due to capillary action, resulting in a rust coloured stain on the surface of the slab, Figure 5.19. Although this did occur on the OPC and OPC+GGBS slabs, it was to a much lesser extent. As discussed in Chryssanthopoulos and Sterritt (2002), the aesthetic appearance may affect the functional and financial performance of a structure and may be an issue in areas where aesthetics are important to owners/managers of structures.

Figure 5.19. Staining of OPC+PFA slabs from pore fluid

5.5.2. Corrosion Rate Measurement

At different stages of the experiment (as will be discussed in Section 5.5.3 and Section 5.5.4), the current and POTs were disconnected from the slabs and the reinforcing bars were broken out. Using the gravimetric weight loss method, according to ASTM G1-90, the weight loss of the bars over the course of the experiment was found (Vu et al., 2005; Vu, 2003). Before pouring the slabs, all reinforcing bars were cleaned and the mass of each bar was recorded. By determining the weight loss due to corrosion, the average corrosion rate of the bars can be calculated. According to the literature (Vu et al., 2005; Vu, 2003; Alonso et al., 1998; Andrade et al., 1993) the actual measured corrosion rate of the bars differs from the set corrosion rate (100μA/cm^2) and is usually higher. This can be due to spalling of the steel at the surface of the bar while the surrounding steel is corroding and oxidising. It can also be attributed to the acidification developed by the progressive corrosion of the steel bars due to the impressed current, which can induce a simultaneous additional corrosion (Vu et al., 2005; Alonso et al., 1998). According to Andrade et al. (1993), the pH values of the corrosion products from this process were measured to be as low as 3.0.

Before carrying out the procedure according to ASTM G1-90, the bars are cleaned, dried and weighed. The specimens are then brushed lightly with a non-metallic brush. Chemical cleaning is then carried out in a bath of hydrochloric acid, hexamethylene tetramine and reagent water, at 20°C for 15 minutes. The hexamethylene tetramine prevents digestion of the base metal (Thomas and Matthews, 2004; Thomas, 1991). This chemical cleaning is followed by a light brushing with a non-metallic brush in reagent water. This procedure is carried out on the specimens several times and the mass of the bar is recorded after each cleaning. Using this procedure, Figure 5.20 was obtained.

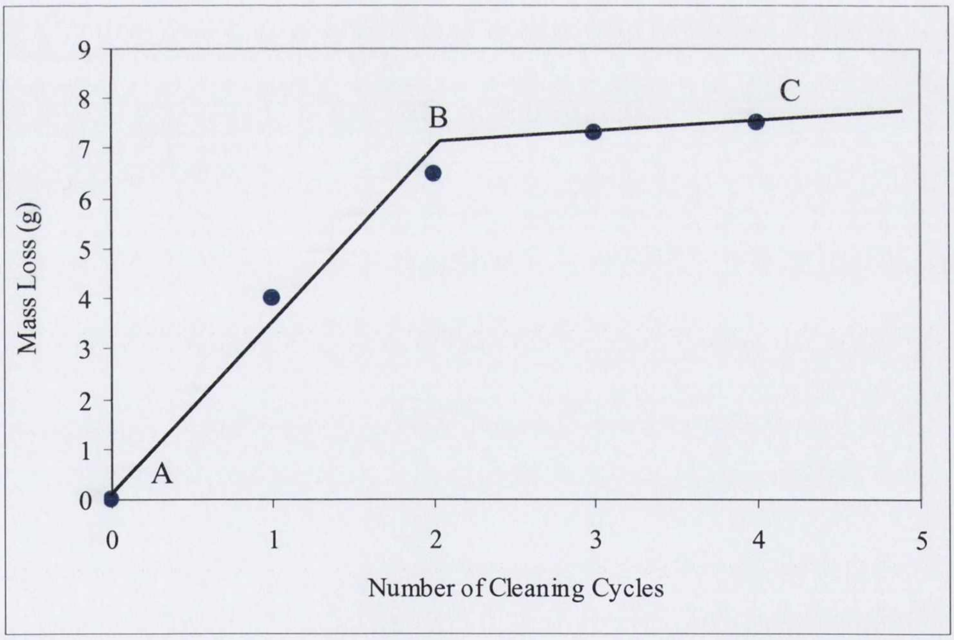


Figure 5.20. Mass loss of corroded specimens resulting from repetitive cleaning cycles (ASTM G1-90)

The mass loss due to corrosion corresponds to point B on the graph. The latter part of the graph corresponds to the mass loss after the corrosion products have been removed.

Figure 5.21(a) shows a corroded bar before cleaning and Figure 5.21(b) shows the same bar after cleaning. Using the graph obtained for a sample corroded reinforcing bar (Figure 5.20), all further reinforcing bars were cleaned by carrying out two cleaning cycles according to the method outlined above.

(a)

(b)

Figure 5.21. Corroded reinforcing bar before and after cleaning

5.5.3. Comparison of crack width and mass loss after fixed period of time

To investigate the reason for the large variation in average crack width in the different mixes and the reason why the cracks in the OPC+PFA were growing so slowly, three slabs (one of each mix, Slab 3, Slab 6 and Slab 9) were disconnected from the power supply after about 1150 hours (48 days) of testing. The POTs were taken off and approximate measurements of the cracks were taken to verify the readings from the POTs. The approximate readings taken using the crack width measurement card verified that the readings from the POTs were correct.

The slabs were then broken open and the reinforcing bars were taken out. From visual observations there appeared to be a similar amount of corrosion products on each of the bars. The bars were then cleaned according to ASTM G1-90, as described in Section 5.5.2. Figure 5.22 illustrates the condition of the bars before and after cleaning. The results for the initial, final and percentage mass loss are given in Table 5.3, along with the corresponding approximate crack width.

Figure 5.22. Steel reinforcing bars before and after cleaning according to ASTM G1-90

Mix (Slab)	Bar Number	Crack Width (mm)	Initial Mass (g)	Final Mass (g)	Mass Loss (g)	Mass Loss (%)	Δ Average Mass Loss wrt OPC (%)
OPC	3.1	0.55	633.62	612.39	21.23	3.35	0
(Slab 3)	3.2	0.45	632.46	611.5	20.96	3.31	
OPC+PFA	6.1	0.35	631.98	611.25	20.73	3.28	-0.05
(Slab 6)	6.2	0.3	631.5	610.75	20.75	3.29	
OPC+GGBS	9.1	1.0	634.91	610.62	24.29	3.83	0.5
(Slab 9)	9.2	0.95	632.73	608.48	24.25	3.83	

Table 5.3. Results for approximate crack width and mass loss for slabs 3, 6 and 9

The results of average mass loss and average crack width for each mix are also illustrated in Figure 5.23. The results confirm that the OPC+PFA slabs perform better with respect to crack growth due to uniform corrosion (i.e. % mass loss) of the reinforcing bars. There is only a maximum cumulative difference of 0.55% in the percentage mass loss for slabs with OPC+PFA and slabs with OPC+GGBS, yet the average crack width varies by over 0.6mm. Therefore, in relation to the SLS of a structure, using an OPC+PFA mix rather than an OPC+GGBS mix with the same initiation time, maintenance would be required less often to repair cracking due to chloride-induced corrosion (e.g. considering a limit crack width of 1.0mm).

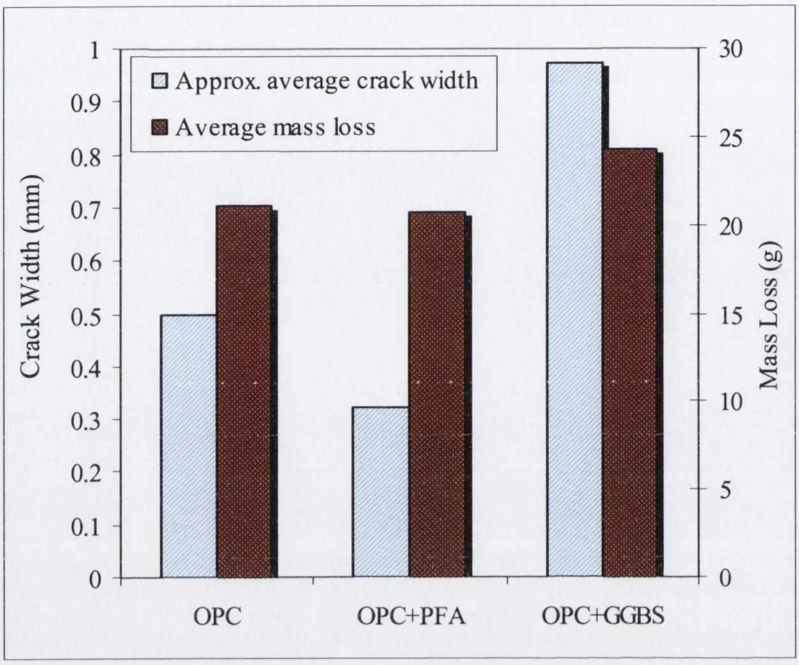


Figure 5.23. Results of approximate average crack width and mass loss for slabs 3, 6 and 9

When the bars were cleaned, pitting corrosion could be seen on each of the bars. There was a lot of pitting on the ends of the bars of Slab 9 (OPC+GGBS). There pits were about 50mm in length and were a maximum of 1.0mm deep. Otherwise, the pitting was quite uniform along the bars, which ultimately led to the same effect as uniform corrosion. The width of the cracks in each of the slabs was quite constant, only varying by about 0.05mm (approximated using crack width measurement card) on any point along the cracks.

5.5.4. Comparison of mass loss for fixed crack width

In addition, it was decided to investigate the percentage mass loss that was needed to create a 1.0mm crack width in each of the concrete mixes (this is considered the limit crack width, as discussed in Section 4.2). On this basis, four more slabs were disconnected and broken after 2250 hours (94 days) when the two remaining OPC slabs had crack widths of approximately 1.0mm. At this stage the approximate crack width of the OPC+GGBS slabs was 2mm. Following this, the remaining two slabs (OPC+PFA, slab 4 and slab 5) were disconnected after 3625 hours (151 days) when the crack widths in the slabs had reached approximately 1.0mm. For each of the slabs the POTs were taken off and again, approximate readings which were taken using the crack width measurement card verified that the readings from the POTs were correct. The slabs were then broken open and the reinforcing bars were taken out. The bars were then cleaned according to ASTM G1-90, as described in Section 5.5.2. Figure 5.24 illustrates the condition of the bars before and after cleaning (slab 4 and slab 5). For the slabs with approximate crack widths of 1.0mm, the results for the initial, final and percentage mass loss are given in Table 5.4.

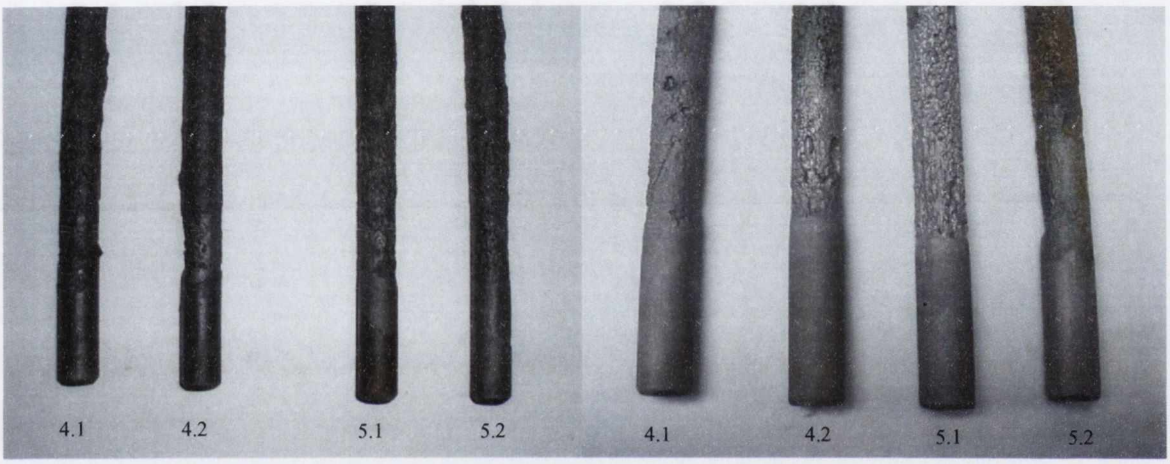


Figure 5.24. Steel reinforcing bars before and after cleaning according to ASTM G1-90

Mix	Name	Bar Number	Crack Width (mm)	Initial Mass (g)	Final Mass (g)	Mass Loss (g)	Mass Loss (%)	Δ Average Mass Loss wrt OPC (%)
OPC	Slab 1	1.1	1.0	632.46	591.82	40.64	6.43	0
		1.2	1.0	632.42	591.96	40.46	6.40	
	Slab 2	2.1	1.0	632.44	592.26	40.18	6.35	
		2.2	1.0	632.99	591.76	41.23	6.51	
OPC + PFA	Slab 4	4.1	0.95	631.26	565.09	66.17	10.48	+4.18
		4.2	0.95	632.59	566.46	66.13	10.45	
	Slab 5	5.1	1.0	633.15	565.79	67.36	10.64	
		5.2	1.0	632.19	563.8	68.39	10.82	
OPC + GGBS	Slab 9	9.1	1.0	634.91	610.62	24.29	3.83	-2.59
		9.2	0.95	632.73	608.48	24.25	3.83	

Table 5.4. Results for approximate crack width and mass loss for slabs 1,2,4,5 and 9

As illustrated in Figure 5.25, the average percentage mass loss of reinforcing bar to produce a 1.0mm crack width is much larger for a slab with 30% PFA. The average percentage mass loss needed to produce a crack width of 1.0mm for slabs made with OPC+PFA is 10.6%, which is 4.18% higher than the percentage mass loss needed for slabs made with OPC alone and 6.77% higher than the percentage mass loss needed for slabs made with OPC+GGBS. This illustrates the higher performance of OPC+PFA slabs in relation to the development of cracks due to chloride-induced corrosion of reinforcing bars. When considering the maintenance management of a structure, this is an important development, especially in relation to the serviceability limit state, where the maximum crack width can dictate when repairs are carried out.

Using a combination of OPC+PFA (30% PFA in this case) in the concrete can result in a longer period between inspections since the time taken to reach a limit crack width is increased in relation to slabs with OPC alone and slabs with OPC+GGBS (50% GGBS in this case). This will be discussed further in Chapter 6. In relation to ultimate limit state conditions, it must also be recognised that structures which are made using blended cements have different cracking properties than structures made with OPC alone. This must be taken into account particularly when estimating the percentage mass loss (and hence remaining structural capacity) of a component based on the corrosion induced crack width in the concrete cover.

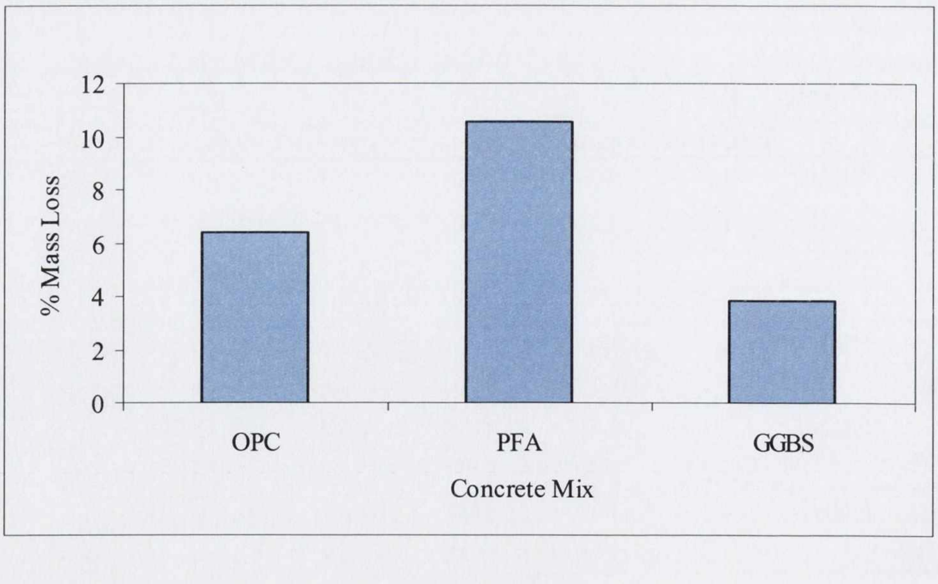


Figure 5.25. Average % mass loss of reinforcing bars to produce 1.0mm crack width for each material

5.5.5. Correction of experimental results using experimental corrosion rate

Using the mass loss from each of the slabs, an experimental corrosion rate for each of the bars can be calculated. As mentioned in Section 5.5.2, the applied corrosion rate (of 100μA/cm^2) can vary from the actual experimental corrosion rate. The experimental corrosion rate can be calculated using Equation 5.1 (Vu, 2003). The calculated results for experimental corrosion rate ($i_{corr(exp)}$) are give in Table 5.5, along with the amended time to the measured crack width for each of the slabs.

$$i_{corr} = \frac{(K \times W)}{0.0116(A \times T \times \rho_s)} \ (\mu A / cm^2) \qquad \text{Equation 5.1}$$

Mix	Slab	Bar Number	Crack Width (mm)	$i_{corr(exp)}$ (μA/cm^2)	Mean $i_{corr(exp)}$ (μA/cm^2)	SD $i_{corr(exp)}$ (μA/cm^2)	Time (hrs)	Corrected Time (hrs)
OPC	Slab 1	1.1	1.0	115.08	115.72	1.51	2250	2589
		1.2	1.0	114.57			2250	2578
	Slab 2	2.1	1.0	113.78			2250	2560
		2.2	1.0	116.75			2250	2627
	Slab 3	3.1	0.55	117.82			1148	1353
		3.2	0.45	116.32			1148	1335
OPC +PFA	Slab 4	4.1	0.95	116.30	116.89	2.01	3625	4216
		4.2	0.95	116.23			3625	4213
	Slab 5	5.1	1.0	118.39			3625	4292
		5.2	1.0	120.20			3625	4357
	Slab 6	6.1	0.35	115.05			1148	1321
		6.2	0.3	115.16			1148	1322
OPC +GGBS	Slab 7	7.1	1.9	122.33	129.28	4.96	2250	2752
		7.2	2.0	125.78			2250	2830
	Slab 8	8.1	1.9	127.74			2250	2874
		8.2	2.0	130.45			2250	2935
	Slab 9	9.1	1.0	134.81			1148	1548
		9.2	0.95	134.58			1148	1545

Table 5.5. Corrected experimental corrosion rate and time to given crack widths

All the results for crack propagation can be altered to give a corresponding result for an actual experimental corrosion rate of 100μA/cm^2 so that all the results can be compared in a more objective way once they all correspond to the same corrosion rate. The results are altered by multiplying the time to a particular crack width by $i_{corr(exp)}/100$ (Vu et al., 2005), to get the times for a corresponding experimental corrosion rate of $100\mu A/cm^2$. This just has the affect of delaying the time to a particular crack width and does not affect the standard deviation or coefficient of variation of the crack width data from the POTs (as presented in Figure 5.15 - Figure 5.17). Figure 5.26 – Figure 5.28 illustrate the original and corrected rate of crack propagation for each of the reinforcing bars in each mix.

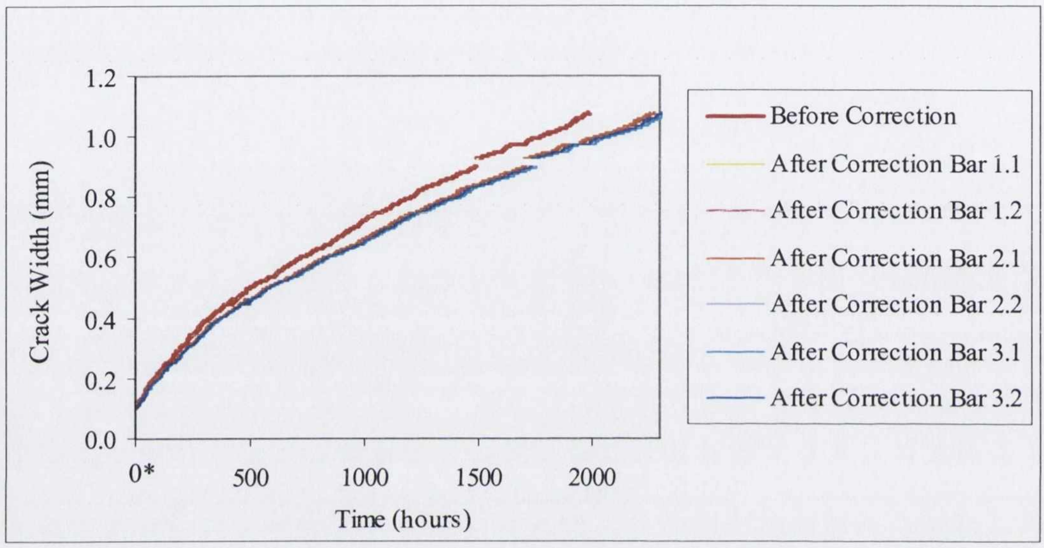


Figure 5.26. Propagation of crack growth for OPC slabs, before and after correction

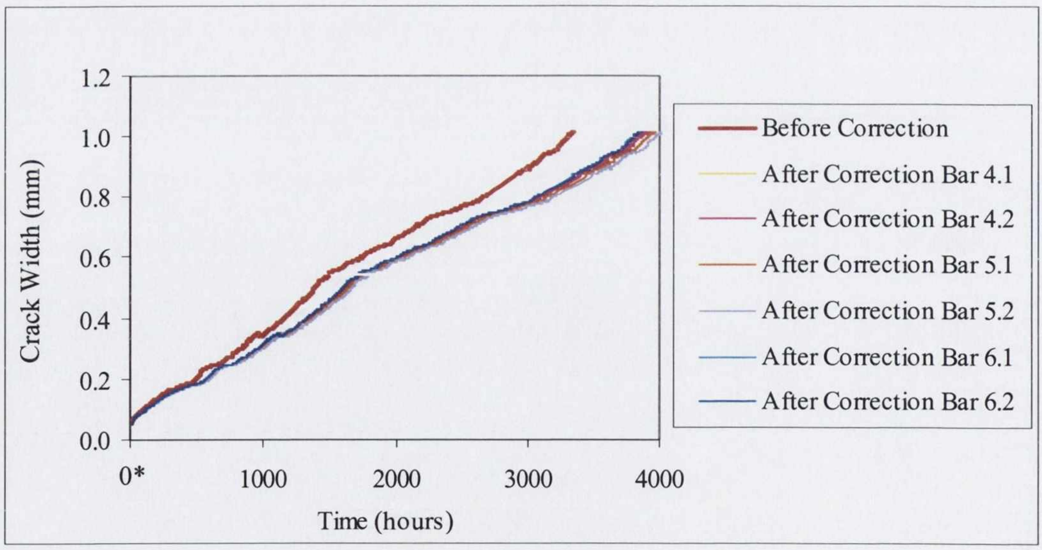


Figure 5.27. Propagation of crack growth for OPC+PFA slabs, before and after correction

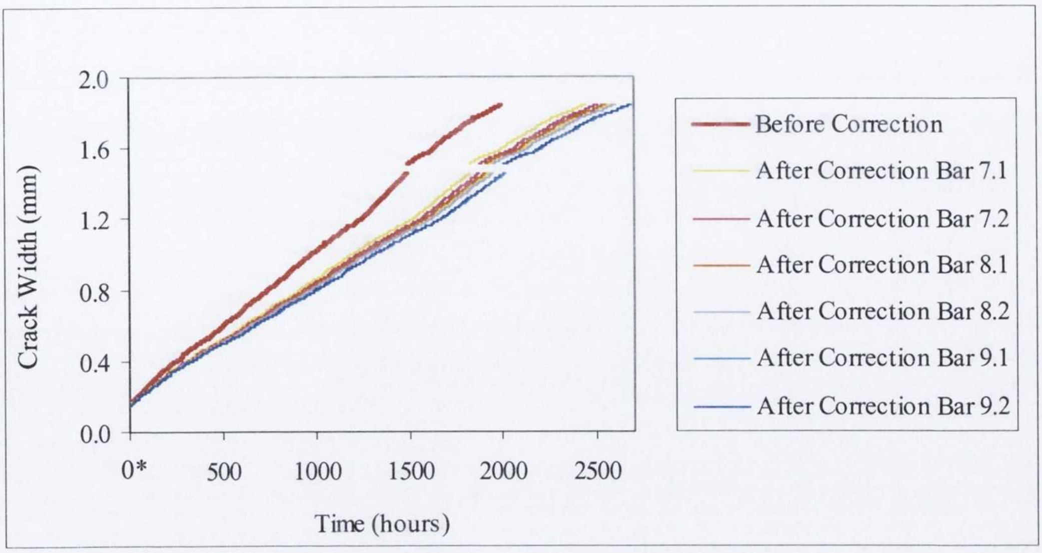


Figure 5.28. Propagation of crack growth for OPC+GGBS slabs, before and after correction

The standard deviation of the corrected time to a mean displacement increases linearly with the mean corrected time, resulting in a coefficient of variation of 1.2%, 1.6% and 3.7%, for OPC, OPC+PFA and OPC+GGBS, respectively. A comparison of the rate of crack growth for the three different mixes can be seen in Figure 5.29 for the average corrected times for crack propagation.

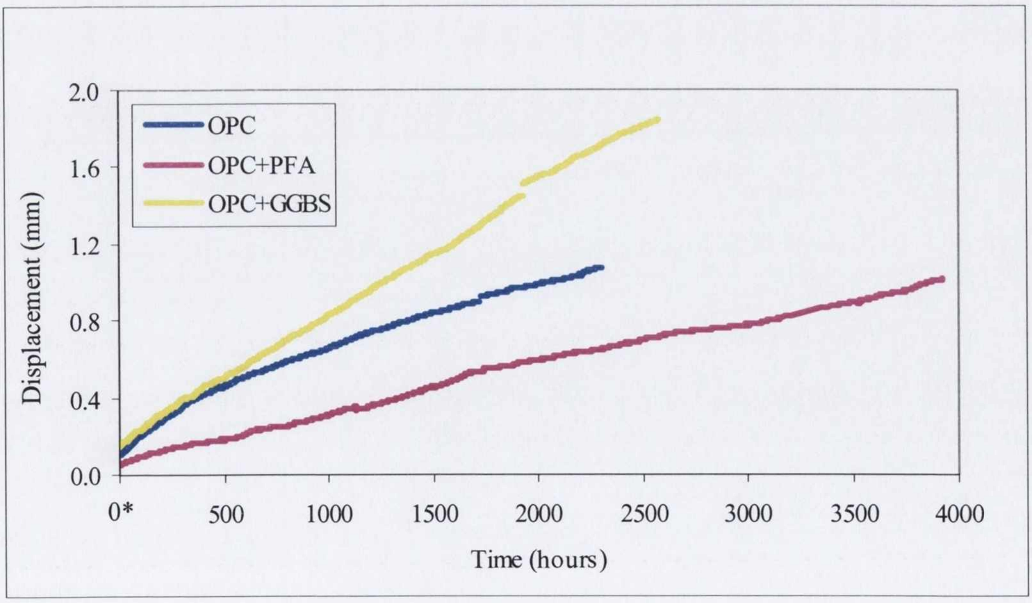


Figure 5.29. Corrected average rate of crack growth over time

The growth in the crack width of the OPC+PFA is still the slowest out of the three mixes, followed by OPC and then OPC+GGBS. However, the difference between the OPC and OPC+GGBS for the corrected rate of crack growth is reduced by 28% for an experimental time of 2000 hours.

A comparison of the behaviour of the different slabs can be made by studying Figure 5.30. For each slab the final crack width (left y-axis, purple columns) and corresponding corrected experimental time to this crack width (right y-axis, yellow columns) are illustrated. It is clear that the time to each crack width for OPC+PFA slabs is much larger than for slabs made with OPC alone and OPC+GGBS.

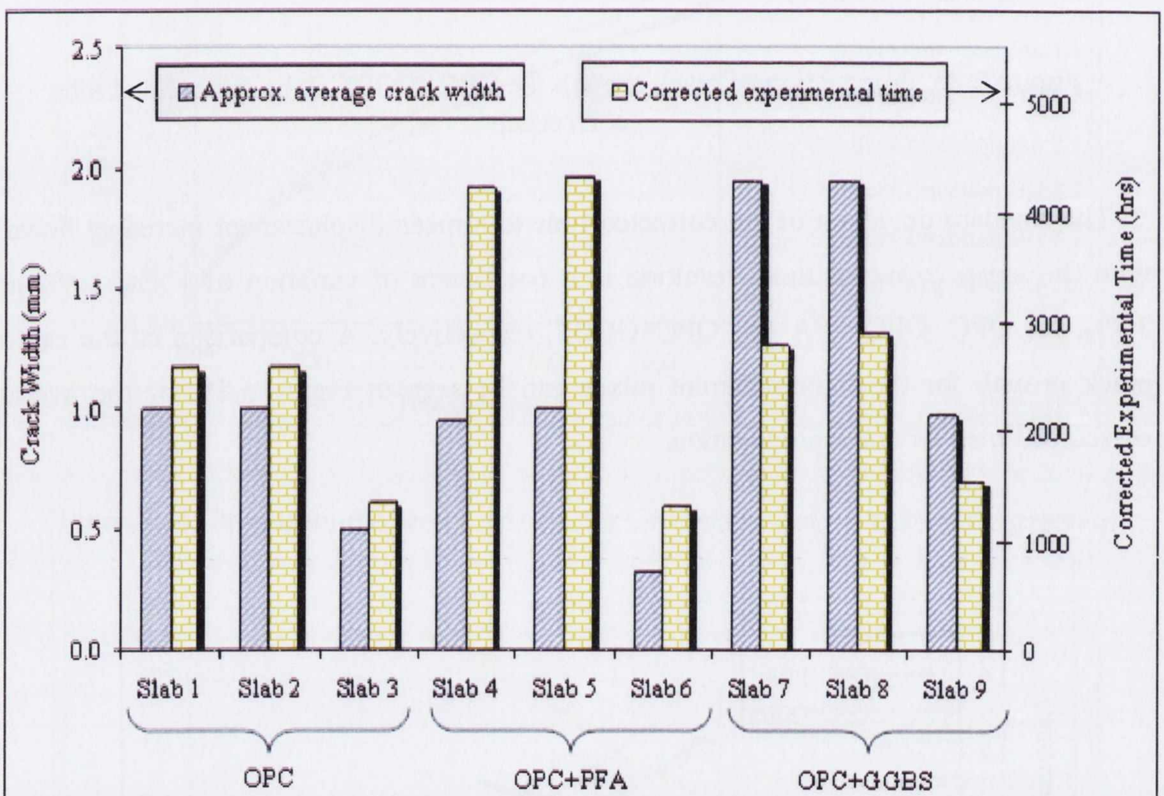


Figure 5.30. Comparison of displacement and corresponding corrected experimental time for each slab

For example, comparing two slabs made from OPC (slab 1 and 2) and two slabs made from OPC+PFA (slab 4 and 5), the average corrected experimental time to an approximate crack width of 1.0mm is 65% higher for the OPC+PFA slabs. In addition, Slabs 1 and 2 (OPC) and slabs 7 and 8 (OPC+GGBS) have similar corrected experimental times, yet the corresponding approximate crack width for the OPC+GGBS slabs (slab 7 and 8) is nearly twice that of OPC slabs (slabs 1 and 2). Similarly, comparing slab 3 and slab 6 with similar corrected experimental times, the corresponding approximate crack width for slab 6 is just 0.65 times the crack width for slab 3. By correcting the experimental times so that they all correspond to a corrosion rate of 100μA/cm^2, these direct comparisons can be made with more accuracy.

5.6. CONCLUSIONS

The results from this study indicate that concrete with the addition of PFA (30% in this case) improves the performance of the concrete in relation to cracking due to chloride-induced corrosion of the reinforcing bars. Surprisingly, the addition of GGBS (50% in this case) had the opposite effect, increasing the rate of crack growth. On this basis, objective 3 of this thesis, which was to carry out an experimental study to investigate the rate of deterioration in different repair materials (OPC, OPC+PFA and OPC+GGBS), has been achieved. By studying the results of corrosion after a fixed period of time (Section 5.5.3) it was evident that for accelerated corrosion using the impressed current method, chloride binding did not affect the experimental results of the rate of crack growth, as the percentage mass loss was similar for the reinforcing bars in each of the slabs. Due to the short period of testing, carbonation was not an issue that needed to be considered when analyzing the results, although it is recognised that this may have an impact on the results over a longer period of testing. If adequate curing is carried out, the addition of PFA or GGBS will lead to a reduced carbonation rate in relation to OPC, which will in turn lead to a reduction in the corrosion rate. However, in the case of inadequate curing, this effect may be reversed. In the case of this thesis, an increase in the corrosion rate will lead to an increase in the growth rate of the defects (i.e. crack width), resulting in a higher value for the growth rate parameter (α) when calibration is carried out (this will be discussed further in Chapter 6).

The slabs with OPC+PFA behaved better than the slabs with OPC only and OPC+GGBS in relation to crack propagation due to corrosion of the reinforcing steel. The curves for the rate of crack growth for OPC and OPC+GGBS were close initially in comparison to the OPC+PFA. It is interesting to note that the concrete cylinders for

OPC+PFA and OPC+GGBS, when tested for tensile splitting strength, indicated that the OPC+PFA slabs had only a marginally higher tensile strength than the OPC+GGBS slabs (0.1MPa, 3.2% of a difference in the mean strength), Figure 5.3. Also, the OPC slabs performed better than the OPC+GGBS slabs even though they had a lower tensile strength than the OPC+GGBS, indicating that it is not only the tensile strength of a concrete which determines cracking behaviour.

The superior performance of the OPC+PFA in relation to concrete cracking is considered to be due to its higher porosity which was discussed in Section 2.3.3. This would also explain why the rate of crack growth in the OPC was slower than the OPC+GGBS, as the porosity of OPC+GGBS is less than a corresponding OPC mix. A similar mass loss due to corrosion was observed for each of the concrete slabs in the first set of results (after about 1150 hours), yet the variation in crack width for the different concrete mixes was very significant. Similarly, for a specific crack width in the slab, the percentage mass loss of the reinforcing bars in the OPC+PFA slabs was much higher than for the other two materials. It is recognised, however, that the corrosion products did travel up through the slab and caused staining on the surface of the slab. This may be undesirable for aesthetic reasons in areas where the concrete is visible.

In relation to the incorporation of the experimental results into the developed maintenance management methodology, the higher performance of the OPC+PFA slabs will result in a lower value for the defect growth rate (α). The OPC+GGBS repair material will have the highest growth rate of the three materials considered (i.e. OPC, OPC+PFA and OPC+GGBS). However, the results of this experiment are from accelerated corrosion testing, and therefore, the results must be converted to real time before being used to develop or compare maintenance management strategies. Once this conversion has been carried out, the rate of crack growth for each of the concrete mixes can be used to determine the growth parameters (α, g) which can then be inputted into the maintenance management model which was developed as part of this thesis. The performance of these different repair materials has been considered in the literature in relation to the initiation phase of chloride-induced corrosion, yet the propagation phase or rate of defect growth has not previously been considered. Therefore, the experimental study carried out as part of this thesis allows the performance of OPC, OPC+PFA and OPC+GGBS concretes to be compared considering the two phases of deterioration, initiation and propagation. It is recognised that further experimental work is required to understand the underlying trends identified in this chapter.

Using information from the literature (i.e. initiation time for each of the mixes) and the results from the experimental study carried out as part of this thesis, the performance of the different repair materials will be directly compared in relation to the expected total costs of inspection, maintenance and repair and the optimum inspection interval, considering both the initiation phase and propagation phase of deterioration. This comparison will be carried out and discussed further in Chapter 6.

CHAPTER 6 – FURTHER MODEL DEVELOPMENT AND PRACTICAL EXAMPLE

CHAPTER 6 - FURTHER MODEL DEVELOPMENT AND PRACTICAL EXAMPLE

6.1. INTRODUCTION

Following on from the experimental results, which have provided information on the relative performance of different repair materials in relation to the rate of crack growth and have facilitated calibration of the growth parameters (α and g), the maintenance management model was further developed to allow the user to input material parameters for a repair material as well as for the original material with which the structure was constructed. This allows the owner/manager of a structure to compare the effectiveness and efficiency of different repair materials (e.g. OPC, OPC+PFA and OPC+GGBS), to select the most appropriate repair material for the repair of defects in the structure being considered (e.g. within particular budgetary constraints) and to determine the optimum combination of inspection techniques to monitor the performance of the repairs over the remaining lifetime of the structure. Also, to enable this model to be used on a wider range of structures a two step deterioration process has now been incorporated, comprising of an initiation phase and a propagation phase.

This form of deterioration (i.e. an initiation and propagation phase) is most commonly associated with corrosion of reinforced concrete, but is also appropriate for modelling the deterioration of other structural materials, such as coated steel. In relation to reinforced concrete, during the initiation phase there is no corrosion of the reinforcing bars but aggressive agents (such as chlorides and carbon dioxide) ingress into the concrete cover towards the steel. Once these aggressive agents break down the passive layer surrounding the steel, the initiation phase ends and the propagation phase of deterioration begins. During the propagation phase corrosion begins and causes cracking of the concrete cover (as discussed in Section 2.3). However, using the developed methodology the initiation phase can be bypassed and set to zero for materials that do not follow this kind of deterioration behaviour (i.e. materials with a propagation phase only).

The experimental study which was carried out as part of this thesis to investigate the rate of crack growth in different repair materials (i.e. OPC, OPC+PFA and OPC+GGBS) due to chloride-induced corrosion of reinforced concrete was described in Chapter 5. As discussed in Chapter 5, only the propagation phase of deterioration was considered as part of this study. Therefore, information on the initiation time of these different concrete mixes is estimated based on information from the literature. On this basis, an example illustrating the capabilities of the maintenance management methodology to compare the efficiency of different repair materials is carried out as part of this chapter using information from the literature (initiation phase) and the experimental results presented in Chapter 5 (propagation phase). This will allow owners/managers of structures to compare the efficiency of different repair materials in terms of both the initiation phase and the propagation phase of deterioration, to determine the most effective repair strategy and subsequent inspection strategy for the structure or group of structures being considered.

6.2. MODELLING REPAIR EFFICIENCY

As described in Section 3.4.1, in this thesis the growth of a defect is described by two parameters. One is α, which describes the growth rate of a defect and therefore controls how quickly a defect moves from one defect group to the next. The other parameter is g, which determines how gradual or sudden the growth of an individual defect is. This parameter controls whether defects develop gradually and just move from one defect group to the next, or whether the growth of a defect is more abrupt. In Section 3.4.1, three forms of growth kinetics were considered, gradual growth (g=5, Figure 3.7), abrupt growth (g=0, Figure 3.10) and very abrupt growth (g=-5, Figure 3.12).

In this chapter, the maintenance management model is further developed to simulate the repair of defects using a different material to the original material that the structure was constructed with. This allows the owner/manager to consider the deterioration of the original structure and select an appropriate repair material for defects by comparing the efficiency of different repair strategies in relation to the expected cost and optimal inspection interval. When the first defect develops in a structure, it does so according to the growth parameters assigned to the original material. As the defect grows inspections are carried out which may lead to repair of the defect. Alternatively, failure can occur, and in this case a repair must also be carried out.

When repair is carried out a new more durable material may be used and any defect that develops in the new material will develop according to the deterioration parameters of the new material. For example, the repair material may have a longer initiation period and a slower rate of crack propagation, as illustrated in Figure 6.1. Therefore, the model can be used to study the deterioration of the original material which was used to build the structure, to study the efficiency of the chosen repair material, where the frequency of inspection and repair is a function of the repair material, and also to determine the optimum combination of inspection techniques for each repair material. All repairs are assumed to be carried out using the same material over the lifetime of the structure.

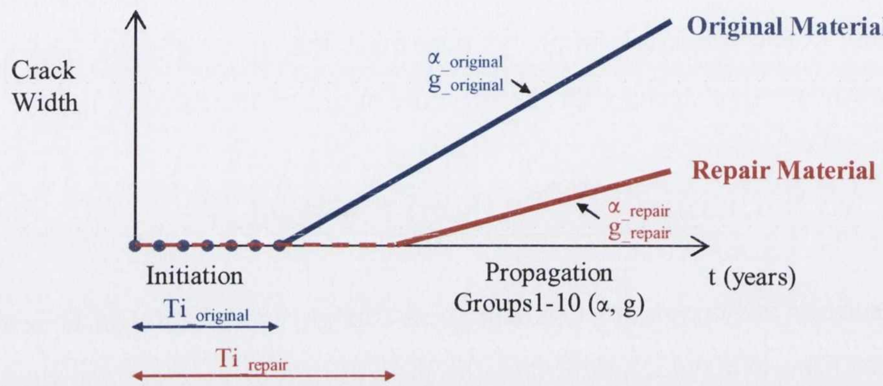


Figure 6.1. Illustration of deterioration behaviour of original material and repair material

Therefore, in the revised model, as a defect is repaired it acquires the new properties of the repair material (i.e. initiation time, α and g) and a record is kept of the number of defects in each material over time. In relation to the propagation phase, a different Markov matrix is developed for each material (using α and g) and is used to model the growth, inspection, repair and failure of the population of defects over time. Figure 6.2 and Figure 6.3 present Markov matrices which could be used to model the behaviour of an original and repair material, respectively. For example, the original material may have a high growth rate of about α=0.8 and moderate/gradual growth kinetics of about g=3, whereas the repair material could have a moderate growth rate of about α=0.5 and gradual growth kinetics of about g=5. The first row in each matrix is highlighted to illustrate the relative difference in the entries of the Markov matrix based on the specific growth parameters (i.e. α and g) of each material.

As discussed in Section 3.4.1, this affects the development of the Markov matrix with the growth rate controlling the diagonal entries and the growth kinetics controlling the other entries in the upper triangular matrix. As defects in the original material are repaired, or fail, the defects are removed from the relevant group in the original material and added to the first group (or initiation phase) of the repair material. The method used to model the initiation phase will be discussed in Section 6.3. The model is run until stabilisation of the number of defects in each group has occurred for each material. The total number of repairs can be calculated from the stabilised number of defects in each group.

$\alpha\_{original} = 0.8$ $g\_{original} = 3$

	To *j*										
		1	2	3	4	5	6	7	8	9	10
From *i*	1	**0.20**	0.67	0.08	0.02	0.01	0.01	0.00	0.00	0.00	0.00
	2		**0.20**	0.67	0.08	0.02	0.01	0.01	0.00	0.00	0.00
	3			**0.20**	0.67	0.08	0.02	0.01	0.01	0.00	0.00
	4				**0.20**	0.67	0.08	0.02	0.01	0.01	0.00
	5					**0.20**	0.67	0.08	0.02	0.01	0.01
	6						**0.20**	0.68	0.08	0.03	0.01
	7							**0.20**	0.69	0.09	0.03
	8								**0.20**	0.71	0.09
	9									**0.20**	0.80
	10										**1.00**

Figure 6.2. Example of a Markov matrix for an original material (with α=0.8, g=3)

$\alpha\_{repair} = 0.5$ $g\_{repair} = 5$

	To *j*										
		1	2	3	4	5	6	7	8	9	10
From *i*	1	**0.50**	0.48	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2		**0.50**	0.48	0.02	0.00	0.00	0.00	0.00	0.00	0.00
	3			**0.50**	0.48	0.02	0.00	0.00	0.00	0.00	0.00
	4				**0.50**	0.48	0.02	0.00	0.00	0.00	0.00
	5					**0.50**	0.48	0.02	0.00	0.00	0.00
	6						**0.50**	0.48	0.02	0.00	0.00
	7							**0.50**	0.48	0.02	0.00
	8								**0.50**	0.48	0.02
	9									**0.50**	0.50
	10										**1.00**

Figure 6.3. Example of a Markov matrix for a repair material (with α=0.5, g=5)

Using this methodology, different repair materials can be compared by analysing the relative cost and number of failures over the lifetime of a structure. Therefore, the maintenance strategy can be optimised not only as a function of the defect deterioration mechanism of the original material, but also with respect to the deterioration behaviour of the chosen repair material. In addition, the model has been improved to allow different growth parameters (i.e. α and g) to be inputted for each defect group, facilitating the modelling of growth rates that vary with defect size. Figure 6.4 illustrates the capability of the improved model in relation to inputting different curves which can simulate the variation in growth rate over time. In the original model, a growth rate was specified for the material and was assumed to be constant for each group, resulting in a linear growth rate of defects over time, as illustrated in Figure 6.4(a). This may not always be accurate, as in some cases, the growth rate can change as the deterioration progresses. For example, considering chloride-induced corrosion of reinforced concrete, as corrosion products build up on the surface of the steel, this can obstruct the ingress of chloride ions and oxygen into the surface of the un-corroded steel, which can lead to a reduction in the corrosion rate. In this case, the rate of crack growth would also reduce over time, as illustrated in Figure 6.4(b). Although in this example it is assumed that the corrosion rate of the defect decreases over time, it is recognised that the corrosion rate can increase as well as decrease, which can also be accommodated within the developed methodology. Therefore, by allowing the user to input different growth parameters for each defect group, many different forms of deterioration behaviour can be simulated using the Markov matrix. A linear growth rate can still be used by inputting the same growth rate for each group. By fitting the deterioration parameters (i.e. α and g) to actual curves which describe the rate of crack width over time (which can be obtained from experimental studies or on-site tests), the deterioration of different materials can be approximated and inputted into the maintenance management model to investigate the efficiency of different repair materials. This procedure will be discussed further in Section 6.4.2.

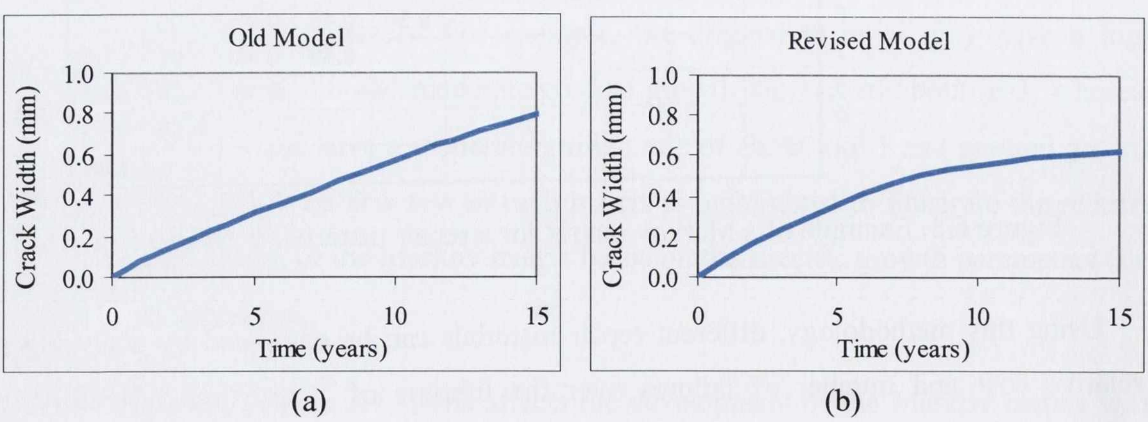


Figure 6.4. Illustration of the capabilities of the old model and revised model in relation to inputting deterioration information

6.3. MODELLING INITIATION PHASE

For some materials such as reinforced concrete there is an initiation phase of deterioration where aggressive agents such as chlorides diffuse into the concrete cover. No cracks develop until the propagation phase begins and corrosion of the reinforcing steel takes place. The initiation phase and propagation phase of deterioration were illustrated in Figure 6.1 in relation to the rate of crack growth over time. It is therefore important to model this initiation time when investigating the repair efficiency of different materials. This can affect the frequency of inspections, repairs and failures, and hence the expected cost over the remaining lifetime of the structure.

The length of this initiation period for different concrete mixes will be discussed further in Section 6.4.1. An initiation time of zero years can also be inputted into the model when considering a material where there is no initiation phase. The maintenance management model has been further developed to take this two step deterioration process into account. This process is illustrated in Figure 6.5.

The initiation time (a deterministic parameter which is specified in years) can be determined based on the properties of the material, for the original material that the structure was constructed with and the new material that the defects are patch repaired with. In a new structure, all defect locations (or hotspot locations) begin in the initiation phase of the original material, Figure 6.5. After time, defects grow and inspections are carried out. When repair or failure occurs, the defects are repaired with some material (which can be a new material or the same as the original material).

When a patch repair is carried out all the old concrete cover is broken out and replaced with a new concrete repair material. It is assumed here that the aggressive agents such as chloride ions and carbon dioxide are completely removed, and therefore, a new initiation phase begins while these aggressive agents again ingress into the new repair concrete. In this case, the initiation time is dictated by the properties of the repair material. Therefore, when a sizing assessment indicates that the defect size is greater than the critical defect size and repairs are carried out, defects move from the defect group they were in to the beginning of the initiation phase of the new material. This process is illustrated in Figure 6.5.

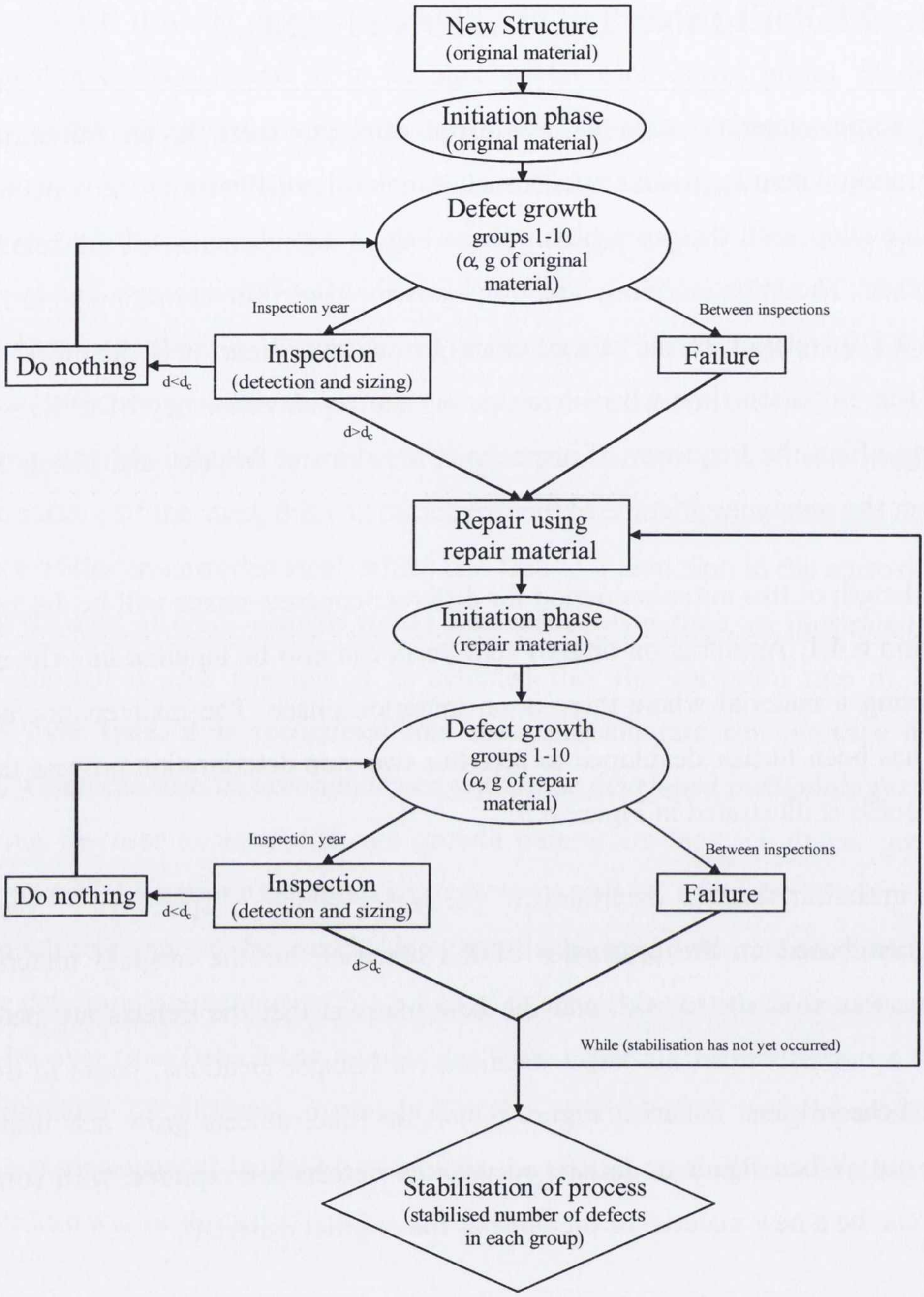


Figure 6.5. Illustration of inspection/repair/failure process of defects when repaired with a new material

To model this behaviour, new matrices have been developed to keep a record of the number of defects in each group in each of the materials over time. As described in Section 3.4.4 and Figure 3.22, the developed model assumes that in any year of simulation the inspection, repair and failure occur at the end of the year in question (i.e. it is assumed that defects may grow throughout the year before inspections, repair or failure occur). Five new matrices were initiated to keep track of the number of defects in each group, and one matrix was initiated to keep track of the number defects in each year of the initiation phase.

- *NoDefects_EndYr_BeforeFailure* – Records the number of defects in each group in each material after the defects have grown throughout the year, before the simulation of failure. This matrix has entries in the years between inspections.

- *NoDefects_EndYr_AfterFailure* – Records the number of defects in each group in each material after the defects have grown throughout the year and after failure has taken place. Therefore the failed defects are taken from the relevant groups and added to the initiation phase of the new material, for the specified initiation time. This matrix has entries in the years between inspections.

- *NoDefects_EndYr_BeforeRepairFailure* – Records the number of defects in each group in each material after the defects have grown throughout the year, before the simulation of repair and failure. This matrix has entries only for inspection years.

- *NoDefects_EndYr_AfterRepairFailure* – Records the number of defects in each group in each material after the defects have grown throughout the year and after repair and failure have taken place. Therefore the repaired/failed defects are taken from the relevant groups and added to the initiation phase of the new material, for the specified initiation time. This matrix has entries only for inspection years.

- *NoDefectsBeginYr* – Records the number of defects in each group in each material at the beginning of each year. This is therefore the number of defects in each group after repair and failure has taken place, but also after defects have been added into the first group of the repair material once the initiation time after a repair or failure has passed. This matrix has entries each year of the simulation.

- *InitiationPhase* – Records the number of defects that are added to the initiation phase of each material over time. These defects are then added to the first group of the repair material once the initiation time has passed.

Figure 6.6 illustrates the procedure for simulating the growth and failure of defects for a year of simulation (Y) between inspections (i.e. $Y \neq n\Delta T$, where n is a positive integer). In this case (between inspections), only 4 of the 6 matrices that were defined are used (i.e. *NoDefects_EndYr_BeforeFailure*, *NoDefects_EndYr_AfterFailure*, *NoDefectsBeginYr* and *InitiationPhase*). For a year of simulation where an inspection takes place (i.e. Y = nΔT), both repairs and failures must be considered and the number of failures is replaced by the number of repairs/failures. In this case the matrices *NoDefects_EndYr_BeforeFailure* and *NoDefects_EndYr_AfterFailure* are replaced with *NoDefects_EndYr_BeforeRepairFailure* and *NoDefects_EndYr_AfterRepairFailure*.

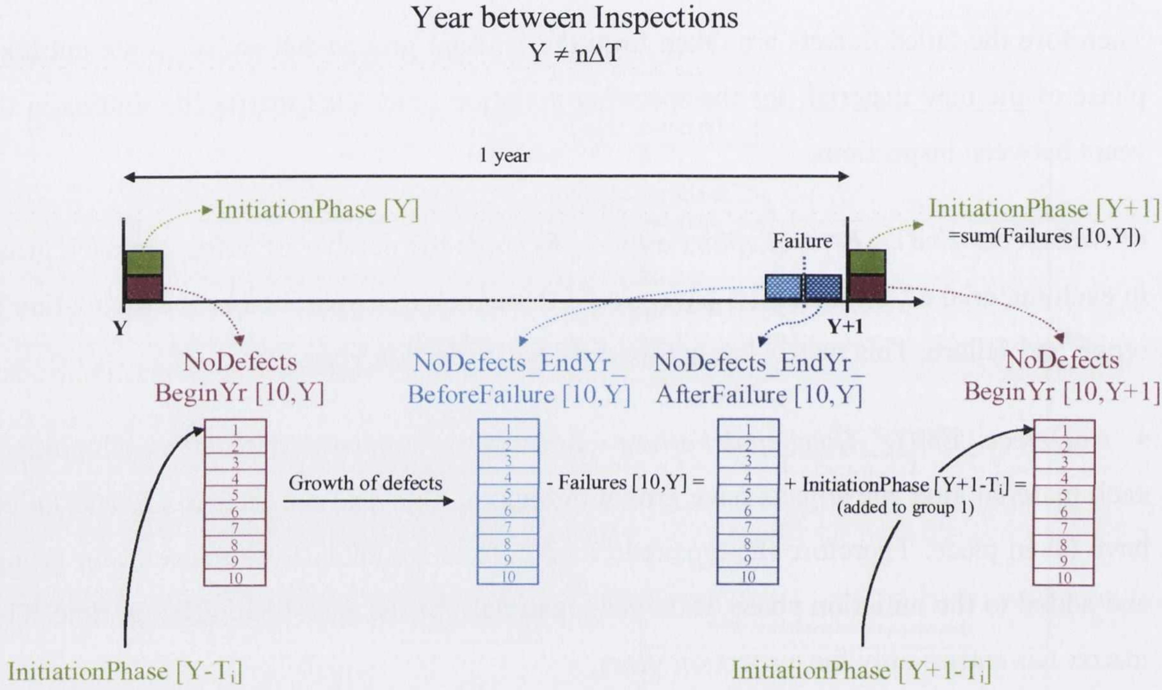


Figure 6.6. Schematic illustration of the simulation of the initiation phase, growth and failure of defects between inspections

A 'while' loop is set up to calculate the stabilised number of defects in each group (as illustrated in Figure 6.5) using the relevant Markov matrix for each year (inspection year or between inspections) and for each material (original or repair material). Using the counter of the loop a record is kept of how many years have passed since repair or failure, and after the initiation period (Ti) the defects are added to the first group (0-0.1mm) of the repair material. The initiation time of the original material just acts as a delay in the running of the program. Until the initiation time of the original material has passed nothing can happen, as there are no defects to grow, be inspected and repaired, or fail. Once the initiation phase (a deterministic parameter which is specified in years) of the original material has passed, defects in the original material gradually deplete, as they are repaired with a new material. By the time stabilisation has occurred, all defects are in the new repair material. The results are therefore not sensitive to the properties of the original material. The stabilised number of defects in each group of the repair material is used to calculate the expected mean annual total cost and expected mean annual number of failures for a population of defects. This can be used to compare the relative benefits of different repair options, taking into account the initiation time of the material as well as the rate of deterioration once the propagation phase has begun.

6.4. EXAMPLE – REINFORCED CONCRETE

The capabilities of the revised maintenance model were investigated using a reinforced concrete example comparing the efficiency of different repair materials. The type of deterioration considered was chloride-induced corrosion of the reinforcing bars, as studied in the experiment outlined in Chapter 5. As discussed in Section 6.2, the deterioration of reinforced concrete due to chloride-induced corrosion is a two step process consisting of an initiation phase and a propagation phase.

Since the objective of the experiment carried out was only to investigate the propagation phase (or rate of crack growth) in reinforced concrete, data concerning the behaviour of the initiation phase of different repair materials has been taken from the literature. Although information in the literature comparing the initiation time of concretes using OPC and blended cements (i.e. OPC+PFA and OPC+GGBS concrete mixes) is quite limited, this available literature was used to estimate the initiation time of the 3 concrete mixes considered as part of this thesis (i.e. OPC, OPC+PFA and OPC+GGBS), and will be discussed further in Section 6.4.1. For each material, the propagation phase of deterioration was simulated using the experimental results. The calibration of the deterioration parameters (i.e. α and g) using the experimental results will be presented in Section 6.4.2. A schematic illustrating the effect of different repair materials on the propagation phase of deterioration is presented in Figure 6.7. For illustration purposes in Figure 6.7, the initiation time is assumed to be equal for each material and the propagation phase is assumed to be linear. As discussed in Section 6.2, non-linear deterioration may also be considered by varying the growth parameters for each defect group. The initiation time for each material may also vary depending on properties such as the permeability of the material and the depth of concrete cover to the reinforcing bars.

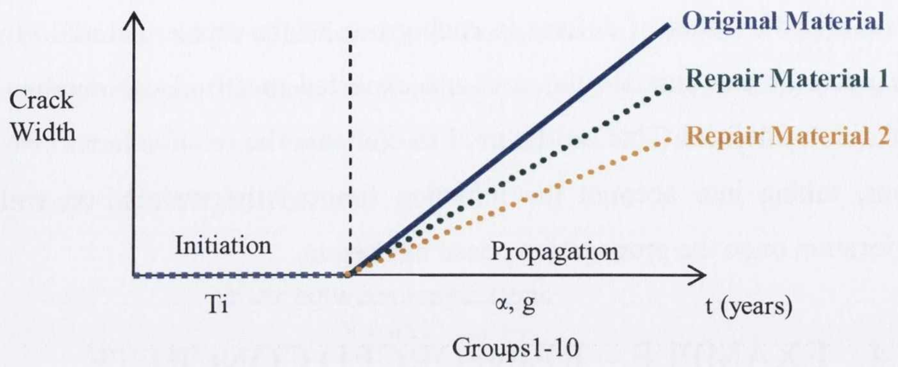


Figure 6.7. Two phase deterioration process

Figure 6.7 presents the growth rate of defects for three different materials, the original material with which the structure was constructed, and two alternative repair materials with a different rate of deterioration. Therefore, once the initiation phase has ended the growth of the cracks in the different materials progresses at different rates. In the experimental study carried out as part of this thesis, the cracks in the OPC+PFA concrete mix progressed at the slowest rate, followed by OPC and OPC+GGBS.

Therefore, in relation to the propagation phase, OPC+PFA is considered to be the most effective repair material of the three mixes considered as part of this study (i.e. OPC, OPC+PFA and OPC+GGBS). However, the initiation phase must also be considered. On this basis, the effect of the initiation phase and the propagation phase on the optimal maintenance strategy and expected total cost will be studied in this example, considering OPC, OPC+PFA and OPC+GGBS concrete mixes. This allows owners/managers of structures to compare the relative benefits and shortcomings of different repair materials, and choose the most suitable repair option for a particular structure or group of structures within possible requirements/constraints.

6.4.1. Initiation Phase – From Literature

With the deterioration of concrete due to corrosion of the reinforcing bars, the initiation phase takes place while the aggressive agents such as chlorides and carbon dioxide diffuse into the concrete cover towards the reinforcing bars (Bertolini et al., 2004; Li, 2003). The initiation phase ends once the steel has been depassivated (Bertolini et al., 2004). For this example, it is assumed that patch repairs of defects are carried out. According to Tilly (2007), patch repairs were chosen in about 60 percent of case histories where repairs needed to be carried out. Although it is recognised that other repairs may also be performed, such as coatings, crack injection, cathodic protection and addition of corrosion inhibitors (Tilly, 2007).

In the case of a patch repair, the aim is to remove the deteriorated concrete, clean and/or replace corroded steel and cover with a new concrete (Tilly, 2007). The new repair concrete material can have a lower permeability than the original material and can therefore be more durable against attack from aggressive agents such as chloride and carbon dioxide. It has been stated in the literature that concretes containing PFA and GGBS have a lower permeability than the equivalent concrete with only an OPC binder (McPolin et al., 2005; Neville, 2005; Bertolini et al., 2004; McCarthy et al., 2001; Hussain and Rasheeduzzafar, 1994; Dhir and Byars, 1993; Thomas, 1991).

McPolin et al. (2005) carried out a study to investigate the rate of ingress of chlorides into different materials using a wetting and drying cycle. Concrete mixes similar to those used in this thesis were investigated (i.e. 100% OPC, 30% PFA and 50% GGBS), also with a water/binder ratio of 0.5. The results indicated that the rate of chloride ingress into concrete with 30% PFA was about 2.1 times slower than the OPC concrete. Also, the rate of chloride ingress into concrete with 50% GGBS was about 2.9 times slower than the OPC concrete.

In relation to PFA, Hussain and Rasheeduzzafar (1994) found similar results. They carried out accelerated corrosion tests and obtained results indicating that the addition of 30% PFA to the concrete increased the initiation time by between 1.9 and 2.5 times (depending on the C_3A content in the cement). In a study carried out by Dhir et al. (1994) slightly lower results were found for the ratio of initiation time for OPC and OPC+PFA concrete using accelerated corrosion tests. For equivalent concretes, the addition of PFA led to an increase in corrosion initiation time of between 1.5 and 2.0 times (Dhir et al., 1994).

One of the mixes studied by Dhir and Byars (1993) was a 30% PFA concrete mix. Figure 6.8 illustrates the results obtained for the predicted time to critical chloride content for an OPC mix and an OPC+PFA (30% PFA) mix for different concrete strengths in a marine environment (with a 50mm cover, and assuming a critical chloride content of 0.4%). Therefore, by interpolating this data (using linear interpolation), the time to a critical chloride content of 0.4% is estimated to be 16 years and 43 years for an OPC mix and an OPC+PFA mix (30% PFA), respectively, with a design strength of 42MPa (and a 28 day curing period).

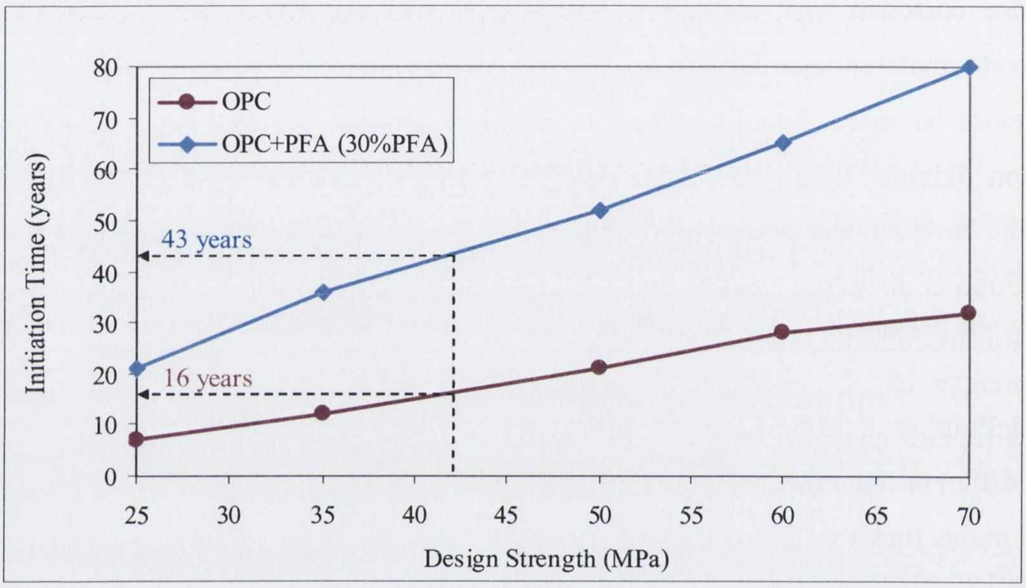


Figure 6.8. Predicted time to critical chloride content for a curing time of 28 days (Dhir and Byars, 1993)

Based on this limited information from the literature above, an estimation of the relative initiation times of the different repair mixes was made. The ratio of the initiation time of the OPC+PFA mix and OPC+GGBS mix to the initiation time of the OPC mix was chosen as 2.0 and 2.8, respectively, based on the ratio of initiation times and diffusion coefficients presented in the literature (McPolin et al., 2005; Dhir et al., 1994; Hussain and Rasheeduzzafar, 1994). It was chosen to simulate a moderately aggressive environment. On this basis, values of initiation time were estimated for each repair material for a concrete cover of 25mm (as in experiment described in Chapter 5) based on the chosen ratios and an estimated initiation time of 10 years for the OPC mix. These values are presented in Table 6.1.

The chosen initiation time for the OPC concrete was based on results from Dhir and Byars (1993) (discussed above) and Mele (2006), where the estimated initiation time is 15 years in OPC concrete with a 30mm cover and a critical chloride concentration of 0.5%. The estimated value for initiation times for OPC and OPC+PFA are less than the interpolated values based on information from Dhir and Byars (1993) because the concrete cover is only 25mm in the case considered as part of this thesis, whereas, the cover is 50mm in Dhir and Byars (1993). An alternative concrete cover for a particular environment may be taken from other sources in the literature (e.g. BS EN 206-1:2006) depending on properties such as the concrete grade, w/c ratio, cement content and cement type, but the relative trend in the initiation times for the three concrete mixes is expected to be consistent. The sensitivity of the results to the estimated value of the initiation time for OPC will be studied in Section 6.4.3.2.

Concrete Mix	Estimated Initiation Time (years)	Ratio to OPC
OPC	10	1.0
OPC+PFA	20	2.0
OPC+GGBS	28	2.8

Table 6.1. Estimated time to corrosion initiation for different repair mixes

6.4.2. Propagation Phase – Results from Experiment

The results from the experiment cannot be directly inputted into the Markov maintenance model as they are from an accelerated corrosion test. These results need to be modified using a conversion factor which relates the accelerated corrosion test to real life structures using the relative corrosion rates. The results from the experiment, for a time to a particular crack width, are therefore converted into an equivalent real time using the assumed corrosion rate for a structure in a particular environment.

For the purpose of this example, a corrosion rate of $3\mu A/cm^2$ was chosen for the real structure, which is considered a moderate corrosion rate (Bertolini et al., 2004). Using the model presented by Vu (2003), Equation 6.1, the time to a particular crack width in a real structure was estimated using this assumed corrosion rate.

$$t_{(real)} = \left[0.93\left(\frac{i_{corr\,(exp)}}{i_{corr\,(real)}}\right)^{1.1}\right] t_{(exp)} \qquad \text{Equation 6.1}$$

The values for $i_{corr(exp)}$ were calculated in the previous chapter using the mass loss from the reinforcing bars after corrosion and were presented in Table 5.5. These values for $i_{corr(exp)}$ correspond to the experimental time to a particular crack width, $t_{(exp)}$. For each bar there was a separate value for $i_{corr(exp)}$ and a separate set of values for $t_{(exp)}$. This resulted in a $t_{(real)}$ value for each bar for each recorded crack width. These results were then averaged for each material to find the mean values for $t_{(real)}$. This model (Vu, 2003) was developed using experimental data from a different source (Saifullah and Clark, 1994) where the maximum ratio of $i_{corr(exp)}/i_{corr(real)}$ was 50.0. Therefore, since the maximum experimental corrosion rate for this study is 135μA/cm^2 and the assumed corrosion rate for a real structure is 3μA/cm^2, the ratio of $i_{corr(exp)}/i_{corr(real)}$ being used in this study (maximum of 45.0) is within the limits of the model. Using this conversion and the experimental results, the curves illustrating crack growth over time were developed for all repair materials, Figure 6.9.

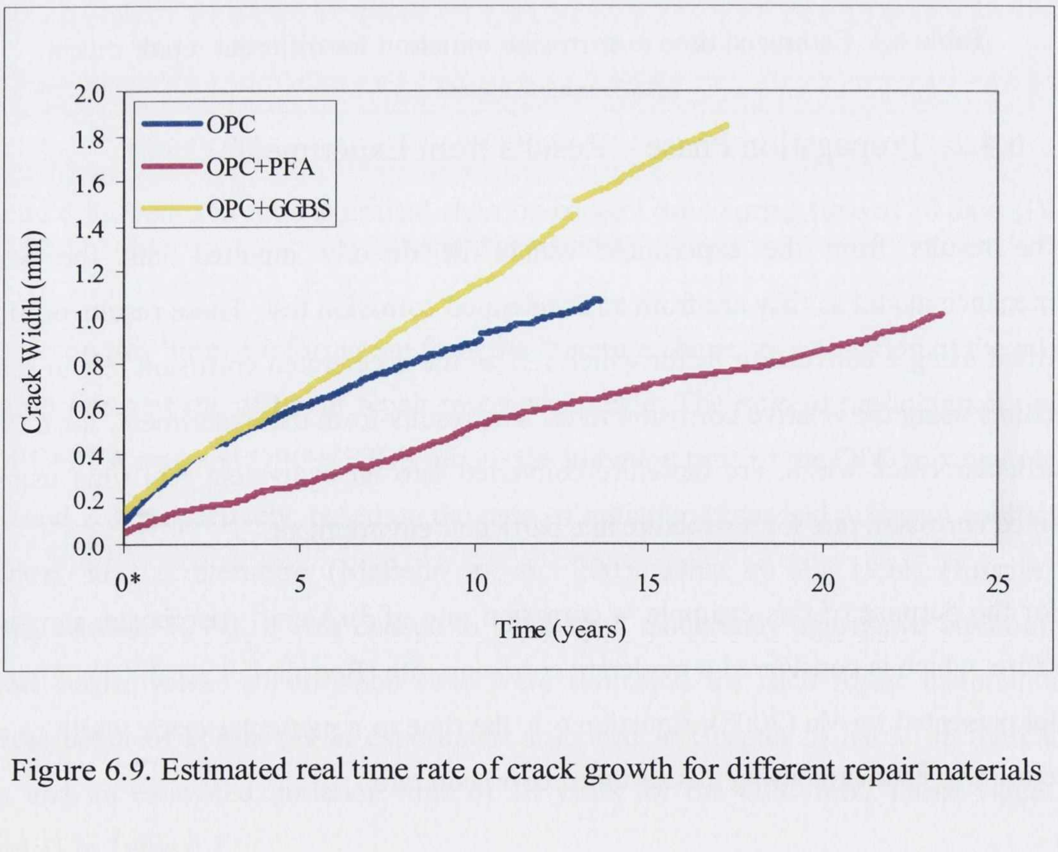


Figure 6.9. Estimated real time rate of crack growth for different repair materials

In addition, Figure 6.10 illustrates the variation in the mean and standard deviation of the real time ($t_{(real)}$) to a mean crack width of 0.2, 0.4, 0.6 0.8 and 1.0mm, for each of the concrete mixes. Considering Equation 6.1 for each reinforcing bar the term in the square brackets is constant resulting in a linear relationship between $t_{(real)}$ and $t_{(exp)}$. For each bar, as the experimental time to a crack width ($t_{(exp)}$) increases the real time to a crack width ($t_{(real)}$) increases linearly. Therefore, as the time increases the scatter in the results from the different reinforcing bars also increases linearly. This results in a linear increase in the standard deviation with the mean real time to a mean crack width, and a constant coefficient of variation of 2.6%, 3.5% and 7.7% for OPC, OPC+PFA, and OPC+GGBS, respectively. The standard deviation of the crack width data for a particular experimental time was discussed in Section 5.5.1, for each of the concrete mixes considered.

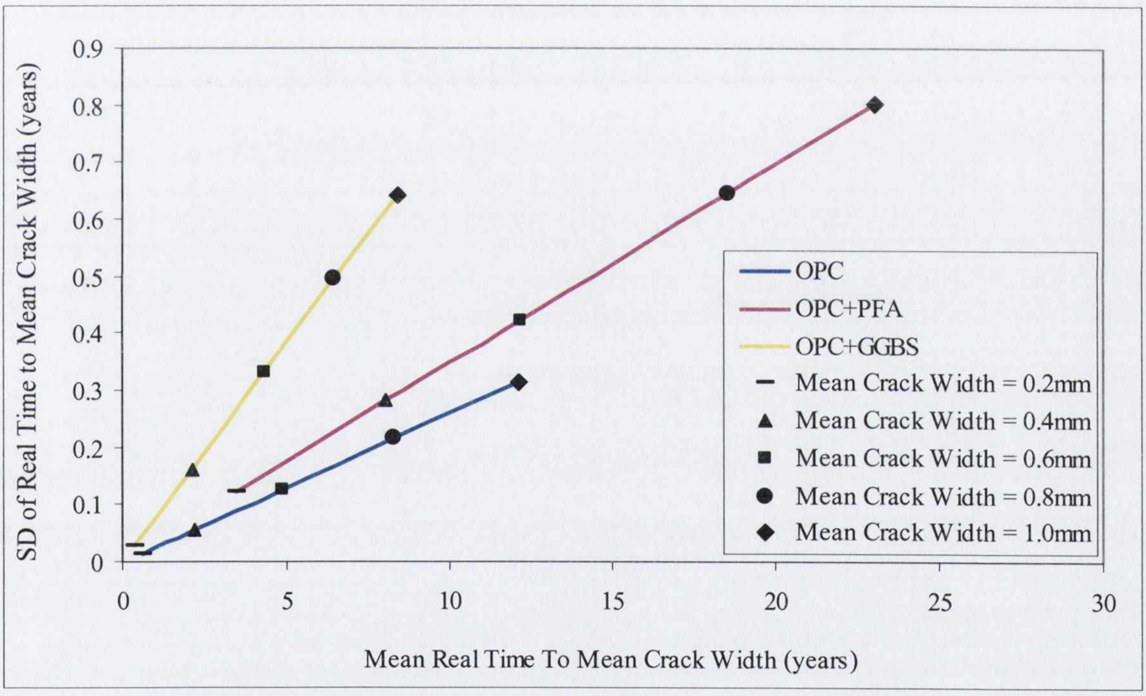


Figure 6.10. Variation in mean and standard deviation of time to a particular displacement for each of the mixes (i.e. OPC, OPC+PFA and OPC+GGBS)

Using the curves of the rate of crack growth for each of the mixes (Figure 6.9), values for α and g were determined to best fit the experimental data. As discussed in Section 6.2, the ability to input different growth parameters for each defect group allows non-linear deterioration curves, like those obtained from the experimental study, to be inputted into the maintenance management model. The simulated growth was estimated for the defects up to a crack width of 1.0mm (as the range of defect sizes being considered for the purpose of this example is 0-1.0mm). These growth parameters are listed in Table 6.2 for each repair material. The α value for the 10th group must be zero. There are no larger defect groups defined in this example, so defects cannot grow any larger once they reach group 10.

	OPC	OPC+PFA	OPC+GGBS
g	3.0	3.0	3.0
α:			
group 1	1	0.4	0.85
group 2	1	0.2	0.9
group 3	1	0.3	0.9
group 4	0.75	0.5	0.8
group 5	0.6	0.6	0.6
group 6	0.55	0.6	0.9
group 7	0.55	0.4	1
group 8	0.6	0.4	1
group 9	0.6	0.4	1

Table 6.2. Growth parameters for different repair materials which were fitted to experimental results

A value of 3.0 was chosen for the g parameter to represent the growth kinetics of the crack growth in reinforced concrete. The growth was gradual, yet the growth of the cracks was not totally smooth. Some discontinuities were apparent on the experimental data for crack growth on the individual curves for each POT, as illustrated in Figure 6.11, so g=3.0 is considered an appropriate value.

The converted experimental rate of crack growth for a real structure (blue) and the simulated crack growth using the input growth parameters (α and g, from Table 6.2) for the maintenance management model (red) are illustrated in Figure 6.12 - Figure 6.14. Since a different value of α can be inputted for each defect group (as discussed in Section 6.2), a very good approximation of the experimental crack growth data can be achieved. The figures presented illustrate that the simulated curves are very close to the experimental data (i.e. with R^2 values of 0.998, 0.995 and 0.999 for OPC, OPC+PFA and OPC+GGBS, respectively).

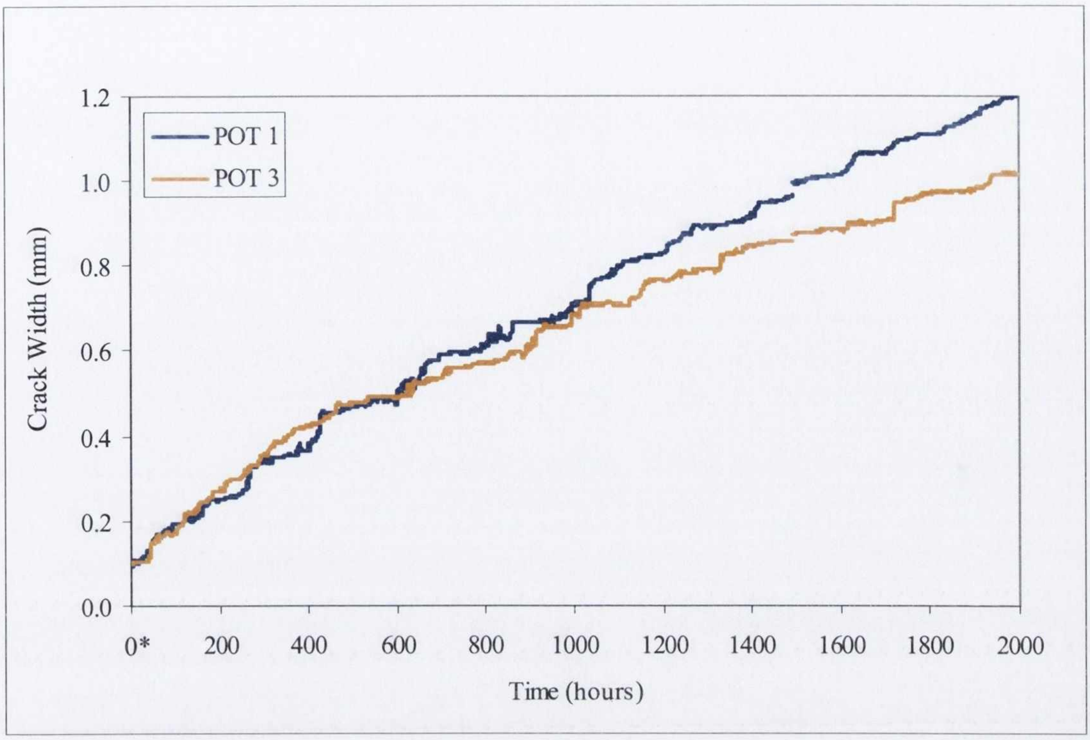


Figure 6.11. Example of growth kinetics of experimental results for POT 1 and POT 3

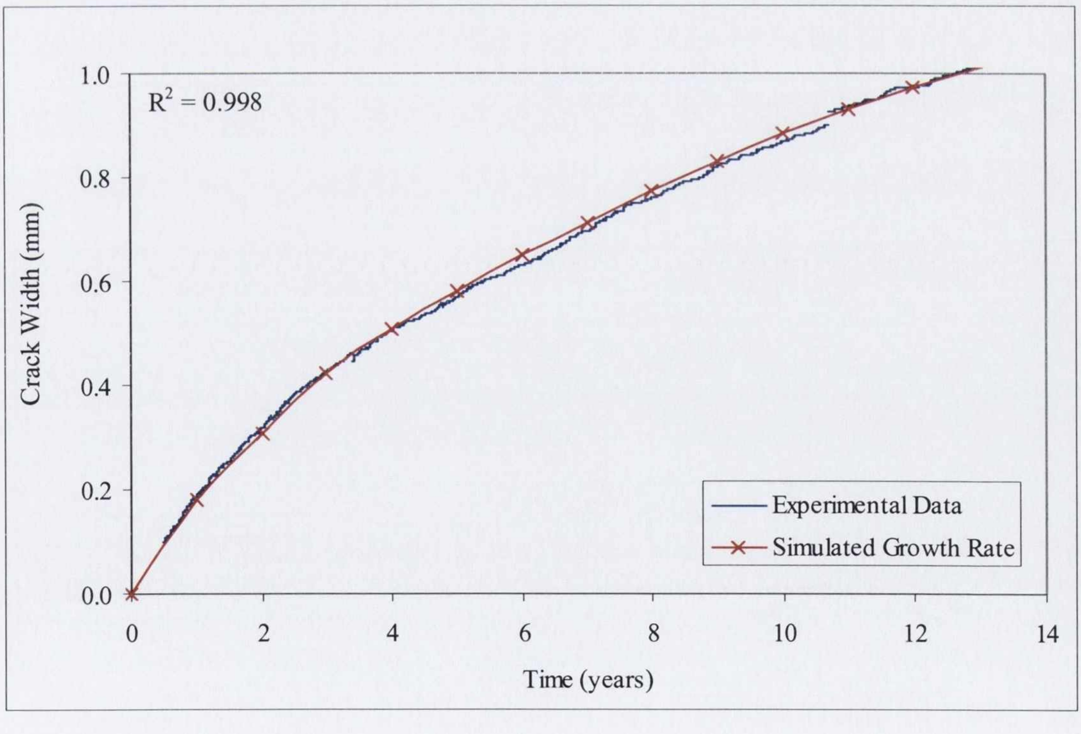


Figure 6.12. Corrected experimental data and simulated growth rate for crack growth in OPC

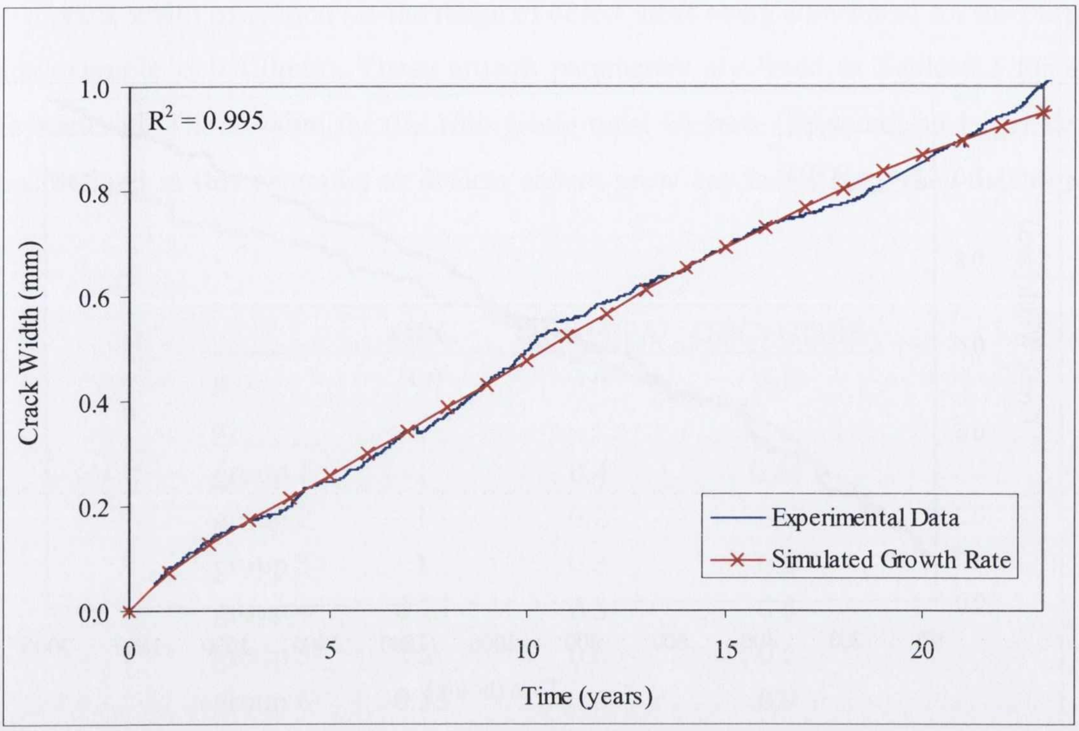


Figure 6.13. Corrected experimental data and simulated growth rate for crack growth in OPC+PFA

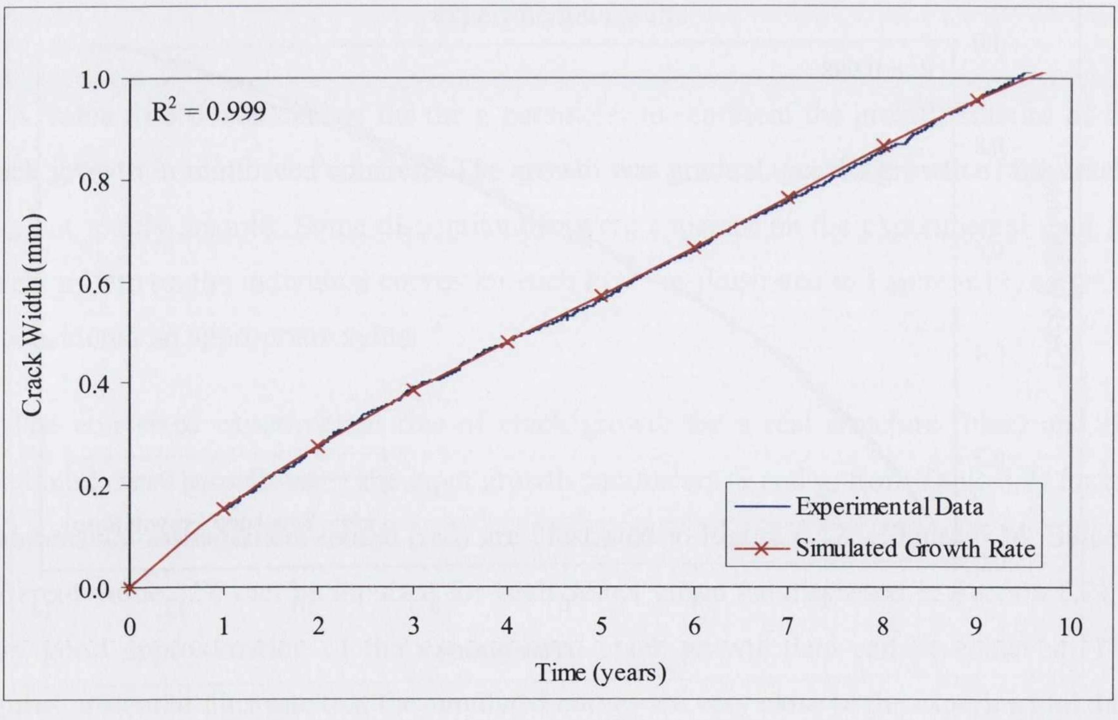


Figure 6.14. Corrected experimental data and simulated growth rate for crack growth in OPC+GGBS

6.4.3. Results from Markov Model

6.4.3.1. Comparison of the behaviour of the three repair materials

To compare the repair efficiency of the three materials considered (i.e. OPC, OPC+PFA and OPC+GGBS) the model simulation was run using the growth parameters which were determined based on the results of the experimental study (and as presented in Table 6.2). All other variables were assigned the values which were presented in Table 4.2. Other than the initiation time and growth parameters, the same parameters were used for each repair material to allow a direct comparison of the repair efficiency to be made. As in Chapter 4, for the purpose of this example, the possible range of defect sizes was subdivided into 10 groups varying from 0mm to 1.0mm. The sensitivity of the model to many of the parameters presented in Table 4.2 was studied in Chapter 4. In this chapter, the effect of different repair materials is studied with the incorporation of an initiation phase. Therefore the sensitivity of the results of the model (i.e. expected cost and number of failures, optimum inspection interval) will be studied in relation to the properties of different repair materials. In addition, the sensitivity of these results to the estimated initiation time will also be studied in Section 6.4.3.2. In this section of the thesis it is assumed that a moderate inspection technique is used for detection (i.e. Q_1=10) and a moderate/high quality inspection technique is used for sizing (i.e. Q_2=20), however, the optimum combination of techniques for each repair material (i.e. OPC, OPC+PFA and OPC+GGBS) will be discussed in Section 6.4.3.3, and the sensitivity of the optimum combination of inspection techniques to variations in the rate of propagation and the initiation time will be studied in Section 6.4.3.4 and Section 6.4.3.5, respectively.

Using the parameters given in Table 4.2 and Table 6.2 the optimum inspection interval for each repair material was determined on the basis of the minimum expected annual total cost of the structure, as illustrated in Figure 6.15 - Figure 6.17. A range of inspection intervals varying from 1 year to 6 years was studied, since principal inspections are recommended every 6 years in the UK (Vassie and Arya, 2006), and from 1 year to 6 years in Ireland (Duffy, 2004). Also, it is assumed that only defects in the propagation phase (i.e. groups 1-10) are inspected. Using OPC as the repair material for a structure results in an optimum inspection interval of 3 years. When repairing with OPC+PFA the optimum inspection interval is longer, at 4 years, and repairing with OPC+GGBS results in an optimum inspection interval of 2 years.

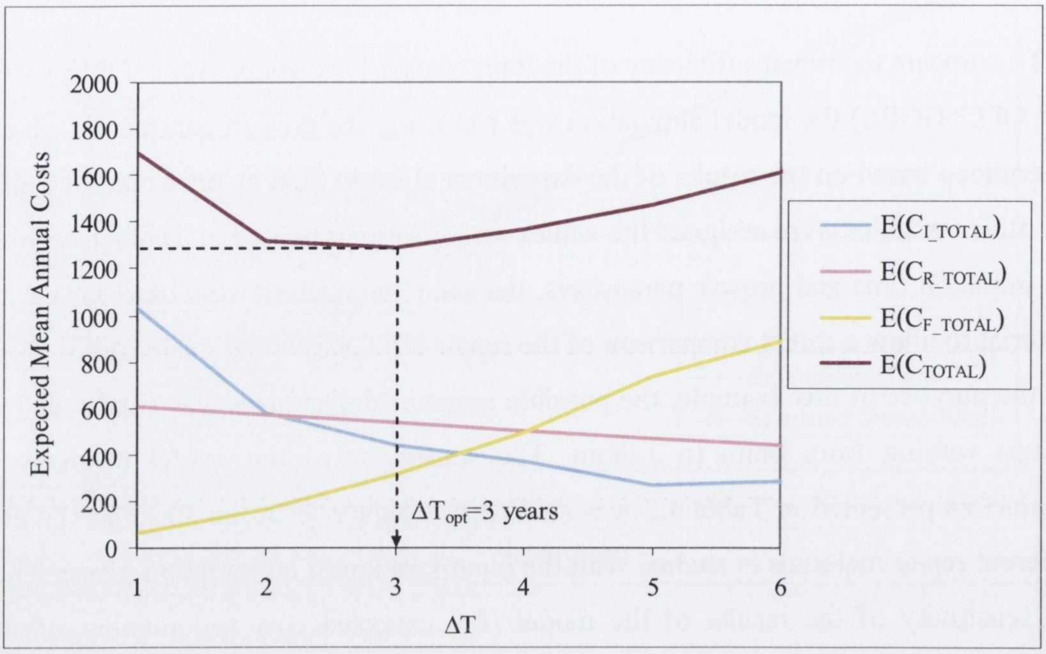


Figure 6.15. Optimum inspection interval for a structure using OPC as the repair material

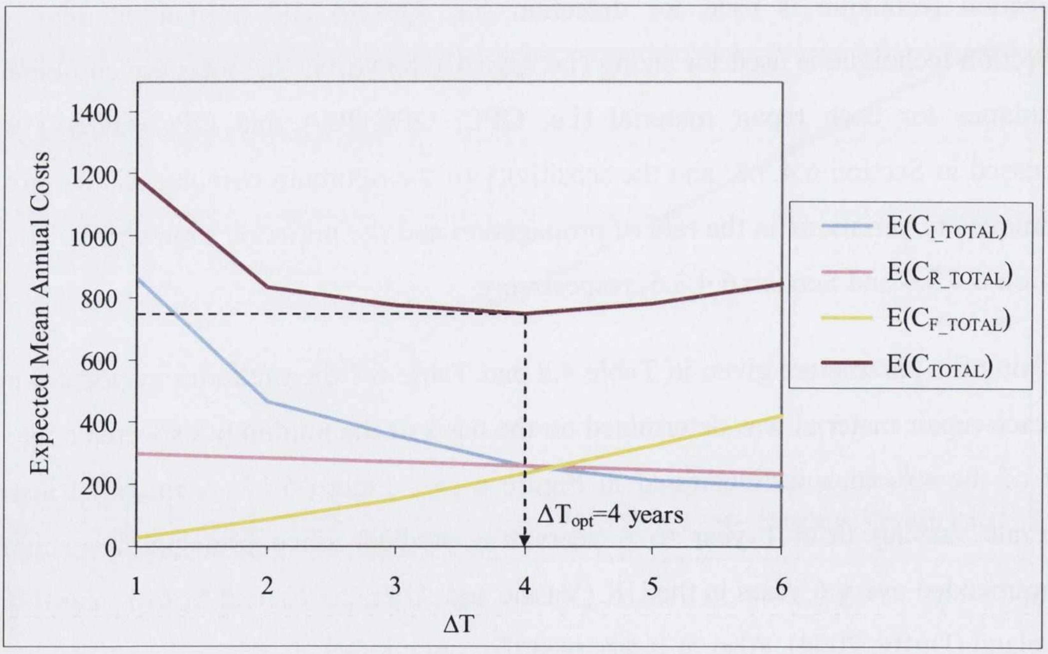


Figure 6.16. Optimum inspection interval for a structure using OPC+PFA as the repair material

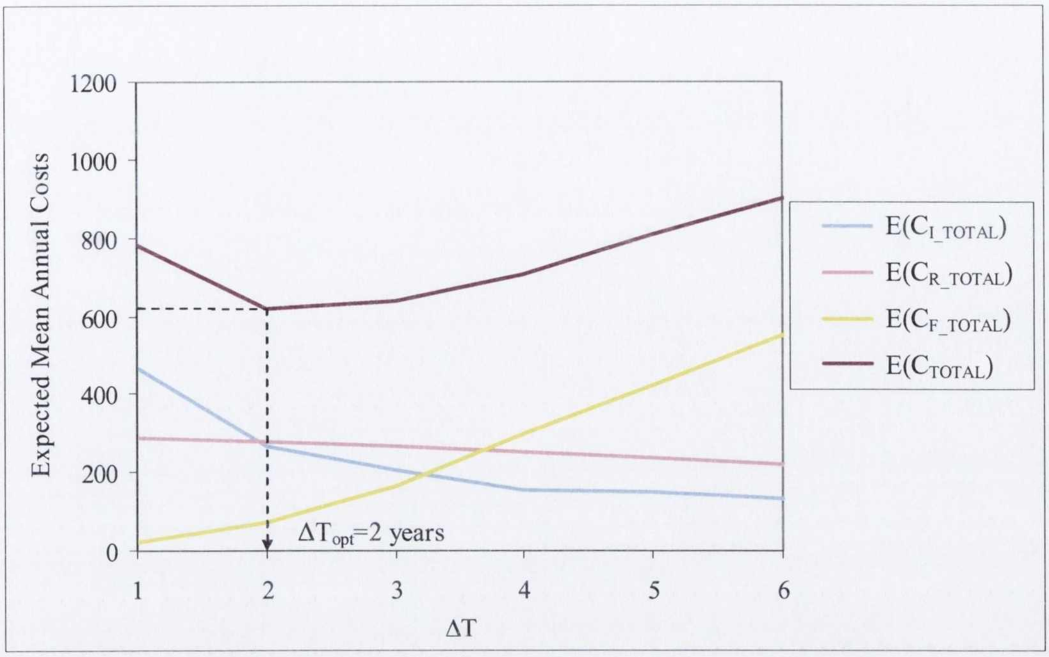


Figure 6.17. Optimum inspection interval for a structure using OPC+GGBS as the repair material

These results indicate that for this particular example the optimum time between inspections is mainly influenced by the growth of the defects once the propagation phase has begun. When considering the propagation phase alone, the OPC+PFA has the best performance followed by OPC and then OPC+GGBS, based on the results of the experimental study that was carried out as part of this thesis (as discussed in Chapter 5). This behaviour is reflected in the optimum inspection interval. Although costing models and cost coefficients can be very subjective, it is interesting to investigate the relative difference in optimum inspection intervals depending on the repair material chosen. For example, even though the initiation time of OPC+GGBS is the longest of the three repair materials, it results in the shortest optimal time between inspections.

Figure 6.18 illustrates that although the optimum inspection interval of a structure which is repaired with OPC+GGBS is the lowest of the three repair materials, it also results in the lowest mean annual expected cost up to an inspection interval of 4 years. This can be attributed to the length of the initiation period. For an inspection interval of greater than 4 years, repairs using OPC+PFA result in the lowest cost. This is both due to a relatively long initiation time (i.e. 20 years compared to 10 year initiation time for OPC), and a slower rate of crack propagation for OPC+PFA (Figure 6.9).

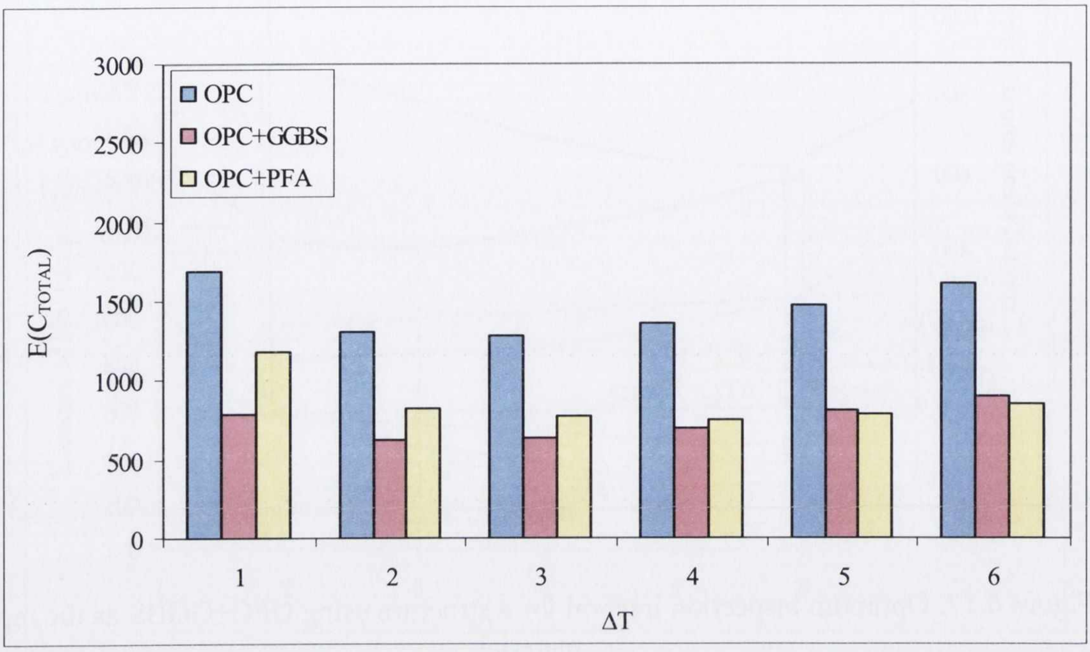


Figure 6.18. Expected mean annual total cost for different repair materials

For short inspection intervals the inspection cost forms a large proportion of the total cost (e.g. for ΔT=1, C_{I_TOTAL} is 61%, 73% and 70% of C_{TOTAL} for OPC, OPC+PFA and OPC+GGBS, respectively). In relation to repairs carried out using OPC+GGBS, the initiation period is longer than the other two repair materials resulting in less inspections being carried out. It is assumed that inspections are only carried out on defects that are in the propagation phase of deterioration (i.e. in groups 1-10). The longer the initiation period, the higher the number of defects in the initiation phase, which results in a lower number of inspections for both detection and sizing. This contributes to the lower cost associated with repairs carried out using OPC+GGBS for an inspection interval of 4 years or less. As the inspection interval increases, however, the failure cost forms a higher proportion of the total cost and the rate of propagation of defects becomes more significant. For larger intervals between inspections defects have a longer time to grow to larger defect groups where failure can occur. Since OPC+GGBS has a higher rate of crack propagation, this results in a higher number of failures for large inspection intervals in comparison with the other two repair materials which are considered as part of this thesis (i.e. OPC, OPC+PFA). Therefore, as the inspection interval increases larger than 4 years, the expected cost of OPC+PFA becomes the lowest of the three materials.

However, it is recognised that the costing models and the cost coefficients are subjective, and that owners/managers of structures will use cost models that are appropriate to the specific structures being considered, which may be different to those outlined in this methodology (Section 4.6, Chapter 3). Therefore, the number of failures resulting from each repair material has also been investigated since the number of failures is independent of costing models and cost coefficients. The results are illustrated in Figure 6.19. Repairs carried out using OPC result in the highest number of failures. This is as expected since OPC has a significantly shorter initiation time (10 years compared with 20 years and 28 years for OPC+PFA and OPC+GGBS, respectively) and a rate of crack growth similar to OPC+GGBS up to a crack width of about 0.6mm (Figure 6.9).

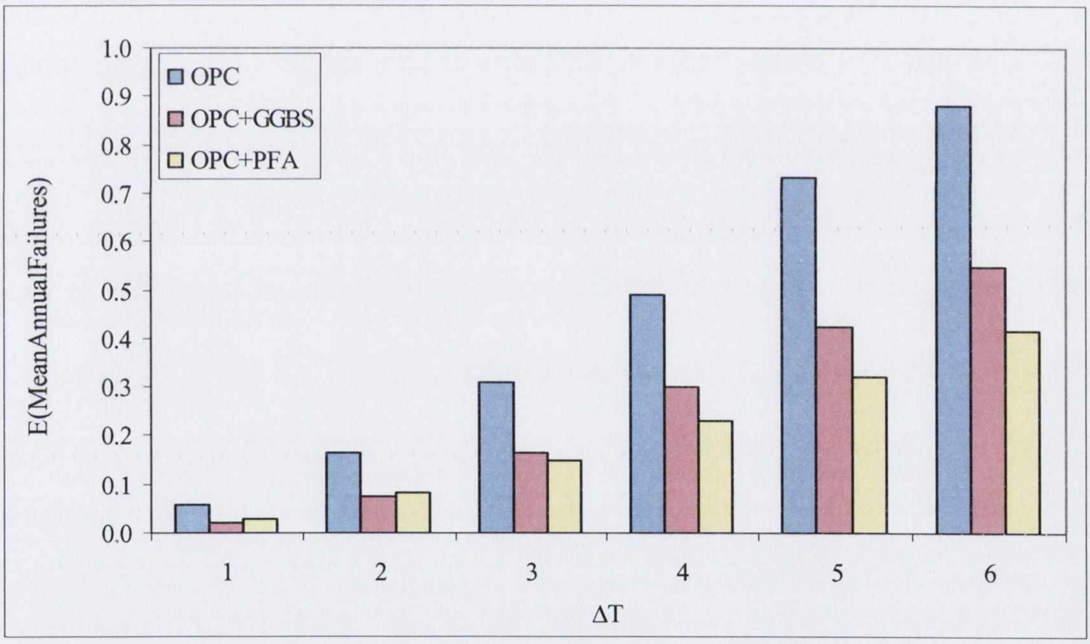


Figure 6.19. Expected mean annual number of failures for different repair materials

For an inspection interval of up to 2 years, the structures repaired using OPC+GGBS result in the lowest number of failures. Since OPC+GGBS has the highest rate of crack growth of the three repair materials, it must be inspected regularly to ensure that the defects are repaired before they reach the critical defect size and result in failure (with an associated failure cost). Also, since the initiation time is so long (28 years), there are fewer defects in the defect groups, so fewer failures occur. However, for longer inspection intervals the high rate of crack growth can lead to a large number of failures between inspections. Therefore, it is beneficial for owners/managers of structures to carry out regular inspections (with short inspection intervals) on structures constructed or repaired with OPC+GGBS to monitor the growth of defects present and avoid costly failures. For an inspection interval of 3 years or more, repairs carried out using OPC+PFA result in the lowest number of failures. Therefore, in relation to principal inspections which are carried out every 6 years in the UK (Vassie and Arya, 2006), it can be concluded that OPC+PFA is the superior repair material when considering OPC, OPC+PFA (30% PFA) and OPC+GGBS (50% GGBS). However, the initiation time can vary depending on characteristics of the material such as the permeability and the depth of concrete cover to the reinforcing bars. In this regard, the sensitivity of these results to the assumed initiation time is studied in Section 6.4.3.2. This gives owners/managers the ability to investigate the implications of varying the length of the initiation time of a repair material in relation to the expected cost, the expected number of failures and the optimum inspection interval.

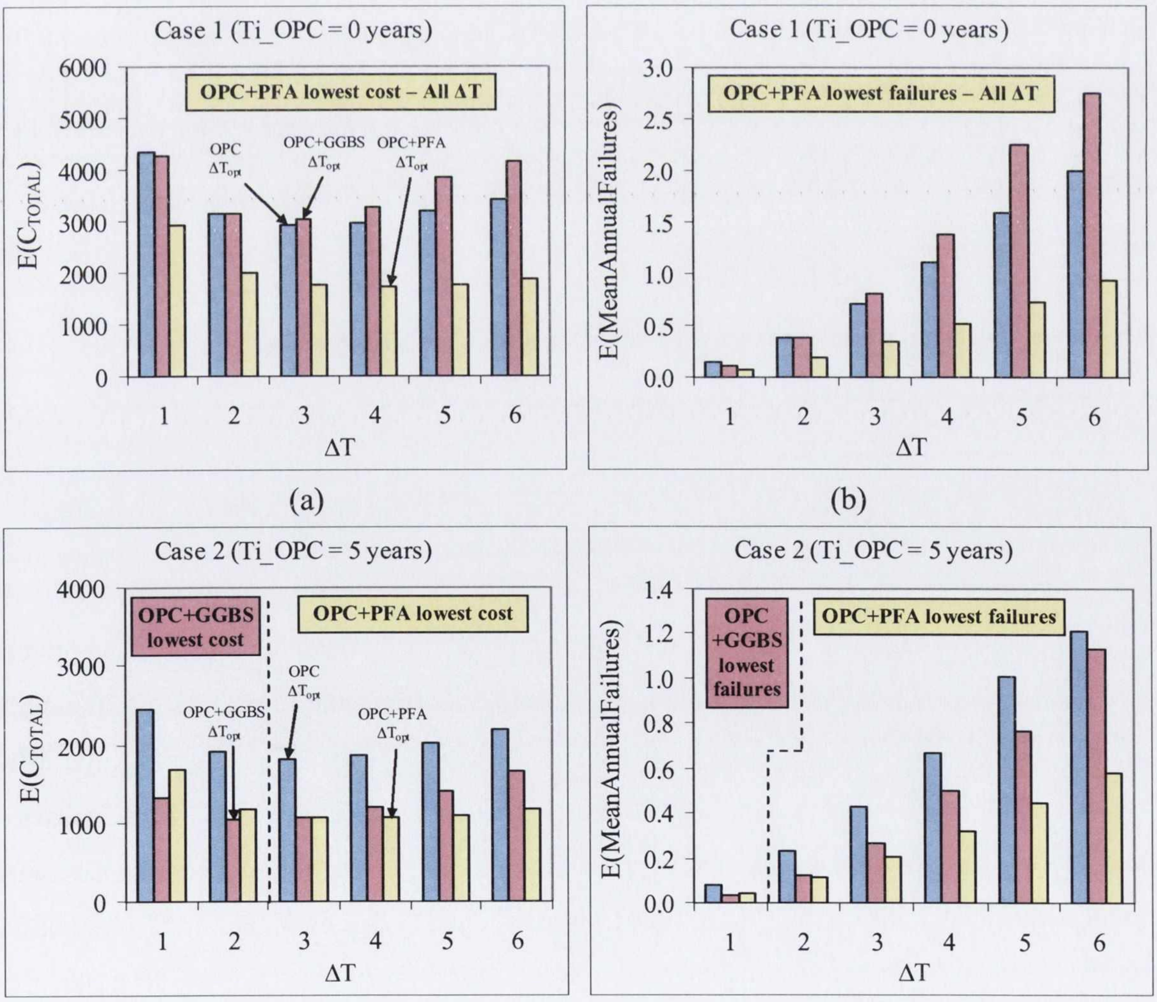
6.4.3.2. Effect of initiation time on results

In Section 6.4.1 the time to corrosion initiation was estimated for each of the repair materials considered as part of this thesis (i.e. OPC, OPC+PFA and OPC+GGBS) based on information in the literature. An initiation time for OPC was estimated at 10 years, although it is recognised that this value may change depending on the properties of the material, the environment and the concrete cover to the reinforcing bars. Again based on the literature, the ratio of initiation time for OPC+PFA and OPC+GGBS to the initiation time for OPC was estimated as 2.0 and 2.8, respectively. However, since the initiation time for OPC (10 years) is based on limited information, the sensitivity of the results to this estimated value has been investigated. It is assumed that the ratios of initiation time for the other repair materials in relation to the initiation time of OPC remain the same (i.e. 2.0 for OPC+PFA and 2.8 for OPC+GGBS). On this basis, Table 6.3 presents the four cases considered as part of this sensitivity study, with the initiation time of the OPC concrete mix varying from 0 years to 15 years in 5 year intervals. Based on the results of this study, owners/managers of structures can consider the effect of varying the initiation time on the relative performance of the different repair materials.

Concrete Mix	**Estimated Initiation Time (years)**			
	Case 1	Case 2	Case 3	Case 4
OPC	0	5	10	15
OPC+PFA	0	10	20	30
OPC+GGBS	0	14	28	42

Table 6.3. Four cases considered in sensitivity study of initiation time

The results are presented in Figure 6.20 for each of the cases considered. Case 3 consists of the original values of the initiation times that were assumed in Section 6.4.3.1. Based on the results of this study, for each assumed initiation time it is interesting to compare the relative merits of the different repair materials in relation to the expected cost and number of failures. Therefore, for each case considered, the repair material which results in the lowest expected cost/number of failures is highlighted on the corresponding figure. The optimum inspection interval for each material is also displayed.



(a) (b)

(c) (d)

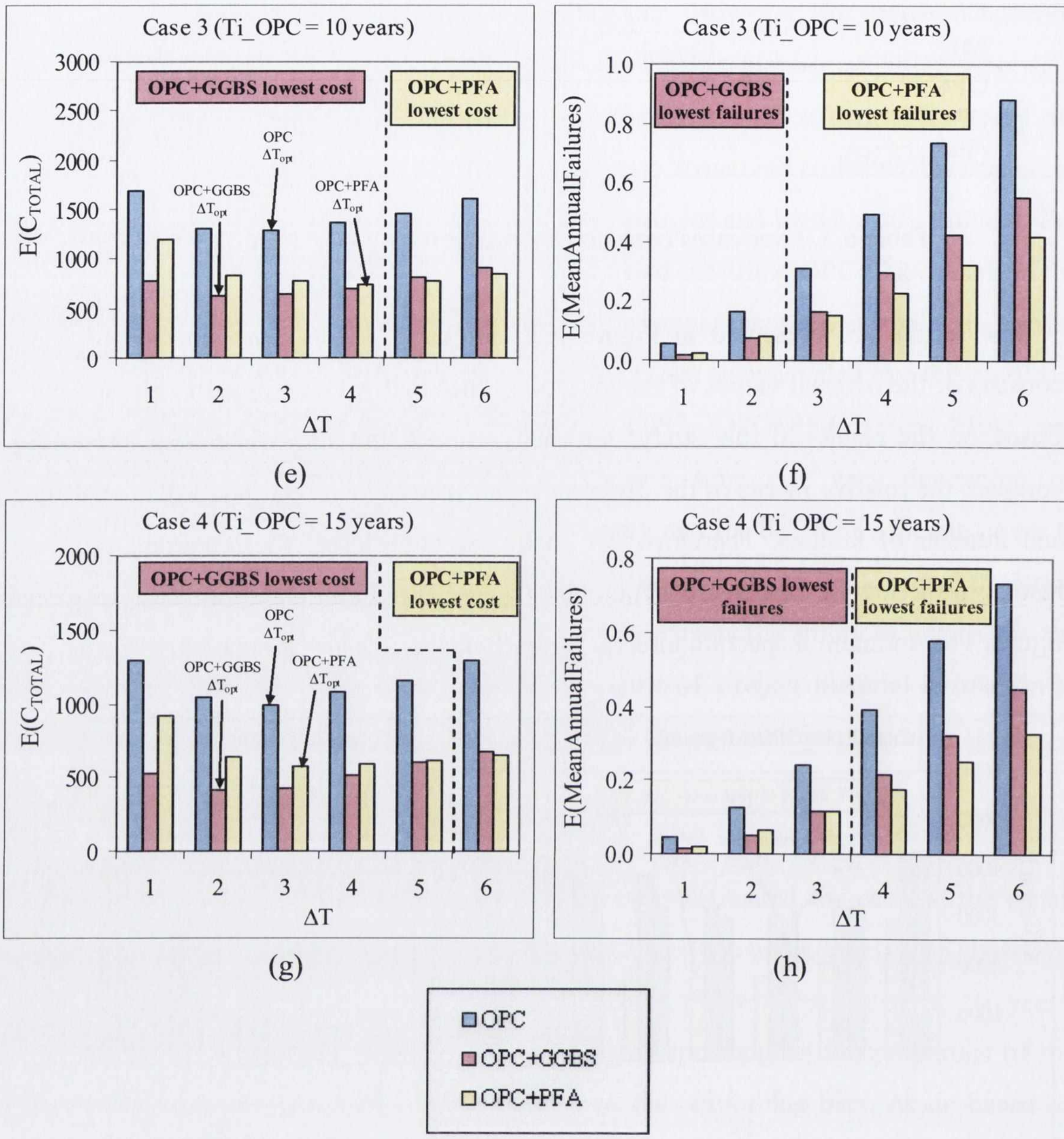


Figure 6.20. Sensitivity of relative efficiency of different repair materials to assumed initiation time

For each material the expected cost and number of failures demonstrate an inverse relationship to the length of initiation time. As the initiation time increases, the number of defects which are in the propagation phase (i.e. groups 1-10) reduces, resulting in a lower number of defects in the larger groups, where failure can occur. This in turn leads to a reduction in the expected cost. Therefore, as expected, when carrying out repairs it is worth considering increasing the initiation time by repairing defects using a material with a lower permeability or by increasing the concrete cover if possible, as this will lead to a reduction in the expected cost and number of failures over the remaining lifetime of the structure.

However, assessing the cost associated with each of these repair options is beyond the scope of this thesis.

Considering the optimum inspection interval (ΔT_{opt}) for each of the materials considered (based on the minimisation of the expected mean annual total cost), for OPC the optimum inspection interval remains at 3 years regardless of the initiation time, whereas, the optimum inspection interval for OPC+PFA is 4 years for the first 3 cases, but reduces by only 1 year to 3 years for case 4 as the initiation time increases. Similarly, the optimum inspection interval for OPC+GGBS is 3 years for case 1, but then reduces to 2 years for the remaining cases considered. As the initiation time increases there are more defects in the initiation phase when inspections are carried out. This reduces the inspection cost since fewer inspections for detection and sizing have to be carried out. Due to this reduction in the inspection cost, more frequent inspections become optimal. However, these results also indicate that the optimum inspection interval, which varies by a maximum of 1 year, is relatively insensitive to the initiation time and that the optimum inspection interval is dictated primarily by the behaviour of the propagation phase of deterioration, as discussed in Section 6.4.3.1.

In this section, the main area of interest is the relative performance of the different repair materials as the initiation times are varied. In the first case, Figure 6.20(a)-(b), no initiation time is considered, therefore, the expected cost and number of failures are influenced only by the deterioration behaviour of the propagation phase for each of the materials. On this basis, OPC+PFA results in the lowest expected cost and number of failures due to the slowest rate of crack propagation of the three concrete mixes considered. However, considering OPC and OPC+GGBS only, OPC results in the highest expected cost and number of failures for an inspection interval of 2 years or less, whereas for larger inspection intervals OPC+GGBS results in the highest expected cost and number of failures (i.e. for $\Delta T>3$years). This can be explained by studying Figure 6.9 where initially OPC has a slightly higher rate of crack growth than OPC+GGBS. This is of interest for a short inspection interval. For larger inspection intervals the rate of crack propagation for larger cracks is of interest and in this case it is higher for OPC+GGBS. This illustrates that the parameters describing the rate of crack propagation for each defect group must be carefully estimated to accurately model the deterioration behaviour of a material, as this may influence the optimal maintenance strategy. This also highlights the importance of the ability to simulate non-linear deterioration of materials, which is incorporated as part of this model, as discussed in Section 6.2.

In the second case, Figure 6.20(c)-(d), the initiation time is increased to 5 years for OPC, resulting in an initiation time of 10 years and 14 years for OPC+PFA and OPC+GGBS, respectively. In this case, due to a longer initiation time (i.e. 14 years), OPC+GGBS results in the lowest expected cost for an inspection interval of 1 or 2 years and the lowest number of failures for an inspection interval of 1 year. Since OPC+PFA has an initiation time of 10 years and the slowest rate of crack propagation, OPC+PFA results in the lowest expected cost and number of failures for all other inspection intervals. This trend continues for case 3 and case 4, Figure 6.20(e)-(h). Since the ratio of initiation time for OPC+PFA and OPC+GGBS to OPC remain the same for each of the cases considered, as the initiation time for OPC increases, the difference between the initiation time of OPC+PFA and OPC+GGBS increases, with diminishes the superior performance of OPC+PFA which has the slowest rate of crack propagation. As discussed in Section 6.4.3.1, for case 3 OPC+GGBS results in the lowest expected cost for an inspection interval of 4 years or less and the lowest number of failures for an inspection interval of 2 years or less. This improved performance of OPC+GGBS increases with the assumed initiation time for OPC and for case 4 OPC+GGBS has the lowest expected cost for an inspection interval of 5 years or less and the lowest number of failures for an inspection interval of 3 years or less. Since there is a higher proportion of defects in the initiation phase for OPC+GGBS (due to a longer initiation period), this results in a lower number of inspections, repairs and failures, which in turn results in a lower expected total cost. However, for longer inspection intervals ($\Delta T \geq 6 \text{years}$), OPC+PFA is still the superior repair material due to a slow rate of crack propagation, which is of primary importance when defects have a long time to grow between inspections.

Ultimately, the results of this study illustrate the sensitivity of the expected cost, the number of failures and the relative performance of the different repair materials to the assumed initiation time. Similar to Section 6.4.3.1, it can be concluded that the optimal maintenance strategies and the relative performance of the different repair materials depends on both the initiation phase and propagation phase of deterioration. In addition, this section highlights how the developed methodology allows owners/managers to study the relative benefits of different repair materials considering the initiation phase and propagation phase of deterioration, and the sensitivity of the results to assumed parameters such as the initiation time. However, thus far, the results have been presented assuming specific inspection qualities for detection (Q_1=10, moderate quality) and sizing (Q_2=20, moderate/high quality). Using this developed methodology it is also possible to determine the optimal inspection quality for each stage of an inspection and to investigate the sensitivity of the optimal combination of inspection techniques to the properties of the repair material being considered.

6.4.3.3. Optimum inspection quality for each repair material

In this thesis a distinction is made between an inspection which is carried out to detect a defect and an inspection which is carried out to size a defect. On this basis, to allow the optimal maintenance strategy to be achieved, the optimum inspection quality must be determined for each stage of an inspection. Using the procedure outlined in Section 4.2.3 the optimum combination of inspection techniques and the associated expected total cost was determined for each of the three repair materials for a range of inspection intervals (i.e. ΔT=1:6). These results are presented in Table 6.4. As in Section 6.4.3.1 an initiation time of 10, 20, 28 years is assumed for OPC, OPC+PFA and OPC+GGBS, respectively. For each repair material the increase in expected cost for each inspection interval with respect to the minimum cost (i.e. for the optimal inspection interval for that repair material) is also presented. This percentage difference is highest for OPC+GGBS due to the high rate of crack propagation once the initiation phase has ended. This can result in a higher increase in the number of failures as the inspection interval is increased.

The results also demonstrate that the optimum combination of inspection techniques depends on the deterioration parameters (i.e. for initiation and propagation) of the repair material being considered, particularly in relation to the sizing assessment. It was concluded in Section 4.2.3 that it is optimal to use a low quality inspection technique for detection (i.e. like a screening exercise) followed by a higher quality inspection technique for the sizing assessment. This is again evident for each of the repair materials, with the optimum inspection quality for detection varying only from Q_1=2 to Q_1=4, both of which are low quality inspection techniques. The only difference in the inspection quality for detection for the three repair materials considered (i.e. OPC, OPC+PFA and OPC+GGBS) is for an inspection interval of 2 years where Q_1=2 for OPC+PFA and Q_1=4 for OPC and OPC+GGBS. The lower optimal inspection quality for OPC+PFA can be attributed to the slower rate of deterioration associated with the OPC+PFA concrete mix. These results demonstrate that the inspection quality for detection is not very sensitive to the chosen repair material.

Concrete Mix	**ΔT**	**Q_1**	**Q_2**	**$E(C_{TOTAL})$**	**Increase wrt Min Cost**
OPC	1	2	8	807	0%
	2	4	12	864	7%
	3	4	16	959	19%
	4	4	18	1100	36%
	5	4	20	1281	59%
	6	4	26	1411	75%
OPC+PFA	1	2	4	438	0%
	2	2	10	468	7%
	3	4	14	517	18%
	4	4	18	564	29%
	5	4	20	633	44%
	6	4	22	713	63%
OPC+GGBS	1	2	6	378	0%
	2	4	14	417	10%
	3	4	18	489	29%
	4	4	20	594	57%
	5	4	20	707	87%
	6	4	20	813	115%

Table 6.4. Optimum inspection quality and associated expected total cost for each repair material for a range of inspection intervals (ΔT=1:6)

The inspection quality for the sizing assessment is more sensitive to the variation in deterioration parameters for the different repair materials. Comparing OPC and OPC+PFA, the optimum inspection quality for OPC+PFA is lower than or equal to the optimal inspection quality for OPC for each inspection interval considered, varying from Q_2=4 (low quality inspection, ΔT=1) to Q_2=22 (moderate/high quality inspection, ΔT=6), Table 6.4. For OPC the optimal inspection quality for sizing varies from Q_2=8 (low quality inspection, ΔT=1) to Q_2=26 (high quality inspection, ΔT=6), Table 6.4. For OPC a higher optimum inspection quality (which is associated with a lower level of noise) is needed since the rate of deterioration is faster than for OPC+PFA (Figure 6.9), which would result in more failures if defects were not repaired. In addition, the initiation time is shorter for OPC, resulting in a higher number of defects in the propagation phase (i.e. groups 1-10) which could result in a higher number of failures if defects were not repaired.

As discussed in Section 6.4.3.2, the rate of crack propagation is fastest initially for OPC (up to a crack width of about 0.3mm). Therefore, for an inspection interval of 1 year, OPC has the highest optimum inspection quality for sizing. Even though OPC+GGBS has the longest initiation time (i.e. 28 years), for an inspection interval of 2 years to 4 years the highest optimal inspection quality is needed for OPC+GGBS due to a faster rate of deterioration for crack widths greater than about 0.3mm (Figure 6.9). However, for longer inspection intervals (i.e. $\Delta T=6$) the inspection quality for sizing for OPC+GGBS is the lowest of the three materials, with $Q_2=20$ (Table 6.4). This can be attributed again to the high rate of deterioration. For an inspection interval of 6 years defects have time to grow to the critical defect size ($d_c=0.62$) between inspections (Figure 6.9), resulting in failures. It is therefore not optimal to use very high quality inspection techniques since failure can still occur between inspections even if a repair has been carried out. Therefore, for a high rate of deterioration it is optimal to carry out inspections more often, where lower quality inspection techniques are also optimal.

These results indicate that the optimum combination of inspection techniques is specific to the material and the deterioration mechanism which is being considered, particularly in relation to the sizing assessment and that the owner/manager of a structure should select the inspection techniques based on the characteristics of the repair material which is chosen. However, since each phase of deterioration (i.e. initiation and propagation) is different for each material considered in this case, a direct comparison of the effect of each phase of deterioration on the optimum combination of inspection techniques cannot be made. Therefore, the sensitivity of each phase has been considered separately by keeping one phase constant while varying the other. This will allow owners/managers to gain an insight into the sensitivity of the optimum inspection quality to changes in the deterioration behaviour of a material or to changes in the material which is chosen to carry out a repair, and to understand how changes in each phase of deterioration can affect the optimal inspection quality.

6.4.3.4. Effect of propagation phase on optimum inspection quality

To study the effect of the rate of propagation on the optimal inspection quality for detection and sizing an initiation time of 0 years was assumed for each repair material, and the parameters which were presented in Table 6.2 were used to define the rate of propagation based on the results of the experimental study. This study will allow owners/managers of structures to be aware of the effect of changes in the rate of crack propagation (e.g. due to changes in material properties, changes in environment) on the optimal inspection quality for each stage of an inspection. The results of this sensitivity study are presented in Table 6.5. In relation to the expected cost, as discussed in Section 6.4.3.3, the increase in the expected cost with the inspection interval is larger for OPC+GGBS (84%) due to the higher rate of crack propagation.

Considering OPC and OPC+PFA, the rate of crack propagation is higher for OPC. Therefore, higher quality inspection techniques which are associated with a lower level of noise are needed to maintain an acceptable level of safety when repairs are carried out using OPC. In relation to the first inspection (i.e. for detection), the inspection quality is the same for both repair materials for inspection intervals of $\Delta T=1$, $\Delta T=3$, $\Delta T=4$ and $\Delta T=5$, but a higher quality inspection technique for detection is optimal for OPC for an inspection interval of $\Delta T=2$ and $\Delta T=6$. This is illustrated in Figure 6.21. This minor change in inspection quality for detection indicates that the inspection quality of the first inspection is relatively insensitive to changes in the rate of crack propagation. A similar conclusion can be made when considering the optimum inspection quality for detection for OPC+GGBS (which is the same as OPC for each ΔT).

This demonstrates that owners/managers may not have to consider changing the inspection technique for detection if there is a change in the concrete mix which is chosen to carry out patch repairs on a structure or group of structures. The maximum difference in the optimum inspection quality (for detection) for a particular inspection interval is $\Delta Q_1=2$. Therefore, although the optimal inspection quality for detection may change slightly, this change would be minimal and rather than purchasing a new inspection technique which has a similar performance to the old technique, it may be more convenient for owners/managers to continue using the optimum inspection technique for the original material.

Concrete Mix	ΔT	Q_1	Q_2	$E(C_{TOTAL})$	Increase wrt Min Cost
OPC	1	2	14	2076	0%
	2	4	16	2085	0%
	3	4	18	2190	6%
	4	4	22	2420	17%
	5	4	24	2768	33%
	6	6	28	3089	49%
OPC+PFA	1	2	8	1119	1%
	2	2	12	1112	0%
	3	4	14	1191	7%
	4	4	18	1292	16%
	5	4	20	1427	28%
	6	4	24	1582	42%
OPC+GGBS	1	2	12	2080	0%
	2	4	16	2129	2%
	3	4	20	2343	13%
	4	4	20	2738	32%
	5	4	20	3427	65%
	6	6	24	3827	84%

Table 6.5. Optimum combination of inspection qualities for each material, considering an initiation time of 0 years for a range of inspection intervals (ΔT=1:6)

However, the optimum quality of the second inspection (i.e. sizing assessment) is more sensitive to changes in the rate of crack propagation. Again, considering OPC and OPC+PFA, the difference in the optimum inspection quality for sizing is highest for an inspection interval of 1 year, where ΔQ_2=6, as illustrated in Figure 6.21(a). This difference reduces to ΔQ_2=4 as the inspection interval increases, Figure 6.21(b)-(f). This reduced difference is considered to be due to the bilinear behaviour of OPC, where the rate of crack propagation changes and cracks develop more slowly after a crack width of about 0.6mm. To maintain an acceptable level of safety, a higher quality inspection technique is needed to size defects when OPC is used as the repair material due to a higher rate of crack propagation compared to OPC+PFA. When a higher quality inspection technique is used, there is a lower level of noise associated with the inspection, resulting in fewer incorrect inspection results. Incorrect inspection results could lead to defects which are greater than the critical defect size being inaccurately sized and consequently not repaired, as discussed in Section 3.3.2.

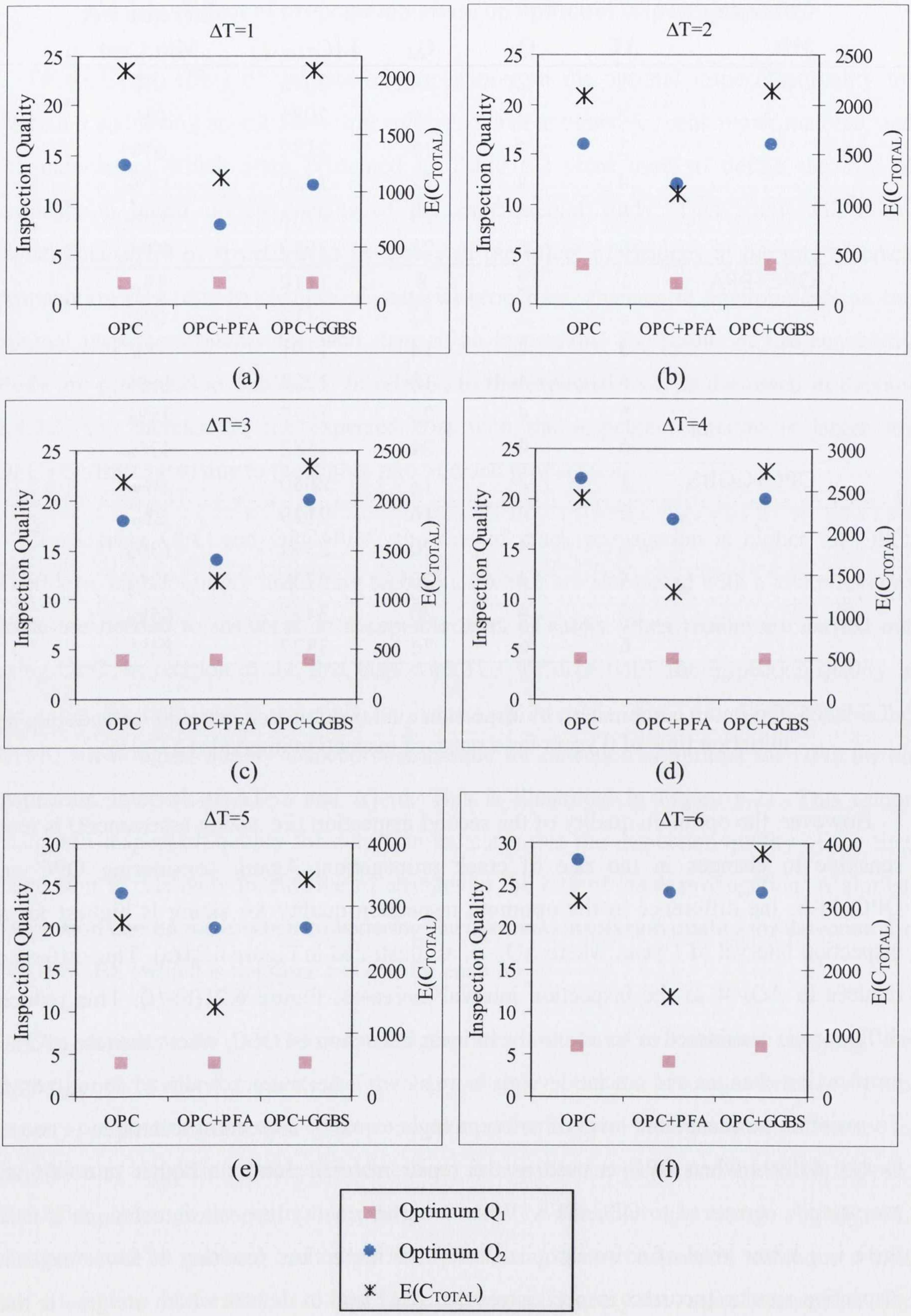


Figure 6.21. Effect of the rate of deterioration on expected cost and optimum inspection quality for a range of inspection intervals (ΔT=1:6)

In relation to OPC and OPC+GGBS, the results are similar to Section 6.4.3.3. For an inspection interval of 1 year the optimum inspection quality for sizing is higher for OPC (Q_2=14) than for OPC+GGBS (Q_2=12), as illustrated in Figure 6.21(a), due to the higher rate of crack propagation initially for OPC. However, since the rate of crack growth is faster in OPC+GGBS after an approximate crack width of 0.3mm, higher quality inspection techniques are needed for the sizing assessment for an inspection interval of 2 years and 3 years, as illustrated in Figure 6.21(b)-(c). However, as the inspection interval is increased further, higher quality inspection techniques are optimal for OPC, even though the rate of crack propagation is slower compared to OPC+GGBS. This can again be attributed to the high rate of deterioration for OPC+GGBS and a high probability of failure between inspections, even if repairs have been carried out following the previous inspection. For example, considering an inspection interval of 6 years, the approximate crack width of OPC+GGBS is 0.67mm (Figure 6.14) and the limit crack width is only 0.62mm. Therefore, considering the rate of crack propagation, it is recommended that structures which were constructed with or repaired using OPC+GGBS are inspected at regular intervals, with less than 6 years between inspections. However, this will depend on the environment, the specific properties of the material and the limit state being considered.

Therefore, an increase in the rate of crack propagation leads to a need for a higher quality inspection for sizing (and for detection to a lesser extent) up to a point, until an increase in the inspection quality does not lead to a significant reduction in the number of failures. At this point, even if repairs are carried out, defects can still fail between inspections due to the high rate of crack propagation, depending on the limit state and critical defect size being considered. To summarise, the results of this section indicate that the optimal inspection quality (in particular the sizing assessment) is influenced by the rate of crack propagation in the different repair materials. However, the maximum difference in optimal inspection quality for sizing is only ΔQ_2=6. Therefore, if an owner/manager of a structure owns/uses a specific inspection technique (with an associated inspection quality), and it is decided to use an alternative repair material, then it may be more convenient to alter the maintenance strategy around the inspection techniques which are available, rather than developing a totally new maintenance strategy in which new inspection techniques will have to be purchased, with which inspectors will have to become trained. For example, consider a structure that has been constructed and repaired with OPC and is inspected annually with a sizing technique corresponding to Q_2=14 (referring to Table 6.5). If it is decided to repair with OPC+PFA rather than OPC in the future, then using the same inspection technique it would be optimal to inspect every 3 years. Therefore, although this may not be the optimal maintenance strategy, as determined from the maintenance management model, this option may be more convenient for the owner/manager, and the capabilities of this developed strategy allows owners/managers to study the implications of such decisions in relation to the associated expected cost and expected number of failures. This ability to provide owners/managers with a range of maintenance options was highlighted in Liu and Frangopol (2005a) (as discussed in Section 2.2.2). This provides owners/managers with more than one maintenance strategy and allows the decision maker to select a maintenance strategy which is most suitable and practical to meet the specific constraints and requirements associated with a particular structure or group of structures. This is possible using the developed methodology.

This section has studied the effect of the rate of crack propagation on the optimal combination of inspection techniques for detection and sizing. However, the effect of the length of the initiation phase is also of interest as this can change, for example, if the permeability of the repair material is altered, or if it is decided to increase the concrete cover to the reinforcing steel. Therefore, the sensitivity of the optimal inspection quality to the initiation time of a repair material was also studied and will be discussed in Section 6.4.3.5.

6.4.3.5. Effect of initiation time on optimum inspection quality

One material was chosen to study the effect of the length of initiation time on the optimum combination of inspection techniques for detection and sizing, and the initiation time was varied from 0 years to 15 years in intervals of 5 years. For the purpose of this sensitivity study the deterioration parameters for the propagation phase of the OPC mix (as in Table 6.2) were used. The results of this study are presented in Table 6.6. In Table 6.6 the optimal inspection qualities for detection and sizing (Q_1 and Q_2) and the expected cost for each inspection interval are presented. For each initiation time considered, the increase in expected cost for each inspection interval with respect to the minimum cost (i.e. for the optimal inspection interval for that initiation time) is also presented.

Initiation Time	ΔT	Q_1	Q_2	$E(C_{TOTAL})$	Increase wrt Min Cost
0 years	1	2	14	2075	0%
	2	4	16	2084	0%
	3	4	18	2189	6%
	4	4	22	2419	17%
	5	4	24	2767	33%
	6	6	28	3089	49%
5 years	1	2	10	1166	0%
	2	4	14	1240	6%
	3	4	18	1347	16%
	4	4	20	1501	29%
	5	4	22	1751	50%
	6	4	28	1936	66%
10 years	1	2	8	806	0%
	2	4	12	864	7%
	3	4	16	959	19%
	4	4	18	1100	36%
	5	4	20	1280	59%
	6	4	26	1410	75%
15 years	1	2	6	614	0%
	2	4	12	677	10%
	3	4	16	742	21%
	4	4	18	871	42%
	5	4	20	1009	64%
	6	4	24	1143	86%

Table 6.6. The variation in optimal inspection quality and expected cost with initiation time for a range of inspection intervals (ΔT=1:6)

In each case the optimal inspection interval is 1 year. Although it is recognised that the costing models used to carry out this study are subjective, the results do indicate that the optimal inspection interval is not sensitive to the initiation time and, as discussed in Section 6.4.3.1, the optimal inspection interval is considered to be influenced primarily by the growth of the defects once the propagation phase has begun. Therefore, if the owner/manager decides to carry out a repair using a higher performance concrete (i.e. with a lower permeability) or a larger concrete cover to the reinforcing bars to increase the time to initiation of corrosion, then the optimal inspection interval is unlikely to change. It is recognised that the sensitivity of the results will be influenced by the properties of the material being considered and the associated environmental conditions. However, using the developed methodology, owners/managers can study the implications of such changes on the expected cost and number of failures.

By studying Table 6.6, it is also apparent that increasing the initiation time of the material (i.e. by producing a more impermeable concrete or increasing the concrete cover) reduces the expected mean annual total cost of a structure for each inspection interval considered, as discussed in Section 6.4.3.2. In addition, it is noted that a longer initiation time results in a higher percentage difference in the expected cost between short ($\Delta T=1$) and long ($\Delta T=6$) inspection intervals. In this case the maximum percentage difference is 49% for an initiation time of 0 years and 86% for an initiation time of 15 years. As the initiation time increases, inspections and resulting repairs have a larger impact on the service life performance of a structure and the difference between structures which are inspected and repaired annually or biannually and those which are inspected less often (i.e. $\Delta T=6$) becomes more evident.

Figure 6.22 also illustrates the inverse relationship between the initiation time and the expected cost. In addition, Figure 6.22(a)-(f) demonstrate that the optimum inspection quality for detection (Q_I) is relatively insensitive to the initiation time, varying only for an inspection interval of 6 years, where the optimum inspection quality decreases from $Q_I=6$ (Ti=0 years) to $Q_I=4$ (Ti=15 years), both of which are low quality inspection techniques. Therefore, similar to the discussion in Section 4.2.3, it is optimal to use a low quality inspection technique for detection (i.e. like a screening exercise), followed by a higher quality inspection technique for the sizing assessment (which dictates whether or not repair is carried out). These results indicate that the length of the initiation time does not affect this conclusion. Therefore, if owners/managers have invested in equipment for the detection of defects, a change in the length of the initiation time will not significantly affect the suitability of the inspection technique.

However, the optimum inspection quality for sizing is more sensitive to changes in the initiation time. The optimal inspection quality for sizing is inversely proportional to the length of the initiation time. To maintain the necessary level of safety, lower quality inspection techniques are adequate when the initiation time is long since defects do not grow during the initiation time, leading to fewer failures over the lifetime of the structure.

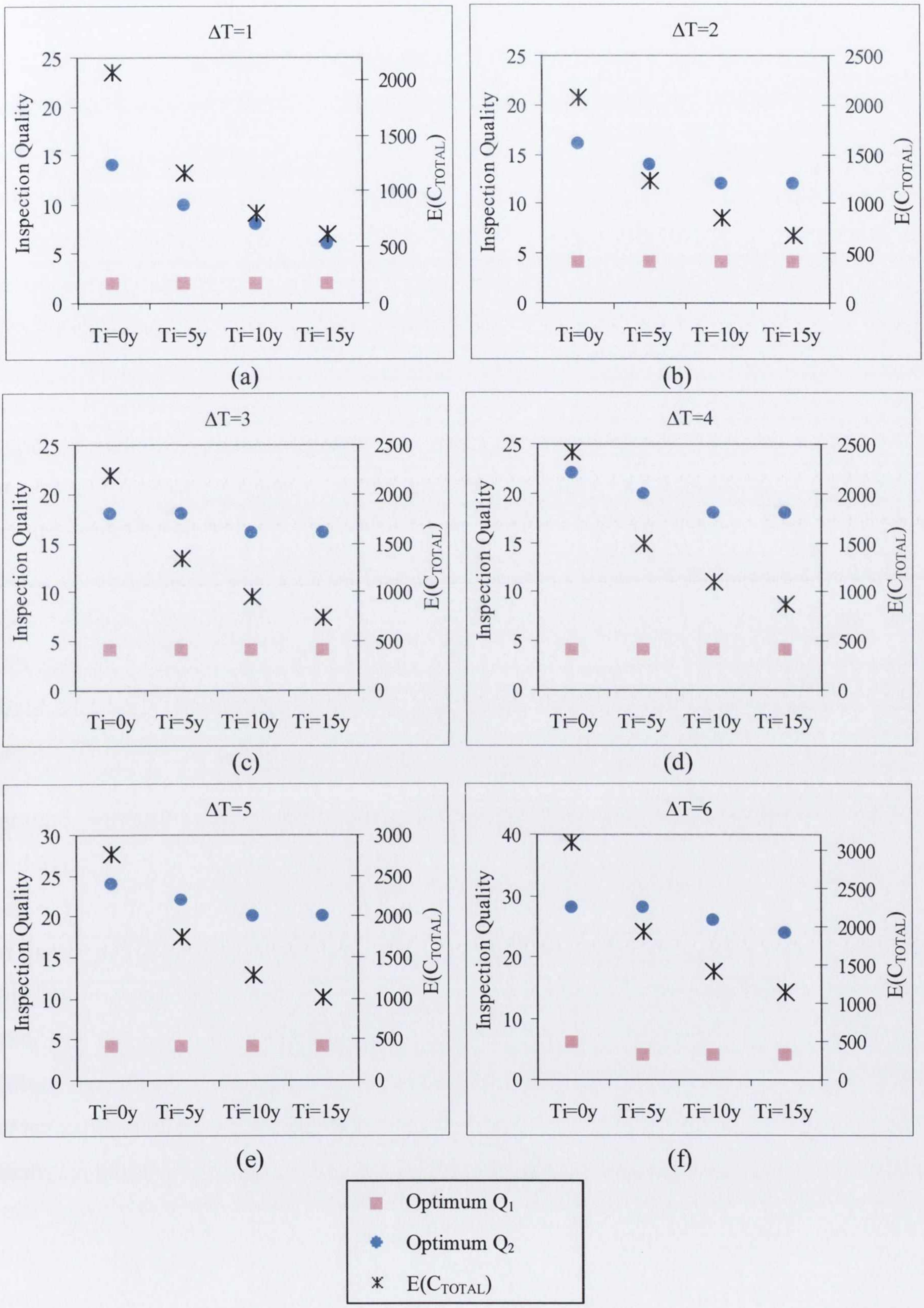


Figure 6.22. Effect of initiation time on expected cost and optimum inspection quality for a range of inspection intervals (ΔT=1:6)

The effect of the initiation time on the number of failures was discussed in Section 6.4.3.2. This sensitivity is most evident for an inspection interval of 1 year, since the number of repairs (and hence the number of defects in the initiation time) is higher for an inspection interval of 1 year. For example, considering an inspection interval of 1 year, the optimum inspection quality for sizing varies from Q_2=14 (moderate inspection quality) for an initiation time of 0 years to Q_2=6 (low inspection quality) for an initiation time of 15 years, with a ΔQ_2=8, whereas for an inspection interval of 6 years the optimum inspection quality for sizing varies only from Q_2=28 (high inspection quality) for an initiation time of 0 years to Q_2=24 (high inspection quality) for an initiation time of 15 years, with a ΔQ_2=4.

In this context, since the difference in the optimal inspection quality is minor for larger inspection intervals, if there is a change in the initiation time it may be possible to increase/reduce the quality of the inspection using the same inspection technique but by changing another parameter, such as reducing human error by using a more experienced inspector for example, or by carrying out the inspection when environmental conditions are more favourable. Otherwise, if the inspections are carried out by a contracted agency, this method allows owners to specify the optimum quality inspection techniques to be used, based on the properties of the repair material and the estimated time to corrosion initiation.

These results indicate that the length of initiation time does affect the optimal combination of inspection techniques. The effect on the inspection quality for detection is minimal but the effect on the inspection quality for sizing can be more significant (i.e. for short inspection intervals). Therefore, if possible, the inspection techniques (for sizing in particular) should be chosen based on the properties of the material in question and the estimated time to corrosion initiation. In addition, the variation in the expected cost with initiation time is considerable. Therefore, when repairing a structure it is worth considering designing a more durable mix with a lower permeability or increasing the concrete cover to the reinforcing steel to increase the time to initiation of corrosion. Although this may lead to a higher repair cost, it may be optimal considering the whole life cost of a structure.

6.5. CONCLUSIONS

This chapter details the advances that have been made in the development of the Markov maintenance model. Growth parameters can now be specified for two materials, the original material from which the structure was built and a repair material with which repairs are carried out. This achieves objective 5 of this thesis, which was to incorporate the ability to simulate the repair of a structure using a different material. This allows the sensitivity of the optimum maintenance strategy to the chosen repair material to be studied. However, using this method of simulation, the results are not sensitive to the properties of the original material. In addition, the ability to simulate non-linear deterioration of defects in the propagation phase was incorporated into the methodology, thereby achieving the second objective of the thesis (part of which was achieved in Chapter 3), which was to develop a method of deterioration modelling with the ability to consider many different forms of defect growth and deterioration kinetics (i.e. abrupt and gradual, linear and non-linear).

It was decided to apply this methodology to a real example such as reinforced concrete. To do so, the two stages in the deterioration of reinforced concrete due to corrosion of the reinforcing bars had to be considered. Therefore, the model was revised and advanced to take into account the initiation phase as well as the propagation phase in the deterioration of a material, allowing the effect of both phases of deterioration to be investigated and achieving objective 4 of this thesis. In this regard, defects that are repaired are inputted into an initiation phase and re-entered into the first group of defect growth once the specified initiation period has passed.

A practical example was carried out by estimating the initiation period for the different repair materials using information from the literature. The experimental results from Chapter 5 were used to model the propagation phase for the three different repair materials studied in the experiment. The input growth parameters needed to carry out the simulation were fitted to the experimental data to give an accurate estimation of the rate of crack growth for the different repair materials, as illustrated in Figure 6.12-Figure 6.14. The example illustrates how the developed methodology can be used to model the behaviour of many different materials using the two phase growth model that has been incorporated, therefore, achieving objective 6 of this thesis. For a material with no initiation period, the initiation time can be specified as zero.

The results of this work indicated that repairs carried out using OPC+PFA resulted in the lowest number of failures for an inspection interval greater than 2 years, for the given set of input parameters used in this example, Section 6.4.3.1 (Figure 6.19). Considering for example an inspection interval of 6 years, which is the interval for principal inspections in the UK (Vassie and Arya, 2006), OPC+PFA is the most efficient repair material with the lowest expected cost and number of failures of the three materials studied as part of this thesis (i.e. OPC, OPC+PFA and OPC+GGBS). However, since the estimated initiation time for OPC was based on limited information from the literature, the sensitivity of the results to this assumed value was investigated in Section 6.4.3.2. The results indicated that the relative performance of the different materials is sensitive to the assumed initiation time. For an initiation time of 0 years OPC+PFA is the most efficient repair material for all inspection intervals (in terms of expected cost), whereas for an initiation time of 15 years (for OPC) OPC+GGBS is the most efficient repair material for an inspection interval of 5 years or less, Figure 6.20. In addition, an inverse relationship exists between the initiation time and the expected cost and number of failures. The variation in the expected cost with initiation time is considerable. Therefore, as expected, when repairing a structure it is worth considering designing a more durable mix with a lower permeability or increasing the concrete cover to the reinforcing steel to increase the time to initiation of corrosion. Although this may lead to a higher repair cost, it may be optimal considering the whole life costing of a structure. It is beyond the scope of this thesis to quantify the cost of different repair options (e.g. increasing concrete cover).

Using the developed methodology the optimum quality inspection tools for detection and sizing can also be determined as a function of the repair material. On this basis, the optimum combination of inspection techniques was determined for each repair material for a range of inspection intervals (i.e. $\Delta T=1:6$). The results of this study indicate that the optimum inspection quality (particularly for sizing) depends on the deterioration properties of the repair material being considered. To further investigate the effect of the deterioration parameters, both the initiation phase and propagation phase were studied independently. Based on the results of this work it can be concluded that the optimum inspection technique for detection is relatively insensitive to both the initiation time and the rate of crack propagation, whereas, the inspection quality for the sizing assessment is more sensitive, particularly in relation to the length of the initiation time. The optimum inspection quality for sizing displays an inverse relationship to the length of the initiation time. In addition, a higher rate of crack propagation requires a higher quality inspection technique for sizing up to a point, where failure is likely to occur between inspections (i.e. for long inspection intervals) even if repair was carried out after the previous inspection. At this point, it is not optimal to use higher quality inspection techniques since the reduction in the number of failures is not significant enough to justify the increased inspection cost. In this case, it is recommended that inspections are carried out at shorter intervals.

This methodology also allows owners/managers to choose the maintenance strategy which is most suitable and practical for the structure or group of structures being considered. Constraints such as specified inspection intervals, availability of particular inspection techniques or trained inspectors may have to be considered. On this basis, parameters such as specific inspection techniques or inspection intervals etc. can be inputted in to the maintenance model, and the optimal solution which is within the constraints provided, can be determined. Alternatively, it is possible to determine the optimum maintenance strategy (i.e. inspection interval, inspection techniques) which is recommended by the maintenance management model and then alter this strategy to meet a specific set of constraints/requirements, so that the maintenance strategy is as close as possible to optimal but still meets the constraints/requirements of the owner/manager. The sensitivity of the results (i.e. the expected cost and number of failures) to such changes in the optimal maintenance strategy can also be assessed using the developed methodology. This provides owners/managers with a range of options when deciding upon the most optimal/suitable maintenance strategy for a particular structure or groups of structures.

In addition, this chapter combines all aspects of this thesis including the ability to simulate (i) many forms of deterioration growth (using α and g), (ii) an initiation phase and propagation phase of deterioration, (iii) probabilistic inspection modelling which allows for the determination of the optimum combination of techniques for both the detection stage and sizing stage of an inspection, (iv) the repair of defects using a repair material which is different from the original material with which the structure was constructed and (v) a practical example using the results of the experimental study which was carried out to investigate the efficiency of different repair materials (i.e. OPC, OPC+PFA and OPC+GGBS) in relation to crack propagation. This chapter also demonstrates, along with Chapter 4, the capabilities of the developed methodology and the sensitivity of the results to the different input parameters, achieving objective 7 of this thesis. The results from this chapter emphasise that both the initiation phase and the propagation phase of defect growth must be taken into account where appropriate when considering the deterioration of a structure to determine the optimum inspection interval, optimal inspection techniques and the optimum repair material for a structure.

CHAPTER 7 – CONCLUSIONS

CHAPTER 7 - CONCLUSIONS

7.1. INTRODUCTION

A lot of research has been carried out in the area of BMS since the Intermodal Surface Transportation Efficiency Act (ISTEA) was passed in 1991 outlining that all US states must adopt a BMS. However, the state of the art review in Chapter 2 highlighted some areas which require further research. In relation to inspection modelling, it was recognised by other researchers that there are two stages to an inspection (Rouhan and Schoefs, 2003; Rouhan and Schoefs, 2000; Zhang and Mahadevan, 2000), detection of defects present and sizing of defects, yet this had not been taken into consideration in the literature. The development of a two stage inspection process as part of a BMS was therefore outlined as one of the main aims of this thesis. In addition, based on the findings of the literature search outlined in Chapter 2, the objectives of this thesis were defined as:

1. The development of a maintenance management methodology which optimises inspection planning by taking into account the two aspects of an inspection, detection of a defect and sizing of a defect.
2. To incorporate the ability to consider many different forms of defect growth and different deterioration kinetics (i.e. abrupt and gradual growth, linear and non-linear).
3. To carry out an experimental study to investigate the rate of deterioration in different repair materials (OPC, OPC+PFA and OPC+GGBS) in relation to the rate of crack growth in reinforced concrete.
4. To incorporate the ability to simulate both the initiation phase and the propagation phase of deterioration using a Markov decision process.
5. To incorporate the ability to simulate the repair of a structure using a different material to the original construction material.
6. To demonstrate the capabilities of the developed maintenance management model using a practical example (i.e. using the results of the experimental study).
7. To perform sensitivity studies in the context of the derived methodology to investigate the effect of changes in the input parameters on the resulting maintenance strategies.

The developments and conclusions of this thesis with regard to the objectives outlined are summarised in the following sections.

7.2. TWO STAGE INSPECTION PROCESS

A maintenance management model was developed in this thesis which, as part of a BMS, aims to optimise maintenance strategies by providing owners/managers of structures with a rational, robust and repeatable tool to allocate restricted maintenance budgets. A novel aspect of this work is that a distinction has been made between the two stages of an inspection, to detect and size defects present in a structure. Since each stage of an inspection is carried out for a distinct purpose, different parameters are used to represent each procedure and both have been incorporated into the developed maintenance management model. By separating these two procedures, not only can the optimal inspection interval be determined, but an optimal maintenance management plan can also be developed by choosing the most suitable inspection technique for each stage of the inspection, whether it is for detection or sizing, rather than using the same inspection technique for both procedures. This developed methodology enables the optimisation of this process by the selection of specific methods at each stage of an inspection as a function of many different factors (such as environment, deterioration mechanism, allowable probability of failure), to deliver maximum benefits. The separation of the inspection process of a structure into two stages enables the investigation to study the effect of both stages of the inspection on the expected annual costs, the expected number of failures and the optimal maintenance plan for the structure. For example, depending on the requirements of the owner/manager, it may be more convenient to use a low quality screening technique for detection and a higher quality inspection technique for sizing to assess which defects should be repaired, as was the case for the examples considered in this thesis. The developed methodology allows for the first time the effect of such decisions to be evaluated.

7.3. DIFFERENT FORMS OF DEFECT GROWTH

As part of the maintenance management model, the growth, failure, inspection and repair of defects was simulated using two Markov transition matrices. Between inspections growth and failure can occur whereas at an inspection year growth, repair and failure can occur. Using this developed framework the optimal inspection interval for an infrastructural element/network can be assessed, based on the minimisation of the expected cost (or number of failures) for a specified limit state.

As discussed in Chapter 2, the developed methodology is considered to be material independent. On this basis, the methodology must have the capability to simulate many forms of deterioration mechanism in different materials and environments. The range of possible deterioration is divided into groups, and the deterioration propagation of a defect is modelled using two parameters (α, g) which can be assigned a value for each group. This development was discussed in Chapter 3. One parameter is associated with the deterioration rate of the defect whereas the other is associated with the deterioration kinetics (i.e. the abruptness of the deterioration mechanism). By introducing these two parameters, the defect could deteriorate by one group or by several groups in one year, allowing many forms of gradual and aggressive deterioration behaviour to be considered. In addition, by specifying deterioration parameters for each group, non-linear deterioration propagation could be simulated, as was demonstrated in Chapter 6. Alternatively, the parameters could be fitted to non-linear (or linear) deterioration curves from laboratory or on-site tests. This flexibility in the developed methodology allows owners/managers of structures to accurately simulate the deterioration of many materials in different environments, and develop the most suitable maintenance strategy for the structure or group of structures being considered.

7.4. EXPERIMENTAL STUDY

Since the addition of PFA and GGBS reduce the permeability of a concrete, it may have been assumed that both have similar deterioration characteristics in relation to the rate of crack propagation, although this has not been previously studied in the literature. The results of the experimental study carried out as part of this thesis demonstrate that the rate of crack propagation in OPC+GGBS is actually faster than in OPC and that OPC+PFA has the slowest rate of crack propagation of the three mixes considered as part of this thesis (i.e. OPC, OPC+PFA and OPC+GGBS). By determining the mass loss of steel reinforcing bars at different points in the experiment, the mass loss for a particular crack width and the crack width for a specific mass loss could be compared for the three concrete mixes. A similar mass loss due to corrosion was observed for each of the concrete slabs in the first set of results, yet the variation in crack width for the different concrete mixes was very significant. OPC+PFA displayed the smallest crack width and OPC+GGBS displayed the largest. Similarly, for a specific crack width in the slab the percentage mass loss of the reinforcing bars in the OPC+PFA slabs was much higher than for the other two materials. In conclusion, the mass loss needed to produce a particular crack width is significantly larger for an OPC+PFA concrete. This superior performance of OPC+PFA in relation to concrete cracking is considered to be due to its higher porosity, as discussed in Chapter 5.

This experimental study was carried out using the impressed current method. Since the results of the study were from an accelerated corrosion test, the results had to be converted to real time before being used to develop or compare maintenance management strategies for the different concrete mixes. Once this conversion had been carried out, the rate of crack growth for each of the concrete mixes could be used to determine the growth parameters (α, g) which could then be inputted into the maintenance management model which was developed as part of this thesis, to study the repair efficiency of three materials considered.

7.5. INCORPORATION OF INITIATION PHASE

Since some materials can demonstrate an initiation phase of deterioration (e.g. reinforced concrete, coated steel) it is important to include this behaviour into a maintenance management model when determining the optimal maintenance strategy for a structure or group of structures. In reinforced concrete, during the initiation phase aggressive agents such as chlorides and carbon dioxide diffuse into the concrete cover, which can then lead to corrosion of the reinforcing bars. Therefore, the deterioration of reinforced concrete is a two phase process, which consists of an initiation phase and a propagation phase. It was highlighted in the literature review that one limitation of a Markov process as a part of a maintenance management model is the inability to simulate an initiation phase. Therefore, as part of the methodology developed in this thesis, one of the objectives was to investigate the incorporation of the initiation phase into the deterioration of a material where the propagation phase is simulated using Markov transition matrices. This objective was achieved in Chapter 6, allowing owners/managers of structures to consider the initiation phase of deterioration when planning maintenance strategies. The initiation time can be inputted in years depending on factors such as the environment and the properties of the material. In addition, an initiation time of 0 years can be specified for deterioration modes/mechanisms which do not demonstrate an initiation phase.

7.6. REPAIR USING DIFFERENT MATERIAL

In practice it is probable that structures will be repaired using a material which is different to the original construction material. It is therefore considered important to incorporate the ability to simulate the repair of a structure using a different material. The new repair material may have different deterioration parameters, which alters the optimal maintenance strategy of a structure once it has been repaired. In this thesis, since the experimental study considered the rate of crack growth in different concrete mixes, it was assumed that patch repairs were carried out in the example considered, and that a defect which develops in that specific critical/hotspot location would deteriorate according to the deterioration characteristics of the repair material. Therefore in Chapter 6, once defects are repaired, different parameters could be specified for the new repair material, and the deterioration of defects in this new material are simulated using these deterioration parameters (i.e. for initiation phase and propagation phase). On this basis, the efficiency of different repair materials could be studied, allowing the owner/manager of a structure to select the optimum repair material and optimum maintenance strategy for the structure, limit state and environment being considered.

7.7. PRACTICAL EXAMPLE

It was considered important to demonstrate the capabilities of the developed methodology and to incorporate all of the advances and achievements of the thesis into a practical example. Since the experimental study considered the rate of crack growth in different concrete mixes, a reinforced concrete example was considered. By fitting the deterioration parameters of the developed maintenance management methodology to the experimental curves of the rate of crack propagation, the repair efficiency of the three concrete mixes could be compared. The bi-linear rate of crack propagation for OPC emphasises the importance of the ability to simulate non-linear behaviour.

Based on information from the literature, initiation times for each of the concrete mixes were estimated and inputted into the model. The efficiency of the three repair materials was compared in relation to the expected cost and number of failures. The results demonstrated that the expected cost and number of failures depend on the initiation phase and the propagation phase, emphasising that both must be considered when planning a maintenance strategy. For the reinforced concrete example considered, OPC+GGBS resulted in the lowest cost for an inspection interval of up to 4 years and the lowest number of failures for an inspection interval of up to 2 years, even though it had the highest rate of crack propagation. OPC+GGBS did however have the longest initiation time. Otherwise, OPC+PFA had the lowest cost ($\Delta T \geq 5$years) and the lowest number of failures ($\Delta T \geq 3$years). The sensitivity of the results to the estimated initiation time was also studied.

Using the developed methodology, it was also possible to determine the optimum inspection interval and optimum inspection quality for both stages of an inspection for each of the materials. The results also demonstrate that the optimum combination of inspection techniques depends on both the initiation phase and the propagation phase of the material being considered, particularly in relation to the sizing assessment. Therefore, when selecting the inspection techniques for an inspection (in particular for the sizing assessment), the deterioration properties of the specific material being considered should be taken into consideration. The sensitivity of the optimum inspection quality for detection and sizing to the length of initiation phase and the rate of defect propagation was also studied.

7.8. SENSITIVITY STUDIES

The sensitivity studies which were carried out as part of this thesis give the owner/manger of a structure the ability to investigate the effect of changes in the input parameters of the model on the overall resulting optimal maintenance strategy (i.e. optimum inspection interval, optimum combination of inspection techniques) and on the expected total cost or number of failures. For example, if there is a change in the environmental conditions surrounding a structure, or if there is a change in usage of the structure resulting in a different limit state being considered, this may have a significant impact on the resulting maintenance strategy.

Initial sensitivity studies were carried out in Chapter 4 to investigate the effects of changes in the modelling parameters (i.e. the number of groups chosen), the growth parameters, the mode of failure and the limit state. The results indicated the importance of accurately defining these parameters. In relation to the deterioration kinetics (i.e. abruptness of the defect growth), the sensitivity study demonstrated that an increase in the inspection quality does not always result in a lower number of failures or a lower cost (e.g. for very abrupt defect growth) and in such cases increasing the inspection quality would be an inefficient use of limited financial resources. It would be more efficient for the owner/manager of the structure to schedule inspections more often to reduce the possible number of failures. When considering the mode of failure, the results illustrated that for a particular allowable probability of failure, sudden failure modes resulted in more failures than progressive failure modes, and that the number of failures also depends on the critical defect size (e.g. as specified by the owner/manager of a structure depending on importance of the structural component). The sensitivity studies carried out also demonstrated that a higher inspection quality is needed for ULS than SLS and that the allowable probability of failure is directly proportional to the optimum inspection interval. Again, these studies illustrated the importance of accurately estimating these parameters to achieve meaningful results from the maintenance model.

Further sensitivity studies were carried out and discussed in Chapter 6 in relation to the effect of the initiation phase and propagation phase on the optimal maintenance strategy, the expected cost and the expected number of failures. The results of these studies indicated that (i) the optimal inspection interval is primarily influenced by the rate of crack propagation, (ii) an increase in the initiation phase can result in a significant reduction in the expected total cost and number of failures and (iii) the optimum inspection quality (in particular for sizing) is affected by both the initiation phase and propagation phase of deterioration.

However it was also recognised that owners/managers of structures are often faced with constraints when deciding on a maintenance strategy, such as specified inspection intervals or available inspection techniques, and that the optimal maintenance strategy which is recommended by a BMS is not always practical. This methodology provides owners/mangers with a tool to find a maintenance strategy which is both efficient and within these specific constraints. The developed maintenance management model can recommend the optimal maintenance strategy in terms of the optimal inspection interval and optimal quality of inspection techniques for detection and sizing. However, based on the sensitivity studies which have been carried out, the implications of changes to the optimal maintenance strategy (i.e. inspection interval and inspection quality) can be investigated, and used to find the most convenient maintenance strategy for a particular structure/group of structures. Alternatively, an owner/manager can specify which inspection techniques are available and the maintenance management model can determine which combination of inspection techniques is optimal from the techniques available (i.e. for detection and sizing), and the corresponding optimal inspection interval. Similarly, for a specific inspection interval, the optimal combination of inspection techniques can be determined. On this basis, owners/managers can be provided with a range of maintenance solutions, allowing the most convenient maintenance strategy to be chosen for the constraints/requirements associated with a particular structure or group of structures being considered.

7.9. SUGGESTIONS FOR FUTURE WORK

Based on the developments of this thesis, there are areas where further research is recommended:

1. An important further development would be to relate the inspection parameters (i.e. PoD/PFA, PGA/PWA) to specific inspection techniques, allowing an inspection tool to be recommended rather than just an inspection quality. An interesting further development would also be to optimise the inspection quality for a structure or group of structures in terms of the importance of a particular route by considering the connectivity of the network.

2. Calibration of parameters which were introduced as part of this model is also recommended, particularly in relating the parameters of the Weibull distribution (which are used to model the probability of failure) to the actual mode of failure

being considered (e.g. sudden or progressive failure) and investigating the most suitable number of defect groups (and associated parameters) for the deterioration rate/mechanism being considered.

3. The development of more accurate costing models including not only direct costs such as inspection, repair and failure costs, but also indirect costs such as user delay costs (which could be related to the importance of a route, as discussed in point 1) and environmental costs. Also, to quantify the cost of different repair options using different materials.

4. To incorporate the ability to repair to any deterioration group and not just to the original condition of the structure (or smallest group), which would allow the optimum repair efficiency for a structure or component to be determined. In addition, inferences regarding the extent of inspections could also be optimised for particular NDTs considering spatial variability due to material properties and environmental conditions.

5. To carry out further experimental studies to determine the effect of the percentage addition of PFA and GGBS on the rate of crack propagation and to optimise the percentage not only with respect to the initiation time but also with respect to propagation behaviour.

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APPENDIX A – LIST OF JOURNAL AND CONFERENCE PAPERS

Journal papers (published/in press):

Breysse, D., Elachachi, S., Sheils, E., Schoefs, F. and O'Connor, A. (2008). "Life cycle cost analysis of ageing structural components based on non destructive condition assessment." *Australian Journal of Structural Engineering*, in press.

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Journal papers (under review):

Sheils, E., O'Connor, A., Breysse, D., Schoefs, F. and Yotte, S. (2009). "Development of a two stage inspection process for the assessment of deteriorating bridge structures." *ASCE Journal of Bridge Engineering*, under review.

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Conference papers:

Sheils, E., Schoefs, F., Breysse, D. and O'Connor, A. (2007) "Comparing efficiency of systematic and conditional maintenance for randomly ageing components." *Proceedings of the 10th International Conference on Applications of Statistics and Probability in Civil Engineering (ICASP10)*, CD-ROM Proceedings, Tokyo, Japan.

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APPENDIX B – ASCE JOURNAL OF BRIDGE ENGINEERING PAPER (UNDER REVIEW)

Development of a two stage inspection process for the assessment of deteriorating bridge structures

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ABSTRACT

Inspection based maintenance strategies can provide an efficient tool for the management of ageing bridge structures subjected to deterioration. Many of these methods rely on quantitative data from inspections, rather than qualitative and subjective data. The focus of this paper is on the development of an inspection based decision scheme, incorporating analysis on the effect of the cost and quality of non-destructive technique (NDT) tools to assess the condition of a bridge structure over its lifetime. For the first time the two aspects of an inspection are considered separately, i.e. detection and sizing. Since each stage of an inspection is carried out for a distinct purpose, different parameters are used to represent each procedure and both have been incorporated into a maintenance management model. The separation of these procedures allows the complex interaction between the two inspection techniques to be studied thereby providing the owner/manager with a decision tool to select the optimum combination of techniques for each phase of the assessment based upon specific budgetary and performance requirements.

Keywords: Inspection, Maintenance, Markov process, Cost analysis, Optimization

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Introduction

Due to the extent of deteriorating infrastructure in the U.S. (about 5,000 bridges become classed as deficient each year), the estimated cost of rehabilitation and repair has been estimated at $1.3 trillion (Enright and Frangopol 1999). "The federally mandated biennial inspection interval is not the most cost-effective maintenance strategy for bridges (Soltani 1995), and bridge repairs are not always performed with life-cycle cost effectiveness in mind" (Enright and Frangopol 1999). As a result, over the last decade a lot of research has been conducted into optimisation of the existing infrastructural resource to develop methods of maintenance management which consider the dual constraint of optimal maintenance budget while maximising efficiency for the required remaining service life (Estes and Frangopol 1999, Faber and Sorensen 2002, Kong and Frangopol 2004, Kong and Frangopol 2005, Lauridsen et al 2006, O'Connor and Eichinger 2006, O'Connor and Enevoldsen 2006, O'Connor and O'Brien 2005, Radojicic et al. 2001, Stewart 2005, Stewart and Mullard 2006). The main objective is to find the optimal maintenance management plan, thereby optimizing the life-cycle cost of the structure. Many of these methods rely on quantitative data from inspections, rather than qualitative and subjective data. Consequently, monitoring and inspections are key aspects in this process (Corotis et al 2005) as the information from these tests can be used to update deterioration models and to derive the optimal economic maintenance strategy for the remaining lifetime of the structure.

The main focus of this paper is on the development of an inspection based decision scheme, incorporating analysis of the effect of the cost and quality of NDT tools to assess the condition of a bridge structure over its lifetime. Two aspects of an inspection, i.e. detection and sizing are considered. The aim is not to compare existing strategies but to suggest a new systematic approach which facilitates quantification the cost and predicts the required maintenance budgets as a function of time. There have been many studies which focus only on the detection stage of an inspection, using various sets of parameters such as Probability of Detection and Probability of False Alarm (Schoefs and Clement 2004, Rouhan and Schoefs 2003), Probability of Detection and Probability of False Indications (Straub and Faber 2003) or Probability of Detection and False Call Probability (Chung et al. 2006, Zhang and Mahadevan 2001) to assess the quality of a particular inspection method. In this study a distinction has been made between an inspection carried out to detect a defect, and an inspection carried out to size a defect. Since each stage of an inspection is carried out for a distinct purpose, different parameters are used to represent each procedure and both have been incorporated into a maintenance management model. In this way, the owner/manager of the infrastructural element/network can assess the implication of the choice of combinations of assessment techniques (for detection and sizing) on the predicted number of failures (i.e. exceedance of specified limit states) within the network. Given that each technique has an associated cost, where higher accuracy implies higher cost, an efficient budgetary planning tool is provided which balances the requirements of safety with assessment/maintenance expenditure.

As part of the new process the first part of an inspection is concerned with the detection of existing defects. The Probability of Detection (PoD) and the Probability of False Alarm (PFA) are used in this study, for a particular NDT tool used in the assessment, to indicate the quality of the inspection method for detection. The second part of an inspection deals with the assessment of the size of the defect knowing that it has already been detected. For this part of the analysis, two new parameters are introduced, Probability of Good Assessment (PGA) and Probability of Wrong Assessment (PWA). In this context it has been necessary to introduce a distinction between good and wrong sizing assessments that lead to repair (PGA_R, PWA_R), and those which lead to no repair (PGA_{NR}, PWA_{NR}).

Using the methodology developed in Rouhan and Schoefs (2003), an events based decision theory is subsequently introduced to look at the effects of an individual good/bad inspection performance. Based on the inspection results, for detection or sizing, a decision is made as to whether further inspection should be carried out, or to repair. For evaluating the cost of the system, and to find the optimum costs, it is useful to investigate whether the decision to carry out a sizing assessment or a repair is correct/incorrect. On this basis, a decision scheme is introduced which considers four inspection events for each of the two stages of an inspection. The probability of these events are evaluated using Bayes Theorem and are subsequently introduced as parameters into cost functions which are used to investigate the effect of cost overrun due to inaccurate inspection results.

In addition to this, for a particular set of input parameters the optimum time between inspections, which results in the lowest annual cost of a particular bridge structure, is determined. By varying the quality of the inspection techniques, the sensitivity of the optimal inspection time to changes in the modelled parameters is assessed, allowing the optimum combination of techniques to be determined for the constraint of optimisation of performance with respect to available budget.

Probabilistic Modelling of Inspection Results

When carrying out an inspection, the information obtained is just a relative estimation of what is present in reality, e.g. cover, electrical potential etc. Therefore, due to the inherent uncertainty associated with inspections, many of the variables involved are modelled stochastically, and the simulation of inspection results should be performed in a probabilistic sense. Given the size of the defect, and the inspection method being used, there is a certain probability of detection (Madsen et al. 1987, Faber and Sorensen 2002, Onoufriou and Frangopol 2002). On this basis, probabilistic methods are described below which are used to model inspection results for detection and sizing assessment, taking this uncertainty into account. Note that the quantification of the on-site performance of inspection is difficult. Generally specific inter-calibration campaigns are needed as were initiated in the offshore field during the ICON project (Barnouin 1993). Other recent works provide data for the probability of detection of the corrosion initiation in concrete (Bonnet et al. 2008, 2009), the probability of detection and false alarm for uniform (Schoefs et al. 2008, 2009) or localized (Pakrashi et al., 2008) corrosion of steel structures. Expert judgment can also be introduced in this regard (Boéro et al. 2008).

Stage 1 - Detection

It is assumed that every ΔT years, an inspection is carried out. The first part of an inspection is concerned with the detection of existing defects. For an individual defect, it is assumed that detection of a defect by the first inspection leads to a further inspection to assess the size of the defect, and that no detection leads to no further action. In this study, the Probability of Detection (PoD) and the Probability of False Alarm (PFA) are the parameters chosen to indicate the quality of an inspection method for detection and are used to assess if a defect will be detected or not when an inspection is carried out, Figure 1. The PoD is the probability that a defect is detected by the inspection, given that a defect is present, Equation 1, and the PFA is the probability that a defect is detected by the inspection, given that no defect greater than the detection threshold is present, Equation 2.

$$\text{PoD} = P\left(\hat{d}_1 \geq d_{min} \mid d \geq d_{min}\right) \quad (1)$$

$$\text{PFA} = P\left(\hat{d}_1 \geq d_{min} \mid d < d_{min}\right) \quad (2)$$

The results of an inspection (and the ability of a method to detect a defect) depend on many different factors, such as the NDT method, the detection threshold (d_{min}), the environment and several conditions of the structure (Breysse et al. 2007), the skill/experience of the operator, the characteristics of the defect, the detection mechanism and primarily on the size of the defect. For a given test, the PoD depends on the defect size (for example the average defect size), the detection threshold and noise. The PFA, however, is independent of the size of the defect and, therefore, depends only on the detection threshold and noise.

Stage 2 - Sizing Assessment

The second part of an inspection deals with the assessment of the size of a defect. This assessment is only carried out if the previous inspection (i.e. performed for detection) has indicated that a defect exists. For this analysis, two new probabilities are defined, the Probability of Good Assessment (PGA) and the Probability of Wrong Assessment (PWA). A repair of the defect is carried out if the inspection indicates that the size of the defect is greater than the critical defect size d_c. The value of d_c will be fixed by the bridge owner/manager, depending on the safety level he/she must/wants to ensure and limited by budgetary constraints. It can for instance be related to the annual probability of failure. There is also a distinction made between good and wrong assessments that lead to repair (subscript $_R$), and those which lead to no repair (subscript $_{NR}$), Equations 3-6, Figure 2.

$$\text{PGA}_R = P\left(\hat{d}_2 \geq d_c \mid d \geq d_c \;\&\; \hat{d}_1 \geq d_{min}\right) \quad (3)$$

$$\text{PGA}_{NR} = P\left(\hat{d}_2 < d_c \mid d < d_c \;\&\; \hat{d}_1 \geq d_{min}\right) \quad (4)$$

$$\text{PWA}_R = P\left(\hat{d}_2 \geq d_c \mid d < d_c \;\&\; \hat{d}_1 \geq d_{min}\right) \quad (5)$$

$$\text{PWA}_{NR} = P\left(\hat{d}_2 < d_c \mid d \geq d_c \;\&\; \hat{d}_1 \geq d_{min}\right) \quad (6)$$

Events Based Decision Theory

As described in Rouhan and Schoefs (2003), an events based decision theory can be used to look at the effects of a good/bad inspection performance. Since there can be various sources of error when performing an inspection, it is useful to investigate the probability that each of the decisions taken (e.g. to carry out a further assessment for sizing or to repair) are correct/incorrect. In this study, a similar method is implemented for detection and sizing, considering four inspection events for each of the two stages of an inspection. The probability of these events are evaluated using Bayes Theorem and are subsequently introduced as parameters into cost functions which are used to investigate the effect of cost overrun due to inaccurate inspection results.

Firstly, in the case of an inspection to detect a defect, a decision on whether to carry out a further assessment is made based on the inspection result, $\hat{d}_1$. It is assumed that detection of a defect by the first inspection leads to a further inspection to assess the size of the defect, and that no detection leads to no further action. This decision on whether or not to carry out a further assessment can never be taken with certainty, and the level of uncertainty depends on the quality of the inspection and the level of the other sources of noise associated with the inspection. To assess this risk, four events are defined for the detection stage of an inspection, labelled E_{1D}, E_{2D}, E_{3D} and E_{4D} respectively. For consistency in sizing assessment, the same methodology is employed. Again, this decision on whether or not to carry out a repair can never be taken with certainty. Therefore, four events are also defined for the sizing assessment stage of an inspection, E_{1A}, E_{2A}, E_{3A} and E_{4A}.

Development of Maintenance Management Model

When managing a bridge structure or a group of bridge structures it is important to be aware of and to have an accurate estimate of the growth of the population of defects present in the structure over time. Assuming that the state of the structure in each time period only depends on the state of the structure and the action applied to it in the preceding period (Ang and Tang 1975), a Markov process may be employed to simulate the growth/evolving deterioration and repair of a population of defects over time (Scherer and Glagola 1994, Roelfstra et al. 2004). A Markov decision process can be a useful tool for controlling and finding the optimal strategy when managing a large scale system (Orcesi and Crémona 2006, Micevski et al. 2002, Poinard et al. 2003).

For the purpose of this assessment the total range of defect sizes is broken into N defect groups (where in reality the ranges of defect size will be function of the particular deterioration mechanism considered e.g. concrete cracking due to chloride induced corrosion, alkali silica reaction or fatigue etc.) and a record is kept each year of the number of defects within each group. Based on the growth rate, and the kinetics of the growth, the probability of moving from one defect group to a larger defect group is assessed, all of which are a function of the particular deterioration mechanism under consideration or which is being monitored/assessed. It is assumed that inspections are carried out every ΔT years. Having ΔT as a parameter of the optimisation has the advantage that (i) in the case where a federally mandated inspection frequency is specified, e.g. $\Delta T = 2$ years then the optimum combination of NDT techniques can be chosen, for inspection phase 1 (i.e. detection) and phase 2 (i.e. sizing), which minimise the probability of failure (i.e. limit state exceedance) with respect to assessment budget or (ii) facilitates determination of the optimal inspection interval ΔT for a particular deterioration mechanism based upon the available inspection budget (and associated budget for the two phases of this inspection), or upon available NDT techniques.

Two Markov matrices are required (size NxN) by the process, one to simulate the growth, repair and failure of the defects at an inspection year, and another to simulate the growth and failure of the defects between inspections (Corotis et al 2005). It is assumed that defects return to the smallest group, e.g. to their initial size after failure/repair. Therefore, at an inspection year, the first column in the matrix is controlled by the probability of repair and the probability of failure given that no repair is carried out, whereas, between inspections, this column is controlled by the probability of failure alone.

For the simulation of the growth of the defects, the objective is to develop the upper triangular part (growth part) of the Markov transition matrix of size NxN using the specified growth parameters. There are two parameters in the model which define the growth of a defect over time and from which all transition probabilities for the growth matrix. The first is α, which describes the growth rate of a defect, which therefore controls how quickly a defect moves from one defect group to the next. The other parameter is g, which determines how gradual or sudden the growth of an individual defect is, Figures 3. Both parameters can be seen to be functions both of the deterioration mechanism under consideration and its agressivity and also of the structure mechanical characteristics (i.e. in a reinforced concrete bridge the rate of crack opening will also be a function of the modulus of rupture of the concrete, while the time to initiation of cracking and of reinforcement corrosion will be a function of the permeability of the concrete, where blended cements will have considerable lower permeability than standard OPC concretes).

To simulate failure between inspections, for each defect group, the probability of failure is calculated to assess the probability that a defect will fail and subsequently be repaired, and will therefore return to the smallest defect group. The annual probability of failure, p_f, is calculated using the Weibull cumulative distribution function (Weibull 1951, Ang and Tang 1984), based on the mean size of the defects in each group, $\bar{d}_i$, Equation 7,

$$p_f(\bar{d}_i) = 1 - \left[\exp - \left(\frac{\bar{d}_i - d_1}{d_{ref\_pf}} \right)^m \right] \qquad (7)$$

Over the lifetime of a bridge structure, inspections and repairs are carried out and, in some cases, failure can occur. At an inspection year, it is assumed that failure will only occur if a repair is not carried out. Therefore, the first column of the Markov matrix is calculated using a combination of the probability of repair, the probability of failure given that no detection has occurred, and the probability of failure given that no repair has been carried out.

To calculate the probability of repair of defects in each group it is necessary to assess the PoD/PFA and PGA/PWA for each group. The PoD and PFA are estimated for each defect group, given the mean and standard deviation of the defects in the group and the quality of the inspection method assumed to be being used, Q_1. It is assumed that the defect size and the noise are normally distributed and non-correlated, and that for detection the quality (and hence cost) of the inspection method is related to the distribution of the noise, σ_{ND}, Equation 8. This allows the owner/manager to take into account the relative costs, reliabilities and capabilities of different inspection methods.

$$\frac{\sigma_{ND}}{d_{ref}} = \frac{1}{Q_1} \qquad (8)$$

Similarly, for each defect group the values of PGA and PWA are estimated, given the mean and standard deviation of the defects in the group and the quality inspection method being used for sizing. It is assumed for assessment also that the quality of the inspection method is related to the distribution of the noise, σ_{NA}, Equation 9,

$$\frac{\sigma_{NA}}{d_{ref}} = \frac{1}{Q_2} \quad (9)$$

When the first inspection is carried out to detect a defect, i.e. as a screening exercises, there can be two decision outcomes. One is to carry out a further assessment, and the other is to do nothing. Similarly, when a second inspection is carried out to assess the size of a defect, there can also be two decision outcomes. One is to repair, in which case the defect returns to the initial defect group, and the other is to carry out no repair. Figure 4 represents this decision process schematically. If at the detection stage, no further assessment is carried out, or if at the assessment stage, no repair is carried out, there is still a remaining probability of failure (which will be larger in the second case as the defect is larger). Similar to the event of repair, if failure occurs the defect returns to the initial defect group. These probabilities are calculated analytically, and are used to calculate the first column of the Markov matrix.

Once both matrices have been formulated, they are used to simulate the growth and repair of a population of defects over time. Each defect group is assumed to have an initial population of defects. Using this methodology, the number of defects in each group is calculated on a yearly basis using the relevant Markov matrix, and the number of defects in each group from the previous year. The model is run until the number of defects in each group reaches a steady state. The stabilised number of defects in each group is then used to calculate the expected annual total cost of the structure.

Cost Functions

In the proposed methodology the expected annual cost of inspections ($E(C_{I\_TOTAL})$), repair ($E(C_{R\_TOTAL})$) and failure ($E(C_{F\_TOTAL})$) are considered, which are summed to find the expected annual total cost of the structure ($E(C_{TOTAL})$). Once the stabilised number of defects in each group has been determined, the expected number of inspections, repairs and failures each year can be calculated by multiplied by the probability of inspection (for detection and sizing), the probability of repair and the probability of failure (between inspections and at an inspection year), respectively.

The cost of an individual inspection, which is directly proportional to the quality of the inspection, is calculated using Equation 10, for inspection 1 (i.e. detection), and Equation 11 for inspection 2 (i.e. sizing). The expected total cost of inspections is the product of the number of inspections and the cost of an individual inspection, which is directly proportional to the quality of the NDT technique selected/prescribed.

$$CI1 = C_o k_I \, {Q_1}/{Q_{ref}} \qquad (10)$$

$$CI2 = C_o k_I \, {Q_2}/{Q_{ref}} \qquad (11)$$

Similarly, the cost of an individual repair (for each group) is calculated, Equation 12, and is then used to find the expected total cost of repair. The cost of an individual repair is proportional to the size of the defect being repaired.

$$CR_i = C_0 k_R \left(\frac{\bar{d}_i}{d_{ref}} \right) \qquad (12)$$

In this study, the cost of an individual failure at an inspection year is equal to the cost of an individual failure between inspections. This cost is calculated using Equation 13.

$$CF = C_o k_F \qquad (13)$$

Knowing the number of failures at the detection stage, the number of failures at the sizing assessment stage (at an inspection year) and the total number of failures between inspections, the expected total cost of failure is calculated. The expected total cost of the structure is calculated by summing these costs. These cost are calculated over one ΔT cycle, therefore, the expected annual costs, ($E(C_{I\_TOTAL})$), ($E(C_{R\_TOTAL})$), ($E(C_{F\_TOTAL})$) and ($E(C_{TOTAL})$) are found by dividing by ΔT.

Results

Using this new methodology, a maintenance management model has been developed, an example of the application of which is presented here. For the purpose of this example, the possible range of defect sizes was subdivided into 10 groups, each modelled statistically with an assumed mean and standard deviation, as outlined in Table 1. These values could be taken to represent the results of a survey performed to measure the magnitude of e.g. fatigue cracks. The standard deviation of the groups represents the scatter of the range of actual sizes of the defects in the group, and is not related to the error in sizing of a defect. It was assumed that the standard deviation of the range of defect sizes in a group is independent of the mean defect size of the group, and a constant value was assumed for each group. In addition, it was assumed initially that there were 100 defects in the smallest defect group, and no defects in all other groups (i.e. taken to represent a new bridge structure), Table 1, although the methodology can consider a structure at any stage of its life.

Initially, for each value of ΔT, the two Markov matrices were used to calculate the stabilised number of defects in each group (directly before inspection or failure), for each year in the ΔT cycle, for a given set of input parameters. The methodology outlined was then used to determine the optimum time between inspections, and subsequently to analyse the effect of the interaction of the quality of the inspection techniques for detection and sizing on the optimum time between inspections and the expected annual total costs of the bridge structure. The effect of the quality of inspections on cost overrun, such as unnecessary repairs, was also investigated using the events based decision theory described. This provides a powerful decision tool for bridge owners/managers in optimising maintenance budget spend as based upon available funds, structural form, deterioration mechanism, environment and limit state considered, it empowers them, rather than performing inspections on e.g. a bi-annual basis, using a range of NDT tools to identify the best combination of techniques to be employed in assessing condition and associated probability of failure (i.e. limit state exceedance).

Optimal Time between Inspections

Table 2 shows the set of parameters assumed in the model for the purpose of this exercise. Using these parameters, the optimum time between inspections is determined on the basis of the minimum expected annual total costs of the bridge structure, ($E(C_{TOTAL})$), which were assessed according to the cost functions outlined in Section 4.6. It is noted here that these are theoretical parameters selected to demonstrate the operability of the methodology. Research is ongoing to identify realistic parameters to be employed based upon the NDT tool, deterioration mechanism, limit state, cost etc (Barnouin 1993, Bonnet et al. 2008, 2009, Schoefs et al. 2008, 2009, Pakrashi et al., 2008, Boéro et al. 2008). Figure 5 shows the results of the analysis, illustrating that, for the case considered, a period of 4-years represents the optimum inspection interval.

As illustrated in Figure 5 the inspection interval has a significant effect on the expected annual total inspection cost, ($E(C_{I\_TOTAL})$), and the expected annual total failure cost, ($E(C_{F\_TOTAL})$). The expected total inspection cost ranges from 60% of the total cost for a 1 year inspection interval, to just 10% of the total cost for a 10 year inspection interval. As expected, an inverse trend emerges for the total failure cost, with the expected total failure cost ranging from just 1.4% of the total cost at a 1 year inspection interval, to 48% of the total cost for a 10 year inspection interval.

The expected total cost of repair, ($E(C_{R\_TOTAL})$), has a significant effect on the expected annual total costs, contributing to 58% of the total cost at the optimal inspection interval ($\Delta T=4$). However, Figure 5 demonstrates that the expected total cost of repair is relatively insensitive to the inspection interval. This is due to the incorporation of the sizing assessment into the analysis, as the second stage of an inspection. Using this methodology it is possible to determine the extent of each repair at the time of an inspection, and to estimate the cost of repair based on the size of the defect, according to Equation 12. For example, if inspections are carried out annually, then it is assumed that large defects are unlikely to develop, and only minor repairs are carried out every year. Whereas if inspections are only carried out every 10 years, it is assumed that quite extensive repairs will be necessary due to larger defects, but these repairs are less frequent. Therefore, there is just a 15% difference between ($E(C_{R\_TOTAL})$) for $\Delta T=1$ and ($E(C_{R\_TOTAL})$) for $\Delta T=10$.

Inspection Quality

Using the methodology developed, it is possible to look at the interaction of the inspection methods for detection and sizing, and see how this affects the optimum inspection interval and the expected annual total costs. This provides the owner/manager of a bridge structure with a useful decision tool when selecting a combination of inspection techniques to be used as part of a maintenance management plan.

Figures 6-7 illustrate how a different combination of inspection techniques can affect the optimal maintenance management plan, and the expected annual costs of the bridge structure. In relation to the first inspection, a higher quality technique, Q_1, reduces the noise associated with the inspection procedure, and therefore, more accurately determines which defects should be further assessed, which consequently reduces the number of failures due to undetected defects. Figure 6 illustrates a direct relationship between the inspection quality for detection and the optimal inspection interval. A similar trend emerges when the quality of the second technique, Q_2, is increased. A better technique reduces the number of failures, as a higher proportion of defects are sized correctly and repaired when necessary.

The owner/manager has a number of options when using this new decision tool. In the case where a convenient inspection interval has been decided upon or specified by authorities, Figure 6 can be used to find a combination of technique qualities with this inspection interval as optimal. Although this can result in a multiple of combinations of techniques, Figure 7 can then be used to determine which of these combinations results in the lowest expected annual costs. For example, if an inspection interval of 4 years is convenient, from Figure 6, there are 6 different combinations of techniques which would be suitable. These 6 combinations are listed in Table 3, with the expected annual total costs for each combination, which are illustrated in Figure 7. In this case, the second option (Q_1=10, Q_2=10) results in the lowest relative cost, and is clearly the most cost efficient combination of techniques for the chosen inspection interval of 4 years.

Alternatively, if the inspection for detection is chosen initially, Figures 6-7 can be used to choose a suitable inspection technique for the sizing assessment. For each inspection quality available for the second assessment it is possible to determine the optimal inspection interval, Figure 6, and the expected annual total costs of the bridge structure, Figure 7. Depending on the structure, it may be more convenient to carry out inspections less often (depending on intangible costs/benefits that have not been incorporated into this model), even though the relative expected annual total cost is higher. Table 4 details a list of options available for a specific inspection quality for detection, Q_1=10. This method interestingly points out that there are two available options (for the quality of the second inspection) that result in an optimal inspection interval of 5 years, yet one option has a relatively lower expected annual total cost than the other, clearly showing it to be the most cost efficient choice. By looking at this interaction of inspection techniques for detection and sizing assessment, an owner/manager can clearly pick the optimum combination of techniques to suit a particular set of requirements.

Furthermore, using the events based decision theory outlined in Section 3, the effect of the quality of inspections on cost overrun, such as unnecessary repair, can also be investigated. Figure 8 illustrates that the total cost of repair is relatively insensitive to the quality of the second inspection, although it is clear that the inspection quality affects the relative breakdown of these costs into necessary and unnecessary repairs. The number of repairs carried out depends on the number of defects that are sized and are found to be greater than the critical defect size, therefore, the cost overrun of unnecessary repairs reduces as more accurate inspections for sizing are carried out. By using a better quality technique, the defects that could lead to failure of a component are repaired, rather than defects that are incorrectly sized, and are not in need of repair. Reducing the number of failures within the bridge structure has the effect of increasing the optimal inspection interval. As discussed previously, although a higher inspection quality results in an increase in the expected annual cost of a bridge structure, Figure 7, the optimal inspection interval is likely to also increase, Figure 6, which can be more convenient for an owner/manager of a structure.

Conclusions

This paper presents bridge owners/managers with a decision tool based upon the subdivision of the assessment process into two phases, (i) detection and (ii) sizing, which can be used in optimal management of infrastructural elements/networks to minimise the probability of failure (i.e. limit state exceedance) within budgetary constraints.

The paper demonstrates that the choice of inspection techniques for detection and sizing has a significant influence on the optimum time between inspections, and hence the minimum annual total cost of the bridge structure. When carrying out an inspection there are two points of interest, the presence of a defect, and the size of a defect present. Since each stage of the inspection has a different purpose, it is necessary to separate these procedures to accurately model an inspection process which is to be incorporated into a maintenance management plan.

The separation of the inspection process of a bridge structure into two stages enables the investigation to study the effect of both stages of the inspection on the expected annual costs of the bridge structure, and the maintenance plan for the structure. The separation of these procedures and the interaction of the two inspection techniques have not previously been considered.

By modelling the two stages of an inspection as separate procedures, using different parameters, the effect of different combinations of techniques can be investigated. The detection process is similar to a screening exercise to determine which defects require further assessment. By producing decision tools similar to Figures 6-7, it is possible to look at the relative benefits of using different quality techniques. Depending on the requirements of the owner/manager, upon the structure considered, its environment and deterioration mechanism, it may be more convenient to use a low quality screening technique for detection and a higher quality inspection technique for sizing to assess which defects should be repaired. The developed methodology allows for the first time, the effect of such decisions to be evaluated quantitatively both with regard to performance (i.e. probability of limit state exceedance) and budgetary cost (i.e. cost of assessment campaign and associated good/bad decisions). The combination of techniques used during an inspection is demonstrated to effect the optimal time between inspections. If an inspection requires partial or total closure of a bridge structure, which can lead to user delays, the owner/manager may prefer to incur a higher annual total cost in return for a longer optimal inspection interval. The results of each combination of techniques can be assessed quantitatively with reference to Figures 6-7, allowing the most cost efficient approach for a given set of requirements to be determined.

Figures 6-7 demonstrates the benefits of the proposed approach. By modelling each stage of an inspection separately, with different parameters, the interaction between these two inspection procedures and the effect of the quality of the individual inspection methods on the optimal maintenance management plan and the annual costs of the bridge structure can be identified.

Finally this methodology can be extended to spatial stochastic fields where inspections can be spatial dependent and the sampling can be different for the kin of inspection. One way to solve this problem, is to base the description of the defect and the error due to NDT on the polynomial chaos expansion (Schoefs 2008)

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Notation

CF = cost of failure for an individual defect

CI1 = cost of an individual inspection for detection

CI2 = cost of an individual inspection for sizing

C_o= initial cost of construction

CR_i = cost of an individual repair

d = actual size of the defect

d_c = critical defect size (a defect size greater than d_c leads to a repair)

di = defect group i

$\overline{d}$ = mean defect size of a group

d_{min} = detection threshold

d_{ref} = reference defect size

d_{ref_pf} = reference defect size for the probability of failure, Weibull law parameter

d_l = limit defect size, Weibull law parameter

$\hat{d}_1$ = size of the detected defect (from inspection 1)

$\hat{d}_2$ = size of the defect from inspection (from inspection 2)

E( ) = annual expectancy of any cost variable

E_{1A} = event 1 for sizing assessment – good sizing, no repair

E_{2A} = event 2 for sizing assessment – wrong sizing, repair

E_{3A} = event 3 for sizing assessment – wrong sizing, no repair

E_{4A} = event 4 for sizing assessment – good sizing, repair

E_{1D} = event 1 for detection – no defect, no detection

E_{2D} = event 2 for detection – no defect, detection

E_{3D} = event 3 for detection – defect, no detection

E_{4D} = event 4 for detection – defect, detection

g = deterioration kinetics parameter

k_F = failure impact coefficient

k_I = inspection cost coefficient

k_R = repair cost coefficient

m = Weibull exponent (to calculate p_f) which determines the spread of the curve

N = total number of groups

NDT= non-destructive technique

PDF = probability density function

PFA = probability of false alarm

PGA_{NR} = probability of a good assessment resulting in no repair

PGA_R = probability of a good assessment resulting in repair

PoD = probability of detection

PWA_{NR} = probability of a wrong assessment resulting in no repair

PWA_R = probability of a wrong assessment resulting in repair

p_f = annual probability of failure

Q_{ref} = reference inspection quality

Q_1 = quality of the inspection method for defect detection

Q_2 = quality of the inspection method for sizing assessment

ΔT = inspection interval in years

α = growth rate of a defect

σd = standard deviation of the defect size in a group

σ_{NA} = standard deviation of noise distribution (for assessment)

σ_{ND} = standard deviation of noise distribution (for detection)

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Table 1. Defect group data used in the model

Defect Group	Range From	Range To	$\overline{d}$	σ_d	Initial Population
d1	0	0.1	0.05	0.02	100
d2	0.1	0.2	0.15	0.02	0
d3	0.2	0.3	0.25	0.02	0
d4	0.3	0.4	0.35	0.02	0
d5	0.4	0.5	0.45	0.02	0
d6	0.5	0.6	0.55	0.02	0
d7	0.6	0.7	0.65	0.02	0
d8	0.7	0.8	0.75	0.02	0
d9	0.8	0.9	0.85	0.02	0
d10	0.9	1	0.95	0.02	0

Table 2. Parameter values used in Markov Maintenance model

Model Properties	Value
Growth rate, α	0.4
Deterioration kinetics parameter, g	3
Reference defect size, d_{ref}	1
Probability of failure exponent, m	3
Limit defect size, d_l	0.4
Reference defect size, d_{ref_pf}	2
Detection threshold, d_{min}	0.35
Quality of inspection for detection, Q_1	10
Mean of noise distribution, n_{mean}	0.3
Critical defect size, d_c	0.62
Quality of inspection for sizing assessment, Q_2	20
Initial construction cost, C_o	1000
Inspection coefficient, k_I	0.01
Reference quality, Q_{ref}	50
Repair coefficient, k_R	0.05
Failure impact coefficient, k_F	1

Table 3. Inspection technique combination 1

$\Delta T=4$		
Q_1	Q_2	$E(C_{TOTAL})$
5	25	446.4
10	10	424.4
15	10	448.4
20	5	473.9
20	10	472.5
25	5	498.4

Table 4. Inspection technique combination 2

$Q_1=10$		
Q_2	ΔT	$E(C_{TOTAL})$
5	3	417.0
10	4	424.4
25	5	467.7
40	5	508.3
50	6	534.9

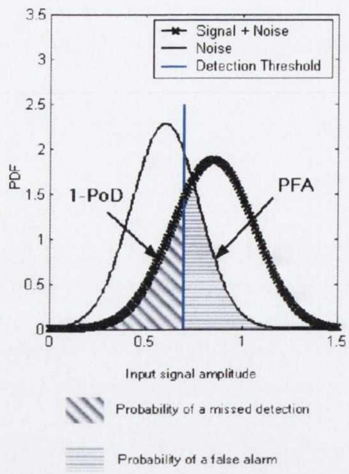

Signal + Noise
Noise
Detection Threshold
PDF
1-PoD
PFA
Input signal amplitude
Probability of a missed detection
Probability of a false alarm

Figure 1

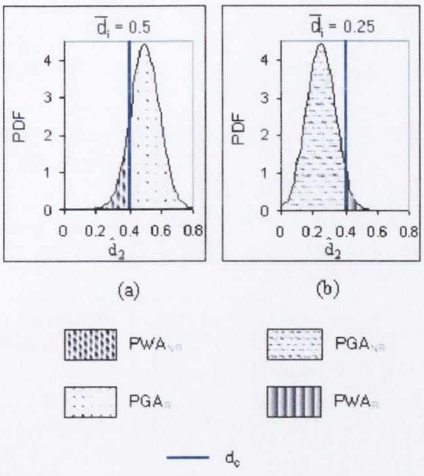
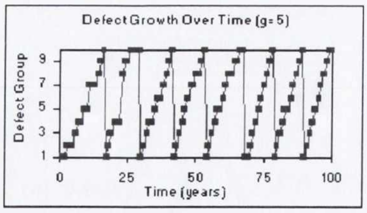
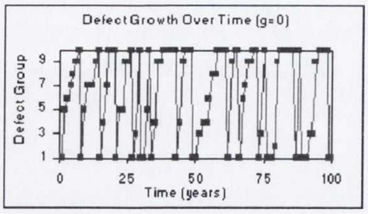


Figure 2



(a) Gradual



(b) Abrupt

Figure 3

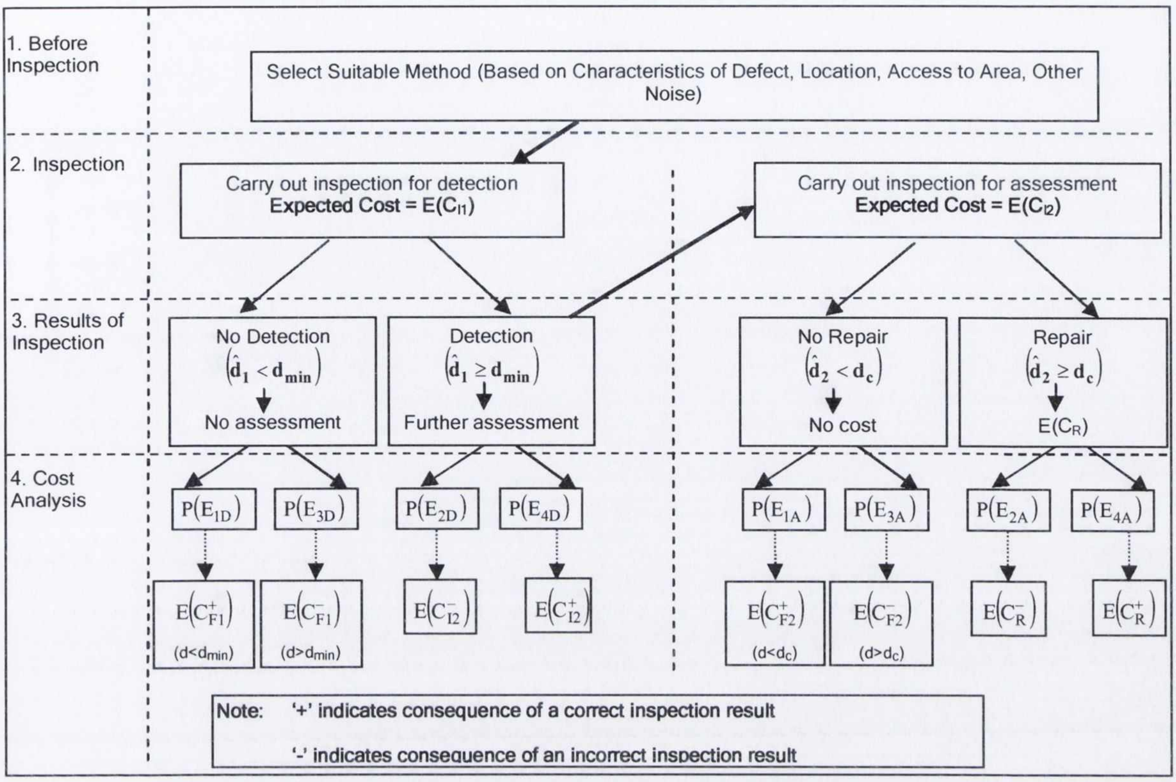

1. Before Inspection
Select Suitable Method (Based on Characteristics of Defect, Location, Access to Area, Other Noise)
2. Inspection
Carry out inspection for detection
Expected Cost = $E(C_{I1})$
Carry out inspection for assessment
Expected Cost = $E(C_{I2})$
3. Results of Inspection
No Detection
$(\hat{d}_1 < d_{min})$
No assessment
Detection
$(\hat{d}_1 \geq d_{min})$
Further assessment
No Repair
$(\hat{d}_2 < d_c)$
No cost
Repair
$(\hat{d}_2 \geq d_c)$
$E(C_R)$
4. Cost Analysis
$P(E_{1D})$
$P(E_{3D})$
$P(E_{2D})$
$P(E_{4D})$
$P(E_{1A})$
$P(E_{3A})$
$P(E_{2A})$
$P(E_{4A})$
$E(C^+_{F1})$
(d<dmin)
$E(C^-_{F1})$
(d>dmin)
$E(C^-_{I2})$
$E(C^+_{I2})$
$E(C^+_{F2})$
(d<dc)
$E(C^-_{F2})$
(d>dc)
$E(C^-_R)$
$E(C^+_R)$
Note: '+' indicates consequence of a correct inspection result
'-' indicates consequence of an incorrect inspection result

Figure 4

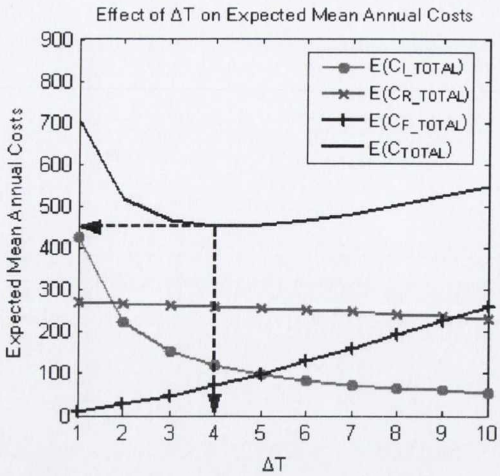

Effect of ΔT on Expected Mean Annual Costs
Expected Mean Annual Costs
ΔT
E(C_I_TOTAL)
E(C_R_TOTAL)
E(C_F_TOTAL)
E(C_TOTAL)
0
100
200
300
400
500
600
700
800
900
1
2
3
4
5
6
7
8
9
10

Figure 5

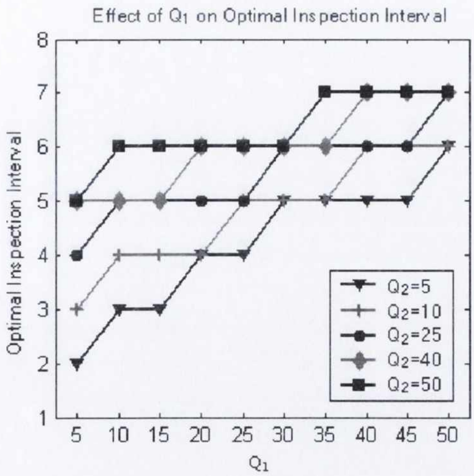
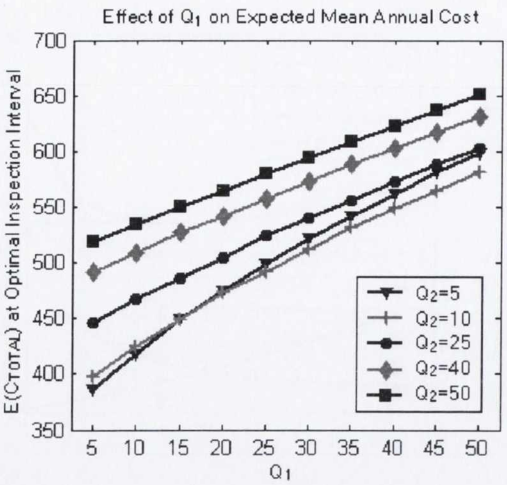

Effect of Q1 on Optimal Inspection Interval
Optimal Inspection Interval
Q1
1
2
3
4
5
6
7
8
5
10
15
20
25
30
35
40
45
50
Q2=5
Q2=10
Q2=25
Q2=40
Q2=50

Figure 6



Effect of Q_1 on Expected Mean Annual Cost
E(CTOTAL) at Optimal Inspection Interval
700
650
600
550
500
450
400
350
5 10 15 20 25 30 35 40 45 50
Q_1
Q_2=5
Q_2=10
Q_2=25
Q_2=40
Q_2=50

Figure 7

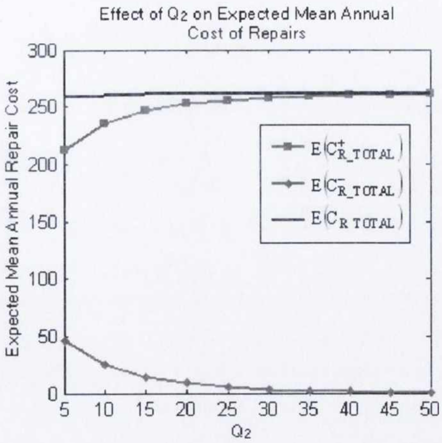

Effect of Q2 on Expected Mean Annual Cost of Repairs
Expected Mean Annual Repair Cost
300
250
200
150
100
50
0
5 10 15 20 25 30 35 40 45 50
Q2
$E(C^{+}_{R\_TOTAL})$
$E(C^{-}_{R\_TOTAL})$
$E(C_{R\_TOTAL})$

Figure 8

FIGURE CAPTIONS

Figure 1. Probabilistic modelling of inspection results for detection

Figure 2. Example of the effect of noise on sizing inspection results

Figure 3. Defect growth kinetics

Figure 4. Inspection outcomes for a defect group

Figure 5. The effect of the time between inspections on expected annual costs

Figure 6. The effect of inspection quality on optimal inspection interval

Figure 7. The effect of inspection quality on expected annual costs

Figure 8. The effect of the quality of inspections for sizing assessment on the expected annual repair cost