SPATIAL TIME-DEPENDENT RELIABILITY ANALYSIS OF CARBONATION INDUCED CORROSION DAMAGE TO RC STRUCTURES UNDER A CHANGING CLIMATE AND COST-BENEFIT ANALYSIS OF CLIMATE ADAPTATION STRATEGIES

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Thesis submitted for the degree of Doctor of Philosophy

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Declaration

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university or other tertiary institution and, to the best of my knowledge and belief, contains no material previously published or written by another person, except where due reference has been made in the text. I give consent to this copy of my thesis, when deposited in the University Library, being made available for loan and photocopying subject to the provisions of the Copyright Act 1968.

Lizhengli Peng

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Table of Contents

Declaration1
Acknowledgement2
List of Publications
Table of Contents
Nomenclature
Abstract
Chapter 1 : Introduction
1.1 Introduction
1.2 Background
1.3 Research significance
1.3.1 Modelling carbonation induced corrosion in RC structures under a changing climate 30
1.3.2 Spatial time-dependent reliability analysis
1 3 3 Cost-benefit analysis of climate adaptation strategies 32
1.4 Outline of the dissertation 32
1.4 Outline of the dissertation 32 1.5 References 35
1.4 Outline of the dissertation 32 1.5 References 35 Chapter 2 : Literature review 38
1.4 Outline of the dissertation 32 1.5 References 35 Chapter 2 : Literature review 38 2.1 Introduction 38
1.4 Outline of the dissertation321.5 References35Chapter 2 : Literature review382.1 Introduction382.2 A changing climate38
1.4 Outline of the dissertation321.5 References35Chapter 2 : Literature review382.1 Introduction382.2 A changing climate382.2.1 Emission scenarios38
1.4 Outline of the dissertation321.5 References35Chapter 2 : Literature review382.1 Introduction382.2 A changing climate382.2.1 Emission scenarios382.2.2 GCMs40
1.4 Outline of the dissertation321.5 References35Chapter 2 : Literature review382.1 Introduction382.2 A changing climate382.2.1 Emission scenarios382.2.2 GCMs402.3 Carbonation induced corrosion42
1.4 Outline of the dissertation321.5 References35Chapter 2 : Literature review382.1 Introduction382.2 A changing climate382.2.1 Emission scenarios382.2.2 GCMs402.3 Carbonation induced corrosion422.3.1 Overview42
1.4 Outline of the dissertation321.5 References35Chapter 2 : Literature review382.1 Introduction382.2 A changing climate382.2.1 Emission scenarios382.2.2 GCMs402.3 Carbonation induced corrosion422.3.1 Overview422.3.2 Carbonation depth models43
1.4 Outline of the dissertation321.5 References35Chapter 2 : Literature review382.1 Introduction382.2 A changing climate382.2.1 Emission scenarios382.2.2 GCMs402.3 Carbonation induced corrosion422.3.1 Overview422.3.2 Carbonation depth models432.3.2.1 Fick's first law of diffusion43
1.4 Outline of the dissertation321.5 References35Chapter 2 : Literature review382.1 Introduction382.2 A changing climate382.2.1 Emission scenarios382.2.2 GCMs402.3 Carbonation induced corrosion422.3.1 Overview422.3.2 Carbonation depth models432.3.2.1 First law of diffusion432.3.2.2 CEB Model (1992)45

2.3.2.4 Yoon et al. (2007)	. 47
2.3.2.5 Stewart et al. (2011)	. 47
2.3.2.6 Summary	. 48
2.4 Corrosion propagation	. 50
2.4.1 General	. 50
2.4.2 Corrosion rate models	. 51
2.5 Crack initiation and propagation models	. 53
2.5.1 General	. 53
2.5.2 Crack initiation models	. 54
2.5.2.1 Liu and Weyers (1998)	. 54
2.5.2.2 El Maaddawy and Soudki (2007)	. 55
2.5.3 Crack propagation models	. 56
2.5.3.1 Vidal et al. (2004)	. 56
2.5.3.2 Vu et al. (2005)	. 57
2.5.3.3 Mullard and Stewart (2010)	. 58
2.6 Reliability analysis	. 60
2.7 Spatial modelling of deterioration	. 61
2.7.1 General	. 61
2.7.2 Spatial variable properties of RC structures	. 62
2.7.3 Random field modelling to predict corrosion-induced cracking in RC structures	. 63
2.7.3.1 Li et al. (2004)	. 63
2.7.3.2 Malioka and Faber (2004)	. 63
2.7.3.3 Sudret et al. (2006)	. 65
2.7.3.4 Stewart and Mullard (2007)	. 66
2.7.3.5 Summary	. 67
2.8 Climate adaptation strategies of RC structures and cost-benefit analysis	. 68
2.8.1 General	. 68
2.8.2 Maintenance strategies	. 70
2.8.2.1 Inspection regime	. 70
2.8.2.2 Damage limit state	. 71
2.8.2.3 Repair techniques	. 72
2.8.3 Adaptation measures	. 73

2.8.4 Life cycle analysis	74
2.8.5 Reliability and cost-benefit analysis of climate adaptation strategies	75
2.8.6 Summary	77
2.9 References	80
Chapter 3 : Carbonation induced corrosion models	
3.1 Introduction	
3.2 Corrosion initiation due to carbonation	
3.2.1 Carbonation depth models	
3.2.2 Temperature effects on the diffusion coefficient	
3.2.3 Relative humidity effects on the diffusion coefficient	
3.2.4 k _{site} parameter	
3.3 Modelling time to corrosion initiation (T _i)	
3.4 Corrosion propagation	
3.5 Crack initiation and propagation models	100
3.6 Time to corrosion damage (T _{sev})	102
3.7 Corrosion loss	103
3.8 Summary	103
3.9 References	104
Chapter 4 : Spatial time-dependent reliability modelling	108
4.1 Introduction	108
4.2 Spatial modelling using random field	108
4.2.1 Stationary and non-stationary random fields	109
4.2.2 Discretisation methods	109
4.2.2.1 General	109
4.2.2.2 Midpoint method	110
4.2.2.3 Spatial averaging method	110
4.2.2.4 Expansion optimal linear estimation method	111
4.2.2.5 Summary	111
4.2.3 Using the midpoint method to discretise a RC surface	112
4.2.4 Correlation function and scale of fluctuation	113
4.2.5 Element size	117
4.3 Stochastic deterioration modelling	118

PhD Thesis – The University of Newcastle, Australia

4.3.1 Model error	118
4.3.2 Concrete properties	119
4.3.2.1 Concrete strength and elastic modulus	119
4.3.2.2 Concrete cover	122
4.3.3 Corrosion initiation model	123
4.3.4 Corrosion propagation model	124
4.3.5 Crack initiation model	125
4.3.6 Crack propagation model	125
4.4 Random field modelling	126
4.4.1 Spatial analysis of RC building slab or bridge deck	127
4.4.2 Spatial analysis of RC beam	128
4.5 Spatial time-dependent reliability analysis	129
4.5.1 Extent of corrosion initiation	129
4.5.2 Probability and extent of corrosion damage	129
4.5.3 Repair threshold	130
4.5.4 Modelling process	131
4.5.5 Summary of modelling parameters	132
4.5.6 Number of Monte Carlo simulations	132
4.6 References	135
Chapter 5 : Climate change impacts on carbonation induced corrosion	140
5.1 Introduction	140
5.2 Climate projections for Australian and Chinese cities	140
5.2.1 Emission scenarios	141
5.2.2 Temperature and RH projections	142
5.3 Durability design specifications for RC structures in two countries	145
5.3.1 Australian building and bridge standard	147
5.3.2 Chinese building and bridge standard	148
5.4 Model parameters	149
5.4.1 Random field parameters	149
5.4.2 Variability of climate projections	154
5.4.3 Year of construction	155
5.5 Applications: Australian RC building slabs and bridge decks	155

PhD Thesis – The University of Newcastle, Australia

5.5.1 Structural configuration for Australian cases	155
5.5.2 Results for Australian RC building slabs and bridge decks	157
5.6 Applications: Chinese RC building slabs and beams and bridge decks	167
5.6.1 Structural configuration for Chinese cases	167
5.6.2 Results for Chinese RC building slabs, beams and bridge decks	168
5.7 Comparison between Australian and Chinese RC structures	183
5.8 Accuracy of carbonation depth model	184
5.9 Sensitivity analysis	186
5.10 Conclusions	189
5.11 References	190
Chapter 6 : Climate adaptation and maintenance strategies and cost assessments	192
6.1 Introduction	192
6.2 Adaptation methods	192
6.2.1 Increase in concrete cover thickness	193
6.2.2 Increasing strength grade	194
6.2.3 Special concrete and corrosion inhibitors	194
6.2.4 Concrete surface coatings	195
6.2.5 Other adaptation measures	196
6.2.6 Summary	197
6.3 Maintenance strategies	198
6.3.1 Inspection interval	198
6.3.2 Damage limit state	199
6.3.3 Maintenance techniques	200
6.4 Costs of adaptation (C _{adapt})	200
6.4.1 C _{adapt} for increasing design cover (A1, A2)	201
6.4.2 C _{adapt} for increasing strength grade (A3, A4)	202
6.5 Costs of damage (C _{damage})	203
6.5.1 Repair costs (C _{repair})	203
6.5.2 User delay costs	204
6.6 Summary	205
6.7 References	207
Chapter 7 : Cost-benefit analysis of climate adaptation strategies	210

PhD Thesis – The University of Newcastle, Australia

7.1 Introduction	210
7.2 Life cycle cost modelling	211
7.3 Decision rules	213
7.3.1 Net Present Value	213
7.3.2 Net Present Value of adaptation strategies	214
7.4 Discount rate	215
7.5 Risk reduction	217
7.6 Break-even analysis	217
7.7 Applications: Australian RC building slabs and beams	218
7.8 Applications: Chinese RC building slabs and beams	222
7.9 Break-even analysis of ΔR and $C_{\text{adapt}}/C_{\text{damage}}$ ratio	232
7.10 Sensitivity analysis of the discount rate	240
7.11 Consideration of co-benefit	242
7.12 Conclusion	243
7.13 References	245
Chapter 8 : Conclusions and recommendations for future work	247
8.1 Summary and conclusions	247
8.2 Recommendations for future work	249
8.3 References	254
Appendix A: Pr(NPV(T)>0) for various C _{adapt} /C _{damage}	256
Appendix B: Discount rate impacts on mean NPV(T) and the Pr(NPV(T)>0)	

Figure 2-1. Projected global annual average CO ₂ concentration for SRES and RCPs (Jubb et al., 2013)
Figure 2-2. Projected annual average temperatures for the six GCM projections for RCP 8.5, RCP 4.5 and Year 2010 emission scenarios, for (a) Sydney and (b) Kupming
Figure 2.2. Projected appual average PH for the six GCM projections for PCD 9.5. PCD 4.5 and
Var 2010 emission scenarios for (a) Sydney and (b) Kunming
Figure 2-4: Distribution of carbonation denth in four buildings in Singapore of varying ages (a) 7
V_{pars} (b) 15 vears (c) 19 vears and (d) 59 vears (Roy et al. 1996)
Figure 2-5: Corrosion induced cracking and spalling (Capin, 2000)
Figure 2-6: RH and temperature effects on the corrosion rate (Brevise et al. 2014) 53
Figure 2-7: Representation of time-variant reliability problem (Stewart & Rosowsky, 1998) 61
Figure 2-8: Probability contours for $Pr(d_{1}(t) \ge V_{1})$ for different surface areas (Stewart &
Figure 2-8. Frobability contours for $F(u_{crack}(t) > \Lambda_{repair}/6)$ for university surface areas (stewart & Mullard, 2007)
Figure 2.0: Homogonous analysis of PC surface (Mullard 2010)
Figure 2-9. Homogenous analysis of NC surface (Mullard, 2010).
Figure 2-10. Spatial analysis of RC surface (Mullard, 2010).
Figure 2-11. Corrosion damaged NC surface (Mulliard, 2010)
Figure 3-1. Temperature effects on CO_2 diffusion coefficient.
Figure 3-2. Investigations of k
Figure 3-5. Investigations of R _{site}
Figure 4-1. Comparison between correlation functions
Figure 4-2: Effect of scale of fluctuation on the correlation coefficient
Figure 4-3. A schematic of a two-dimensional RC bridge deck which has been discretised into a
number (k) of spatially correlated elements of size Δ (Mullard, 2010)
rigure 4-4. Discretisation of one-dimensional RC component into K spatially correlated elements.
Figure 4-5: Convergence of d _{crack} (t) with increased Monte Carlo simulations
Figure 4-6: Convergence of probability of first repair with increased Monte Carlo simulations.133
Figure 5-1. Projected global annual average CO ₂ concentration for RCP 8.5, RCP 4.5 and Year
2010 emission scenarios
Figure 5-2: Temperature projections of six GCMs for Sydney, Canberra, Kunming, Xiamen and
Jinan under RCP 8.5, RCP 4.5 and Year 2010144
Figure 5-3: RH projections of six GCMs for Sydney, Canberra, Kunming, Xiamen and Jinan under
RCP 8.5, RCP 4.5 and Year 2010
Figure 5-4. Climatic zones of Australia defined by the Australian concrete code AS3600 (2009).
Figure 5-5. Climatic zones of China defined by the Chinese code JTG.D62-2004
Figure 5-6: Influence of Scale of fluctuation on the probability of 1 st maintenance (Mullard, 2010).
Figure 5-7. Influence of element size on the probability of 1 st maintenance (Mullard, 2010) 152

Figure 5-8. Simulation of spatially distributed corrosion process showing three typical Monte Carlo realisations for cast in-situ sheltered RC slab in Kunming, RCP 8.5 (Peng & Stewart, 2014b). Figure 5-9. Mean extent of surface corrosion damage for each GCM and the average for cast insitu sheltered RC slabs in Sydney and Kunming, RCP 8.5 emission scenario....... 154 Figure 5-10. Probability contours for extent of surface corrosion damage for each GCM and the mean for cast in-situ sheltered RC slabs in Sydney and Kunming, RCP 8.5 emission scenario... 154 Figure 5-11: Mean extent of surface corrosion damage for cast in-situ sheltered RC building slabs Figure 5-14. Mean carbonation depth of building slabs and bridge decks in Sydney and Canberra, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios. 158 Figure 5-15. Mean extent of corrosion initiation of building slabs in Sydney and Canberra, under Figure 5-16. Mean extent of corrosion initiation of bridge decks in Sydney and Canberra, under Figure 5-17. Mean extent of corrosion damage of building slabs in Sydney and Canberra, under Figure 5-18. Mean extent of corrosion damage of bridge decks in Sydney and Canberra, under Figure 5-19. Simulation histogram of the extent of surface corrosion damage for cast in-situ sheltered RC building slabs in Sydney in 2040, 2070 and 2100, under RCP 4.5 emission scenario. Figure 5-20. Probability contours for the extent of surface corrosion damage for cast in-situ sheltered building slabs in Sydney, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios. 166 Figure 5-21. RC beam for spatial analysis......168 Figure 5-22. Mean carbonation depth of buildings and bridges in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios. 169 Figure 5-23. Mean extent of corrosion initiation of building slabs in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios. 171 Figure 5-24. Mean extent of corrosion initiation of building beams in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios. 172 Figure 5-25. Mean extent of corrosion initiation of bridge decks in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios. 174 Figure 5-26. Mean extent of corrosion damage of building slabs in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios. 177 Figure 5-27. Mean extent of corrosion damage of building beams in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios. 177 Figure 5-28. Mean extent of corrosion damage of bridge decks in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios. 179

Figure 5-29. Simulation histogram of the extent of surface corrosion damage for cast in-situ sheltered RC building slabs in Kunming at 2040, 2070 and 2100, under RCP 4.5 emission scenario. Figure 5-30. Probability contours for the extent of surface corrosion damage for cast in-situ sheltered building slabs in Kunming, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios. Figure 5-32. Effects of climate input change on carbonation depth and extent of corrosion Figure 5-33. Effects of concrete quality change on carbonation depth and the extent of corrosion Figure 7-1: Mean extent of surface corrosion damage of BAU and four adaptation strategies for Figure 7-2: Mean extent of surface corrosion damage of BAU and four adaptation strategies for RC buildings in Sydney under RCP 8.5 and RCP 4.5...... 220 Figure 7-3: Expected damage costs $(\$/m^2)$ of BAU and four adaptation strategies for RC buildings Figure 7-4: Mean extent of surface corrosion damage of BAU and four adaptation strategies for RC slabs and beams in Kunming, Xiamen and Jinan under RCP 8.5...... 224 Figure 7-5: Mean extent of surface corrosion damage of BAU and four adaptation strategies for RC slabs and beams in Xiamen under RCP 8.5 and RCP 4.5. Figure 7-6: Mean E_{damage} (\$/m²) of BAU and four adaptation strategies for RC slabs and beams in Figure 7-7: Simulation histogram of NPV of the four adaptation strategies for RC slabs (D=100 mm) in Xiamen under the RCP 8.5 emission scenario...... 228 Figure 7-8: Time-dependent mean NPV and Pr(NPV>0) of four adaptation strategies for RC building slabs (D=100 mm and D=300 mm) and beams in Xiamen under the RCP 8.5...... 230 Figure 7-9: Effects of the four adaptation strategies on the histogram of ΔR for RC slabs (D=100 Figure 7-10: Influence of discount rates (r) on both the mean NPV and the Pr(NPV>0) of adaptation strategy A1 (cover + 5 mm) for RC slabs (D = 100 mm) in Xiamen under RCP 8.5... 240

Table 2-1: Atmospheric-ocean general circulation models applied in this study (CMIP5, 2014). 41
Table 2-2: Carbonation corrosion rates (i _{corr-20}) for various exposures (DuraCrete, 1998)
Table 2-3: Spatial variability of corrosion of a RC bridge (Fazio et al., 1999)
Table 3-1: Mean parameter values (Yoon et al., 2007). 92
Table 3-2: Definition of urban areas for k _{site}
Table 3-3. Examples of studies of measuring CO ₂ concentration in sites with summary details of
measurements and key results
Table 3-4: Statistical parameters for k _{site}
Table 3-5: Carbonation corrosion rates (i _{corr(ref)}) for various exposures (DuraCrete, 1998)
Table 4-1: Correlation function data used in Figure 4-1. 115
Table 4-2: Existing estimates of scale of fluctuation 117
Table 4-3. Statistical parameters for concrete compressive strength (fc(28)) in Australia (Forster
et al., 2013)
Table 4-4: Statistical parameters for concrete compressive strength for highway bridges in China.
Table 4-5: Statistical parameters for concrete compressive strength for buildings in China 121
Table 4-6: statistics of concrete cover of RC structures in Australia
Table 4-7: Mean parameter values (Yoon et al., 2007). 124
Table 4-8: Random field parameters
Table 4-9: Statistical parameters for corrosion parameters, material properties and dimensions.
Table 5-1: Australian durability design requirements (AS3600, 2009) and deterioration models
for carbonation corrosion
Table 5-2: Exposure classifications (GB50010-2010, 2010) for Chinese buildings
Table 5-3: Chinese durability design specifications (GB50010-2010, 2010) and deterioration
models for building structures
Table 5-4: Chinese durability design specifications (JTG.D62-2004, 2004) and deterioration
models for bridge structures
Table 5-5: Exposure classifications for RC buildings and bridges in five cities
Table 6-1. Data and relationships for the assessment of adaptation costs of A1 and A2 201
Table 6-2. Adaptation costs of A1 and A2 for various structural elements. 202
Table 6-3: Adaptation costs of A3 and A4 for various structural elements
Table 6-4: Expected costs of patch repair. 204
Table 6-5: Costs of adaptation strategies A1 to A4 and damage for RC structural elements 206
Table 7-1: Durability design requirements of Australian RC buildings for BAU and four adaptation
strategies
Table 7-2: Costs of adaptation strategies A1 to A4 and damage for RC structural elements in
Australia
Table 7-3: Mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs
(D=100 mm and D=300 mm) and RC beams in two Australian cities under RCP 8.5, RCP 4.5 and
Year 2015

Table 7-4: Durability design requirements of Chinese RC buildings for BAU and four adaptation Table 7-5: Costs of adaptation strategies A1 to A4 and damage for RC structural elements in Table 7-6: Mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs (D=100 mm and D=300 mm) and RC beams in three Chinese cities under RCP 8.5, RCP 4.5 and Year 2015...... 232 Table 7-7: ΔR that required to make Pr(NPV>0) larger than X% for RC slabs in Xiamen under RCP Table 7-8: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various C_{adapt}/C_{damage} for RC Table 7-9: Pr(NPV(T)>0) of adaptation strategies A1 to A4 using C_{adapt}/C_{damage} from Table 6-5 for RC building slabs and beams in Xiamen under RCP 8.5, RCP 4.5 and Year 2015 scenarios. 238 Table 7-10: The Maximum C_{adapt}/C_{damage} ratio of four adaptation strategies to make Table 7-11: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs (D=100 mm and D=300 mm) and RC beams in Xiamen under RCP Table A-1: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various C_{adapt}/C_{damage} for RC Table A-2: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various C_{adapt}/C_{damage} for RC Table A-3: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various C_{adapt}/C_{damage} for RC building slabs and beams in Kunming under RCP 8.5, RCP 4.5 and Year 2015 scenarios. 259 Table A-4: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various Cadapt/Cdamage for RC Table A-5: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various C_{adapt}/C_{damage} for RC Table B-1: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs and RC beams in Sydney under RCP 8.5, RCP 4.5 and Year 2015. Table B-2: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs and RC beams in Canberra under RCP 8.5, RCP 4.5 and Year 2015. Table B-3: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs and RC beams in Kunming under RCP 8.5, RCP 4.5 and Year 2015. Table B-4: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs and RC beams in Xiamen under RCP 8.5, RCP 4.5 and Year 2015. Table B-5: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs and RC beams in Jinan under RCP 8.5, RCP 4.5 and Year 2015.267

Nomenclature

А	= penetrated area (m^2) ;
a	= binding capacity for CO_2 (kg CO_2/m^3);
a'	= inner radius;
$B(n-1, N, \theta(t))$	= the cumulative Binomial distribution;
B(t)	= benefits at time t ;
b	= a model parameter defined as $b = \theta/2$ where θ is the scale of fluctuation;
b'	= outer radius;
Cs	= CO_2 concentration at the surface (kg CO_2/m^3);
C_{CO_2}	= atmospheric CO_2 concentration (g cm ⁻³);
С	= the concrete cover (mm);
C _D	= the design cost;
C _C	= the construction cost (materials and labour);
C _{QA}	= the cost of quality assurance/control;
C _{IN}	= the cost of inspections;
C _{PM}	= the cost of preventative maintenance;
C _{CM}	= the cost of corrective maintenance;
C _{USE}	= the user delay cost;
C _F	= the cost of durability failure;
C _{adapt}	= defined as the extra costs to take adaptation measures for unit area
$(\$/m^2);$	
C _{damage}	= the cost of damage including maintenance and repair costs, user delay
and disruption	n costs, and other direct or indirect losses arising from damage to
infrastructure	(\$/m ²);
a ()	

 $C_{co_2}(t)$ = the time-dependent increase in atmospheric CO₂ concentration (10⁻³ kg/m³);

 C_e = cement content (kg/m³);

 $C_aO = C_aO$ content in cement;

C _{i, j}	= the value of the covariance matrix;
COV _{ME}	= the coefficient of variation of model error;
COV _{T/C}	= the coefficient of variation obtained directly from the comparison of
the actual and	predicted values;
COV _{test}	= the coefficient of variation of measurement of the data;
COV _{spec}	= the coefficient of variation of the specimen properties;
C _{mean}	= the mean concrete cover (mm);
C _{nom}	= the nominal or design cover (mm);
C _{repair}	= direct costs of repair of corrosion damage $(\%/m^2)$;
C _{cv}	= construction cost per unit volume;
C(t)	= costs at time t;
c ₁	= external concentration of carbon dioxide (g/m^3) ;
c ₂	= the concentration of carbon dioxide at the carbonation front in g/m^3 ;
c	= a model parameter defined as $c = \theta/4$;
D _{CO2}	= the diffusion coefficient for carbon dioxide through carbonated concrete
in m^2/s ;	
D _{eff}	= effective diffusion coefficient at defined compaction, curing and
	· · ·
environmental	conditions (m^2/s) ;
environmental D_1	conditions (m^2/s) ; = CO ₂ diffusion coefficient after one year;
environmental D ₁ D _{CO2} (t)	 conditions (m²/s); = CO₂ diffusion coefficient after one year; = the time-dependent CO₂ diffusion coefficient;
environmental D ₁ D _{CO2} (t) D _{bar}	 conditions (m²/s); = CO₂ diffusion coefficient after one year; = the time-dependent CO₂ diffusion coefficient; = diameter of the steel reinforcing bar (mm);
environmental D ₁ D _{CO2} (t) D _{bar} D	 conditions (m²/s); = CO₂ diffusion coefficient after one year; = the time-dependent CO₂ diffusion coefficient; = diameter of the steel reinforcing bar (mm); = slab depth or beam width;
environmental D ₁ D _{CO2} (t) D _{bar} D DH	 conditions (m²/s); = CO₂ diffusion coefficient after one year; = the time-dependent CO₂ diffusion coefficient; = diameter of the steel reinforcing bar (mm); = slab depth or beam width; = degree of hydration;
environmental D ₁ D _{CO2} (t) D _{bar} D DH d_x and d_y	 conditions (m²/s); = CO₂ diffusion coefficient after one year; = the time-dependent CO₂ diffusion coefficient; = diameter of the steel reinforcing bar (mm); = slab depth or beam width; = degree of hydration; = the correlation lengths in the x and y axes, respectively;
environmental D ₁ D _{CO2} (t) D _{bar} D DH d_x and d_y d	conditions (m^2/s) ; = CO ₂ diffusion coefficient after one year; = the time-dependent CO ₂ diffusion coefficient; = diameter of the steel reinforcing bar (mm); = slab depth or beam width; = degree of hydration; = the correlation lengths in the x and y axes, respectively; = the correlation lengths defined as $d = \theta/\sqrt{\pi}$;
environmental D ₁ D _{CO2} (t) D _{bar} D DH d_x and d_y d $d_{crack}(t)$	conditions (m^2/s) ; = CO ₂ diffusion coefficient after one year; = the time-dependent CO ₂ diffusion coefficient; = diameter of the steel reinforcing bar (mm); = slab depth or beam width; = degree of hydration; = the correlation lengths in the x and y axes, respectively; = the correlation lengths defined as $d = \theta/\sqrt{\pi}$; = the extent of the concrete surface severe corrosion damage at time t;
environmental D ₁ D _{CO2} (t) D _{bar} D DH d_x and d_y d $d_{crack}(t)$ E	conditions (m^2/s) ; = CO ₂ diffusion coefficient after one year; = the time-dependent CO ₂ diffusion coefficient; = diameter of the steel reinforcing bar (mm); = slab depth or beam width; = degree of hydration; = the correlation lengths in the x and y axes, respectively; = the correlation lengths defined as $d = \theta/\sqrt{\pi}$; = the extent of the concrete surface severe corrosion damage at time t; = the activation energy of the diffusion process (40 kJ/mol);
environmental D ₁ D _{CO2} (t) D _{bar} D DH d_x and d_y d $d_{crack}(t)$ E E _{ef}	conditions (m ² /s); = CO ₂ diffusion coefficient after one year; = the time-dependent CO ₂ diffusion coefficient; = diameter of the steel reinforcing bar (mm); = slab depth or beam width; = degree of hydration; = the correlation lengths in the x and y axes, respectively; = the correlation lengths defined as $d = \theta/\sqrt{\pi}$; = the extent of the concrete surface severe corrosion damage at time t; = the activation energy of the diffusion process (40 kJ/mol); = the effective elastic modulus of concrete in (MPa);
environmental D ₁ D _{CO2} (t) D _{bar} D DH d_x and d_y d $d_{crack}(t)$ E E_{ef} E_c	conditions (m ² /s); = CO ₂ diffusion coefficient after one year; = the time-dependent CO ₂ diffusion coefficient; = diameter of the steel reinforcing bar (mm); = slab depth or beam width; = degree of hydration; = the correlation lengths in the x and y axes, respectively; = the correlation lengths defined as $d = \theta/\sqrt{\pi}$; = the extent of the concrete surface severe corrosion damage at time t; = the activation energy of the diffusion process (40 kJ/mol); = the effective elastic modulus of concrete in (MPa); = the elastic modulus of concrete;

 $E_{SF}(T)$ = the expected cost of repair or rehabilitation of corrosion-induced damage during service life T;

$E[C_M]$	= the expected cost of maintenance;
$E[C_U]$	= the expected user cost due to maintenance disruptions;
$E[C_F]$	= the expected cost of failure during the service life of the bridge;
$E_{W(X)\boldsymbol{v}}$	= the N \times 1 vector containing the covariance of W(X) with the elements of
w ;	
E _c (t)	= the time-dependent in-situ elastic modulus;
E _{ef}	= the effective elastic modulus of concrete;
E _{damage} (T)	= the expected cost of repair or rehabilitation (maintenance) of corrosion-
induced dama	age during service life T;
E _{damage-BAU} (t)	= the expected damage costs for business as usual(existing practice);
Edamage-adaptation	n(t) = the expected damage costs for adaptation measures;
$F_R(x)$	= the cumulative distribution function of R structural resistances;
FV	= the future cost;
F'c	= the nominal design concrete compressive strength;
F_1	= a random variable linking cube/cylinder strength to nominal
compressive s	strength;
F_2	= a random variable relating 28-day in-situ strength to cube/cylinder
strength;	
\mathbf{f}_{t}	= concrete tensile (splitting) strength (MPa);
f_R	= the probability density functions of R structural resistances;
$\mathbf{f}_{\mathbf{S}}$	= the probability density functions of S structural load effect;
f _T (t)	= time-dependent change in diffusion coefficient due to changes in
temperature;	
f _{RH} (t)	= time-dependent change in diffusion coefficient due to changes in RH;
$f_t(t)$	= time-dependent in-situ concrete splitting tensile strength;
f _c (t)	= time-dependent in-situ concrete compressive strength;
f	= the fluctuation scale;
f _c (28)	= the 28-day in-situ compressive strength;
f' _t	= the design concrete tensile strength (MPa);

$f_{d_{crack}}\left(d_{crack},t\right)$	= the multi-dimensional probability distribution of $d_{crack}(t)$;	
GDP	= Gross Domestic Product (\$ trillion);	
G	= the limit state function;	
$g_j(X, t)$	= the limit state function of element j;	
g _e	= a constant equal to 2.5;	
h	= equals to $\theta \sqrt{2}$;	
i _{corr-20}	= the corrosion rate at 20 °C in $1 \mu\text{A/cm}^2$;	
i _{corr}	= the corrosion current density in $1 \mu A/cm^2$;	
i _{corr(real)}	= the corrosion rate of the structure in the field (=100 μ A/cm ²);	
i _{corr(exp)}	= $100 \ \mu\text{A/cm}^2$ is the accelerated corrosion rate ;	
i	= the number of inspection;	
i _{corr(ref)}	= corrosion current density at reference state ($\mu A / cm^2)$ and is time-	
invariant;		
i _{corr} (t)	= time-dependent corrosion rate;	
K ₁	= constant parameter which considers the influence of execution on D_{eff}	
(e.g. influence	of curing);	
K ₂	= constant parameter which considers the influence of the environment on	
D _{eff} (e.g. realis	tic moisture history at the concrete surface during use);	
K	= parameter of temperature impacts on corrosion rate;	
K _{N,n}	= the number of combinations of elements that exceed critical percentage;	
k_{urban} = the in	ncreased CO_2 levels in urban environments;	
k _p	= the rate of rust production;	
k _R	= a rate of loading correction factor;	
k _c	= the confinement factor that represents an increase in crack propagation	
around external (edge) reinforcing bars due to the lack of concrete confinement;		
k	= the total number of elements of the concrete surface;	
k _{site}	= factor to account for increased CO_2 levels in non-remote environments;	
1	= the length of beam;	
L	= the lower triangular matrix of the covariance matrix calculated by a	
Cholesky deco	omposition (or similar);	
Μ	= respective molar masses in (kg/mol);	

M_{st}	= the mass of corroded steel depends on the type of corrosion product;				
$ME(r_{crack})$	= model error of crack propagation model;				
M _{CaO}	= molar mass of C_aO and equal to 56 g/mol;				
M_{CO_2}	= molar mass of CO_2 equal to 44 g/mol;				
m	= a mass of carbon dioxide (g);				
m _i	= the number of elements in the i^{th} combination;				
Ν	= the total number of elements;				
n _y	= the number of years in the future at which the cost is incurred;				
n _c	= constant parameter which considers the influence of meso climatic				
conditions (e.g. orientation and placing of structure);					
n _d	= the age factor of CO_2 diffusion coefficient;				
n _m	= an age factor of microclimatic conditions which related to the frequency				
of wetting-drying cycles;					
n _e	=the number of elements that exceed critical degradation;				
n	= the number of damage incidents;				
\mathbf{P}_{f}	= the probability if durability failure;				
$p_{s,n}(t)$	= the probability of the n^{th} damage incidence before time t;				
RH	= relative humidity (%);				
R	= the gas constant $(8.314 \times 10^{-3} \text{ kJ/mol}^{-3} \text{ K});$				
r_{crack} = the r	ate of crack propagation (mm/hr);				
r	= the discount rate;				
S	= the space interval;				
S	= the location of the spatial random variable within the random field;				
Ti	= Corrosion initiation, the time from construction completed to the time				
when carbon dioxide or chlorides, etc. induced de-passivation of the protective alkaline					
film around the reinforcing bar (year);					
t _{1st}	= Crack initiation, the time from corrosion initiation to the time when the				
first visible ci	rack of width of approximately 0.05 mm can be observed on the concrete				
surface (year)	, ,				
T(t)	= the temperature at time t ($^{\circ}$ C);				
T _{sev}	= the time to corrosion damage;				

Т	= the service life of the structure (years);			
t	= time in service;			
t ₀	= reference period, e.g. 1 year;			
t _{sev}	= the time for a crack to propagate from crack initiation to a limit crack width;			
$V_{f_c(28)}, V_{F_1}$ and	V_{F_2} = the COV of the 28-day in-situ concrete compressive			
strength, F ₁ an	ad F_2 , respectively;			
W	= the crack width (mm);			
W	= a vector representing the value of the random field for each of N			
elements;				
X(s)	= the multidimensional random field;			
X_i	= the location of the centroid of the element i;			
Х	= the averaging interval;			
X_0	= the distance from the end of the first element to the start of the second			
element;				
X_1	= the distance from the start of the first element to the start of the second			
element;				
X_2	= the distance from the start of the first element to the end of the second			
element;				
X ₃	= the distance from the end of the first element to the end of the second			
element;				
X	= thickness of penetrated concrete layer (m);			
$x_{c}(t)$	= the carbonation depth at time t;			
Z	= a vector of uncorrelated standard random variables;			
$\alpha_{\rm H}$	= a degree of hydration;			
ΔA_s	= the loss of steel cross section in (mm^2) ;			
ΔA_{s0}	= the loss of steel cross section that is needed for crack initiation in (mm^2) ;			
ΔB	= co-benefit of adaptation strategies;			
Δt	= the time between inspections;			
$\Delta P_{s,i}$	= the probability of damage incident between the $(i-1)^{th}$ and i^{th} inspections;			
$\Delta P_{f,i}$	= the probability that the extent of damage exceeds the repair threshold			
between the (i-1) th and i th inspections;				

$\Delta R(t)$	= the p	proportional	reduction	in	damage	losses	due	to	an	adaptation
measure;										

δ_s	= the thickness of corrosion products to generate tensile stresses;						
δ_0	= the thickness of the pore band around the reinforcing interface (μ m);						
ζ_{i}	= the independent standard normal variates (zero mean, unit variance and						
zero correlations);							
θ	=the scale of fluctuation;						
$\theta(t)$	= the probability of failure of the individual elements at time t;						
θ_i	= the eigenvalues of the covariance matrix;						
θ_x and θ_y	= the scales of fluctuation for a two-dimensional random field in x						
and y axes, respectively;							
ρ_{rust}	= the density of the corrosion products (kg/m^3) ;						
$\rho(\tau)$	=the correlation coefficient between elements;						
ρ_0	= the common correlation between all elements;						
$\rho_{i,j}$	= the value of the correlation function between points i and j;						
σ	= the standard deviation of the random field;						
τ	= the distance between two elements in the random field;						
τ_x and τ_y	= the distances between the centroid of element i and j in the x and y axes,						
respectively;							
υ	= Poisson's ratio of concrete;						
ψ_{cp}	= the concrete cover cracking parameter;						
φ _{cr}	= the creep coefficient;						

 ϕ_i = the length of the element in a one-dimensional random field or the area of the element in a two dimension random field.

Abstract

The long term performance of infrastructure is an important consideration for asset owners, particularly in relation to reinforced concrete (RC) structures subject to corrosion. This thesis focuses on management of RC structures subject to carbonation induced corrosion under a changing climate. A changing climate may lead to increases in atmospheric CO_2 concentration, and changes in temperature and relative humidity (RH), especially in the longer term, will accelerate the deterioration processes and consequently decline the safety, serviceability and durability of RC infrastructure. Therefore, modelling the deterioration process of RC structures under a changing climate and estimating cost effectiveness of climate adaption strategies can provide very useful information in decision making for the management of RC structures in corrosive environments.

While there is much research on corrosion-induced deterioration of concrete structures, there is relatively little research on how deterioration can be affected by a changing climate. In this thesis, an improved carbonation induced corrosion model is developed by considering time-dependent atmospheric CO_2 concentration, local temperature and RH effects. A new parameter k_{site} is introduced to take account local differences in CO_2 concentration based on recorded data from various observation sites around the world. Future climates may be influenced by various factors which make projections difficult. Therefore, high and medium emission scenarios (i.e. RCP 8.5 and RCP 4.5), as well as a reference scenario, are used to cover the full range of possible outcomes.

Many corrosion parameters and concrete properties governing the corrosion process are uncertain. Moreover, due to the spatial variability of workmanship, and environmental and other factors, it is recognised that the material and dimensional properties of concrete structures will not be homogeneous. So, it is necessary to model the spatial variability of the parameters in order to be able to characterise not only the probability of degradation, but also the extent of damage. This information is useful in optimising maintenance strategies. Random field is used in this thesis to model the spatial variability of corrosion damage. The method of discretisation and the random field parameters of element size, scale of fluctuation and correlation function are fully discussed in here.

In addition, a cost-benefit analysis of climate adaptation strategies is developed based on spatial time-dependent reliability analysis described above. Climate adaptation strategies such as increasing concrete cover and upgrading concrete strength, as well as a maintenance strategy are defined. The cost-effectiveness of an adaptation strategy is measured in terms of Net Present Value (NPV). Both the mean NPV and the probability of NPV exceeds zero can be calculated to provide useful information for decision makers.

RC beams and slabs for bridges and buildings in two Australian cities and three Chinese cities are investigated in the thesis. Durability design requirements, climate projections of specific locations and statistics of real structures' parameters (such as concrete cover and concrete compressive strength) of RC structures in Australia and China are considered in order to make practical predictions. Cost data for adaptation and corrosion damages are based on local market prices of the two countries. Monte-Carlo simulation is used as the computational method to do the spatial time-dependent reliability analysis which includes the time-dependent climate scenarios and deterioration processes, as well as a large number of random variables and spatial random fields of material properties and dimensions. Sensitivity analysis is performed in this thesis to estimate the relative influences of the considered random variables. Break-even analysis is also conducted to provide a straight forward measure for decision makers to quickly determine if an adaptation strategy is cost effective or not.

The overall results indicate that the reliability framework is well suited to predict carbonation induced corrosion damage of RC structures under a changing climate and assessing the cost-effectiveness of climate adaptation strategies. Moreover, the framework can easily adapt to updates or adjustments of information. The results and analysis can greatly assist designers and asset owner or operators in improving and optimising the management of RC structures in corrosive environments.

Chapter 1 : Introduction

1.1 Introduction

The long term performance of infrastructure is an important consideration for asset owners, particularly in relation to reinforced concrete (RC) structures subject to corrosion. A leading cause of deterioration in RC structures is poor concrete durability (low concrete quality and/or insufficient cover), leading to reinforcement corrosion and subsequent cracking and spalling of the concrete surface. The reduction in structural capacity at the time of severe cracking is estimated to be no more than 10% to 20% (Stewart & Val, 2003). Moreover, the surrounding environment will influence the deterioration rate of RC structures. Increases of atmospheric CO_2 concentrations and temperature, and changes of relative humidity due to climate change will lead to an acceleration of deterioration processes. Consequently, the safety, serviceability and durability of concrete infrastructure will decline at a higher rate, especially over longer terms. When optimising the lifetime performance of RC structures under a changing climate, corrosion induced cracking becomes an important and costly mode of failure.

The prediction of the likelihood and extent of corrosion damage is, therefore, paramount in effectively managing corroding RC structures. Due to workmanship, and environmental and material variability, the dimensional and structural properties of the RC structure are non-homogenous and will vary spatially in time and space for most structural components (Fazio et al., 1999). Not including spatial variability in the reliability analysis oversimplifies structural characterisation and can lead to significant underestimation of failure probabilities (Stewart, 2004). Further, it is of considerable benefit that a spatial time-dependent reliability model allows the extent of corrosion damage to be predicted. As such, it is especially useful in estimating the time at which repair actions will be required, and the corresponding cost (as the cost of repairs is a function of the extent of damage).

This is clearly an important issue. The cost of corrosion worldwide is estimated to exceed \$1.8 trillion annually, which is equivalent to 3% to 4% of the Gross Domestic Product

(GDP) of industrialised countries (Schmitt, 2009). In Japan, the maintenance and repair budget for infrastructure is predicted to double by 2015 (Nasu et al., 2004). Not only direct losses, but indirect losses such as user delays due to repair actions can be immense, especially for critical infrastructure like bridges linking cities and airports. So it is clear that optimising the management of deteriorating RC structures can have significant economic benefit. Climate adaptation strategies conducted at the design stage are expected to reduce repair areas and times, and therefore, decrease corrosion damage losses including repair costs and associated indirect losses. However, the cost of adaptation strategies, cost of damage and discount rate used to balance future and present well-being are country, site and structure specific. Thus, the selection of an adaptation strategy for a given item of infrastructure must be made with regards to pre-defined key performance criteria so that the outcomes satisfy the objectives.

An integrated reliability and cost-benefit analysis must be conducted that incorporates all key design/maintenance parameters to allow designers and asset owners to investigate the consequences of different design specifications or adaptation strategies. This analysis must accurately and realistically model all aspects of the deterioration and repair process, and provide output that can be readily used to aid in decision making for the management of RC structures in corrosive environments.

1.2 Background

Concrete is the predominant building type and has been used in many critical infrastructures in many countries. Infrastructure is vital to human settlement, because most human activities rely on transportation, buildings, energy and communication. Enormous wealth is locked up in infrastructure. Take Australia for instance: more than \$1.1 trillion has been invested in factories, homes, ports and other infrastructure assets, which is equal to three times the national budget or the GDP of Australia. China is developing rapidly, especially in a big demand of construction of infrastructure. More than half a trillion dollars has been invested since 2008 to improve physical infrastructure in China, including upgrading road and railway networks, etc. The primary building type for most critical infrastructure is concrete construction; hence the behaviour of concrete

construction is vital to a nation's essential economic activities and services. Not only in Australia and China, but most countries including the United States, Canada and Europe depend on concrete infrastructure for economic and social well-being.

Sound concrete could provide an alkaline environment that can protect the reinforcement bar embedded in the concrete from corrosion by a passive film. However, as time goes by, the passive film may be destroyed, and the reinforcement starts to corrode. There are two primary processes which cause corrosion of embedded steel. These are carbonation and chloride ion penetration. Carbonation induced corrosion is due to the CO₂ in the atmosphere diffusing into the concrete, and therefore the pH decreases due to chemical reactions. Therefore, carbonation induced corrosion can happen anywhere, especially in the places where the temperature is high, and the relative humidity (RH) is around 60-80%. However, carbonation induced corrosion is a time-consuming process; it usually takes decades to reach a critical level. Chloride-induced corrosion of reinforcing bars is the main cause of deterioration of RC structures in onshore and offshore marine environments or areas using de-icing salts during winter. When chloride concentration reaches a threshold, corrosion takes place. In this study, carbonation induced corrosion of the reinforcing steel in concrete structures is studied in detail.

Once corrosion of reinforcing bar is initiated in a RC structure, the expansion of corrosion products start to generate mechanical stress in the surrounding concrete causing concrete surface cracking and spalling, which results in inevitable damage of the structure (Melchers et al., 2005; Fang & Yang, 2011; Malumbela et al., 2011).

The U.S. Bureau of Standards reports that about \$300 billion is lost due to corrosion annually in the US, and carbonation induced corrosion accounted for 40% of those losses (Huang et al., 1983, Koch et al., 2001). In the UK, about 36% of concrete buildings need to be rebuilt or replaced because of carbonation induced corrosion (Huang et al., 1983). In 1985, a durability survey found that carbonation induced corrosion accounted for 47.5% of the pier, breastwork, and the girder damage based in more than 40 small and medium-

sized RC sluice structures in China (Huang et al., 1983). Therefore, the problem of carbonation induced corrosion is clearly a major and costly source of RC deterioration.

The deterioration rate of RC structures depends on the employed construction processes and the composition of the materials, as well as the environment. Increases in atmospheric CO_2 concentrations and temperature, and changes in RH due to a changing climate will, especially in the long run, lead to an acceleration of deterioration. As a result, the performance, safety and durability of concrete infrastructure may decline faster. The U.N. Intergovernmental Panel on Climate Change (IPCC) reports that industrial development has increased the CO_2 concentration in the atmosphere, hence, the temperature will increase due to greenhouse effects. The IPCC Fifth Assessment Report indicates that the annual average temperature change from 1901-2012 for grid-point values from all over the world ranges from -0.53 to 2.50 °C, and observed global annual average temperature increases by 0.61 °C from 1850–1900 to 1986–2005 with 5–95% of confidence interval: 0.55 to 0.67 $^{\circ}$ C (IPCC, 2014). CO₂ concentration in the atmosphere is projected to increase from 369 ppm in 2000 to 936 ppm after 100 years, which yields a range of temperature increases of $4.0-6.1 \,^{\circ}{\rm C}$ by 2100 (Rogelj et al., 2012). Rising temperatures will increase the rate of carbonation, as well as the corrosion rate. For instance, corrosion rates will increase up to 15% if the temperature rises by only $2 \, \mathbb{C}$ (Stewart et al., 2011, 2012). Clearly, concrete infrastructure in higher temperature environments suffers more severe carbonation induced corrosion (Roy et al., 1996).

While there is much research on corrosion-induced deterioration of concrete structures, there is relatively little research on how deterioration can be affected by a changing climate. Talukdar et al. (2012) predicted carbonation depths in Canada for several climate change scenarios, but did so using a deterministic model and assuming RH is constant. Stewart et al. (2011, 2012) developed reliability-based methods to predict corrosion induced concrete deterioration under a changing climate in Australia. It was shown by Stewart et al. (2011, 2012) that the additional carbonation-induced damage risk for the A1FI (high) emission scenario can be 16% higher if there are no changes to design or construction of concrete structures. Stewart and Peng (2010) used a simplified corrosion-

induced deterioration model and IPCC (2007) projected CO_2 concentration and temperature data to conduct a life-cycle cost analysis to assess the cost-effectiveness of increasing design concrete cover as an adaptation measure to mitigate the effects of carbonation on concrete. This preliminary study found that increasing design concrete cover may not be beneficial, but the results were sensitive to repair and user disruption costs. However, these models still depend on relatively straightforward deterioration models and they ignored the effect of RH changes on the deterioration process. The effects of climate change on chloride-induced corrosion appear to have also been the subject of relatively little research; nevertheless, Bastidas-Arteaga et al. (2010, 2013) calculated a 5–15% increase in the probability of corrosion initiation and a shortened service life of up to 15 years due to climate change.

Many corrosion parameters and concrete properties governing the corrosion process are uncertain, and it is important to take into account the uncertainties when modelling each phase of corrosion. Therefore, it is ideal to use reliability analysis to simulate the corrosion process. Fazio et al. (1999) measured a 39-year-old bridge deck and found significant spatial variability in chloride content, concrete cover, concrete strength and corrosion damage. For example, the corrosion rate was found to be different by as much as two orders of magnitude across the bridge deck, and concrete cover varied from less than 25 mm to over 50 mm. Moreover, due to the spatial variability of workmanship, and environmental and other factors, it is recognised that the material and dimensional properties of concrete structures will not be homogeneous. Moreover, it is necessary to model the spatial variability of the parameters in order to be able to characterise not only the probability of degradation, but also the extent of damage (Vu & Stewart, 2005; Darmawan & Stewart, 2007; Stewart & Mullard, 2007; Sudret, 2008; Na et al., 2012; Papakonstantinou & Shinozuka, 2013). The extent of damage is an important variable that characterises the performance of the structure, and may be used in optimising maintenance strategies (Li et al., 2004; Stewart, 2006; Mullard & Stewart, 2009, 2012).

1.3 Research significance

This thesis will improve the existing stochastic carbonation modelling by including the effects of changing climate on carbonation induced corrosion. Spatial time-dependent reliability analysis is used to simulate the concrete properties, corrosion parameters and climate projections, etc. The main contributions can be summarised as:

- The existing carbonation model is improved by considering the effects of a changing climate. The climate parameters are simulated as time-dependent variables.
- Climate change effects on the corrosion rate are also considered.
- Random field is used to model the spatial variability of corrosion damage to the concrete surface.
- A spatial time-dependent reliability analysis is adapted to predict the probability and extent of corrosion damage.
- Durability design requirements, climate projections of specific locations and statistics of real structures' parameters are considered to predict climate change effects on RC structures in Australia and China.
- The effects of climate adaptation strategies on new structures are modelled by spatial time-dependent reliability analysis, and the probability and extent of corrosion damage and expected damage costs are predicted. This will allow comparison between adaptation strategies and distribution of net present value.
- Break-even analysis is applied to estimate break-even values.

Detailed descriptions of the key elements of this thesis are presented as follows.

1.3.1 Modelling carbonation induced corrosion in RC structures under a changing climate

Modelling of the processes involved in corrosion damage of RC structures under a changing climate is the primary component of a predictive analysis. As discussed, existing carbonation and corrosion models are not appropriate for stochastic analysis as they are either too computationally expensive or do not consider key parameters.

Therefore, an accurate, robust carbonation induced corrosion model including climate change effects that are suitable for use in reliability-based analysis is needed.

This thesis develops on improved carbonation depth and corrosion propagation models that will be used to predict the extent of severe corrosion-induced cover cracking for RC structures in carbonated environments under a changing climate. Carbonation depth can be influenced by CO_2 concentration in the atmosphere, temperature and relative humidity. A new parameter k_{site} is introduced to take account location of differences in CO_2 concentration based on recorded data from various observation sites around the world. The time-dependent temperature and RH effects on carbonation are modelled as well. When carbonation depth reaches the reinforcing bar, corrosion starts. In the corrosion propagation stage, the corrosion rate is critical for the performance of RC structures, and it can also be affected by temperature and RH. The carbonation and corrosion models presented herein are an essential tool for estimating the time-dependent performance of RC structures subject to carbonation induced corrosion under a changing climate.

1.3.2 Spatial time-dependent reliability analysis

Concrete properties and dimensions will vary spatially over a RC structure due to a number of factors including material variations and workmanship. Incorporating spatial variability is an important aspect of modelling corrosion damage in RC structures (Stewart et al., 2007). Ignoring the spatially variable characteristics can lead to non-conservative probabilities of failure. Random fields are typically used to represent the spatially variable properties of the RC structure (Darmawan & Stewart, 2007; Na et al., 2012; Papakonstantinou & Shinozuka, 2013; Stewart & Mullard, 2007; Sudret, 2008; Vu & Stewart, 2005). The structure (or structural component) is discretised into a number of elements, and a random variable is used to represent the random field over each element. Each of these random variables is statistically correlated based on the correlation function of the corresponding random field. This study will use random fields to model spatial variability of concrete cover, concrete strength, CO_2 diffusion coefficient and binding capacity. The likelihood and extent of corrosion damage can then be estimated at any

time throughout the service life of RC slabs and beams. The impacts of climate change will be expressed in terms of:

- mean extent of corrosion initiation and damage $(d_{ini} \text{ and } d_{crack})$
- mean time of first repair
- 5th percentile of time of first repair.

1.3.3 Cost-benefit analysis of climate adaptation strategies

As discussed, the new and existing models are integrated into a spatial time-dependent reliability analysis. This type of analysis allows the statistical uncertainty of the various processes to be fully captured and provides the entire distribution of results so that the uncertainty of the predictive outcomes can be characterised.

The inclusion of spatial variability allows the extent of damage at a given time to be known and thus the timing of repair actions can be predicted. By modelling the likelihood and extent of corrosion damage, damage losses of RC structures' corrosion damage based on direct losses expressed by cost of repair actions and indirect losses can be assessed. Four climate adaptation strategies are considered, namely increased concrete cover by 5 mm and 10 mm, and increased concrete strength by 1 grade and 2 grades. Climate adaptation strategies can be evaluated by reducing damage losses during the service life of a RC structure. Monte-Carlo event-based simulation (MCS) analysis is used to predict the performance of each adaptation strategy. A cost-benefit analysis of each adaptation strategies based on these results can be conducted using decision rules such as mean Net Present Values (NPV) and Pr(NPV>0), etc. The results will be shown in terms of:

- expected damage costs and risk reduction (ΔR) due to adaptation strategies
- the NPV and Pr(NPV>0) of adaptation strategies
- \bullet the break-even analysis of ΔR and $C_{adapt}\!/C_{damage}$ ratio.

1.4 Outline of the dissertation

The outline of this dissertation is:

• Chapter Two: a literature review. This chapter lists climate emission scenarios that have been developed and the projections of possible temperature, RH and CO₂ concentrations in the future. The models to simulate corrosion processes are

described and discussed, including carbonation induced corrosion models, corrosion propagation models, crack initiation, and crack propagation models. Modelling of the spatial variability of RC structures is considered and discussed. Spatial time-dependent reliability analysis for predicting corrosion damage of RC structures is also discussed. Climate adaptation and maintenance strategies are reviewed, as well as life cycle cost of RC structures and cost-benefit analysis.

- Chapter Three: carbonation induced corrosion models. This chapter describes the development of carbonation induced corrosion initiation and corrosion propagation models. Climate change effects including temperature and RH effects on the diffusion coefficient and corrosion rate are presented, as well as site effects of CO₂ concentration on carbonation models. The models used to simulate crack initiation and crack propagation are also described.
- Chapter Four: spatial time-dependent reliability analysis. This chapter describes the use of random fields to model spatial variability in RC structures. The method of discretisation for random fields is discussed, and the random field parameters of element size, scale of fluctuation and correlation function are described in relation to the analysis conducted in this thesis. The spatial time-dependent reliability model developed in this thesis is described in detail. All deterministic, stochastic, spatial and spatially dependent variables are described. Statistical parameters are defined, and a description of the modelling process of structural deterioration is given.
- Chapter Five: climate change impacts on corrosion damage of RC structures. This chapter presents results of the spatial time-dependent reliability model for RC slabs and beams in two Australian cities (Sydney and Canberra) and three Chinese cities (Kunming, Xiamen and Jinan) under two emission scenarios. Durability design requirements of these two countries, various climate conditions, microclimate conditions and the exposure environment, as well as different construction methods, are included in this analysis. The likelihood and extent of corrosion damage are predicted during 2010 to 2100. Sensitivity analysis of climate variables and concrete quality factors are also conducted.

- Chapter Six: climate adaptation and maintenance strategies and cost assessments. This chapter discusses typical adaptation methods for corrosion damaged RC structures under a changing climate. Four adaptation strategies are defined. Maintenance strategy is assumed to be the same for each alternative for comparison purposes. The durability performance of adaptation strategies and repair actions is estimated so that they can be integrated into the spatial time-dependent reliability analysis. The cost associated with adaptation strategies and damages including repair actions and user delays are estimated based on published data, and this will allow the cost-benefit analysis of adaptation strategies.
- Chapter Seven: cost-benefit analysis of climate adaptation strategies. Adaptation strategies and repair actions are integrated with the spatial time-dependent reliability analysis, so expected damage costs of business as usual and four adaptation strategies can be estimated. Decision rules of cost-benefit analysis including the mean NPV and Pr(NPV>0) are used. The four adaptation strategies applied on RC slabs and beams in the five cities under two climate emission scenarios are compared. Break-even analysis of ΔR and C_{adapt}/C_{damage} ratio is conducted accordingly. Sensitivity analysis of the discount rate is presented.
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Chapter 2 : Literature review

2.1 Introduction

In order to predict the impact of climate change on the performance of RC infrastructure, it is important to accurately simulate the corrosion process and time-dependent climate change. The time up to severe corrosion damage of RC structures can be divided into three stages:

i. Corrosion initiation - the time from construction completed to carbonation induced depassivation of the protective alkaline film around the reinforcing bar;

ii. Crack initiation - the time from corrosion initiation to when the first visible crack of width of approximately 0.05 mm can be observed on the concrete surface; andiii. Crack propagation - the time from crack initiation to the time when the crack reaches a limit crack width.

The performance of RC structures under a changing climate may be improved significantly by adaptation strategies implemented at the design stage. Therefore, this chapter will introduce the issues that can influence the deterioration of RC structures, and present existing models and researches relating to the carbonation induced corrosion process.

2.2 A changing climate

According to the IPCC Fifth Assessment Report, observed data of annual average temperature change from 1901-2012 for grid-point values from all over the world ranges from -0.53 to $2.50 \,$ °C, and observed global annual average temperature increases by 0.61 °C from 1850–1900 to 1986–2005 with 5–95% of confidence interval: 0.55 to 0.67 °C (IPCC, 2014). The IPCC Fourth Assessment Report (IPCC, 2007) indicated that the CO₂ concentration in the atmosphere increased significantly from 280 ppm to 380 ppm during the period from 1750 to 2005 with an increasing trend.

2.2.1 Emission scenarios

Future climates may be influenced by changes in technology, energy, population, economy, land use and agriculture, etc., which makes projections difficult and

comprehensive. IPCC projects future climate by defining carbon emission scenarios. The special Report on Emission Scenarios (SRES) defined four scenario families who are A1, A2, B1 and B2 (IPCC, 2000) and were used in the IPCC Third and Fourth Assessment Reports. The A1 scenarios are characterised by very fast economic growth, the global population peaking in 2050 and then declining, and the rapid spread of new and more improved technologies, as well as a reduction in regional differences in income and way of life. Sub-categories of A1 scenarios include A1FI and A1B, which indicate fossil intensive and a balance across all energy sources, respectively. The A2 scenarios are of a more heterogeneous world characterised by the preservation of local identities, a continuous increase of population, regionally oriented economic development, more independent per capita economic growth and technological change. The B1 scenarios assume the same population trend as A1, but rapid change in economic structures towards information and service economy, reduction in material intensity, and the introduction of environmental friendly and resource efficient technologies. The B2 scenarios emphasise local solutions to economic, social and environmental sustainability, a continuous increase of the global population at a rate slower than A2, median levels of economic development and less fast and more diverse technological change than those in B1 and A1.

The IPCC Fifth Assessment Report (AR5) use RCPs (IPCC, 2013). The four RCPs, RCP 2.6, RCP 4.5, RCP 6, and RCP 8.5, represent a likely range of radiative forcing values in the year 2100 to be 2.6, 4.5, 6, and 8.5 W/m², respectively, where RCP 8.5, RCP 6.0 and RCP 4.5 are roughly equivalent to A1FI, A1B, and A1B to B1 emission scenarios respectively (Inman, 2011). The selected RCPs were considered to be representative of the literature, and included a low forcing level (RCP 2.6), two medium scenarios (RCP 4.5/RCP 6) and one high emission scenario (RCP 8.5) (Moss et al., 2010). The projected global annual average CO₂ concentration for SRES and RCPs were presented as Figure 2-1.



Figure 2-1. Projected global annual average CO₂ concentration for SRES and RCPs (Jubb et al., 2013).

2.2.2 GCMs

Various Atmosphere-Ocean General Circulation Models (AOGCMs) have been developed that depend on physical principles at the continental scale to project spatially dependent temperature and RH changes in the future under emission scenarios. Selecting AOGCMs to be used in the impact assessment is not an unimportant task, given the variety of models. The future scenarios use the past runs based on historical data from 1850-2005 and adjust the chemical forcing it to match the RCP guidelines to see how it turns. Weather data are normalised to meet actual observations in the last few years for each site. Note that each location is represented by a single grid point; this simplification may overlook local geographical effects. Weather systems cannot be completely modelled due to chaos components, and climate models are even harder due to the scale of data, so no single model can be considered the best. Each climate research group puts forward a model that most closely matches the observational record according to their criteria, and every model contains some bias due to sub-grid scale parameterisation of the physics and limited real data. Each individual simulation is considered as a possible representation of the weather, and using the multiple models is a way to find the full range of possible values. Therefore, it is necessary to use various models to consider the

uncertainties of models in any impact assessment. In the current study, climate projections of six GCMs are used, see Table 2-1.

Models	Developers
BCC-CSM1.1	Beijing Climate Center, China Meteorological Administration
MIROC5	Atmosphere and Ocean Research Institute (The University of Tokyo), National Institute for Environmental Studies, and Japan Agency for Marine-Earth Science and Technology
IPSL-CM5A-LR	Institut Pierre-Simon Laplace
CSIRO-MK3.6.0	Commonwealth Scientific and Industrial Research Organization in collaboration with Queensland Climate Change Centre of Excellence
CNRM-CM5	Centre National de Recherches M & éorologiques / Centre Europ én de Recherche et Formation Avanc é en Calcul Scientifique
ACCESS1.0	Commonwealth Scientific and Industrial Research Organization (CSIRO) and Bureau of Meteorology (BOM), Australia

 Table 2-1: Atmospheric-ocean general circulation models applied in this study (CMIP5, 2014)

The projected annual average temperatures and RH for the six GCM projections, for RCP 8.5 and RCP 4.5 emission scenarios for Sydney and Kunming are shown in Figure 2-2 and Figure 2-3, respectively. A considerable variability between model projections can be found, which makes climate projections particularly challenging. On the other hand, the large variations could cover the full range of possible values. Clearly, temperature and RH projections for Sydney and Kunming are similar. The projected annual average temperatures and RH for all five cities are given in Chapter 5.



Figure 2-2. Projected annual average temperatures for the six GCM projections for RCP 8.5, RCP 4.5 and Year 2010 emission scenarios, for (a) Sydney and (b) Kunming.



Figure 2-3. Projected annual average RH for the six GCM projections for RCP 8.5, RCP 4.5 and Year 2010 emission scenarios, for (a) Sydney and (b) Kunming.

2.3 Carbonation induced corrosion

2.3.1 Overview

Generally, sound concrete can prevent the embedded reinforcing bar from corroding. A passive layer can be formed on the steel surface due to calcium hydroxide contained in the concrete which can provide a highly alkaline environment of a minimum pH value of 12. However, concrete as a porous material allows carbon dioxide, oxygen and moisture, etc., to penetrate into the concrete itself, and then induces chemical or physical interactions with the alkaline compounds of the hydrated cement paste. These interactions can cause deterioration of the concrete and embedded reinforcing bar, and is accompanied with a drop of the pH of the concrete pore fluid to a value of about 8.0 (Parrott, 1987; Hilsdorf & Kropp, 2004). Therefore, the protective barrier provided by the passive layer on the reinforcing surface breaks down and corrosion starts. A simplified reaction equation is:

$$CO_2 + Ca(OH)_2 \rightarrow CaCO_3 + H_2O$$

2-1

Atmospheric carbon dioxide can react with the cement hydrates only if there is enough pore water first to dissolve it and it is necessary for the CO_2 to diffuse through the carbonated surface layer to reach the reaction zone (Hilsdorf & Kropp, 2004). This gaseous diffusion is a slow process if the concrete pores are saturated with water and the maximum carbonation rate is observed when moisture content is intermediate. The carbonation rate can be dominated by carbon dioxide diffusion or by moisture content, and it rises with temperature, carbon dioxide concentration and porosity. Variations of microstructure, when induced by carbonation under normal exposure conditions, change with time as carbonation depth increases (Parrott, 1987). The models to describe the carbonation depth are normally focused on the diffusion process, as well as chemical components of concrete and exposure conditions etc., and some are discussed in detail below.

2.3.2 Carbonation depth models

2.3.2.1 Fick's first law of diffusion

A common method to estimate carbonation depth into RC structures is by the use of Fick's first law. Based on Fick's first law of diffusion (Crank, 1956), the rate of diffusing of carbon dioxide through a unit area of the concrete section is assumed to be proportional to the carbon dioxide concentration gradient measured to the section. Therefore, the time-dependent depth of carbonation can be described by Fick's law of diffusion. The amount of CO_2 diffusing through a concrete cover by time period dt is given by

$$m = -D_{CO_2} A \frac{c_1 - c_2}{x} dt$$
 2-2

where m is a mass of carbon dioxide in g; D_{CO2} is the diffusion coefficient for carbon dioxide through carbonated concrete in m²/s; A is penetrated area m²; c₁ is the external concentration of carbon dioxide g/m³; c₂ is the concentration of carbon dioxide at the carbonation front in g/m³; and x is thickness of penetrated concrete layer in m.

The CO₂ diffused into the concrete then reacts with the available alkaline compounds at the carbonation front. The amount of CO₂ a (g/m^3) is needed for the carbonation of these alkaline compounds contained in a unit volume of concrete, and the mass of CO₂ that is needed to increase the depth of carbonation by an increment dx is given as m=aAdx (Hilsdorf & Kropp, 2004). Then Eq. 2-2 can be written as:

$$aAdx = -D_{CO_2}A\frac{c_1 - c_2}{x}dt$$
 or $xdx = -\frac{D_{CO_2}}{a}(c_1 - c_2)dt$ 2-3

Integration of Eq. 2-3, then $x^2 = \frac{2D_{CO_2}}{a}(c_2 - c_1)t$ or $x = \sqrt{\frac{2D_{CO_2}}{a}(c_2 - c_1)t}$ which is the well-known equation x=C \sqrt{t} , if all constant parameters are integrated as a single constant C.

Several simplifications have been made in the derivation of equations above (Hilsdorf & Kropp, 2004):

- 1. The carbon dioxide diffusion coefficient has been taken as a constant in time and space. For a concrete structure in service, however, D may be influenced by numerous factors, which is also related to the microstructure of concrete, such as curing condition, moisture content, temperature, location, service time and mixing parameters, etc. As time goes by, cement particles contained in concrete are hydrated continuously and Ca(OH)₂ crystals are converted into CaCO₃ due to carbonation attacking the concrete microstructure at the same time (Deceukelaire & Vannieuwenburg, 1993). These two processes tend to slow down CO₂ diffusion with elapsed time because the carbonation of Ca(OH)₂ increases solid volume inside the concrete. Moreover, the porosity will be reduced due to the saturation of free pores by hydration reaction products. These provide less free pore space for CO₂ diffusion.
- 2. The derivation implies that the non-carbonated material and carbonated material is separated by a sharp reaction front which means all alkaline compounds have been transformed within the distance of the carbonation depth. In reality, the formation of carbonates develops in a transition zone.
- 3. The parameter binding capacity defined as the amount of carbon dioxide a (g/m^3) that is needed for carbonation of the alkaline compounds contained in a unit volume of cementations material is assumed to be constant. The parameter binding capacity depends on the cement type, CaO content, mix proportion, pozzolanic additions, as well as hydration degree of the cement. It may not reasonable to compute the parameter binding capacity for given cement, assuming

a total carbonation of all alkaline compounds. The value of binding capacity is likely to be a function of time, as well as exposure conditions.

A carbonation model based on Fick's law of diffusion that models the carbonation process is subject to many limitations and differs from field conditions. Modifications and improvements of the model are needed. The majority of carbonation models have been formulated by applying suitable modifications to Fick's first law of diffusion (DuraCrete, 1998; Tuutti, 1982; Richardson, 1988; Hakkinen, 1993; Bakker, 1994; Parrott, 1994; CEB, 1992, etc.).

2.3.2.2 CEB Model (1992)

The model developed by the CEB Task Group V (1992) is given as follows:

$$x_c = \sqrt{\frac{2K_1K_2D_{eff}C_s}{a}}\sqrt{t}\left(\frac{t_0}{t}\right)^{n_c}$$
2-4

In which,

$$a = 0.75 \cdot C_a O \cdot C_e \cdot DH \cdot \frac{M_{CO_2}}{M_{CaO}}$$
2-5

where x_c is carbonation depth, D_{eff} is effective diffusion coefficient at defined compaction, curing and environmental conditions (m²/s); a is binding capacity for CO₂ (kg CO₂/m³); C_s is CO₂ concentration at the surface (kg CO₂/m³); t is time in service (s); t₀ is reference period, e.g. 1 year; K₁ is constant parameter which considers the impact of execution on D_{eff} (e.g. influence of curing); K₂ is constant parameter which considers the influence of the environment on D_{eff} (e.g. realistic RH history at the concrete surface during use); n_c is constant parameter which considers the influence of meso climatic conditions (e.g. orientation and placing of structure); C_aO is C_aO content in cement (% by weight); C_e is cement content (kg/m³); DH is degree of hydration; M is respective molar masses in (kg/mol). The value proposed by the task group for K₁, K₂ and n as given depends on exposure class and curing conditions. The CEB (1992) model is one of those models modified from Fick's first law of diffusion. This model considers chemical content, carbonation degrees, exposure class and moisture conditions, etc. However, the time and spatial effects of these factors are not considered.

46

2.3.2.3 Papadakis et al. (1991)

Papadakis et al. (1991) consider the physicochemical processes involved in concrete carbonation. A model is proposed based on differential mass balances of gaseous CO_2 , solid and dissolved $Ca(OH)_2$, CSH, and un-hydrated silicates, which account for the production, diffusion, and consumption of these substances. The mathematical form of the model is presented as follows:

$$x_{c} \frac{\mathrm{d} x_{c}}{\mathrm{d} t} = \frac{D_{e,CO_{2}}^{c} [\mathrm{CO}_{2}]^{0}}{[\mathrm{Ca}(\mathrm{OH})_{2}]^{0} + 3[\mathrm{CSH}]^{0} + 3[\mathrm{C}_{3}\mathrm{S}]^{0} + 2[\mathrm{C}_{2}\mathrm{S}]^{0}}$$
2-6

This equation can be integrated, under the initial condition xc=0 at t=0, yielding

$$x_{c} = \sqrt{\frac{2D_{eff}^{c} [CO_{2}]^{0}}{[Ca(OH)_{2}]^{0} + 3[CSH]^{0} + 3[C_{3}S]^{0} + 2[C_{2}S]^{0}}t}$$
 2-7

where D_{eff} is effective diffusivity (in m²/s); $[CO_2]^0$, $[Ca(OH)_2]^0$, $[CSH]^0$, $[C_3S]^0$ and $[C_2S]^0$ are 'initial' concentrations of CO₂, Ca(OH)₂, CSH, C₃S and C₂S at t=0, respectively.

Papadakis et al. (1991) has considered the chemical composition of cement, water/cement ratio and aggregate/cement ratio, as well as CO_2 concentration and RH. However, environmental conditions such as temperature are not included, and this is a very complicated model which contains many parameters (including idealised or empirical variables), a large amount of input data, which have to be calculated, tested or estimated for different field conditions and concrete types, are not always easily obtained and can always lead to an increased margin for error in parameter estimations. Further, coupled partial differential equations sets are usually solved numerically that are computationally expensive. Some other models of this kind can be found from the literature (Saetta et al., 1993; Van Balen & Van Gemert, 1994; Saetta et al., 1995; Roy et al., 1999; Bary & Sellier, 2004; Saetta & Vitaliani, 2004; Talukdar et al., 2012; de Larrard, 2013, etc.).

2.3.2.4 Yoon et al. (2007)

Yoon et al. (2007) made some modifications to the CEB (1996) model by considering time-dependent CO_2 diffusion coefficient, as well as RH effects on CO_2 diffusion coefficient. The model for carbonation depth is:

$$x_{c} = \sqrt{\frac{2D_{CO_{2}}(t)}{a(CO_{2})}} C_{CO_{2}}(t) t \left(\frac{t_{0}}{t}\right)^{n_{m}}$$
2-8

where the CO₂ diffusion coefficient $D_{CO_2}(t) = D_1 t^{-n_d}$; D₁ is CO₂ diffusion coefficient after one year; n_d is age factor of CO₂ diffusion coefficient; n_m is an age factor of microclimatic conditions which related to the frequency of wetting-drying cycles; C_{CO_2} is atmospheric CO₂ concentration (g cm⁻³); and a is the binding capacity for CO₂ which is the amount of CO₂ for complete carbonation. The parameters D₁ and n_d depend on the water/cement ratio w/c; for details see Yoon et al. (2007).

The Yoon et al. (2007) model is a simplified point-in-time predictive model which assumes the CO_2 concentration and CO_2 diffusion coefficient are constant for the time period from 0 to t years.

2.3.2.5 Stewart et al. (2011)

Stewart et al. (2011) made some improvements on Yoon et al.'s (2007) model by including time-dependent temperature effects and increasing CO_2 levels in urban environments, as well as the accumulated effects of CO_2 concentration change. The model is:

$$x_{c} \approx \sqrt{\frac{2f_{T}(t)D_{CO_{2}}(t)}{a}k_{urban}\int_{2000}^{t}C_{CO_{2}}(t)dt} \left(\frac{1}{t-1999}\right)^{n_{m}} t \ge 2000.$$
 2-9

where $C_{CO2}(t)$ is the time-dependent atmospheric CO₂ concentration (10⁻³ kg/m³). Elevated CO₂ levels in urban environments are observed due to higher pollution, exhaust fumes, etc., k_{urban} is introduced to consider the increased CO₂ levels in urban environments and assumed to be normally distributed with mean of 1.15 and Coefficient of Variation (COV) of 0.10. The effects of time-dependent temperature on the diffusion coefficient $f_T(t)$ when compared to a temperature of 20 °C is:

$$f_T(t) \approx e^{\frac{E}{R} \left(\frac{1}{293} - \frac{1}{273 + T_{av}(t)}\right)}$$
 and $T_{av}(t) = \frac{\sum_{i=2000}^{T} T(t)}{t - 1999}$ 2-10

where T(t) is the temperature at time t in \mathcal{C} , E is the activation energy of the diffusion process (40 kJ/mol – (DuraCrete, 2000; Yoon et al., 2007)), and R is the gas constant (8.314×10⁻³ kJ/mol·K).

Stewart et al. (2011) calculated carbonation depths due to enhanced atmospheric CO_2 concentration conditions using the average CO_2 concentration over the time period instead of the peak value at time t as assumed by Yoon et al. (2007). Further, this model is convenient to be included in the reliability analysis. Therefore, this model is improved herein.

2.3.2.6 Summary

The physical and chemical composition of real RC structures is highly varied and nonuniform, so it is clear that the accurate prediction of the rate of carbonation can be problematic. The carbonation of concrete structures is investigated, and the carbonation depths measured from real structures are presented as Figure 2-4 (Roy et al., 1996). Figure 2-4 shows the variability of collected data for the carbonation depth from four buildings with different service years. It is obvious from these figures that a large amount of scatter exists in the published data based on field tests. Further, to include all influential variables would require a large amount of data to be known about the structure (many of which cannot be easily inferred directly from testing) and it is likely that the resulting model would be computationally expensive. On the other hand, environmental factors and concrete characteristics are not constant with time because they may vary due to climate change and the progress of carbonation. Models that allow for carbonation depth, temperature and relative humidity have been discussed. Although models that account for more variables have the potential to produce a more accurate prediction, any inherent accuracy of the model can be made redundant by the estimation of unknown parameters.

The one-dimensional model based on Fick's law is efficient and robust and has a proven record of accuracy based on experimental testing of real structures. However, long-term effects of climate change, time-dependent environmental factors and concrete properties are critical in predicting performance of RC structures during their long service lives. As such, a 1-dimensional diffusion model will be used herein to predict the rate of carbonation in RC structures but with some modifications of time-dependent environmental factors and concrete properties. Detailed descriptions are given in Chapter 3.



Figure 2-4: Distribution of carbonation depth in four buildings in Singapore of varying ages (a) 7 years, (b) 15 years, (c) 19 years and (d) 59 years (Roy et al., 1996).

2.4 Corrosion propagation

2.4.1 General

The corrosion process of RC structures is shown as Figure 2-5. Eventually, corrosion may cause RC surface to have severe cracking and spalling.



Figure 2-5: Corrosion induced cracking and spalling (Canin, 2009).

The chemical reactions of the corrosion process are the same no matter if corrosion is induced by carbonation or chlorides which are basically an anodic and a cathodic reaction. The corrosion of steel starts by dissolving in the pore moisture and releasing electrons which can be described as an anodic reaction:

$$Fe \rightarrow Fe^{2+} + 2e^{-}$$
 2-11

The electrons were consumed by a cathodic reaction with water and oxygen:

$$2e^{-} + H_2O + \frac{1}{2}O_2 \rightarrow 2OH^{-}$$
 2-12

The anodic and cathodic reactions are just the beginning of producing hydrated ferric oxide (rust) but several more chemical reactions are needed. There are several ways to express these reactions and Broomfield (2007) shows the chemical reactions that lead to the formation of rust on a steel bar embedded in concrete by the following equations:

$$Fe^{2+} + 2OH^- \rightarrow Fe(OH)_2$$
 (Ferrous hydroxide) 2-13

$$4Fe(OH)_2 + O_2 + 2H_2O \rightarrow 4Fe(OH)_3$$
 (Ferric hydroxide) 2-14

Because the volume of unhydrated (Fe₂O₃) and hydrated ferric oxide (Fe₂O₃ H₂O) are about two to ten times greater than steel, the rust starts to expand around the reinforcing bar to cause the concrete cover to crack and spall. The corrosion rate can be defined by the electronic current flow of corrosion reactions (in μ A/cm²) or the loss of the bar section per year (in μ m/year, 1 μ A/cm² = 0.0116 mm/year).

2.4.2 Corrosion rate models

A number of models predicting corrosion rate can be found in the literature (Morinaga, 1989; Liu & Weyers, 1996; Yalscyn & Ergun, 1996; Yokozeki et al., 1997; Vu & Stewart, 2000; Raupach, 2006). These, however, were considered inappropriate for considering the climate change effects on the corrosion rate, such as exposure conditions, temperature and RH effects, etc. A practical approach to the model corrosion rate is to use an empirical relationship incorporating the parameters known to govern the process as well as exposure classes.

DuraCrete (1998) proposed a corrosion model based on this approach. The corrosion rate is assumed to have lognormally distributed variables with statistical parameters at 20 °C; see Table 2-2. These values consider the concrete grades for the corresponding exposure classes.

Mean	Standard deviation	Distribution
0.0^{a}	0.0	Lognormal
$0.345 \ \mu A/cm^2$	$0.259 \ \mu A/cm^2$	Lognormal
$0.172 \ \mu A/cm^2$	$0.086 \ \mu A/cm^2$	Lognormal
$0.431 \ \mu A/cm^2$	$0.259 \ \mu A/cm^2$	Lognormal
	Mean 0.0 ^a 0.345 μA/cm ² 0.172 μA/cm ² 0.431 μA/cm ²	$\begin{tabular}{ c c c c c c c c c c c c c c c c c c c$

Table 2-2: Carbonation corrosion rates (icorr-20) for various exposures (DuraCrete, 1998).

Note $-a^{*}$: assume negligible = 0.1 μ A/cm².

An increase in temperature will accelerate the corrosion rate. DuraCrete (2000) takes into account the temperature effects on the corrosion rate as:

$$i_{corr}(t) = i_{corr-20} [1 + K(T(t) - 20)]$$
 2-16

where $i_{corr-20}$ is the corrosion rate at 20 °C as given in Table 2-2, K is the parameter of temperature impacts on the corrosion rate, and K=0.025 when T(t)<20 °C and K=0.073

when T(t)>20 °C. DuraCrete (2000) notes that Eq. 2-16 is a close correlation to the Arrhenius equation, at least for temperature below 20 °C, but may be conservative when T(t)>20 °C. A 2 °C temperature increase will accelerate the corrosion rate by 15%.

However, the corrosion rate can be significantly influenced by RH. There is evidence to show that the corrosion rate for carbonation or chlorides becomes negligible when RH(t) is less than about 50% (Gonzalez et al., 1980; Neville, 1995). The optimum RH for corrosion is 70-80%, the corrosion rate then decreases because the concrete pores are filled with fluid that hinders diffusion of oxygen through the concrete (Neville, 1995). However, there is a large variability for corrosion rates at the same RH, and it is still possible for the corrosion rate to be high when RH is low (Andrade et al., 2002).

An empirical model that includes both RH and temperature effects on the corrosion rate is proposed by Breysse et al. (2014) based on a multi-linear regression approach on the data series recorded both in the laboratory (under well-controlled ambient conditions) and on site, on full-scale Portuguese bridge in a real environment (Breysse et al., 2007). The model shows that a lower RH will decrease the corrosion rate when RH is less than the reference RH:

$$i_{corr}(RH,T) = i_{corr(ref)}(RH_{ref},T_{ref})e^{0.0312(RH-RH_{ref})}e^{-4736\left(\frac{1}{273.15+T}-\frac{1}{273.15+T_{ref}}\right)}$$
2-17

where $i_{corr(ref)}$ is the corrosion current density at reference state ($\mu A/cm^2$), and a reference state is RH_{ref}=80% and T_{ref}=20 °C.

Note that the choice of reference values is arbitrary and has no impact. It is a simple mean used to compare measurements at different moments. The temperature and RH effects on the corrosion rate modelled by Breysse et al. (2014) model can be shown as Figure 2-6.



Figure 2-6: RH and temperature effects on the corrosion rate (Breysse et al., 2014).

2.5 Crack initiation and propagation models

2.5.1 General

Existing studies have shown that the timing of cracking in RC structures is significantly influenced by several variables, such as: concrete cover, bar diameter, concrete strength and corrosion rate (Rasheeduzzafar et al., 1992; Liu & Weyers, 1996; Vu et al., 2005; Webster & Clark, 2000; Williamson & Clark, 2000; Du et al., 2006; Rodriguez et al., 1996; Kitsutaka & Nakamura, 1999; Stewart, 1996).

Considerable research has been undertaken on the corrosion-induced cracking process. The work included experimental, numerical and analytical studies. The experimental studies (Liu & Weyers, 1998; Rasheeduzzafar et al., 1992; Andrade et al., 1993; Alonso et al., 1998; Vidal et al., 2004; Zhang et al., 2010) were mainly focused on testing the critical amount of the steel corrosion needed for cracking concrete covers and developing corresponding empirical models. Numerical studies were mainly based on the use of finite element methods to investigate the corrosion-induced cracking process and predict the corrosion of reinforcing steel when the concrete cover has cracks. A number of numerical studies (Molina et al., 1993; Hansen & Saouma, 1999; Du et al., 2006; Val et al., 2009) have also been undertaken to investigate corrosion induced cracking based on finite element models and 2D formulations. Unfortunately, these models demand excessive computational time, even for small academic structure models. Both crack initiation and propagation can be simulated, but the computational time for crack propagation is usually significantly increased compared to the crack initiation stage. Several analytical models (Bazant, 1979; Liu & Weyers, 1998; Bhargava et al., 2005; Li et al., 2006; Pantazopoulou & Papoulia, 2001; Chernin et al., 2010; EI Maaddawy & Soudki, 2007; Kim et al., 2010; Lu et al., 2011; Malumbela et al., 2011; Zhao et al., 2011) were also developed for corrosion-induced concrete cracking.

2.5.2 Crack initiation models

Numerous models for the time of first cracking and crack propagation for corroding RC structures have been developed (Bazant, 1979; Rodriguez et al., 1996; Liu & Weyers, 1998; Webster & Clark, 2000; Vidal et al., 2004; Pantazopoulou & Papoulia, 2001; Wang & Liu, 2004; Thoft-Christensen, 2005; Vu et al., 2005; Bhargava et al., 2006; El Maaddawy & Soudki, 2007; Li et al., 2007a; Zhong et al., 2010; Reale & O'Connor, 2012), some of which will be described herein.

2.5.2.1 Liu and Weyers (1998)

Liu and Weyers (1998) modified an existing model (Bazant, 1979) based on experimental corrosion tests on 44 RC slabs using high dosages of chlorides into admixes. Two effects were included: (i) the decreasing corrosion rate with time; (ii) a 'porous zone' around the reinforcement interface. Therefore, to cause a first crack, the amount of corrosion products should be enough to fill the porous zone first, and then generate the critical tensile stresses in the surrounding concrete. The critical amount of corrosion products (M_{crit}) that are needed to cause crack initiation is given by:

$$M_{crit} = \rho_{rust} \left(\pi \left(\delta_s + \delta_0 \right) D_{bar} + \frac{M_{st}}{\rho_{st}} \right)$$
2-18

where ρ_{rust} is the density of the corrosion products (kg/m³); δ_s is the thickness of corrosion products to generate tensile stresses; δ_0 is the thickness of the pore band around the reinforcing interface, 'porous zone'; D_{bar} is the reinforcing bar diameter; and M_{st} is

the mass of corroded steel that depends on the type of corrosion product ($M_{st} = \alpha_1 M_{crit}$ where $\alpha_1 = 0.523$ for Fe(OH)₃ or 0.622 for Fe(OH)₂).

The thickness of corrosion products δ_s is expressed as:

$$\delta_{s} = \frac{Cf_{t}}{E_{ef}} \left(\frac{a'^{2} + b'^{2}}{b'^{2} - a'^{2}} + v \right)$$
2-19

where C is the concrete cover (mm); f_t is the tensile strength of concrete (MPa); v is Poisson's ratio of concrete; E_{ef} is the effective elastic modulus of concrete in (MPa) and equal to $E_c/(1+\phi_{cr})$, where E_c is the elastic modulus of concrete and ϕ_{cr} is the creep coefficient. The inner radius a' = $(D_{bar} + 2\delta_0)/2$ and outer radius b'= C + a'.

Then the time to first crack is:

$$T_{1st} = \frac{M_{crit}^2}{2k_p}$$
 2-20

where k_p is the rate of rust production given as:

$$k_p = \frac{0.098}{1.08} \left(\frac{1}{\alpha}\right) \pi D_{bar} i_{corr}$$
 2-21

Vu (2003) found that the Liu and Weyers (1998) model has a reasonable fit to experimental data. However, El Maaddawy and Soudki (2007) suggest that the assumption of a decreasing rate of rust production with time used by the Liu and Weyers (1998) model underestimates the mass loss of steel, therefore, an overestimation in the time to first crack. Chernin and Val (2008) deem that the Liu and Weyers (1998) model violates Faraday's law of electrolysis.

2.5.2.2 El Maaddawy and Soudki (2007)

El Maaddawy and Soudki (2007) further developed the Liu and Weyer's (1998) model by using Faraday's Law to estimate the rate of rust production. Liu and Weyer's (1998) assumption of the 'porous zone' and the concrete around the reinforcing bar as a thickwalled cylinder was merged into their model. Concrete cracking is assumed to happen when the tensile stress caused by the expansion of corrosion products in the circumferential direction reaches the tensile strength of the concrete. Some assumptions are made: corrosion products need to fill the porous zone firstly before inducing internal stresses around the bar; the internal stresses caused by corrosion products are uniform; and the volume expansion is caused by corrosion only. Consequently, the time to first crack, t_{1st} , is:

$$t_{1st} = \left[\frac{7117.5(D_{bar} + 2\delta_0)(1 + \nu + \psi)}{365i_{corr}E_{ef}}\right] \cdot \left[\frac{2Cf_t}{D_{bar}} + \frac{2\delta_0E_{ef}}{(1 + \nu + \psi)(D_{bar} + 2\delta_0)}\right]$$
2-22

where D_{bar} is the diameter of the steel reinforcing bar (mm); δ_0 is the thickness of the porous zone around the steel reinforcing bar (µm); i_{corr} is the corrosion current density in μ A/cm²; ν is Poisson's ratio; C is the concrete cover (mm); f_t is the concrete tensile (splitting) strength (MPa); E_{ef} is the effective elastic modulus of concrete (MPa) and is equal to E_c/(1+ ϕ_{cr}), where E_c is the elastic modulus of concrete and ϕ_{cr} is the creep coefficient; and

$$\psi = \frac{\left(D_{bar} + 2\delta_o\right)^2}{2C\left(C + D_{bar} + 2\delta_o\right)}$$
2-23

The thickness of the 'porous zone' (δ_0) is usually in the range of 10-20 µm and can be described by a normal distribution with mean equal to 15 µm and COV of 0.1.

The El Maaddawy and Soudki (2007) model accounts for the concrete cover, bar diameter and concrete strength, and comparison between experimental and predicted times to first cracking shows a reasonable agreement, and as such, it will be used in the reliability analysis presented in this study.

2.5.3 Crack propagation models

2.5.3.1 Vidal et al. (2004)

Vidal et al. (2004) introduced a set of relationships relating the distribution of reinforcement corrosion to the width of cover crack. This work depends on experimental results measured from the longitudinal reinforcements of two beams which are naturally corroded for periods of 14 and 17 years. These experimental results are compared with existing models relating crack width to attack penetration. However, such models only

partially predict actual experimental data. Vidal et al. (2004) proposed a model using the parameter of reinforcement cross-section loss.

Vidal et al. (2004) proposed a model based on experimental results of two RC beams that were stored in a saline environment over periods of 14 and 17 years. The following empirical model was obtained:

$$w_{\rm lim} = 0.0575(\Delta \mathbf{A}_s - \Delta \mathbf{A}_{s0})$$
 2-24

where 0.0575 is from the regression; ΔA_s is the loss of steel cross section in mm² and ΔA_{s0} is the loss of steel cross section that is needed for crack initiation in mm². The loss of steel cross section is defined as:

$$\Delta A_s = \frac{\pi}{4} \left(2\alpha_p x D_{bar} - \alpha^2 x^2 \right)$$
 2-25

where α_p is a pit concentration factor; D_{bar} is the bar diameter and x is the attack penetration.

The loss of steel cross section that is needed for crack initiation ΔA_{s0} is estimated by Alonso et al.'s (1998) model as the following equation:

$$\Delta \mathbf{A}_{s0} = \mathbf{A}_{s} \left[1 - \left[1 - \frac{\alpha_{p}}{D_{bar}} \left(7.53 + 9.32 \frac{C}{D_{bar}} \right) 10^{-3} \right] \right]$$
 2-26

where C is the concrete cover. By combining the equations above, the time from crack initiation to the time when the crack reaches a given limit crack width t_{sev} can be estimated.

The Vidal et al. (2004) model was based on long-term experiments and considered concrete cover and bar diameter. But the model does not account for concrete strength which has been proved to be a significant factor in predicting the time of crack propagation.

2.5.3.2 Vu et al. (2005)

Vu et al. (2005) conducted an accelerated corrosion test on eight RC slabs. They proposed an empirical model to predict the time for a crack to propagate from crack initiation to a limit

crack width of 1.0 mm by the considering concrete cover (C) and water/cement ratio (w/c). The model is:

$$t_{ser} = A \left(\frac{C}{w/c}\right)^{B}$$
 2-27

where t_{ser} is the time for a crack to propagate from crack initiation to a limit crack width. Parameters A and B are empirical constants for a range of limit crack widths and a bar diameter of 16 mm, e.g. for a limit crack width of 1.0 mm, A and B are equal to 700 and 0.23, respectively.

A correction factor for the loading rate (k_R) was developed to consider the high rate of current applied in the accelerated corrosion test, and thus, the time to severe cracking is defined as:

$$t_{sev} = t_{1st} + k_R \frac{0.0114}{i_{corr(real)}} A \left(\frac{C}{w/c}\right)^B$$
2-28

where t_{sev} is the time from corrosion initiation to severe cracking (years); t_{1st} is the time from corrosion initiation to crack initiation (years), and $i_{corr(real)}$ is the corrosion rate of the structure in the field (=100 μ A/cm²).

The Vu et al. (2005) model was based on a series of accelerated corrosion tests of experimental RC slabs with a number of parallel reinforcing bars and the empirical model accounts for concrete strength and cover in predicting the timing of crack propagation. However, the model is recommended for prediction of cracking in RC structures with 16 mm bar diameters only.

2.5.3.3 Mullard and Stewart (2010)

Mullard and Stewart (2010) made some modifications to the Vu et al. (2005) model based on experimental results by incorporating effects of concrete cover, reinforcing bar diameter and concrete tensile strength on corrosion to predict the time for a crack to develop from initiation to a given limit crack width. The model can account for the time-dependent corrosion rate, the effect of reinforcement confinement and the high rate of loading used in the model development. The model is:

$$T_{sev} = k_R \frac{w - 0.05}{k_c ME(r_{crack}) r_{crack}} \left(\frac{0.0114}{i_{corr(real)}}\right) \quad 0.25 \le k_R \le 1, k_c \ge 1.0, w \le 1.0mm$$
2-29

where

$$\mathbf{r}_{crack} = 0.0008 e^{-1.7\psi_{cp}} \qquad 0.1 \le \psi_{cp} \le 1.0$$

$$k_R \approx 0.95 \left[\exp\left(-\frac{0.3i_{corr(exp)}}{i_{corr(real)}}\right) - \frac{i_{corr(exp)}}{2500i_{corr(real)}} + 0.3 \right]$$
2-30
2-30

and where $i_{corr(real)}$ is the corrosion rate of the structure in the field (μ A/cm²) and is timeinvariant; ψ_{cp} is the concrete cover cracking parameter and is equal to cover / (bar diameter × concrete tensile strength); r_{crack} is the rate of crack propagation in mm/hr; w is the crack width (mm); ME(r_{crack}) is the model error of the crack propagation model; k_R is a rate of loading correction factor, $i_{corr(exp)}=100 \ \mu$ A/cm² is the accelerated corrosion rate applied to work out r_{crack} ; and k_c is the confinement factor that considers an increase in crack propagation around external (edge) reinforcing bars due to the lack of concrete confinement. The occurrence of a confinement effect on crack propagation was observed when cracks over bars located at the edges of a RC surface crack at a higher rate to those in internal locations (Mullard & Stewart, 2010).

Some finite element (FE) models can be found in the literature (Molina et al., 1993; Thoft-Christensen, 2005; Du et al., 2006; Ahmed et al., 2007). But the diffusion of corrosion products into the pores of the concrete is generally not considered by these models. Further, these models are computationally expensive and, on the other hand, have not increased accuracy in predicting crack propagation markedly; therefore, they are not recommended for inclusion in a stochastic reliability analysis.

Mullard and Stewart (2010) developed a crack propagation model that includes the key influencing variables of corrosion rate, cover, bar diameter, concrete strength and reinforcement confinement. The model is fast and robust enough to include within a reliability analysis, and as such, it is used in this study. Chapter 3 will describe the models that are used to simulate corrosion process in details, including carbonation depth, corrosion initiation, crack inanition and crack propagation under a changing climate.

2.6 Reliability analysis

Because all the factors influencing corrosion have inherent uncertainties, such as uncertainties in material properties, environmental conditions and structural dimensions, these can influence the performance of reinforced concrete significantly. To treat these uncertainties as deterministic would severely limit the value of a predictive model and does not fit real physical properties; therefore, reliability analysis is a critical method to model RC structures.

Using probabilistic methods in structural reliability analysis has been described in detail, see Melchers (1999) and Stewart and Melchers (1997). This method has been widely used to predict the corrosion-induced deterioration, serviceability and safety of RC structures (Stewart & Rosowsky, 1998; Faber & Sorensen, 2002; Bentz, 2003; Li et al., 2004; Malioka & Faber, 2004; Karimi et al., 2005; Li et al., 2007b; O'Connor & Enevoldsen, 2007; Val & Trapper, 2008; Mullard & Stewart, 2009; Mullard, 2010; Peng & Stewart, 2014a, b). The reliability of the structure is often defined as a limit state function as:

G = R-S

2-32

where G is the limit state function; R and S are the structural resistances and the load effect, respectively.

The 'Limit State (LS)' defines 'Failure' in the structural reliability analysis. However, in the reliability analysis 'Failure' does not necessarily refer to structural collapse, but commonly it means a state when the structure exceeds a predefined limit. For example, if the LS to be considered is the corrosion initiation, then 'Failure' will happen when carbonation depth is equal to the concrete cover. Therefore, the 'LS' is a boundary between the desired and undesired condition of the structure (Nowak & Collins, 2000).

The properties of structural resistance and load effect can be described by their probability distribution functions and, assuming structural resistance and load effect are statistically independent, the probability of failure is:

$$P_{f} = P(R - S \le 0) = \int_{-\infty}^{+\infty} F_{R}(x) f_{S}(x) dx$$
2-33

where f_R and f_S are the probability density functions of R and S respectively and $F_R(x)$ is the cumulative distribution function of f_R . R and/or S may also be time-dependent functions as shown in Figure 2-7.

A novel reliability analysis allows the spatial variability of the concrete parameters, as well as time-variant nature of concrete properties (such as the diffusion coefficient) and climate conditions (CO_2 concentration, temperature and RH) to be considered. The spatial-time-dependent reliability analysis provides a realistic model of corrosion-induced deterioration of RC structures. Hence, a spatial-time-dependent reliability analysis will be used in the present study, and this is described in detail in Chapter 4.



Figure 2-7: Representation of time-variant reliability problem (Stewart & Rosowsky, 1998)

2.7 Spatial modelling of deterioration

2.7.1 General

The deterioration of RC structures is not homogeneous (Stewart et al., 2007). Variation in material properties, loading, structural dimensions and environmental exposure conditions can lead to spatially variable deterioration. For example, Leyland and Castler (2006) observed that the coating deterioration on steel structures is spatially variable, and Stewart et al. (2003) showed that deterioration due to reinforcement corrosion is spatially

variable for RC bridge decks. This section will discuss the spatial variability of corrosion damage of RC structures and review existing research in this area.

2.7.2 Spatial variable properties of RC structures

Reinforced concrete construction involves various processes including concrete batching, onsite dimensional placement, steel rebar setout, consolidation of the wet concrete and curing. These processes may introduce substantial variability of concrete quality and structural dimension due to variations in work practices. Further, concrete material properties are not homogeneous. Fazio et al. (1999) conducted an on-site survey of a 39-year-old RC bridge that had been exposed to de-icing salts and found considerable spatial variability of concrete properties, the corrosion rate and cover. The results of this survey are summarised in Table 2-3. Clearly, the spatial variability of concrete strength, concrete cover, CO_2 diffusion coefficient and the CO_2 binding capacity will affect the corrosion induced cracking of a RC structure and result in spatially distributed corrosion induced cracking. Thus, spatially variables must be included in corrosion predictive models to achieve more realistic results.

phenomena	criteria	proportion of deck (%)
	below 1 µm/yr	-
	1 to 10 μm/yr	1
corrosion rate (mean=52 μm/yr, COV=0.55)	10 to 45 μ m/yr	47
	45 to 100 µm/yr	44
	100 to 1000 $\mu m/yr$	7
	below 25 mm	29
cover	25 to 40 mm	30
(mean= 35 mm, COV=0.48)	40 to 50 mm	26
	above 50 mm	15
	below 20 MPa	29
compressive strength	20 to 30 MPa	29
(mean=27 MPa, COV=0.36)	30 to 40 MPa	29
	above 40 MPa	12

Table 2-3: Spatial variability of corrosion of a RC bridge (Fazio et al., 1999)

2.7.3 Random field modelling to predict corrosion-induced cracking in RC structures

The using of random fields to represent spatial variables (Vanmarcke, 1983) has been widely applied to simulate the spatial deterioration of RC structures (Sterritt et al., 2001; Li et al., 2004; Malioka & Faber, 2004; Karimi et al., 2005; Val, 2005; Vu & Stewart, 2005; Sudret et al., 2006; Stewart & Mullard, 2007; Sudret, 2008; Darmawan & Stewart, 2007; Na et al., 2012; Papakonstantinou & Shinozuka, 2013). Some key studies in this area will now be discussed.

2.7.3.1 Li et al. (2004)

Li et al. (2004) used random fields to represent spatial variability in a probabilistic based reliability analysis that predicts the time to corrosion induced cracking for a RC bridge beam. The spatial variability of the RC bridge beam is modelled as a 1-dimensional random field, and the beam is discretised into 54 elements of size 0.8 m \times 0.65 m. Each element is represented by a random variable calculated at the centre of the element and assumed to be constant over the whole element. The spatial correlation between elements is:

$$\rho(\tau) = \rho_0 + (1 - \rho_0) \exp\left[-\left(\frac{\tau_x}{\theta}\right)^2\right]$$
2-34

where $\rho(\tau)$ is the correlation coefficient between elements; τ_x is the distance between elements (m); ρ_0 is the common correlation between all elements and θ is the fluctuation scale (m).

Li et al. (2004) treated three parameters as random spatial variables in the probabilistic reliability analysis, namely surface chloride concentration, average corrosion rate and wetness period of the environment. A reasonable agreement between the model predictions of the reliability analysis and the observed data from a bridge was found.

2.7.3.2 Malioka and Faber (2004)

Malioka and Faber (2004) suggest that due to variability in concrete batches and workmanship during construction of RC structures, the corrosion initiation and propagation are spatially variable. The spatial variability of concrete permeability is then

modelled by a random field, and the random field is considered isotropic. The spatial variability model was integrated into a reliability analysis to predict the percentage of corrosion degradation at a specified point in time. A 2-dimensional structural member (such as a wall) is divided into a number of small areas based on a correlation study, and the correlation function is defined as:

$$\rho_{xx} = e^{-\frac{\sqrt{t_x^2 + t_y^2}}{h}}$$
2-35

where h is equal to $\theta\sqrt{2}$, θ is the scale of fluctuation and τ_x and τ_y is the cartesian distance between areas.

The probability that the percentage of elements exceed a critical degree of degradation can be described as following:

$$P(d(t) \ge n_e / N) = P\left(\bigcup_{i=j}^{K_{N,n_e}} \bigcap_{j=1}^{m_i} \{g_j(X,t) \le 0\}\right)$$
2-36

where n_e is the number of areas (of size $r \times r$) that exceed critical degradation; N is the total number of areas; $K_{N,ne}$ is the number of combinations of areas that exceed the critical percentage; m_i is the number of areas in the ith combination and $g_j(X, t)$ is the limit state function of area j. Malioka and Faber (2004) suggest that assuming each area is statistically independent, then the binomial theorem can be used to simplify Eq. 2-36 as:

$$P(d(t) \ge n_e / N) = 1 - B(n_e - 1, N, \theta(t))$$
2-37

where $B(n_e-1, N, \theta(t))$ is the cumulative Binomial distribution and $\theta(t)$ is the probability of failure of the individual elements at time t.

Malioka and Faber (2004) use independent areas to model spatial variability over a larger surface, this assumption can be justified in some circumstances (between pour breaks in a RC slab for example), however, the true spatial variability of a larger surface may be diluted.

2.7.3.3 Sudret et al. (2006)

Sudret et al. (2006) introduced the concept of 'damaged length' as an indicator of the mean portion of corrosion damage for a RC beam with spatially variable concrete properties and the expected damaged length is defined as:

$$\overline{L_d(t)} = l \cdot P_f(s, t)$$
2-38

where l is the length of beam and $P_f(s, t)$ is the point-in-space probability of failure from a non-spatial analysis, and for a given limit state criteria, the probability of failure is given by:

$$P_f(s,t) = P\left(g\left(X(s),t\right) \le 0\right)$$
2-39

where X(s) is the multidimensional random field; s is the location of the spatial random variable within the random field; t is time and g represents the limit state function where the failure limit state is represented by $g \le 0$.

The authors believe that by using the equation of 'damaged length' they can represent the mean fraction of the beam that is damaged and negate the need for discretisation and random field when calculating the mean value of the damaged length. This equation essentially means that the average damaged fraction of the beam is equal to the point-in-space probability of failure. The authors assert that using Eq. 2-38 can avoid the discretisation and random field definition to work out the mean damaged length.

Sudret et al. (2006) compared results of the damaged length based on Eq. 2-38 with those calculated by a random field method using full discretisation of the beam into spatially correlated variables. Concrete cover and corrosion current density are assumed as spatially variables in a random field approach, and a correlation function is defined as:

$$\rho(\tau) = e^{-\pi(\tau)^2/\theta^2}$$
 2-40

where τ is the distance between two elements in a random field and θ is the scale of fluctuation.

The results showed good agreement on the estimation of damaged length for both methods. However, the former method has the benefit of negating the use of discretisation of a random field.

2.7.3.4 Stewart and Mullard (2007)

Stewart and Mullard (2007) used random fields to model the spatial variability of corrosion damage for a RC bridge deck. A time-dependent reliability analysis is then used to predict the probability and extent of corrosion damage for a RC bridge deck. The correlation between elements ($\rho(\tau)$) within the random field is defined based on an exponential autocorrelation function as following:

$$\rho(\tau) = \exp\left(-\left(\frac{|\tau_x|}{d_x}\right)^2 - \left(\frac{|\tau_y|}{d_y}\right)^2\right)$$
 2-41

where $\tau_x = x_i - x_j$ and $\tau_y = y_i - y_j$ are the distances between the centroid of element i and j in the x and y axes, respectively; d_x and d_y are the correlation lengths in the x and y axes, respectively. The mean extent of a concrete surface subject to corrosion damage (μ_{dcrack}) is calculated as:

$$\mu_{d_{crack}}(t) = \sum_{j=1}^{k} \frac{1}{k} \Pr\left(t > T_{i(j)} + T_{sev(j)}\right) \times 100\%$$
2-42

where $T_{i(j)}$ and $T_{sev(j)}$ are time to corrosion initiation and the time to severe cracking of element j, respectively; and k is the total number of elements of the concrete surface. Figure 2-8 shows the probability contours of the extent of corrosion damage for bridge deck surface areas of 36 m² and 900 m² from the spatial time-dependent reliability analysis. It is obvious that the likelihood and extent of corrosion damage can be calculated, and this information is critical for further analysis of maintenance strategies, such as deciding when the first repair should happen.



Figure 2-8: Probability contours for Pr(d_{crack}(t)≥X_{repair}%) for different surface areas (Stewart & Mullard, 2007).

2.7.3.5 Summary

To model deterioration as a spatially variable (rather than homogeneous) allows more detailed information to be known about the time-based performance of the structure and thus it is likely to improve the predictive models in terms of the timing and extent of corrosion damage. For example, a RC structure modelled as homogeneous is either fully undamaged or fully damaged at any given time (refer Figure 2-9) and this type of analysis is clearly not suited for predicting the timing of maintenance actions where the extent of the damage is often the limit state for repairs. A spatially variable analysis, however, allows the extent of damage to be predicted with respect to time (refer Figure 2-10) and thus, the timing and extent of maintenance actions can be predicted. A more accurate damage costs assessment will be available. Further, a homogeneous analysis can lead to the underestimation of probabilities of failure (Stewart, 2006). The results from the spatial time-dependent reliability analysis can be integrated with a life-cycle cost

analysis to optimise maintenance strategies and durability design specifications (or adaptation strategies) for RC structures, to provide a tool to assist designers in selecting design specifications and possible maintenance strategies to better manage RC structures in terms of service life and life-cycle cost. The spatial time-dependent reliability analysis used in this study is fully described in Chapter 4.







Figure 2-10: Spatial analysis of RC surface (Mullard, 2010).

2.8 Climate adaptation strategies of RC structures and cost-benefit analysis

2.8.1 General

It has been well reported that corrosion-induced deterioration is one of the most significant and costly durability problems for RC structures. According to Yang et al.

(2006), in the United States alone there are 600,000 existing bridges that are considered to be deficient and in need of repair or replacement. Mirza (2006) reports that in 1996, Canada's municipal infrastructure deficit was CAN\$44 billion, and it was about CAN\$100 billion for all infrastructure under the various jurisdictions — federal, provincial, and others; and these infrastructure deficits are CAN\$60 billion and CAN\$125 billion, respectively, in 2006. Given the magnitude of the problem, the economic impact of deteriorating RC structures is a crucial issue, and any strategies that can reduce this effect is, therefore, highly valuable, even excluding the additional damage caused by a changing climate. For example, Yunovivh et al. (2001) estimated that improved maintenance practices in the US can reduce the annual expenditure on RC bridge decks by up to 46%. Mirza (2006) predicted that Canada's infrastructure deficit can be reduced from CAN\$1 trillion to CAN\$260 billion by 2065, if 2% of infrastructure construction costs were invested to maintenance in 2006.

There are various ways to reduce the costs of deterioration of RC structures under a changing climate. Maintenance strategies could be one solution, but some losses will still occur, and these methods will only slow down the damage process by fixing damaged members. This approach is usually applied to big projects such as bridges where frequent inspection is difficult, and a certain extent of damage is tolerable, moreover, a budget for maintenance is expected and prepared. However, for normal RC buildings, we expect them to work perfectly and with less repairs as possible during their life span or even longer. Therefore, taking account of climate adaptation strategies would be more favourable, including improving the durability design requirements and applying protection methods, etc. However, to choose an optimal strategy is not easy, it involves enormous work, including assessing damage risks, efficiency and expenditure of adaptation or maintenance methods, possible costs of RC structural damage, and choosing cost-benefit criterion, etc.

In this study, climate adaptation strategies are the primary focus. In order to assess the economic efficiency of adaptation strategies, a life cycle analysis of each alternative will be adapted, and then a cost-benefit analysis is used to find out the most efficient

adaptation strategy by comparing net present values. The principle is that if a certain adaptation strategy is applied, then the reduced damage losses are the benefits. The adaptation strategy is cost effective if the benefit is higher than the cost to conduct it. Therefore, the maintenance measures are taken into account to work out the damage losses for comparison purposes. The following sections will describe and discuss both the maintenance measures and adaptation strategies based on existing practice and literature.

2.8.2 Maintenance strategies

Maintenance strategies are a key part of the lifetime management of RC structures and much research has been conducted in this area to optimise both performance and resource expenditure (Englund et al., 1999; Liu & Frangapol, 2004; Thoft-Christensen, 2004; Li et al., 2005; Yang et al., 2006). Generally, the following criteria need to be defined and integrated into a systematic management plan:

• Inspection regime – this includes the inspection interval and the type of inspection that is conducted.

• Damage limit state – this is required to define the condition of the structure when it is deemed that maintenance/ repair is necessary.

• Repair technique – this will determine the type of repair or maintenance action that is implemented once the damage limit state has been exceeded.

2.8.2.1 Inspection regime

The most common method of inspection is a periodic visual inspection (Li, 2004; Nasu et al., 2004; Zandonini et al., 2004; Broomfield, 2007; Sommer et al., 1993) with various levels of thoroughness and detail depending on such things as the type of infrastructure, the age of the infrastructure and the time between inspections. There are other methods to be employed to gather information on the condition of RC infrastructure. These can include such things as remote sensors that record information on corrosion-induced deterioration or further testing of RC structures to determine the extent of corrosion related activity that was observed during a visual inspection. Broomfield (2007) describes a range of inspection techniques for assessing the condition of RC structures, e.g. halfcell potential mapping, measuring resistivity, corrosion rate measurement. impact/ultrasonics, petrography and radar, etc. New techniques for monitoring corrosion are being developed rapidly. For example, fibre optics sensors can be used to monitor cracking, and an acoustic emission technique has been proposed to monitor the onset of corrosion and first cracking in RC structures.

Usually, the time between inspections depends on the controlling authority's policy or the prior experience of the asset owner/operator. For RC structures in particularly aggressive environments, it may be necessary to conduct more regular inspections to detect damage. Similarly, elements of the RC structure that are more structurally significant may require more regular inspections. Further, the inspection interval relies on the type of inspection interval of one or more years is most likely best suited for monitoring corrosion induced damage, and this has been recommended by others (e.g., de Brito and Branco (2004) suggest 15 months; Li (2004) suggests regular inspections twice yearly by non-technical staff and detailed technical inspection every five to ten years by a specialist; Nasu et al. (1993) conclude that inspection intervals vary from one country to another, ranging from a visual inspection four times a year to a main inspection every 6 years. However, inspection intervals should be determined based on the individual structure and the environment in which it is located.

2.8.2.2 Damage limit state

The damage limit state is defined as the amount of damage that is observed at the time it is deemed that repairs are necessary, and it is usually set by the asset owner/operator based on prior experience. As the primary method of determining the condition of a RC structure is a visual inspection (refer Section 2.8.2.1), it follows that the damage limit state is typically based on the observation of corrosion induced damage. For example, the damage limit state could be reached when a given percentage of the bridge deck is visually observed to have cracked.

A number of different definitions of the damage limit state can be found in the literature relating to corrosion-induced deterioration of RC structures. The Netherlands road authority defined the damage limit state as when 0.5% - 1.5% of a concrete surface is
visibly damaged (Directoraat-Gerneraal-Rijkswaterstaat, 2000). Li (2004) proposed that patch repair is required when 5% of the concrete surface is visibly damaged. Englund et al. (1999) suggest patch repair for 5% of concrete surface damage and replacement of damaged area when 30% of the structure is visibly damaged by corrosion. Fitch et al. (1995) recommend 12% of concrete surface damage for a complete rehabilitative overlay. Stewart (2006) proposed a varying damage limit state ranging from 0.5 % to 2.5 % based on the type of RC element for patch repair.

It is proposed that the damage limit state used in the reliability analysis presented herein will be based on the extent of corrosion damage on a RC surface and it is expected that this can be determined through regular visual inspections.

2.8.2.3 Repair techniques

Repairs are generally conducted on RC structures once a defined damage limit state has been exceeded (see Section 2.8.2.2) and there are numerous methods available to both reinstate the damaged material and prevent further deterioration of the structure, i.e. patch repairs; surface treatments; corrosion inhibitors and special concrete; electrochemical systems; and reinforcing coatings.

Patch repair is the most common technique to repair corrosion damage in RC structures (BRE, 2003; Canisius & Waleed, 2004; Zhang & Mailvaganam, 2006) and Figure 2-11 shows a corrosion damaged RC surface that is suitable for patch repair. The repair technique involves the removal of damaged concrete and replacement with a suitable repair material. The three main stages in the patch repair process are:

i. Removal of the damaged concrete;

ii. Preparation of the steel and substrate (including the possible removal and replacement of the reinforcing bar that has lost excessive cross-section) and;

iii. Application of the repair material.



Figure 2-11: Corrosion damaged RC surface (Mullard, 2010).

2.8.3 Adaptation measures

Adaptation measures to corrosion damaged reinforced concrete can make a big difference in improving the durability and economic performance of the infrastructure. Adaptation techniques must be selected based on the efficiency and specific environmental and structural conditions whilst also being economically viable both in the short and long term.

Adaptation measures are generally conducted on RC structures at the design stage or before corrosion starts, and there are numerous methods available. For RC structures subject to carbonation induced corrosion there are five main categories of adaptation techniques:

- i. Increase concrete cover;
- ii. Increase concrete grade;
- iii. Corrosion inhibitors and special concrete;
- iv. Concrete surface coatings; and

v. Other adaptation strategies, such as reinforcing coatings, stainless steel and a galvanised reinforcement, etc.

Adaptation measures will be discussed in detail in Chapter 6.

2.8.4 Life cycle analysis

The life cycle costs for a RC structure are typically calculated as the sum of all construction related costs plus the cost of maintenance or 'failure' of the infrastructure. For example, Stewart (2006) suggests that the total life cycle costs of a RC structure can be defined by the following equation:

$$LCC(T) = C_D + C_C + C_{OA} + C_{IN}(T) + E_{SF}(T)$$
 2-43

where C_D is the design cost, C_C is the construction cost (materials and labour), C_{QA} is the cost of quality assurance/control, $C_{IN}(T)$ is the cost of inspections and $E_{SF}(T)$ is the expected cost of repair or rehabilitation of corrosion-induced damage during service life T.

Li (2004) uses a life cycle cost analysis to optimise the maintenance strategy only and proposes that the total maintenance cost (CL) over the lifetime of a RC structure can be defined as:

$$C_{L} = (C_{PM} + C_{IN} + C_{CM} + C_{USE}) + (C_{F} \cdot P_{F})$$
2-44

where C_{PM} is the cost of preventative maintenance, C_{IN} is the cost of inspections, C_{CM} is the cost of corrective maintenance, C_{USE} is the user delay cost, C_F is the cost of durability failure and P_F is the probability if durability failure.

Thoft-Christensen (2004) suggests that the estimation of expected service life costs for a RC bridge, E[C], can be described by:

$$E[C] = E[C_M] + E[C_U] + E[C_F]$$
2-45

where $E[C_M]$ is the expected cost of maintenance, $E[C_U]$ is the expected user cost due to maintenance disruptions and $E[C_F]$ is the expected cost of failure during the service life of the bridge.

Lizhengli Peng

The models discussed above are in principle all similar in that they include costs that will be incurred throughout the life of the infrastructure. Therefore, the effect of time on the value of that money must be accounted for. To enable a more meaningful comparison of design, construction and damage loss costs it is common to use a 'discount rate' so that all costs can be expressed in terms of a common notion such as present value, annualised cost or future cost (Corotis & Gransberg, 2005). Describing future costs by their value in present day value is a common technique used in the cost-benefit analysis and this has been defined by, amongst others, Matsumoto and Frangopol (2004), Stewart (2006) and Bastidas-Arteaga and Stewart (2014 a, b). The present value (PV) of the cost incurred in the future can be described by:

$$PV = \frac{FV}{\left(1+r\right)^{n_y}}$$
 2-46

where FV is the future cost, r is the discount rate and n_y is the number of years in the future at which the cost is incurred.

2.8.5 Reliability and cost-benefit analysis of climate adaptation strategies

Reliability analysis provides decision makers with a visual representation of the:

• expected value and range of variability due to risk and uncertainty on each of the variables modelled;

• relationships between these variables and estimated possible outcomes; and

• expected value and range of the possible outcomes, representing the combined effect of the multiple sources of uncertainty.

The impacts of different climate adaptation strategies on the durability performance of RC structures can be assessed using a reliability analysis. Previous studies using this type of analysis can be found in the literature (e.g., Englund et al., 1999; Val & Stewart, 2003; Li, 2004; Liu & Frangapol, 2004; Matsumoto & Frangopol, 2004; Petcherdchoo et al., 2004; Thoft-Christensen, 2004; Li et al., 2005; Straub & Faber, 2005; Stewart, 2006; Yang et al., 2006; Li et al., 2007; Mullard, 2010; Stewart & Peng, 2010; Stewart et al., 2014; Bastidas-Arteaga & Stewart, 2014a, b), although most work assumes homogenous

RC structures and does not include the spatial variability of corrosion damage. Bastidas-Arteaga and Stewart's (2014a, b) approach will be described herein.

Bastidas-Arteaga and Stewart (2014a, b) assumed that corrosion damage is always detected when the structure is inspected then the expected damage cost $E_{damage}(T)$ is the product of probability of corrosion damage and damage costs, i.e.,

$$E_{damage}(T) = \sum_{n=1}^{T/\Delta t} \sum_{i=n}^{T/\Delta t} \left[p_{s,n}(i\Delta t) - p_{s,n}(i\Delta t - \Delta t) \right] \frac{C_{damage}}{(1+r)^{i\Delta t}}$$
2-47

where Δt is the time between inspections, n is the number of damage incidents, i is the number of inspection, $p_{s,n}(t)$ the probability of the nth damage incidence before time t, C_{damage} is the cost of damage including maintenance and repair costs, user delay and disruption costs, and other direct or indirect losses arising from damage to infrastructure. For instance, an asset owner should be able to quantify the unit repair cost (\$/m²), and if the area of damage is known then repair cost can be estimated.

In addition, the time-dependent damage risks of the repaired material will not be the same as the original material $p_s(t)$ due to changed temperature and humidity at the time of repair (i.e., when the concrete is new). Hence, the damage risk for repaired (new) concrete exposed to the environment for the first time at time of repair, $t_{rep} = i\Delta t$, will change depending on the new climatic conditions and time of repairs:

$$p_{si}(i\Delta t, t) = \Pr[t \ge T_{sev,i}]$$
2-48

where $T_{sev, i}$ is the time to severe cracking when new concrete is exposed to the environment for the first time after repair. Eq. 2-47 can be written in a different form as:

$$E_{damage}(T) = \sum_{i=1}^{T/\Delta t} \Delta P_{s,i} \frac{C_{damage}}{(1+r)^{i\Delta t}}$$
2-49

where $\Delta P_{s,i}$ is the probability of damage incident between the (i-1)th and ith inspections which is a function of time since the last repair which is affected by damage risks for original and repaired concrete Δp_s (0,t) and $\Delta p_{s,i}$ (i $\Delta t,t$), respectively. Eq. 2-49 can be readily solved by Monte-Carlo simulation methods. Cost-benefit analysis of climate adaptation strategies can be conducted using various indicators. Generally, the net present value (NPV) is used to make a decision. If the NPV is positive then, the project should be adopted. However, there might be some restrictions in using net present value, i.e. the impact of budget constraints; complementarity among projects; the interaction of budget constraints and project timing choice; and comparison of projects with different lengths of life (COA, 2006). Other decision rules may apply, but should be used with caution, such as the internal rate of return, the benefit-cost ratio (BCR); and the payback period (COA 2006).

The NPV of a proposal can be calculated as:

$$NPV = \sum_{t=0}^{t} B(t) - \sum_{t=0}^{t} C(t)$$
 2-50

where B(t) is benefit at time t expressed in present value, C(t) is cost at time t expressed in present value. Another widely used indicator is BCR:

$$BCR = \frac{\sum_{t=0}^{t} B(t)}{\sum_{t=0}^{t} C(t)}$$
2-51

Bastidas-Arteaga and Stewart (2014a, b) assumes that many input variables are random variables and so BCR as the output of the analysis is also variable. This allows 10^{th} and 90^{th} percent lower and upper bounds of BCR to be estimated, as well as the probability that an adaptation measure is cost-effective at time T denoted herein as Pr(BCR>1).

However, this study assumed homogenous RC structures and did not include the spatial variability of corrosion damage, and this can lead to oversimplify the repair strategy. On the other hand, using BCR only as indicators to decide whether the climate adaptation is cost effective may be misleading. NPV should be the primary decision rule.

2.8.6 Summary

In order to compare the economic performance of RC structures, it will be necessary to include all cost information from design and construction right through to failure. As the

cost is often a key performance indicator, this will allow optimised decision making for both design and management of RC infrastructure. Life cycle analysis has been discussed, and the life cycle cost (LCC) of each adaptation strategies can be compared with the LCC of 'business as usual' strategy, to work out whether the adaptation strategy is cost effective or not. Alternatively, the assessment of 'cost' and 'benefit' of adaptation strategies can be expressed as cost of adaptation strategies and reduced damages losses, respectively. The cost of adaptation strategies is associated with design and construction, while damage loss costs are associated with structural failure including maintenance/repair costs and indirect losses. When the 'benefit' is higher than the 'cost', then this adaptation strategy is cost effective.

As discussed in Section 2.8.1, this study is mainly focused on the cost-effectiveness of adaptation strategies, so maintenance strategies will be simplified and are assumed to be identical for the assessment of damage costs of each adaptation strategy, see chapter 6. Studies about various maintenance strategies can be found in other works (Mullard, 2010; Mullard & Stewart, 2012; Chen & Alani, 2013; Frangopol, 2010; Papakonstantinou & Shinozuka, 2014; Sheils et al., 2010; Mori & Ellingwood, 1994; Sommer et al., 1993).

The design and construction costs are relatively easy to quantify and as such, the focus of research should be on the prediction of the cost of failure. The cost of failure is a function of the probability of failure, and the estimation of these is, therefore, a primary aim of the current study. To estimate the cost of failure accurately and realisticly, the following key issues need to be addressed:

• Deterioration modelling – the mechanisms and processes of the corrosion process must be accurately modelled so that the timing of corrosion damage can be predicted.

• Spatial Variability – the spatial nature of corrosion damage over a RC surface must be modelled so that both the likelihood and extent of corrosion damage can be estimated at a given time.

• Influence of adaptation efficiencies – the integration of adaptation strategies into the modelling process will influence the probability and extent of corrosion damage and therefore must be included.

• Indirect costs – such as business and economic disruption, etc. can be significant (Stewart et al., 2014; Yunovich et al., 2001) and should, therefore, be included in the cost of failure.

The processes described above form part of the key research development of this study and were critical to an accurate cost-benefit analysis. A cost-benefit analysis will be integrated into the spatial time-dependent reliability analysis conducted herein, see Chapter 7.

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86

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88

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Chapter 3 : Carbonation induced corrosion models

3.1 Introduction

This chapter presents models that will be used to predict carbonation depth and the timing of corrosion initiation and corrosion damage for RC structures in the atmosphere under a changing climate. An improved model for carbonation depth and models for the corrosion process (corrosion initiation, crack initiation and severe corrosion) considering climate change are presented and discussed. The preliminary models of carbonation depth developed by CEB (1992), Yoon et al. (2007) and Stewart et al. (2011) are improved by four aspects: (1) CO₂ levels, temperature and RH are time-dependent variables, (2) a correction factor k_{site} is introduced to take into account increased CO₂ levels in non-remote environments, (3) time-dependent change in the diffusion coefficient due to changes in temperature, and (4) time-dependent change in the diffusion coefficient due to changes in RH. Time-dependent temperature and RH effects on the corrosion rate are also included.

3.2 Corrosion initiation due to carbonation

3.2.1 Carbonation depth models

As discussed in the literature review, some improvements of climate change effects are required for existing carbonation models. The improvements are mainly based on Fick's first law, considering time-dependent temperature and RH, etc. effects on the diffusion coefficient. Because the CO_2 diffusion process takes a long time, carbonation depth is dominated by diffusion, and it can be regarded as a steady state. For a RC structure built in 2010, the carbonation depth at year t is predicted as:

$$x_{c}(t) = \sqrt{\frac{2D_{CO_{2}}(t)}{a}} \int_{2010}^{t} f_{T}(t) f_{RH}(t) k_{site} C_{CO_{2}}(t) dt} \left(\frac{t_{0}}{t - 2009}\right)^{n_{m}} \quad t \ge 2010$$
3-1

where t is defined in calendar years starting from 2010, assuming the buildings goes into service in the year 2010; $x_c(t)$ is the carbonation depth at time t; $C_{CO_2}(t)$ is the timedependent increase in atmospheric CO₂ concentration (10⁻³kg/m³) as shown in Figure 5-1 (conversion factor 1 ppm = 0.0019×10⁻³ kg/m³); k_{site} is the factor to account for increased CO₂ levels in non-remote environments; $f_T(t)$ is time-dependent change in the diffusion coefficient due to changes in temperature; $f_{RH}(t)$ is the time-dependent change in the diffusion coefficient due to changes in RH; t_0 is one year; the age factor for microclimatic conditions n_m is related to the frequency of wet and dry cycles, and is equal to 0 for sheltered outdoor exposures, and is equal to 0.12 for unsheltered outdoor exposures. The CO₂ diffusion coefficient in concrete calibrated for time period 2010 to t is:

$$D_{CO_{2}}(t) = D_{1}(t - 2009)^{-n_{d}}$$
3-2

where D_1 is CO_2 diffusion coefficient at t=2010; and n_d is the age factor for the CO_2 diffusion coefficient. The model error of carbonation depth prediction model will be considered by including the variability of climate projections and concrete material properties.

The mean values for D_1 and n_d are given in Table 3-1 as a function of the water/cement ratio (w/c), and Yoon et al. (2007) provided estimates of maximum (95th percentile) values for D_1 and n_d . The standard deviation for D_1 , and COV for n_d are assumed to be about 0.15 and 0.12 respectively (Stewart et al., 2011). These statistics represent model error (or accuracy). The CO₂ diffusion coefficient is less than 5×10^{-4} cm²s⁻¹ which is appropriate for good quality concrete (Sanjuan & del Olmo, 2001). These parameters are based on T=20 °C and RH=65%.

w/c	$D_1 \times 10^{-4} \text{ cm}^2 \text{s}^{-1}$	n _d
0.45	0.65	0.218
0.50	1.24	0.235
0.55	2.22	0.240

Table 3-1: Mean parameter values (Yoon et al., 2007).

Note - for intermediate values use linear interpolation.

The amount of carbon dioxide that is needed for carbonation of the alkaline compounds contained in a unit volume of concrete is defined as parameter 'a' (in g/m^3), and is also referred to as the concrete binding capacity. The parameter binding capacity depends on chemical compositions of concrete, such as cement composition, type and concentration of mineral additions and mix proportions of the concrete. Methods to calculate the parameter binding capacity from data on the type of the cement, the mix proportions of the concrete and the type, as well as the amount of pozzolanic additions, are given, for

example, in the paper by Papadakis et al. (1992). For a given concrete, the parameter binding capacity relies on the degree of hydration of the cement. There might be a gradient of the degree of hydration from the inside towards the concrete surface owing to curing effects. Additionally, the CO_2 concentration has an effect on the parameter binding capacity. For example, carbonation reactions occur only for high CO_2 concentration, under atmospheric CO_2 concentration some hydroxide does not undergo carbonation. In this study, parameter binding capacity for Ordinary Portland Cement (OPC) concrete is assumed to be:

$$a = 0.75C_e C_a O \alpha_H \frac{M_{CO_2}}{M_{C_aO}}$$
3-3

where C_e is cement content (kg/m³); C_aO is C_aO content in cement (0.65); M_{CaO} is molar mass of C_aO equal to 56 g/mol; and M_{CO_2} is molar mass of CO_2 equal to 44 g/mol; α_H is a degree of hydration, the degree of hydration for OPC after more than 400 days is estimated as (de Larrard, 1999):

$$\alpha_{\rm H} \approx 1 - e^{-3.38 {\rm w/c}}$$

3.2.2 Temperature effects on the diffusion coefficient

A higher temperature will result in an increase in the diffusion coefficient due to increased molecular activity, and these can lead to increased carbonation depths. The effect of temperature on the diffusion coefficient is commonly modelled using the Arrhenius Law (e.g., Saeki et al., 1991; Saetta et al., 1993; Song et al., 2006), where the time-dependent temperature change effects on the diffusion coefficient compared to 20 % is:

$$f_T(t) = e^{\frac{E}{R} \left(\frac{1}{293.15} - \frac{1}{273.15 + T(t)}\right)}$$
3-5

where T(t) is the temperature at time t in \mathcal{C} , E is the activation energy of the CO₂ diffusion process (40 kJ/mol) (Kada-Benameur et al., 2000), and R is the gas constant (8.314×10⁻³ kJ/mol^{-K}). A 2 \mathcal{C} temperature rise will increase the diffusion coefficient by



Figure 3-1: Temperature effects on CO₂ diffusion coefficient.

3.2.3 Relative humidity effects on the diffusion coefficient

The RH effect on the CO₂ diffusion coefficient is a tricky problem. Richardson (1988) described the effects of relative humidity and rainfall as following: "if the relative humidity equals 25% and less, then insufficient water is available for carbonation to commence. On the other hand, the concrete carbonation reaction is very slow at very high relative humidity. Therefore, the limits of RH model can be defined as 25%". Verbeck (1958) supported this point, as well as that the rate of carbonation is practically nil at RH of 25% and 100%. These can be explained because gaseous diffusion will slow down when concrete pores are filled with water, and the maximum carbonation rate is observed when moisture content is intermediate. The optimal condition for chemical progress of carbonation in concrete is found when RH is 50% to 70% (Russell et al., 2001; Muller & Sickert, 1995).

In this study, the influence of RH on the CO_2 diffusion coefficient is taken into account using a model recommended by the fib Model Code for Service Life Design (fib, 2006) with a modification on the lower bound:

$$f_{RH}(t) = 0 \qquad \qquad RH(t) \le 25\%$$

$$f_{RH}(t) = \left(\frac{1 - \left(\frac{RH(t)}{100}\right)^{f_e}}{1 - \left(\frac{RH_{ref}}{100}\right)^{f_e}}\right)^{g_e} \qquad RH(t) > 25\%$$

where RH_{ref} is the reference condition when RH equals 65% and the temperature equals 20 °C, f_e is a constant equal to 5.0, and g_e is a constant equal to 2.5. The RH factor f_{RH} exceeds one when RH is less than 65% and reduces to zero when RH is equal to 100%. Because there is insufficient water for carbonation to take place when RH is below 25% (Richardson, 1988; Verbeck, 1958); hence, a lower limit of RH=25% is set. The relationship between f_{RH} and RH is shown in Figure 3-2.



Figure 3-2. RH effects on the diffusion coefficient

3.2.4 k_{site} parameter

The CO_2 concentration predicted by the IPCC (see Figure 2-1 and Figure 5-1) is the global mean value which is collected at remote marine sea level locations, such as Mauna Loa in Hawaii and Wisconsin in the Great Lakes region. However, the CO_2 level in urban, suburban and rural areas is usually higher than the global mean value because of anthropogenic and natural sources. Many studies have shown elevated CO_2 levels due to higher pollution and exhaust fumes, etc. in urban environments. Stewart et al. (2002) recorded CO_2 concentrations of as high as 575 ppm in Brno (Czech Republic), with CO_2

3-6

concentrations higher near street level. George et al. (2007) found that CO_2 concentrations in an urban area (Baltimore) were on average 16% higher than a rural area, and increases of 21-31% were reported in the literature. Day et al. (2002) observed an average enhancement over the course of the day in CO_2 concentration near an urban centre (Phoenix) of 19 ppm. A new factor, k_{site} is introduced as

$$k_{site} = \frac{CO_2 concentrationatsite}{globalmeanCO_2 concentration}$$
3-7

Since the size of every city is different, the definitions of rural, suburban and urban areas are based on the size and population of the city. Table 3-2 defines an urban area based on population and distance from the central business district (CBD) or downtown area. If the city's population is less than 0.5 million then, the city is considered as suburban. A rural area is defined as the area outside of cities and towns, and typically much of the land is devoted to agriculture. For an area that is neither rural nor urban, it is categorised as suburban.

Table 3-2: Definition of urban areas for k_{site}.

	Population (million)				
	0.5-1	1-2	2-5	5-10	>10
Distance from CBD or downtown	$\leq 2 \text{ km}$	≤3 km	≤5 km	≤7 km	≤10 km

Recorded CO₂ measurements listed in Table 3-3 were obtained for rural, suburban and urban locations in the China, U.S., U.K., Italy, Poland, Japan, and Kuwait (Aikawa et al., 1995; George et al., 2009; Gratani & Varone, 2005; Han et al., 2009; Idso et al., 2002; Kuc & Zimnoch, 1998; Moriwaki & Kanda, 2004; Nasrallah et al., 2003; Rigby et al., 2008; Wang et al., 2002; Wang et al., 2010; Zhang et al., 2008) and values of k_{site} were calculated for these three location categories. Figure 3-3 presents the estimated k_{site} from the above references, and Table 3-4 shows the statistical parameters for k_{site} obtained from the data. Although the sample size is relatively small, as expected, k_{site} increases for urban areas, most likely due to higher pollution levels and population density.

References	Location	Height	Year	Average CO_2 concentration (ppm)	World average at the same period (ppm)
(Gratani & Varone, 2005)	5 different traffic level zones in Rome	2m above the ground	1995-2004	suburban:405±34 urban: 477±53	369
(Idso et al., 2002)	a residential area of a suburb of Phoenix, AZ	2m above the ground	1 Jan to 31 Dec 2000	suburban: 390.2±0.2	368.78
(Nasrallah et al., 2003)	Al-Jahra, 35 Km west from the CBD of Kuwait City	10m	17 Jun 1996 to May 2001	rural:369.19	365.959
(Zhang et al., 2008)	3 GAW regional stations, 3 cooperative stations, a started CO ₂ flask sampling, at least 50 Km from cities, China		2003-2006	rural: 378.4±1.7, 382±5.3, 383.8±6.4, 383.8±4.1, 390.3±8.1, 380.3±3.2, 385.3±3.4	377.5449,377.4743,3 77.7414,377.7819,37 7.7819,377.2935,377. 4743
(Wang et al., 2010)	in Miyun County, 100 Km northeast of Beijing urban centre	6m above the ground	Dec 2004 to Feb 2009	rural: 392.17, 397.02, 399.62,400.35	378.75, 380.89, 382.69, 384.8
(Wang et al., 2002)	Beijing meteorological 325 m tower, between 3 rd and 4 th ring road of Beijing	32m above the ground	1993-2000	urban:376.417 ±21.7,392.428 ±1 6.9,412.568 ±25.6,400.081 ±27.2 ,404.21 ±34,388.501 ±29.6,399.4 77 ±24,403.304 ±29.9	356.58, 358.03, 359.84, 361.62, 362.73, 365.47, 367.56, 368.78
(Han et al., 2009)	Peking University, between 4 th and 5 th ring road of Beijing	15m above the ground	Nov 2005 to Oct 2006	urban:438±37	378.75
(Moriwaki & Kanda, 2004)	residential area in Kugahara, about 10 Km from CBD of Tokyo, Japan	29m above the ground	May 2001 to Apr 2002	suburban:386.12	370.35
(Rigby et al., 2008)	Queen's tower on the imperial college campus in south Kensington, 5 Km to the west of the geometrical centre of London	87m	Jul 2006 to the end of Jun 2007	urban:396.305	381.7225
(George et al., 2009)	Baltimore city centre; a city park on the outskirts of Baltimore, 11 Km from the Baltimore city centre site; an organic farm, 87 Km from the Baltimore city centre site		2002-2006	rural:412±42,395±25,423±42,43 9±40,402±36 suburban:435±45,447±52,441±3 7,439±26,449±36 urban: 476±24,537±20,477±23,481±23 448±22	372.39, 374.93, 376.74, 378.72, 380.89
(Kuc & Zimnoch, 1998)	Krakow (Southern Poland) west of city bordering sports grounds	17m above ground	1983-1994	suburban:373	351.17
(Aikawa et al., 1995)	Nagoya, Japan	6	Nov 1990- Dec 1993	urban:381,382,377	355.48,356.29,356.99
(Derwent et al., 1995)	Kerb of Exhibition Road, South Kensington, London, UK	5m above the ground	Jul 1991- Jun 1992	urban:411.5	355.54

Table 3-3. Examples of studies of measuring CO₂ concentration in sites with summary details of measurements and key results.



Note - Beijing (1) and (2) means CO_2 observation made at two different locations in Beijing, the same for London (1) and (2).

Figure 3-3. Investigations of k_{site}.

Location	Sample size	Mean	COV
Remote	-	1.00	-
Rural	3	1.05	0.04
Suburban	5	1.07	0.06
Urban	8	1.14	0.08

Table 3-4: Statistical parameters for k_{site}.

3.3 Modelling time to corrosion initiation (T_i)

As time goes by, carbonation depth increases. Yoon et al. (2007) report that the corrosion may commence when the distance between the carbonation front and the reinforcement bar surface is less than 1-5 mm. However, durability design specifications tend to ignore this effect (DuraCrete, 2000; fib, 2006). Thus, corrosion initiates (T_i) when carbonation depth reaches the reinforcing bar, where T_i is the year when the carbonation depth exceeds concrete cover.

3.4 Corrosion propagation

As discussed in chapter 2, the carbonation induced corrosion rate may vary from negligible values to a maximum which highly depends on concrete microstructures, exposure conditions and atmospheric situations (DuraCrete, 1998). DuraCrete (1998)

suggests a series of values for corrosion rates. These values take into account the concrete grades suggested for the corresponding exposure classes. However, DuraCrete (2000) indicates that the standard deviation in Table 3-5 does not include the variation caused by changes of RH and temperature. Therefore, it is reasonable to consider their effects separately.

Exposure class	Mean	Standard deviation	Distribution		
C1 - Dry	0.0^{a}	0.0	Lognormal		
C2 - Wet- rarely dry (unsheltered)	$0.345 \ \mu A/cm^2$	$0.259 \ \mu A/cm^2$	Lognormal		
C3 - Moderate humidity (sheltered)	$0.172 \ \mu A/cm^2$	$0.086 \ \mu A/cm^2$	Lognormal		
C4 - Cyclic wet-dry (unsheltered)	$0.431 \ \mu A/cm^2$	$0.259 \ \mu A/cm^2$	Lognormal		
$N_{1} = \frac{a}{1} + \frac{1}{11} + $					

Table 3-5: Carbonation corrosion rates (i_{corr(ref)}) for various exposures (DuraCrete, 1998).

Note - ^a: assume negligible = $0.1 \ \mu\text{A/cm}^2$. $1 \ \mu\text{A/cm}^2$ = $0.0116 \ \text{mm/year}$.

Like many chemical reactions, the reactions of corrosion occur faster at higher temperatures, and it has been commonly observed that increased temperature will increase the corrosion rate (Neville, 1995). It is very common to use the Arrhenius-equation to model the temperature effects on the corrosion rate (DuraCrete, 2000; Chrisp et al., 2001), as Eq. 2-16 shows.

Because the corrosion reactions consume oxygen and water, the reactions will be slowed down when RH is either too high or too low. A very high humidity in concrete may reduce oxygen diffusion through the concrete while water shortage in concrete also declines the corrosion rate. The corrosion rate for carbonation or chlorides becomes negligible when RH is less than about 50% (Gonzalez et al., 1980; Neville, 1995). The optimum RH for corrosion is 70-80%, and then the corrosion rate decreases as RH increases (Neville, 1995).

An empirical model that includes both RH and temperature effects on the corrosion rate is proposed by Breysse et al. (2014):

$$i_{corr}(RH,T) = i_{corr(ref)}(RH_{ref},T_{ref})e^{0.0312(RH-RH_{ref})}e^{-4736\left(\frac{1}{273.15+T}-\frac{1}{273.15+T_{ref}}\right)}$$
3-8

where a reference state is RH_{ref} =80% and T_{ref} =20 °C.

It must be noted that the choice of reference values is arbitrary and has no impact. It is a simple mean used to compare measurements at different moments. The corrosion rate at a reference state is assumed to be lognormally distributed with statistical parameters shown in Table 3-5, and the temperature and RH effects on the corrosion rate are modelled as in the Breysse et al. (2014) model.

There is limited data on time-dependent effects on the corrosion rate for carbonated RC structures. Hence, the present analysis ignores time effects on the corrosion rate. Vu and Stewart (2000) suggest that this could be a conservative assumption as a corrosion rate will generally reduce with time due to the production of rust that can hinder the corrosion reactions.

3.5 Crack initiation and propagation models

Corrosion-induced concrete cover cracking and damage can be found along the steel bar. The various stages of crack growth can be described in three stages:

- 1. T_i time to corrosion initiation
- 2. t_{1st} time from corrosion initiation to crack initiation (visible crack widths of 0.05 mm)
- 3. t_{sev} time for a crack to propagate from crack initiation to a limit crack width.

The time to corrosion damage (severe cracking or spalling) is thus $T_{sev} = T_i + t_{1st} + t_{sev.}$ As discussed in Chapter 2, El Maaddawy and Souki's (2007) model is used here to predict the time from corrosion initiation T_i to crack initiation t_{1st} . The model has considered concrete strength, cover and bar diameter. The model assumes that corrosion products need to fill a porous zone firstly around the steel reinforcing bar before they induce expansive pressure on the surrounding concrete. Concrete cracking is assumed to happen when the tensile stress caused by the expansion of corrosion products in the circumferential direction reaches the tensile strength of the concrete. The internal radial pressure to cause concrete cracking was related to the steel mass loss. Faraday's law was used to estimate the time that is needed for steel mass loss.

The time from corrosion initiation to crack initiation (t_{1st} in years) is calculated by the method proposed by El Maaddawy and Soudki (2007) for the condition of constant $i_{corr(ref)}$:

$$t_{\rm 1st} = \left[\frac{7117.5(D_{bar} + 2\delta_0)(1 + \nu + \psi)}{365i_{corr(ref)}E_{\rm ef}}\right] \cdot \left[\frac{2Cf_{\rm t}}{D_{bar}} + \frac{2\delta_0E_{\rm ef}}{(1 + \nu + \psi)(D_{bar} + 2\delta_0)}\right]$$
3-9

Lizhengli Peng

where D_{bar} is the diameter of the steel reinforcing bar (mm); δ_0 is the thickness of the porous zone around the steel reinforcing bar; v is Poisson's ratio of concrete (0.2); C is the concrete cover (mm); f_t is the concrete tensile (splitting) strength (MPa); E_{ef} is the effective elastic modulus of concrete (MPa) and is equal to $[E_c/(1+\phi_{cr})]$, E_c is the elastic modulus of concrete, ϕ_{cr} is the concrete creep coefficient; $i_{corr(ref)}$ is the corrosion current density at reference state ($\mu A / cm^2$); and

$$\psi = \frac{\left(D_{bar} + 2\delta_o\right)^2}{2C\left(C + D_{bar} + 2\delta_o\right)}$$
3-10

where the thickness of the 'porous zone' (δ_0) is typically in the range of 10-20 µm (Thoft-Christensen, 2000) and can be described by random variables of a normal distribution with mean of 15 µm and COV of 0.1 (Stewart et al., 2011), which means there is a 99.9% probability that δ_0 is between 10 - 20 µm.

However, the time to crack initiation is very short compared to the time to corrosion initiation and the time of crack propagation (Stewart & Mullard, 2007). Therefore, the corrosion initiation (T_i) and crack propagation (t_{sev}) models are more critical than the model for crack initiation (t_{1st}) in the accuracy of the prediction of the corrosion damage.

When the concrete crack reaches a limit crack width, the concrete structure is severely damaged. The standard for how 'severe' the crack width is may vary and rely on individual conditions and asset owner policies. A limit crack width between 0.15 and 0.4 mm is considered necessary for durability or aesthetic limit states (Andrade et al., 1993; ACI Committee, 2001). Sakai et al. (1999) claim that a limit crack width of 0.8 mm is appropriate for aesthetic requirements. However, concrete structures in China with a corrosion induced crack width larger than 1.0 mm are regarded as not suitable for any further usage, and maintenance and repair measures have to be taken (GB50292, 1999). In this study, concrete structures will be regarded as 'severely cracked' when crack width reaches a crack width of 1.0 mm. Mullard and Stewart (2011) modelled the rate of crack propagation which can predict the time for a crack to develop from crack initiation of 0.05 mm to a limit crack width (t_{sev}). The time for cracking to reach a crack width of w mm is:

$$t_{sev} = k_R \frac{w - 0.05}{k_c ME(r_{crack}) r_{crack}} \left(\frac{0.0114}{i_{corr(ref)}}\right) \quad 0.25 \le k_R \le 1, k_c \ge 1.0, w \le 1.0mm$$
3-11

where r_{crack} is the rate of crack propagation (in mm/hr) and is equal to:

$$r_{crack} = 0.0008 e^{-1.7\psi_{cp}}$$
 $0.1 \le \psi_{cp} \le 1.0$ 3-12

 ψ_{cp} is the concrete cover cracking parameter and can be expressed as:

$$\psi_{cp} = \frac{C}{D_{bar} f_t}$$
3-13

 k_R is a rate of loading correction factor where $i_{corr(exp)}=100 \ \mu A/cm^2$ and is the accelerated corrosion rate used to derive r_{crack} :

$$k_R \approx 0.95 \left[\exp\left(-\frac{0.3i_{corr(exp)}}{i_{corr(ref)}}\right) - \frac{i_{corr(exp)}}{2500i_{corr(ref)}} + 0.3 \right]$$
3-14

where $i_{corr(ref)}$ is the corrosion current density (μ A/cm²) at the reference state and is constant with time; w is the crack width (mm); ME(r_{crack}) is the model error of the crack propagation model; C is the concrete cover in mm; D_{bar} is the reinforcing bar diameter in mm; f_t is the concrete tensile strength in MPa; and k_c is the confinement factor which takes into account an increase in crack propagation due to the lack of concrete confinement around external (edge) reinforcing bars. The occurrence of a confinement effect on crack propagation was observed when cracks over bars located at the edges of a RC surface crack at a higher rate to those in interior locations (Mullard & Stewart, 2011). If the reinforcing bar is located in the interior then $k_c=1$, but for rebars located at edges and corners of RC structures then k_c varies from 1.20 to 1.45. Although the data is limited, there appears to be a trend where k_c increases as ψ_{cp} increases. In this study, k_c is assumed to be 1.0 for all kinds of structural members.

A statistical study of model accuracy to account for variability between experimental data and model prediction is essential for stochastic or reliability studies in terms of model error. Hence, the statistics for model error for r_{crack} (ME_{rcrack}) are: mean(ME_{rcrack}) = 1.04 and COV(ME_{rcrack}) = 0.09 (Mullar & Stewart, 2010).

3.6 Time to corrosion damage (T_{sev})

After corrosion initiation T_i , the steel bar begins to generate corrosion rust. Since the corrosion rate is a time-dependent function of temperature and RH, then the time to

that is needed to cause severe cracking (m_{corr}) for a constant corrosion rate is $m_{corr}=0.0116 \times i_{corr(ref)} \times (T_{sev}-T_i) = 0.0116 \times i_{corr(ref)} \times (t_{1st}+t_{sev})$ where t_{sev} is the time to severe cracking calculated for the constant corrosion rate $i_{corr(ref)}$. The time to corrosion damage for a time-dependent corrosion rate (T_{sev}) is when corrosion products amounts (M_{corr}) for time-dependent corrosion rates equal to the amounts (m_{corr}) for constant corrosion rates. It follows that T_{sev} is obtained from solving the unknown T_{sev} from the following equation:

$$M_{corr} = m_{corr} = \int_{T_i}^{T_{sev}} i_{corr}(t)dt$$
3-15

where $i_{corr}(t)$ is the time-dependent corrosion rate given as Eq. 3-8.

3.7 Corrosion loss

Reduction of the steel section with time (reduction in diameter in mm) can be used to represent the time-dependent corrosion loss which is calculated as

$$\Delta d = 2 \times 0.0116 \int_{T_i}^{t} i_{corr}(t) dt$$
 3-16

where $i_{corr}(t)$ is the time-dependent corrosion rate given as Eq. 3-8.

3.8 Summary

The time of corrosion induced cracking for RC structures in carbonated environments is dominated by corrosion initiation, the corrosion rate, crack initiation and crack propagation. Models which predict the carbonation and corrosion processes are improved by considering time-dependent temperature, RH and CO₂ concentration to predict the time of corrosion initiation, crack initiation and crack propagation. The model is able to account for climate change effects on the diffusion coefficient and corrosion rate, as well as structural characters such as cover, reinforcing bar diameter and concrete strength, etc. Due to the inherent uncertainty of concrete structures, a reliability analysis is critical to predicting future performance of RC structures under a changing climate. Detailed description of the spatial time-dependent reliability analysis is presented in Chapter 4.

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Chapter 4 : Spatial time-dependent reliability modelling

4.1 Introduction

As discussed in the literature review, the inherent variability of RC structures due to material properties and quality control in construction can result in corrosion damage being spatially variable. Moreover, modelling material and dimensional parameters as homogeneous can lead to non-conservative predictions of failure for RC structures in corrosive environments (Mullard, 2010). Also, a spatial analysis can provide more detailed information on the extent of damage at a given point in time to be estimated allowing more detailed analysis of maintenance strategies and damage cost estimation. As such, incorporating spatial variability into predictive models is important, particularly for deterioration processes.

This chapter will describe the spatial time-dependent reliability analysis conducted in this study and will include descriptive information on the following components of the analysis:

- Random field modelling
- Deterioration modelling

The following sections will describe the key components of using random fields to represent spatial variability in RC structures, as well as a full description of all stochastic and deterministic variables for each model and finally, a description of the Monte Carlo Simulation technique used for the reliability analysis is given.

4.2 Spatial modelling using random field

Random fields can be used to represent spatial variability of continuous media (Stewart & Mullard, 2007) and their application to deteriorating RC structures is now well established (e.g., Englund & Sørensen, 1998; Li, 2004; Mullard, 2010; Mullard & Stewart, 2009; Stewart, 2004, 2006; Vu, 2003; Darmawan & Stewart, 2007; Na et al., 2012; Papakonstantinou & Shinozuka, 2013; Stewart & Mullard, 2007; Sudret, 2008; Vu & Stewart, 2005). Methods for defining and discretising random fields are described in detail elsewhere (Haldar & Mahadevan, 2000; Matthies et al., 1997;

Sudret & Der Kiureghian, 2000; Vanmarcke, 1983) and the aspects relevant to this study will be discussed herein.

The random field method can be used to discretise a surface or volume into a number of elements, and each element is a spatially correlated random variable to represent the properties of the random field. Each of the elements is statistically correlated based on the correlation function, and thus the random field can be completely defined by its mean (μ), standard deviation (σ) and correlation function ($\rho(\tau)$).

4.2.1 Stationary and non-stationary random fields

A random field with a constant mean and variance over the entire random field (i.e., the correlation function is only related to the distance between elements) is considered as a 'stationary' or homogeneous random field (Haldar & Mahadevan, 2000). On the other hand, a non-stationary random field has a mean that varies in space, i.e. a big RC bridge may have a varying mean value for chloride surface concentration depending on the location and orientation of the surface element (Li et al., 2004). Similarly, the mean concrete strength in a RC column may vary with height due to the influence of self-weight compaction (Zhu et al., 2001).

Typically, RC surfaces have been modelled as stationary random fields, and this assumption can be justified if the mean would not reasonably be expected to vary within the chosen random field. For example, Malioka and Faber (2004) defined separate random field 'zones' over which the statistical characteristics of the spatial variables could be considered homogenous. Stationary random fields will be used only to model spatial variability in this study.

4.2.2 Discretisation methods

4.2.2.1 General

A random field is discretised into a number of elements. The value of the random field over the element is represented by a correlated random variable (i.e., assume there is no spatial variability within the element). The discretisation methods of a random field can be categorized as three main kinds as following (Sudret & Der Kiureghian, 2000):

• Point discretisation –the random variable of the element is represented by the value of the random field at one or a few selected locations within the element. The midpoint method (Shinozuka & Dasgupta, 1986; Der Kiureghian & Ke, 1988), the shape function method (Vanmarcke, 1983), the integration point method (Brenner & Bucher, 1995; Matthies et al., 1997), and the optimal linear estimation method (Li & Der Kiureghian, 1993) can be categorised into this kind.

• Average discretisation – the spatial average of the random field over an element is used to define a random variable for the element, including the spatial average method (Vanmarcke, 1983), and the weighted integral method (Deodatis, 1991).

• Series expansion methods –the random field is divided into a set of random variables, including the orthogonal series expansion method (Zhang & Ellingwood, 1994), and the expansion optimal linear estimation (Li & Der Kiureghian, 1993).

4.2.2.2 Midpoint method

The midpoint method is using the value at the centroid of the element to represent the value of the random field over the entire element. The random field W(X) is represented over an element i as:

$W_i = W(X_i)$

where X_i is the location of the centroid of the element i. The random field W(X) can then be entirely defined by a random vector who's mean, μ , and covariance matrix, C_{aa} , are calculated from the mean, standard deviation and correlation function of W(X).

4.2.2.3 Spatial averaging method

The value of the random field is defined by the spatial average over the element in the spatial averaging method. The random field W(X) over an element i can be expressed as:

$$W_i = \frac{1}{\varphi_i} \int_{\varphi_i} W(X) d\varphi_i$$
 4-2

where ϕ_i is the length of the element in a one-dimensional random field or the area of the element in a two dimension random field. For a one-dimensional random field, Eq. 4-2 can be written as:

$$W_{i} = \frac{1}{X} \int_{-X/2}^{X/2} W(X) dX$$
 4-3

Lizhengli Peng

4-1

where X is the averaging interval. The vector representing the spatial random variables over the random field is then defined by the value of the random field at each element.

The covariance between two spatial averages in the random field W(X) is expressed as:

$$COV(W_{X}, W_{X}) = \frac{\sigma^{2}}{2} \Big[X_{0}^{2} \gamma(X_{0}) - X_{1}^{2} \gamma(X_{1}) + X_{2}^{2} \gamma(X_{2}) - X_{3}^{2} \gamma(X_{3}) \Big]$$
4-4

where $\gamma(X)$ is the variance function. X_0 is the distance from the end of the first element to the start of the second element; X_1 is the distance from the start of the first element to the start of the second element; X_2 is the distance from the start of the first element to the end of the second element; and X_3 is the distance from the end of the first element to the end of the second element (Vanmarcke, 1983).

4.2.2.4 Expansion optimal linear estimation method

The expansion optimal linear estimation method was developed by Li and Der Kiureghian (1993) as an extension of the optimal linear estimation method (which was previously proposed by the same authors). Assuming that the random field W(X) is Gaussian, the spectral decomposition of a random vector **w** can be expressed as:

$$w = \mu + \sum_{i=1}^{N} \zeta_i \sqrt{\theta_i} \varphi_i$$
4-5

where **w** is a vector representing the value of the random field for each of N elements, ζ_i are the independent standard normal varieties (zero mean, unit variance and zero correlations), θ_i are the eigenvalues of the covariance matrix and φ_i are the eigenvectors of the covariance matrix. The random field can then be represented by:

$$W(X) = \mu(X) + \sum_{i=1}^{r} \frac{\zeta_i}{\sqrt{\theta_i}} \varphi_i^T E_{W(X)_v}$$
4-6

where $\mathbf{E}_{W(X)\mathbf{v}}$ is the N × 1 vector containing the covariance of W(X) with the elements of w.

4.2.2.5 Summary

There are various methods for the discretisation of a random field, and some have been introduced above (Vanmarcke, 1983). The midpoint method is a simple, robust and the most widely used method of discretisation. The covariance matrix is positive definite and can be easily computed, moreover, the distribution function in the discretised and continuous case is same, thus no restriction to Gaussian random fields (Matthies et al., 1997). However, it has been noted that the midpoint method tends to over-represent the variability of the random field (Sudret & Der Kiureghian, 2000). The spatial averaging method is another convenient method of random field discretisation although it has been suggested to under-represent the local variance (Der Kiureghian & Ke, 1988). The expansion optimal linear estimation method has been found to be an efficient method of discretisation (Li & Der Kiureghian, 1993) if the exact eigenvalues and eigenfunctions are known, but this is not possible for many correlation models.

The midpoint method is numerically robust and simple to incorporate into a stochastic reliability analysis; therefore, it will be used for the discretisation of random fields in this study. The following sections will describe the midpoint method and the corresponding correlation function and spatial parameters in detail.

4.2.3 Using the midpoint method to discretise a RC surface

For midpoint method, the covariance matrix and a vector of independent standardised normal random variables can be used to calculate a vector (**X**) of a normal random field. The RC surface which is modelled as a continuous media w(x) is then discretised into m values $w(x_i)$, i = 1, 2, ..., m where x_i is the location of the centroid of element i, and m is the total number of elements. The vector then has m correlated random variables with zero mean and unit standard deviation. The covariance between two midpoint values $w(x_i)$ and $w(x_j)$ for the random field is defined in terms of the covariance matrix as follow:

$$C_{i,j} = \rho_{i,j}\sigma^2 \tag{4-7}$$

where $C_{i, j}$ is the value of the covariance matrix, σ is the standard deviation of the random field and $\rho_{i, j}$ is the value of the correlation function between points i and j. The vector $w = [w_1, w_2, ..., w_m]$ can then be calculated as:

where L represents the lower triangular matrix of the covariance matrix calculated by a Cholesky decomposition (or similar), and Z is a vector of uncorrelated standard random variables. Thus, once the Cholesky decomposition is conducted for a given random field, the spatially correlated random variables $w = [w_1, w_2, \dots, w_m]$ can be generated from the input of standard random variables.

4.2.4 Correlation function and scale of fluctuation

The spatial correlation between elements in a random field can be defined by the correlation function (also known as the autocorrelation function) which describes the exact pattern of decay of the variance (Haldar & Mahadevan, 2000). The correlation function $\rho(\tau)$ defines the correlation coefficient between two elements separated by distance τ in a random field. The value of the correlation coefficient approaches unity when distance τ between two elements is small enough and approaches zero or close to zero as the distance τ increases. Various types of correlation functions have been proposed (Haldar & Mahadevan, 2000; Li et al., 2004; Vanmarcke, 1983) such as triangular, exponential and Gaussian functions.

The triangular correlation function (Vanmarcke, 1983) decreases linearly from unity to zero as τ goes from 0 to a where a is equal to the scale of fluctuation θ . As such the triangular correlation function can be defined as:

$$\rho(\tau) = \begin{cases} 1 - \frac{|\tau|}{a}, |\tau| \le a \\ 0, |\tau| \ge a \end{cases}$$
4-9

The exponential correlation function (Vanmarcke, 1983) can be described as:

$$\rho(\tau) = \exp^{-|\tau|/b}$$
 4-10

where b is a model parameter defined as $b = \theta/2$ where θ is the scale of fluctuation. Vanmarcke (1983) described the correlation function associated with a second-order regressive process as:

$$\rho(\tau) = \left[1 + \frac{|\tau|}{c}\right] \exp^{-|\tau|/c}$$
4-11

where c is a model parameter defined as $c = \theta/4$. The Gaussian (or squared exponential) correlation function (Vanmarcke, 1983) can be defined as:

$$\rho(\tau) = \exp^{-\left(\frac{|\tau|}{d}\right)^2}$$
 4-12

where d is a model parameter defined as $d = \theta / \sqrt{\pi}$.

Li et al. (2004) use an empirically determined correlation function based on the Gaussian function in Eq. 4-12. A factor ρ_0 is introduced to account for a common source of correlation amongst the elements of a random field and thus the correlation between elements can be described by:

$$\rho(\tau) = \rho_0 + (1 - \rho_0) \exp^{-\left(\frac{|\tau|}{f}\right)^2}$$
4-13

where f is defined by Li et al. (2004) as the fluctuation scale. Note that the definition of the fluctuation scale (f) is not thoroughly examined by the authors and its relation to the scale of fluctuation, θ , as defined by Vanmarcke (1983), is unclear. The definition of the scale of fluctuation is described is the following.

The scale of fluctuation (θ) represents the distance within which correlation persists in a random field (Vanmarcke, 1983). Physically, for two points located within the distance θ the corresponding values of the material property is likely to be either both higher or both lower than the mean value μ . Random fields with a large value of θ have very uniform material properties across space, while a small value of θ indicates more random material properties across space. Vanmarcke (1983) defines the scale of fluctuation θ as:

$$\theta = \lim_{S \to \infty} S\gamma(S)$$
 4-14

where $\gamma(S)$ is the variance function for the vector of spatial random variables and S is the space interval.

Vanmarcke (1983) defines the scale of fluctuation as $\theta = d/\sqrt{\pi}$, where d is the spatial correlation parameter in Eq.4-12 for the Gaussian correlation function. The term d has been described elsewhere as the correlation length (Vu, 2003) or the correlation radius and even referred to (erroneously) as the scale of fluctuation (Li et al., 2004). Haldar and Mahadevan (2000) replace the term d in Eq.4-12 with a term which they refer to as the correlation length. Note that the term 'correlation length' is not interchangeable with the term 'scale of fluctuation'.

A comparison of correlation functions is shown in Figure 4-1 using the model parameter data from Table 4-1, which assumes a scale of fluctuation of $\theta = 2.0$. The

triangular correlation function (as expected from Eq.4-9) has zero correlations outside the limits of the scale of fluctuation ($\tau \ge 2$) whilst the other functions show a decaying correlation coefficient of varying magnitude for this value of τ . The correlation function defined by Eq. 4-13, using a value of $\rho_0 = 0.3$, has a much stronger correlation at $\tau = 2$, and this is the result of the common source of correlation that is represented by a factor ρ_0 . If no common source of correlation is modelled in Eq. 4-13 (i.e. $\rho_0 = 0.0$), then Eq. 4-13 still has a relatively high correlation coefficient at $\tau = 2$, and this is due to the imprecise definition of the model parameters.

 Table 4-1: Correlation function data used in Figure 4-1.

Eq. 4-9	Eq. 4-10	Eq. 4-11	Eq. 4-12	Eq. 4-13	Eq. 4-13*
θ=2=a	$\theta = 2 = 2b$	$\theta = 2 = 4c$	$\theta = 2 = d\sqrt{\pi}$	ρ ₀ =0.0 f=2	ρ ₀ =0.3 f=2



Figure 4-1: Comparison between correlation functions.

The validation and calibration of correlation functions demands large amounts of spatial data that is hard to achieve. Therefore, engineering judgement or expert experience is required to match a correlation function to a particular random field. The Gaussian (or squared exponential) correlation function has been widely used in the spatial modelling of corroding of RC structures (Haldar & Mahadevan, 2000; Li & Der Kiureghian, 1993; Mullard, 2010; Stewart, 2006; Sudret et al., 2007), and as such it shall be used in this study. Thus, for random fields with up to three dimensions, the Gaussian correlation function is described as:

$$\rho(\tau) = \exp\left(-\left(\frac{\tau_x}{d_x}\right)^2 - \left(\frac{\tau_y}{d_y}\right)^2 - \left(\frac{\tau_z}{d_z}\right)^2\right)$$
4-15

where $\tau_x = x_i - x_j$, $\tau_y = y_i - y_j$ and $\tau_z = z_i - z_j$ are the distances between the centroid of element i and j in the x, y and z axes respectively, $d_x = \theta_x / \sqrt{\pi}$, $d_y = \theta_y / \sqrt{\pi}$ and $d_z = \theta_z / \sqrt{\pi}$ where $\theta = 0.5$ is the scale of fluctuation.

Vanmarcke (1983) has proposed various methods to calculate the scale of fluctuation for stationary random fields. These methods, however, require large amounts of spatial data and although this has been collected in some cases (O'Connor and Kenshel, 2013; Kenshel, 2009). The spatial parameters defining the correlation function are estimated based on engineering judgement, and the terminology is not overly important since the mathematical and structural characteristics are consistently represented. As such the definitions and terminologies described in Eq. 4-12 and Eq. 4-15 will be used herein.

The effect of the scale of fluctuation on the Gaussian correlation function for a onedimensional random field is shown in Figure 4-2. As defined by the correlation function, the value of the correlation coefficient is unity when $\tau = 0$ and reduces to approximately 0.05 when elements are separated by a distance of the scale of fluctuation.



Figure 4-2: Effect of scale of fluctuation on the correlation coefficient.

A summary of the values for the scale of fluctuation used by others to describe the spatial variability of concrete dimensions and properties is shown in Table 4-2.

Values for the scale of fluctuation are given for the CO₂ diffusion coefficient (D_{CO2}), CO₂ binding capacity, concrete strength (f_c), corrosion rate (i_{corr}), concrete cover, water/cement ratio (w/c) and modulus of elasticity (E). From the data presented in Table 4-2, it would seem that the scale of fluctuation of approximately 2.0 m and 1.0 m is likely to represent the spatial properties of a RC surface. The value of the scale of fluctuation, however, should be chosen in conjunction with an appropriate random field element size, so that mathematical and computational difficulties are avoided. The size of the random field elements is discussed in Section 4.2.5 and the influence on the reliability analysis of both the element size and the scale of fluctuation are investigated in Chapter 5.

Defense	Spatial variable									
References	D _{CO2}	binding capacity	f_c	i _{corr}	cover	w/c	Е			
Sudret (2008)	2.0 m				2.0 m					
Strauss and Bergmeister (2004)				4.0 m						
Li et al. (2004)				2.0 m						
Sudret et al. (2007)				1.0 m	1.0 m					
Kersner et al. (1998)						0.5 m	2.0 m			
Na et al. (2011)	3.0 m	3.0 m			3.0 m					
Mullard (2010)			2.0 m		2.0 m					

Table 4-2: Existing estimates of scale of fluctuation

4.2.5 Element size

The definition of element size is usually based on practical and analytical considerations (Sterritt et al., 2001). Using a fine mesh can increase the computational effort significantly but for little improvement in accuracy. Moreover, the smaller element size indicates that elements close to each other are highly correlated which may cause numerical and computational difficulties in the orthogonalisation of the covariance matrix. On the other hand, too large an element size can lead to an underestimation of the influence of random variability, and the correlation between elements becomes negligible. The element size relative to the scale of fluctuation is, therefore, an important consideration when defining a random field, and this is discussed further in Chapter 5.

Der Kiureghian and Ke (1988) suggested that the element size should be one-quarter to one-half of correlation length and Sterritt et al. (2001) suggested that the element size should be 0.1 to 0.75 m. In terms of numerical precision, Sudret and Der Kiureghian (2000) observed that the error for a Gaussian correlation function is small as long as the ratio of length of the element to scale of fluctuation is less than about 0.28. Therefore, for the scale of fluctuation of 2.0 m, an element size of 0.5 m or less would be sufficient to negate error. Sensitivity analysis of element size and scale of fluctuation are discussed in Chapter 5.

4.3 Stochastic deterioration modelling

4.3.1 Model error

Predictive models often have an uncertainty associated with them based on many factors including insufficient experimental data and an imperfect model fit of the predicted data to the actual physical behaviour. This modelling uncertainty can be incorporated into a reliability analysis by introducing a model error to represent the ratio between the actual value and the predicted model response (Melchers 1999). A probabilistic variable, Model Error (ME), will, therefore, be included in the analysis, and it is defined by its mean and standard deviation calculated from the following relationship:

$$ME = \frac{\text{actual value}}{\text{predicted value}}$$
4-16

where the actual value represents the value obtained from physical testing or measurement, and the predicted value represents the value obtained from the predictive model.

As well as the uncertainty calculated based on the difference between the model prediction values and the actual values (i.e. uncertainty based on the 'fit' of the model), there is also uncertainty based on the test data used in the formulation of the model. This uncertainty is expressed by Ellingwood et al. (1980) and the COV of the model which takes into account testing and specimen uncertainty, and is described by the following equation:

$$COV_{ME} = \sqrt{COV_{T/C}^2 - COV_{test}^2 - COV_{spec}^2}$$
4-17

where COV_{ME} is the COV of model error, $COV_{T/C}$ is the COV obtained directly from the comparison of the actual and predicted values, COV_{test} represents the uncertainties in the test set-up measurements and COV_{spec} represents uncertainties in the specimen properties.

Eq. 4-16 was used by Mullard (2010) to estimate the model error for the crack propagation model. For other models described and used in this study, where estimations of ME have been made in the literature, the statistical parameters for ME are introduced and subsequently incorporated into the spatial time-dependent reliability analysis.

4.3.2 Concrete properties

4.3.2.1 Concrete strength and elastic modulus

Bartlett and MacGregor (1996) proposed a model for 28-day in-situ compressive strength $f_c(28)$ as:

$$f_c(28) = F_1 F_2 F_c$$
 4-18

$$V_{f_c(28)}^2 = V_{F_1}^2 + V_{F_2}^2$$
4-19

where F'_c is the nominal design concrete compressive strength; F_1 is a random variable linking cube/cylinder strength to nominal compressive strength, F_2 is a random variable relating 28-day in-situ strength to cube/cylinder strength, and $V_{f_c(28)}$, V_{F_1} and V_{F_2} are the COV of the 28-day in-situ concrete compressive strength, F_1 and F_2 respectively. The statistics are mean(F_2) =0.85 and V_{F_2} =0.10 for site-cast concrete (Wiśniewski et al., 2012).

Pham (1985) compiled the Australian statistics for concrete compressive strength based on collected data from 231 jobs for a period of 19 years (1962-1981), and reported the statistics for mean($f_c(28)/F'_c$) and COV($f_c(28)/F'_c$) as 1.03 and 0.18, respectively. Foster et al. (2013) obtained the statistics for mean($f_c(28)/F'_c$) and COV($f_c(28)/F'_c$) from new concrete data since 2001 for all cities and regional areas in Australia. The normally distributed statistical parameters for Australian concrete compressive strength $f_c(28)$ are presented as Table 4-3. The mean compressive strength is 3% higher and the COV is 16% lower than the old material data developed by Pham (1985) when $F'_c <50$ MPa. This suggests that new material data reflects enhanced concrete quality. Wiśniewski et al. (2012) reviewed concrete compressive strength data in Australia, Europe, Canada and USA and recommended that mean(f'_c/F'_c)=1.02 and COV(f'_c/F'_c)=0.12 for site-cast concrete.

Table 4-3. Statistical parameters for concrete compressive strength (fc(28)) in Australia (Forster
et al., 2013)

Grade ^a	F'c	Bias Factor	COV	Grade	F'c	Bias Factor	COV
N20	20 MPa	1.069	0.173	N50	50 MPa	1.063	0.154
N25	25 MPa	1.052	0.156	S65	65 MPa	1.084	0.154
N32	32 MPa	1.061	0.152	S80	80 MPa	0.965	0.131
N40	40 MPa	1.069	0.151				

Note - ^a: 'N' represents normal class concrete and 'S' means standard strength grades, sample size could be either 100mm or 150 mm diameter cylinder.

A statistical analysis of 28-day cube strengths taken from Chinese structures is used to infer mean(F_1) and V_{F_1} . More than thirty thousand test data were obtained. Hu et al. (2008) took cube strengths from 373 typical multi-storey and high-rise buildings built in 2003 in Xi'an. Li (2008) investigated more than 1,000 residences built after 2003 in the Xi'an and Xianyang areas. Parameter estimation and hypothesis testing of the probability distribution indicate that the strength of concrete specimens do not reject the normal distribution. The statistical parameters for F_1 and V_{F_1} are calculated and shown in Table 4-4 and Table 4-5.

 Table 4-4: Statistical parameters for concrete compressive strength for highway bridges in China.

	C15	C20	C25	C30	C40	C50
Sample size	1560	9087	6432	5780	5734	2009
Mean cube strength (MPa) ^a	17.42	22.95	27.94	31.90	41.45	48.03
Mean (F ₁₎	1.34	1.28	1.21	1.14	1.09	1.00
Mean $\frac{f_c(28)}{F_c} = F_1 \times F_2$	1.14	1.08	1.03	0.97	0.93	0.85
V _{F1}	0.283	0.236	0.193	0.178	0.158	0.137
$V_{f_c(28)} = \sqrt{V_{F_1}^2 + V_{F_2}^2}$	0.300	0.256	0.217	0.204	0.187	0.170

Note - a: concrete strengths for cube specimen size of $200 \times 200 \times 200$ mm are corrected to $150 \times 150 \times 150$ mm specimen according to JTG D62-2004 (2004).

	C15	C20	C25	C30	C35	C40	C45	C50
Sample size	952	8222	5736	4536	2030	912	160	94
Mean cube strength (MPa)	18.6	23.0	26.5	31.3	35.6	39.8	45.9	55.1
Mean(F ₁)	1.24	1.15	1.06	1.04	1.02	1.00	1.02	1.10
Mean $\frac{f_c(28)}{F_c} = F_1 \times F_2$	1.05	0.98	0.90	0.89	0.86	0.85	0.87	0.94
V_{F1}	0.230	0.197	0.168	0.165	0.227	0.179	0.198	0.242
$V_{f_c(28)} = \sqrt{V_{F_1}^2 + V_{F_2}^2}$	0.251	0.221	0.196	0.193	0.248	0.205	0.222	0.262

Table 4-5: Statistical parameters for concrete compressive strength for buildings in China.

The 28-day in-situ concrete strengths of RC structures in China are lower than those in Australia (Stewart et al., 2011; Foster et al., 2013) and other western countries (Wiśniewski et al., 2012). For example, for design compressive strength of 25 MPa, the mean and the COV for 28-day in-situ concrete strength in Chinese buildings are 2% lower and 39% higher than Australian concrete, respectively.

It is well established that the concrete compressive strength will continue to gain strength with time when compared to the values obtained from cylinder testing at 28 days (Warner et al., 1998). As corrosion damage typically occurs many years into the service life of a RC structure, it is important to estimate the compressive strength of the concrete at this time. Accordingly, the effect of time on compressive strength is estimated herein and the concrete compressive strength was calculated at one year using the American Concrete Institute (ACI) method which was described by Stewart (1996) as:

$$f_c = 1.162 f_c(28)$$
 4-20

Time-dependent increases in strength beyond one year are not considered in the present analysis.

The concrete elastic modulus and the concrete splitting tensile strength can be estimated based on the value of the concrete compressive strength. Mirza et al. (1979) proposed the following models:

$$E_c(t) = 4600\sqrt{f_c(t)}$$
 4-21

Lizhengli Peng

$$f_t(t) = 0.53\sqrt{f_c(t)}$$
 4-22

where $E_c(t)$, $f_t(t)$ and $f_c(t)$ are the time-dependent in-situ elastic modulus, timedependent in-situ concrete splitting tensile strength and time-dependent in-situ concrete compressive strength respectively. The model error estimated for the predictive models for concrete elastic modulus and concrete splitting tensile strength (Eq. 4-21 and 4-22), according to Mirza et al. (1979), has the following statistical parameters: mean = 1.0; and COV = 0.12 and 0.13, respectively. The value of the splitting tensile strength and the elastic modulus are dependent on the concrete compressive strength and are thus also stochastic variables.

4.3.2.2 Concrete cover

It is well established that the actual concrete cover for a RC structure will vary from the nominated design value, and this uncertainty has been studied in the literature (Fazio et al., 1999; McGee, 1999; Mirza & MacGregor, 1979; Stewart & Attard, 1999; Van Daveer, 1975). Concrete cover is an influential parameter in terms of corrosion initiation and propagation (see Chapter 3) and as such the statistical uncertainty of the concrete cover should be included in the modelling of corrosion damage for RC structures.

Van Daveer (1975) analysed results from a study of 42 typical bridge decks in the United States and found that the variability of the top cover follows a normal distribution where the mean deviates +3.2 mm from the design value with a standard deviation of 9.5 mm. It is noted that these statistical parameters indicate a slightly higher degree of certainty than others have suggested based on cover measurements from building slabs (Mirza & MacGregor, 1979) but this is most likely because of the higher level of quality control present in the construction of bridge decks.

McGee (1999) surveyed 143 RC bridges built between 1931 and 1997 in Tasmania, Australia. More than 24,000 measurements were made on cast in-situ, precast bridges and precast culvert. The variables of concrete cover can be described by a normal distribution, and the statistics are presented as Table 4-6. These values can be regarded as representatives of RC structures in Australia.

Table 4-6: statistics of concrete cover of RC structures in Australia

Element type	Mean cover	Std deviation
in situ	Specified + 6mm	11.5mm
precast	Specified + 3mm	9.7mm

More than 54,000 test data for concrete cover were collected for RC structures built in the 1980s in 13 large Chinese cities including Beijing, Jinan, Shanghai, Guangzhou, etc. (RGRRCSM, 1985). According to the construction schedule and site conditions, test components were randomly selected. The statistical results of the data do not reject the normal distribution. The suggested statistical parameters for concrete cover of RC beams and site-cast RC slabs have low mean value and high variability as:

$$C_{mean} = 0.90C_{com}$$

where C_{mean} is the mean concrete cover (mm), C_{nom} is the nominal or design cover (mm) and COV=0.30. However, suggested statistics parameters for concrete cover of precast RC slabs in China are: $C_{mean} = 1.30C_{com}$ with COV=0.25.

4.3.3 Corrosion initiation model

When carbonation depth reaches the reinforcing bar, corrosion initiates (T_i), where T_i is the year when the carbonation depth exceeds the concrete cover, as discussed in Chapter 2 and 3. The carbonation process is considered as a steady state process modelled by Fick's First Law and included time-dependent CO₂ concentration, temperature and RH effects. The carbonation depth model is modified as

$$x_{c}(t) = \sqrt{\frac{2D_{CO_{2}}(t)}{a}} \int_{2010}^{t} f_{T}(t) f_{RH}(t) k_{site} C_{CO_{2}}(t) dt} \left(\frac{t_{0}}{t - 2009}\right)^{n_{m}} \quad t \ge 2010$$
4-24

$$D(t) = D_1 (t - 2009)^{-n_d}$$
 4-25

$$a = 0.75C_e C_a O \alpha_H \frac{M_{CO_2}}{M_{C_a O}}$$

$$4-26$$

where t is defined in calendar years starting from 2010 (assuming the building went into service in the year 2010); $x_c(t)$ is the carbonation depth at time t; $C_{CO_2}(t)$ is the time-dependent increase in atmospheric CO₂ concentration (10⁻³kg/m³) as shown in Figure 5-1 (using the conversion factor 1 ppm = 0.0019×10⁻³ kg/m³); k_{site} is the factor to account for increased CO₂ levels in non-remote environments; $f_T(t)$ is the timedependent change in the diffusion coefficient due to changes in temperature; $f_{RH}(t)$ is the time-dependent change in the diffusion coefficient due to changes in RH; $D_{CO2}(t)$ is the CO₂ diffusion coefficient in concrete calibrated for the time period 2010 to t; D_1 is the CO₂ diffusion coefficient at t=2010; n_d is the age factor for the CO₂ diffusion coefficient; t_0 is one year; C_aO is C_aO content in cement (0.65); C_e is cement content (kg/m³); α_H is a degree of hydration, the degree of hydration for OPC after more than 400 days is estimated as Eq. 3-4 (de Larrard, 1999); M_{CaO} is the molar mass of C_aO and equal to 56 g/mol; and M_{CO_2} is the molar mass of CO₂ equal to 44 g/mol. The age factor for microclimatic conditions n_m is related to the frequency of wet and dry cycles, and is equal to 0 for sheltered outdoor exposures, and is equal to 0.12 for unsheltered outdoor exposures.

The mean values for D_1 and n_d are given in Table 4-7 as a function of water/cement ratio (w/c), and Yoon et al. (2007) provided estimates of maximum (95th percentile) values for D_1 and n_d . The standard deviation for D_1 and COV for n_d are assumed to be about 0.15 and 0.12, respectively (Stewart et al., 2011). These statistics represent model error (or accuracy). These parameters are based on T=20 °C and RH=65%.

Table 4-7: Mean parameter values (Yoon et al., 2007).

w/c	$D_1 \times 10^{-4} \text{ cm}^2 \text{s}^{-1}$	n _d
0.45	0.65	0.218
0.50	1.24	0.235
0.55	2.22	0.240

Note - for intermediate values use linear interpolation.

4.3.4 Corrosion propagation model

A model that includes RH and temperature effects on the corrosion rate proposed by Breysse et al. (2014) shows that a lower RH will decrease the corrosion rate when RH is less than the reference RH:

$$i_{corr}(RH,T) = i_{corr(ref)}(RH_{ref},T_{ref})e^{0.0312(RH-RH_{ref})}e^{-4736\left(\frac{1}{273.15+T}-\frac{1}{273.15+T_{ref}}\right)}$$
4-27

where a reference state is RH_{ref} =80% and T_{ref} =20 °C.

The reference corrosion rate is denoted as $i_{corr(ref)}$ and is assumed to be log normally distributed with statistical parameters given by DuraCrete (1998), see Table 3-5, where the corrosion rate of 1 μ A/cm² = 0.0116 mm/year. These values take account

the concrete grades for the corresponding exposure classes. DuraCrete (2000) indicates that the standard deviation in Table 3-5 does not include the variation caused by changes of RH and temperature.

4.3.5 Crack initiation model

The crack initiation (t_{1st}) model proposed by El Maaddawy and Souki (2007) is used herein.

$$t_{1st} = \left[\frac{7117.5(D_{bar} + 2\delta_0)(1 + \nu + \psi)}{365i_{corr(ref)}E_{ef}}\right] \cdot \left[\frac{2Cf_t}{D_{bar}} + \frac{2\delta_0 E_{ef}}{(1 + \nu + \psi)(D_{bar} + 2\delta_0)}\right]$$
4-28

where t_{1st} is the time from corrosion initiation to first cracking of 0.05 mm width, D_{bar} is the reinforcing bar diameter (mm), δ_0 is the thickness of the porous zone around the reinforcing bar (µm), v is Poisson's ratio of concrete, C is the concrete cover (mm), f'_t is the design concrete tensile strength (MPa), $i_{corr(ref)}$ corrosion current density at reference state (µA / cm²) and E_{ef} is the effective elastic modulus of concrete defined as:

$$E_{ef} = \frac{E_c}{\left(1 + \varphi_{cr}\right)}$$
4-29

where E_c is the concrete elastic modulus (MPa) and ϕ_{cr} is the creep coefficient. Ψ is defined as:

$$\psi = \frac{\left(D_{bar} + 2\delta_o\right)^2}{2C\left(C + D_{bar} + 2\delta_o\right)}$$
4-30

The thickness of the porous zone (δ_0) is usually in the range of 10-20 µm and is assumed to be variables of a normal distribution with mean of 15 µm and COV of 0.1.

4.3.6 Crack propagation model

Mullard and Stewart (2010) have modelled the rate of crack propagation that can predict the time to severe damage (t_{sev}), i.e. for a crack to develop from crack initiation of 0.05 mm to a limit crack width. The time for cracking to reach a crack width of w mm is:

$$t_{sev} = k_R \frac{w - 0.05}{k_c ME(r_{crack}) r_{crack}} \left(\frac{0.0114}{i_{corr(ref)}}\right) \quad 0.25 \le k_R \le 1, k_c \ge 1.0, w \le 1.0mm$$
4-31

where

Lizhengli Peng

$$\psi_{cp} = \frac{C}{D_{bar} f_t}$$
4-32

$$r_{crack} = 0.0008 e^{-1.7\psi_{cp}} \qquad 0.1 \le \psi_{cp} \le 1.0$$
4-33

$$k_R \approx 0.95 \left[\exp\left(-\frac{0.3i_{corr(exp)}}{i_{corr(ref)}}\right) - \frac{i_{corr(exp)}}{2500i_{corr(ref)}} + 0.3 \right]$$
4-34

where $i_{corr(ref)}$ is the corrosion current density at the reference state and time-invariant in $\mu A/cm^2$; ψ_{cp} is the concrete cover cracking parameter; r_{crack} is the rate of crack propagation in mm/hr; ME(r_{crack}) is crack propagation model error; w is the crack width (mm); C is concrete cover in mm. D_{bar} is the reinforcing bar diameter in mm; f_t is the concrete tensile strength in MPa; k_R is a correction factor for the loading rate where $i_{corr(exp)}=100 \ \mu A/cm^2$ is the accelerated corrosion rate used to derive r_{crack} ; and k_c is the confinement factor that considers an increase in crack propagation around external (edge) reinforcing bars due to the lack of concrete confinement. The model error for the time to severe cracking (t_{sev}) is estimated based on the accuracy of the predictive model for r_{crack} . The calculated statistical parameters for the model error of the crack propagation model (MEr_{crack}) are mean=1.04, COV=0.09.

4.4 Random field modelling

It has been discussed that concrete properties and dimensions will vary spatially over a RC structure due to a number of factors including workmanship, material variations and environment. As such, random fields are used herein to describe the spatially variable features of the RC structure in a corrosive environment. The structure (or structural component) is discretised into a number of elements, and a random variable is used to represent the random field over each element. Each of these random variables is statistically correlated based on the correlation function of the corresponding random field. The following parameters are considered as spatially variable:

- concrete cover;
- concrete strength;
- diffusion coefficient and;
- binding capacity.

These parameters were chosen as they are the key variables in the models presented for corrosion initiation, corrosion propagation, crack initiation and crack propagation and therefore adequately model the spatial variability of corrosion damage in RC structures.

4.4.1 Spatial analysis of RC building slab or bridge deck

Figure 4-3 shows a schematic of a two-dimensional RC building slab or bridge deck which has been discretised into a number (k) of spatially correlated elements of size Δ . This analysis considers the spatial variability of corrosion damage on one face of the slab or deck (in this case the top) but a similar study could be conducted to model corrosion damage that could occur on an exposed soffit. It is appropriate to use a twodimensional random field for a RC slab or deck (or a similar surface such as building slabs) as it is expected that the spatial properties will vary with both length and width of the structure.



Figure 4-3. A schematic of a two-dimensional RC bridge deck which has been discretised into a number (k) of spatially correlated elements of size Δ (Mullard, 2010).

The Gaussian correlation function has been widely used in engineering applications, including spatial modelling of RC structures, and as such it shall be used herein to model the spatial correlation of elements within a random field (Der Kiureghian & Ke, 1988; Sudret, 2008; Mullard & Stewart, 2009). For the Gaussian (or squared exponential) correlation function used herein see Eq. 4-12. Once the stochastic random field is defined, Monte Carlo simulation methods are used to generate random variables for each element, and the spatially variable parameters of each element are then determined using the correlation function. Monte Carlo simulation can also predict the deteriorated state of each element (T_i , T_{sev}) at every time interval.

Lizhengli Peng

The correlation characteristics of the random variables of each element can be defined in terms of the covariance matrix as follows:

$$C_{i,j} = \rho_{i,j} \times \sigma^2 \tag{4-35}$$

where $C_{i, j}$ is the value of the covariance matrix, σ is the standard deviation of the random field and $\rho_{i, j}$ is the value of the correlation function between points i and j. The value of the random field for every element can be represented by a vector $\mathbf{w} = [\mathbf{w}_1, \mathbf{w}_2, \dots \mathbf{w}_m]$ and it can be calculated by:

$$\mathbf{w} = \mathbf{L}\mathbf{Z}$$
 4-36

where \mathbf{L} is the lower triangular matrix of the covariance matrix formed using a Cholesky decomposition and \mathbf{Z} is a vector of uncorrelated standard random variables which can be generated using traditional Monte Carlo methods. In this case, the random field is assumed to be stationary and as such, the mean and standard deviation are constant for all elements within the random field. This assumption is justified provided the area of analysis is chosen to avoid obvious differences in the expected mean value of the parameters within the random field such as different contaminate exposure conditions and varying construction practices or materials.

4.4.2 Spatial analysis of RC beam

Due to the relatively small width dimension in a typical RC beam (approximately 0.12 – 0.3 m) it can be assumed that any spatial variability in this direction will be negligible or of such a micro-nature to prohibit meaningful analysis. As such, a series of statistically independent one-dimensional random fields will be used to model the spatially variable properties along the span of a RC beam only. Further, because only the bottom face of the beam may be exposed to the environment, the bottom face of the beam will be studied herein. Figure 4-4 shows a schematic of the bottom face of a RC beam which has been discretised into a number (k) of spatially correlated elements of size Δ .



Figure 4-4. Discretisation of one-dimensional RC component into k spatially correlated elements.

In this analysis, the bottom face of the RC beam is treated as a statistically independent random field for four random field parameters. The method of discretisation and analysis of the random fields for the RC column is the same as that described for the RC building slab and bridge deck (see Section 4.4.1), except the random fields are assumed to act in one-dimension only. For example, the correlation function in one-dimension becomes:

$$\rho(\tau) = \exp\left[-\left(\frac{|\tau_x|^2}{d_x^2}\right)\right]$$
4-37

where τ_x is the distance between the centroid of correlated elements in the x direction and $d_x = \theta_x / \sqrt{\pi}$ where θ is the scale of fluctuation.

4.5 Spatial time-dependent reliability analysis

4.5.1 Extent of corrosion initiation

Corrosion will take place when the carbonation depth reaches the reinforcing bar surface, and as such the extent of concrete surface elements that have been corrosion initiated at time t ($d_{ini}(t)$) can be described as:

$$d_{ini}(t) = \frac{n[x_c(t) \ge \text{cover}]}{k} \times 100\%$$
4-38

where $x_c(t)$ is the carbonation depth predicted from Eq. 4-24.

4.5.2 Probability and extent of corrosion damage

The time to corrosion damage is defined as the time when the concrete cover reaches a limit crack width of 1.0 mm. Therefore, the extent of the concrete surface severe corrosion damage at time t ($d_{crack}(t)$) for each Monte Carlo simulation realisation is

$$d_{\text{crack}}\left(t\right) = \frac{n\left[t > T_{sev(j)}\right]}{k} \times 100\%$$
4-39

where $T_{sev(j)}$ is the time to corrosion damage of element j, and n[] denotes the number of elements for which t> $T_{sev(j)}$.

The probability that at least x% of the concrete surface has been damaged at time t is

$$\Pr\left(d_{crack}\left(t\right) \ge x\%\right) = \int_{x\%}^{100\%} f_{d_{crack}}\left(d_{crack},t\right) \mathrm{d} d_{crack}$$
4-40

where $f_{d_{crack}}(d_{crack},t)$ is the multi-dimensional probability distribution of $d_{crack}(t)$ obtained from a Monte Carlo simulation analysis for each time step t, the value of $d_{crack}(t)$ is calculated and after many simulations the probability distribution $f_{d_{crack}}(d_{crack},t)$ for every combination of t and $d_{crack}(t)$ will be inferred. As will be shown later, $f_{d_{crack}}(d_{crack},t)$ is highly non-Gaussian and so percentile values of $f_{d_{crack}}(d_{crack},t)$ can only be predicted from simulation methods (Stewart & Mullard, 2007).

4.5.3 Repair threshold

The repair threshold, X_{repair} (also referred to as the damage limit state), will be considered as a static parameter of the maintenance strategy in that the defined limit state will apply to all maintenance interventions throughout the life of the structure for any given analysis. The influence of the repair threshold, however, will be investigated over a range of practical values from 0.5 % to 5 % extent of corrosion damage for patch repairs and extents of 12 % and 20 % for full rehabilitative overlay (Englund et al., 1999; Fitch et al., 1995; Li, 2004; Stewart, 2006; Directoraat-Gerneraal-Rijkswaterstaat, 2000; Mullard & Stewart, 2011). The probability of severe corrosion damage for any repair threshold can be represented by $Pr(d_{crack}(t) \ge X_{repair})$. As shown in Figure 2-8, for example, that for repair threshold set at 1%, then there is a 90% probability for A = 36 m² that damage will occur between 16 years and 110 years (probability of occurrence between 0.05 and 0.95). So the time to first repair could be as little as 16 years. If a less stringent repair threshold ($X_{repair} = 2.5\%$) is selected, the first repair actions will be delayed by at least 4 years. Repair threshold X_{repair} is a critical parameter for maintenance strategies.

4.5.4 Modelling process

The spatial time-dependent reliability analysis presented herein contains a number of highly non-linear limit state functions and a large number of random variables, spatial random fields and spatially dependent random variables. Therefore, a closed form solution is not tractable, and a Monte Carlo Simulation is used in the analysis to provide the predictive outcomes. The process is described as the following steps;

• The RC structure (or area of the structure) that is to be analysed is discretised into a number of elements of size Δ . The midpoint method is used to discretise the nominated area.

• A random field is created for every spatial variable where the value of the random field for each element is defined by a spatially correlated random variable generated from the corresponding probabilistic distribution. The correlation coefficient for each element is determined by Eq. 4-35, which is then used to form the covariance matrix (Eq. 4-36). A vector of spatially correlated random variables is then generated for the corresponding random field (Eq. 4-37). The value of the spatial variable (concrete cover, concrete strength, CO_2 diffusion coefficient and binding capacity) is then defined based on the statistical parameters of the underlying distribution and the spatially correlated random variable for each element in the random field.

• Once all random fields, spatially dependent, standard random and deterministic variables are defined (Section 4.3.2 to Section 4.3.4), the stochastic deterioration models are used to predict the timing of corrosion damage for each element (Section 4.3.5 and Section 4.3.6). The extent of corrosion initiation and corrosion damage at a given time can be calculated and checked against the damage limit state (Eq. 4-38 and Eq. 4-39). The repair threshold is defined as being exceeded when a given percentage of the RC surface exhibits cover cracking to a limit crack width of 1.0 mm. The extent of corrosion damage exceeding the repair threshold ($d_{crack} \ge X_{repair}$) at time t can be described by Eq. 4-40.

• When the damage limit state has been exceeded, the first maintenance action will take place as defined by a maintenance strategy (see Chapter 6).

4.5.5 Summary of modelling parameters

The spatial and spatially dependent random variables, standard random variables and deterministic variables, as well as the model errors used in the spatial time-dependent reliability analysis, are summarised in Table 4-8 and Table 4-9.

Parameter	Distribution	Mean	COV	θ (m)
Concrete cover	Truncated normal	Table 4-9	Table 4-9	2
Concrete strength fc(28)	Truncated normal	Table 4-9	Table 4-9	1
Diffusion coefficient D ₁	Lognormal	Table 4-7	σ=0.15	2
Binding capacity a	Lognormal	Eq. 4-26	0.3	2

Tuble 1 21 Studisticul pur unicelli s for correston pur unicelli sy muter fui proper ties und unicelisions	Table 4-9	: Statistical	parameters f	for corrosion	parameters,	material	properties a	and dimensions.
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Parameters	Mean	COV	Distribution	Reference
concrete cover				
Australia: cast in situ	C_{nom} +6 mm	σ=11.5 mm	Truncated normal ^a	McGee (1999)
Australia: precast	$C_{nom}\!\!+\!\!3\ mm$	σ=9.7 mm	Truncated normal ^a	McGee (1999)
China: beams and cast in situ slabs	0.90C _{nom}	0.300	Truncated normal ^a	RGRRCSM (1985)
China: precast slabs	$1.30C_{nom}$	0.250	Truncated normal ^a	RGRRCSM (1985)
f _c (28)				
Australia: N32	1.06F'c	0.152	Truncated normal ^b	Foster et al. (2013)
China: C20	0.98F'c	0.221	Truncated normal ^b	Peng and Stewart (2014)
China: C25	0.90F'c	0.196	Truncated normal ^b	Peng and Stewart (2014)
China: C30	0.98F'c	0.193	Truncated normal ^b	Peng and Stewart (2014)
\mathbf{f}_{t}	$0.53(f_c)^{0.5}$	0.13	Normal	Mirza et al. (1979)
E _c	$4600(f_c)^{0.5}$	0.12	Normal	Mirza et al. (1979)
n _d	0.242	0.12	Normal	Stewart et al. (2011)
ME(r _{crack})	1.04	0.09	Normal	Mullard and Stewart (2010)
δ_0	15 µm	0.1	Normal	Stewart et al. (2011)
k _{site}				Peng and Stewart (2014)
Urban area	1.14	0.08	Truncated normal ^c	Peng and Stewart (2014)
Suburban area	1.07	0.06	Truncated normal ^c	Peng and Stewart (2014)
Rural area	1.05	0.04	Truncated normal ^c	Peng and Stewart (2014)

Notes - ^a: truncated at 8 mm. ^b: truncated at 0 MPa. ^c: truncated at 1.0.

4.5.6 Number of Monte Carlo simulations

Melchers (1999) suggests that the accuracy of a Monte Carlo simulation analysis can be indicated by the convergence of the predicted outcome as the number of samples is increased. Figure 4-5 shows the predicted mean value of the extent of cracking (d_{crack}) for a 36 m² RC surface with an element size of $\Delta = 0.5$ m and scale of fluctuation as presented in Table 4-8. It can be seen that the predicted value of d_{crack} rapidly converges above 1000 simulations. Similarly, Figure 4-6 shows that the predicted value for the probability of the first repair for sheltered cast in-situ RC slabs in Kunming, RCP8.5 (as described by Eq. 4-41, refer to Chapter 5 for a full description and assume a repair threshold of 5%), converges rapidly as the number of Monte Carlo simulations exceeds 1000.

$$\Pr(d_{crack}(t) \ge x\%) = \int_{x\%}^{100\%} f_{d_{crack}}(d_{crack}, t) dd_{crack}$$
4-41



Figure 4-5: Convergence of d_{crack}(t) with increased Monte Carlo simulations.



Figure 4-6: Convergence of probability of first repair with increased Monte Carlo simulations.

Studies conducted using Monte Carlo simulation techniques can be time intensive as the models typically include a large number of spatial and stochastic parameters. The analyses conducted herein make use of random fields to model spatial variability and as such the mathematical manipulation of large matrices is required. The time taken to analyse a 36 m² RC surface, using 10,000 Monte Carlo simulations is approximately 6 minutes (using an Intel(R) Core(TM) i7 CPU 870 @2.93GHz 16.0 GB (RAM) 64-bit Operating System). It is clear from Figure 4-5 and Figure 4-6 that an increase in the number of Monte Carlo simulations above 10,000 will not provide a discernible increase in accuracy of the predictive outcomes. As such, all results produced herein will be calculated using 10,000 run cycles of the spatial time-dependent reliability analysis.

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Chapter 5 : Climate change impacts on carbonation induced corrosion

5.1 Introduction

This chapter will present results of climate change impacts on deterioration of RC structures from the spatial time-dependent reliability analysis. The influence of key parameters on the corrosion-induced deterioration of RC structures will be investigated, and results presented in terms of the likelihood and extent of severe cracking.

RC slabs and beams of buildings and RC bridge decks in two Australian cities, Sydney and Canberra, and three Chinese cities, Kunming, Xiamen and Jinan, are investigated and presented in detail in this chapter. Cast in-situ and precast construction methods, sheltered and unsheltered microclimates conditions, durability design requirements and various climate characters, etc. are considered.

This chapter seeks to provide insights into the possible influences of a changing climate on the corrosion induced damage risks and durability of concrete structures in Australia and China, under various climate change scenarios. The impacts of climate change will be expressed in terms of:

- (i) mean extent of corrosion initiation and damage (d_{ini} and d_{crack});
- (ii) mean time of first repair;
- (iii) 5^{th} percentile of time of first repair.

This will have similar implications for other countries since the primary environmental driver to increased concrete deterioration is CO_2 concentration, temperature and RH, which will affect all concrete infrastructure globally.

5.2 Climate projections for Australian and Chinese cities

Five cities are selected: two Australian cities, Sydney and Canberra (Figure 5-4); and three Chinese cities, Kunming, Xiamen and Jinan (Figure 5-5). Most of the populations in Australia are distributed near the coastline. Sydney, as the capital city of the state of New South Wales, has the largest population in Australia. It has a population of 4.7 million and is on the southeast coast of Australia. Canberra is the capital city of Australia. With a population of 0.4 million, it is the largest inland city

in Australia and the eighth-largest city overall. Sydney and Canberra are representatives of coastal and inland cities, respectively. China has the largest population in the world. Kunming as the largest and the capital city of Yunnan Province in Southwest China has a population of 6.4 million. Xiamen is a major city located on the southeast coast of China whose population is more than five million people. Jinan located in eastern China, is the capital of Shandong province. The city, has about 6.8 million people, and is located about 400 kilometres south of Beijing. Kunming is chosen as a representative of an inland temperate area. Xiamen is regarded as a typical coastal temperate city. Jinan represents an inland city in a cold area. The climate in these five cities ranges from temperate climate, hot and humid climate to cold and dry climate.

5.2.1 Emission scenarios

RCP 8.5 and RCP 4.5 emission scenarios are included in this study, representing high and medium emission scenarios, respectively. Because all cases studies presented later on are assumed to be constructed by 2010, an emission scenario based on the Year 2010 CO_2 level of 389 ppm is also taken into account to provide a reference for other emission scenarios. Figure 5-1 describes the IPCC projection of the annual average CO_2 concentrations from 2010 based on selected emission scenarios from different modelling teams/models, specifically related to RCP 8.5 and RCP 4.5 CO_2 stabilisation scenarios (Clarke et al., 2007; Meinshausen et al., 2011; Riahi et al., 2007; Smith & Wigley, 2006; Wise et al., 2009). Note that a recent study shows that current emissions are tracking slightly above RCP 8.5 (Peters et al., 2013). If this trend persists, then it is likely that CO_2 concentrations will reach 1,000 ppm by the end of this century. The CO_2 concentrations correction parameter k_{site} of urban will apply to Kunming, Xiamen, Jinan and the coastal part of Sydney. On the other hand, k_{site} of suburban is used for Canberra and the outer suburbs of Sydney.



Figure 5-1. Projected global annual average CO₂ concentration for RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.

5.2.2 Temperature and RH projections

As discussed in Section 2.2.2, six GCMs are used to project future temperature and RH, i.e., BCC-CSM1.1, MIROC5, IPSL-CM5A-LR, CSIRO-MK3.6.0, CNRM-CM5, ACCESS1.0. Figure 5-2 and Figure 5-3 show the projections of changes in temperature and relative humidity in five cities, i.e., two Australian cities Sydney and Canberra. and three Chinese cities - Kunming, Xiamen and Jinan, representing different regional climate conditions in Australia and China, ranging from temperate, hot humid climate to cold dry climate. The projections in the figures are based on the six GCMs as listed in Table 2-1, with each of them plotted in dashed curves. The average value of all the six models is plotted by the solid line curves in the figures. The changes in temperature and relative humidity since 2010 are described in degrees Celsius and percentage respectively. As seen in Figure 5-2 and Figure 5-3, the average trend of temperature increases in the five cities, following the global warming projection, while the average trend of relative humidity decreases. It was also found that the increases in the air temperature in a hot/warm regional climate are higher than those in a cold climate. All the projections based on the six climate models and two emission scenarios will be used in the simulation of concrete deterioration under climate change.



d. Temperature projections for Xiamen




Figure 5-2: Temperature projections of six GCMs for Sydney, Canberra, Kunming, Xiamen and Jinan under RCP 8.5, RCP 4.5 and Year 2010.



Relative humidity projections for Kunming

PhD Thesis - The University of Newcastle, Australia



e. Relative humidity projections for Jinan

Figure 5-3: RH projections of six GCMs for Sydney, Canberra, Kunming, Xiamen and Jinan under RCP 8.5, RCP 4.5 and Year 2010.

5.3 Durability design specifications for RC structures in two countries

According to the national code, requirements of concrete cover and concrete properties are not constant. Generally, code requires concrete covers ranging from 15 mm to 50 mm depending on the grade of concrete, and the exposure class. The Australian Concrete Structures Code AS3600 (2009) classifies environmental exposure in Australia as three climatic zones (arid, temperate and tropical), see Figure 5-4. Similarly, the environmental exposures in China are also classified into three climatic areas, as severe cold area, cold area and temperate area (GB50010-2010, 2010; Peng & Stewart, 2014a), see Figure 5-5. Most people in Australia live close to the coast, so chloride induced corrosion is the cause of concrete corrosion that is of the most concern. However, for China, about 86% of the population live in temperate and cold zones. Therefore, the results of RC structures in selected sites of Kunming, Xiamen and Jinan can be typical for most RC buildings. On the other hand, the microclimate conditions concern the position of structural elements in relation to the fluctuating water level (and the period such elements stay dry) such as 'aboveground'

'in-ground' and 'maritime', and pH, sulphates and chlorides content in the water or soil. RC slabs and beams in a concrete structure and RC bridge decks that is 50 km from the coast is studied herein (e.g., in an outer suburb of Sydney) to simplify the problem. The results for RC slabs can be applied to roofs, floors, panels and walls, etc. Note that RC structures are assumed to use OPC concrete, and standard formwork and compaction.



Figure 5-4. Climatic zones of Australia defined by the Australian concrete code AS3600 (2009).



Figure 5-5. Climatic zones of China defined by the Chinese code JTG.D62-2004.

Concrete inside buildings in a dry environment, concrete surfaces subject to long-term rain in unsheltered exposures, and RC permanently submerged in water generally have a low carbonation rate. However, concrete inside buildings with moderate air humidity, and external concrete sheltered from rain have a higher likelihood of carbonation (Richardson, 1988). After corrosion initiates, more humid exposure increases corrosion propagation. Therefore, the spatial time-dependent reliability analysis to follow will focus on corrosion predictions of sheltered and unsheltered RC slabs. Note that RC buildings without air conditioning are more critical, such as warehouses and factories, due to their exposure to changes in the environment.

5.3.1 Australian building and bridge standard

The durability design requirements specified in AS3600 (2009) are about minimum concrete cover and concrete compressive strength. Table 5-1 shows the durability design specifications from the AS3600 exposure classifications A1 to C for carbonation, and assume standard formwork and compaction.

Table 5-1 also shows the parameter values for the deterioration models. The exposure classifications of most relevance are near-coastal (B1: 1-50 km from coast) and coastal excluding tidal and splash zones (B2: <1 km from coast) for the most parts of Sydney, and A2 (>50 km from coast) for Canberra and outer western suburbs of Sydney. B1 and A2 are two very different durability design requirements with design cover for B1 being 40mm due to its near coastal location, and 30 mm cover for A2 due to its inland location. Note that RC structures close to the shore (B1 and B2) in Sydney have higher concrete cover and are not susceptible to carbonation induced corrosion.

For RC bridges, the design life is usually 100 years which is longer than most of the buildings' lifespan of 40-60 years. According to AS5100.5 (2004), the main differences of durability requirements between bridges and buildings are concrete cover. The covers for bridges are commonly 5-10 mm higher than buildings. For example, exposure classification A1 and A2 for RC buildings are combined as exposure classification A for RC bridges, and the durability design requirements can be referred in Table 5-1.

Exposure	Class	Cover (mm)	F' _c (MPa)	w/c ^d ratio	C_e^a (kg/m ³)	mean D ₁	mean n _d
Members in exterior environment	s:						
Inland (> 50 km from the coast):							
non-industrial and arid climate	A1	20	20	0.56	320	2.22	0.240
non-industrial and temperate	Δ2	30	25	0.56	320	2 22	0 240
climate	112	50	25	0.50	520	2.22	0.240
For bridges (A1+A2)	А	35	25	0.56	320	2.22	0.240
non-industrial and tropical	B1	40	32	0.50	320	1.24	0.235
non-industrial and any climate	B 1	40	32	0.50	320	1.24	0.235
Near-coastal (1-50 km), any climate	B1	40	32	0.50	320	1.24	0.235
Coastal (< 1 km, excluding tidal and splash zones), any climate	B2	45	40	0.46	370	0.65	0.218
Surfaces of members in water: in tidal or splash zone	C^{e}	50	50	0.40	420	0.47 ^b	0.19 ^c

 Table 5-1: Australian durability design requirements (AS3600, 2009) and deterioration models for carbonation corrosion.

Notes - a: for minimum cement content required in AS 5100.5, b: the conservative value - highest value based on polynomial, exponential and power extrapolations from Table 3-1, c: the conservative value - lowest value relied on polynomial, exponential and power extrapolations from Table 3-1, d: based on maximum w/c ratio defined in AS 5100.5, e: AS3600-2001.

5.3.2 Chinese building and bridge standard

Exposure classifications for buildings in China are divided into five classes, see Table 5-2. Durability requirements for RC buildings in China (GB50010-2010, 2010) such as minimum concrete cover and concrete compressive strength are presented in Table 5-3. The exposure classifications for RC slabs in the three Chinese cities are exposure I and IIa for sheltered and unsheltered environments, respectively. Note that the concrete compressive strengths in Chinese standards are based on cube compressive strengths.

Class	Exposure
Ι	Indoor dry environment; or environment contact with non-erosive still water
IIa	Indoor humid environment; or open-air environment in the temperate area; or environment of directly contacted with non-erosive water or soil in the temperate area; or environment below the frost line and directly contacted with non-erosive water or soil in severe cold or cold area
IIb	Cyclic wet-dry environments; or area with water level fluctuation; or open-air environment in severe cold or cold area; or environment above frost line and directly contacted with non-erosive water or soil in severe cold or cold area
IIIa	Environment influenced by de-icing salts; or environment in severe cold or cold area with water level fluctuation in winter season; or near-coastal environment
IIIb	Saline soil environment; or environment influenced by de-icing salts; or coastal environment

Table 5-2: Exposure classifications (GB50010-2010, 2010) for Chinese buildings.

Class	F'c ^a (MPa)	w/c ratio	C_e^b (kg/m ³)	Mean D ₁	Mean n _d	Cover/bar diameter (mm)	
						Slab	Beam
Ι	C20	0.60	225	2.65	0.242	20/12	25/16
IIa	C25	0.55	250	2.22	0.240	25/12	30/16
IIb	C30	0.50	275	1.24	0.235	25/16	35/24
IIIa	C35	0.45	300	0.65	0.218	30/20	40/28
IIIb	C40	0.40	325	0.47	0.190	40/28	50/32

 Table 5-3: Chinese durability design specifications (GB50010-2010, 2010) and deterioration models for building structures.

Note - ^a: C represents cube compressive strength. ^b: from GB50010-2002 (2002).

Table 5-4 presents the Chinese durability design requirements for bridges (JTG.D62-2004, 2004). Clearly, the environmental categories for Chinese RC bridges are defined differently, and like Australia, the requirements are generally higher than that for buildings. The exposure classification for concrete bridges in Xiamen is exposure II, and exposure I for bridges in Kunming and Jinan.

 Table 5-4: Chinese durability design specifications (JTG.D62-2004, 2004) and deterioration models for bridge structures.

Exposure	Class	F'c ^a (MPa)	w/c ratio	C _e (kg/m ³)	Mean D ₁	Mean n _d	Cover (mm)	Bar diameter (mm)
Temperate environment or cold area; contact with non-erosive water or soil	Ι	C25	0.55	275	2.22	0.240	30	20
Severe cold environment	II	C30	0.50	300	1.24	0.235	40	28
Sea water environment	III	C35	0.45	300	0.65	0.218	45	32
Environment affected by erosive materials	IV	C35	0.40	325	0.47	0.190	45	32

Note - ^a: C represents cube compressive strength.

5.4 Model parameters

5.4.1 Random field parameters

Two critical parameters, the scale of fluctuation and the discretised element size, impacts on the likelihood and extent of corrosion damage are investigated and will now be discussed.

The scale of fluctuation is defined as the distance over which correlation persists in a random field (see Chapter 4). A large scale of fluctuation corresponds to a highly correlated random field, and if the scale of fluctuation is sufficiently large, the random

field could be regarded as perfectly correlated. A perfectly correlated random field (i.e. no spatial variability) corresponds to the case where the RC surface is considered homogeneous, and this has been shown to underestimate the probability of failure (Stewart, 2006). A small scale of fluctuation, on the other hand, can overestimate the variability and result in higher probabilities of failure. Mullard (2010) presented the effect of scale of fluctuation on the probability of first maintenance ($Pr(d_{crack}(t)\geq X_{repair})$) for a 25 m² RC surface with a discretised element size of 0.5 m for a repair threshold of 1%; see Figure 5-6. It can be seen that as the scale of fluctuation increases, the probability of the first maintenance decreases. This is expected as the variability or 'randomness' of the modelled surface is decreasing to the point of being represented by a single random variable (homogeneity). Conversely, as the scale of fluctuation decreases, the modelled surface approaches an uncorrelated series of elements, thus increasing the variability and therefore the probability of the first maintenance.

As discussed in Chapter 4, the estimation of the scale of fluctuation can be calculated from existing data or estimated using engineering judgement. There is a limited amount of data from real structures and a range of estimates found in the literature (based on both data and engineering judgement) that is presented in Table 4-2. Given the available data, the scale of fluctuation of 1.0 and 2.0 m is identified as a reasonable value and will be used in the current study.



Figure 5-6: Influence of Scale of fluctuation on the probability of 1st maintenance (Mullard, 2010).

The selection of the size of the discretised element in a random field is important, and this has been discussed in Chapter 4. The value of the random field is assumed to be constant over the discretised element and a large element size can, therefore, significantly underestimate the spatial variability. Conversely, for the same scale of fluctuation, a small element size can overestimate the spatial variability and also create numerical problems associated with the decomposition of a large covariance matrix.

As discussed in Chapter 4, a selection of element size is based on practical and analytical considerations (Sterritt et al., 2001). It must be expected in practice that the spatial properties of the structure being analysed will not vary over the element length. For example, in a random field for concrete cover, a very big element size would not be appropriate as cover can vary significantly over small distances. The size of the element must also take into account the physical and analytical relationship with the scale of fluctuation of the random field. For example, Sudret and Der Kiuraghian (2000) suggest that for a Gaussian correlation function, an error is negligible when the ratio of element size to scale of fluctuation is below approximately 0.28. Thus for a scale of fluctuation of 2.0 m, an appropriate element size would be less than approximately 0.57 m. Figure 5-7 shows the probability of the first maintenance for a 100 m² RC surface with a range of element sizes and a scale of fluctuation equal to 2.0 m. It can be seen that the probability converges somewhat as the element size decreases from 0.67 m to 0.4 m. For element sizes smaller than 0.4 m, computational problems can arise (as well as the likelihood of overestimating the spatial characteristics of the random field) whilst for element sizes greater than 0.67 m, possible error is introduced (Haldar & Mahadevan, 2000; Sudret & Der Kiureghian, 2000) along with the likelihood of underestimating the spatial variability of the random field. The important consideration here, however, is that the element size chosen reflects the physical characteristics of the random field being modelled and is analytically and physically compatible with the chosen scale of fluctuation. It has been established from the discussions in Chapter 4 and the current section that an element size of 0.5 m satisfies both these criteria and will, therefore, be the element size used in the spatial time-dependent reliability analysis.



Figure 5-7. Influence of element size on the probability of 1st maintenance (Mullard, 2010).

To demonstrate the spatially-distributed carbonation induced corrosion progress on a two-dimensional surface, the carbonation depth, time to corrosion initiation T_i , time to first cracking T_{1st} , and time to severe corrosion damage T_{sev} of each element are calculated. Figure 5-8 shows three typical Monte Carlo realisations. The area where corrosion started, cracking initiated and severe corrosion damaged is demonstrated in every ten years after 2070. Each realisation shows a rather different corrosion progression in time and space. It also shows the clustering of damage. For the spatial time-dependent reliability analysis, the results such as the extent of surface corrosion damage were calculated at each time step for 10,000 simulation runs.

(a)	(b)	(c)
2080:		
2090:		
2100		
2100:		
Corrosion not initiated.	rosion initiated. Crack initiated.	Severe corrosion damage.

Figure 5-8. Simulation of spatially distributed corrosion process showing three typical Monte Carlo realisations for cast in-situ sheltered RC slab in Kunming, RCP 8.5 (Peng & Stewart, 2014b).

5.4.2 Variability of climate projections

Even though the trend of temperature and RH projections for Sydney and Kunming are similar, the variations are quite different, and the variability of six GCM projections models is also considerable, therefore, simulations are run separately based on each GCM projection model. The effect of six GCMs temperature and RH projections on the extent of surface corrosion damage is shown in Figure 5-9, for RCP 8.5 emission scenario and cast in-situ sheltered RC slabs in Kunming and Sydney. There is only a 1.5% difference between the maximum and minimum predicted extent of surface corrosion damage which is a rather low variability. Similarly, there is negligible variability for the percentile of the extent of corrosion damage of six GCMs, as shown in Figure 5-10. The histograms of the extent of corrosion damage of six GCMs also show no noticeable differences. Therefore, it is reasonable to present the average of the simulation results of six GCMs.



Figure 5-9. Mean extent of surface corrosion damage for each GCM and the average for cast insitu sheltered RC slabs in Sydney and Kunming, RCP 8.5 emission scenario.



Figure 5-10. Probability contours for extent of surface corrosion damage for each GCM and the mean for cast in-situ sheltered RC slabs in Sydney and Kunming, RCP 8.5 emission scenario.

5.4.3 Year of construction

In this chapter, RC structures are assumed to be built in 2010. However, they can be built in any year as long as the climate projections are available. For example, to study climate adaptation strategies conducted at design stage, construction year at 2015 is appropriate (see Chapter 7). Figure 5-11 shows the time-dependent mean extent of corrosion damage of cast in-situ sheltered RC slabs built in 2010 and 2015 in Kunming under RCP 8.5, RCP 4.5 and reference (climate condition is assumed to be constant as Year 2010 or Year 2015 level) scenarios. As expected, the results of the extent of corrosion damage for RC slabs built in 2015 have almost the same trend as results for slabs build in 2010 but with 5 year's delay. The differences between these trends are mainly due to climate conditions changes with time.



Figure 5-11: Mean extent of surface corrosion damage for cast in-situ sheltered RC building slabs in Kunming.

5.5 Applications: Australian RC building slabs and bridge decks

5.5.1 Structural configuration for Australian cases

The RC building slab considered is a $6m \times 6m$ slab built in 2010. Both sheltered (e.g. floors, balconies and roofs) and unsheltered (e.g. facades, balconies and roofs) microclimate conditions, precast and cast-in-situ construction methods are included. The durability design requirements for Australian RC structures such as concrete cover, concrete strength, bar diameter is presented in Table 5-1. As discussed in Section 5.3.1, RC building slabs in Sydney and Canberra are under exposure

classification A2 (>50 km from coast) will be analysed herein. The reference carbonation corrosion rate for sheltered members is C3, and C4 for unsheltered ones; see Table 3-5. On the other hand, the age factor for microclimate conditions n_m in Eq. 3-1 is 0.12 for unsheltered outdoor exposures and is 0 for sheltered exposures.



Figure 5-12. Typical building structures used for analysis of RC building slabs.

Similarly, a RC bridge deck is assumed as a 10 m \times 10 m slab built in 2010. Both precast and cast-in-situ construction methods, as well as sheltered (e.g. sheltered pedestrian bridge), and the unsheltered microclimate condition are included. RC bridge decks in Sydney and Canberra are under exposure classification A (>50 km from the coast).



Figure 5-13. Typical bridge structures used for analysis of RC bridge decks.

Unless otherwise noted, the parameters used in the analysis of the RC building slab and bridge deck are those shown below (for a full description of all variables and statistical parameters, refer to Chapter 4):

• Discretised element size $\Delta = 0.5$ m

- Scale of fluctuation for concrete cover $\theta = 2.0 \text{ m}$
- Scale of fluctuation for concrete compressive strength $\theta = 1.0$ m
- Scale of fluctuation for CO_2 diffusion coefficient $\theta = 2.0$ m
- Scale of fluctuation for concrete binding capacity $\theta = 2.0$ m
- Size of limit crack width $w_{lim} = 1.0 \text{ mm}$

5.5.2 Results for Australian RC building slabs and bridge decks

The mean carbonation depths of sheltered and unsheltered RC building slabs and bridge decks in Sydney and Canberra are shown in Figure 5-14 and are based on the average of six GCM projections. Both precast and cast-in-situ construction methods will lead to the same mean carbonation depth. For unsheltered outdoor exposures, the age factor for microclimate conditions n_m is 0.12. Hence, the mean carbonation depths for unsheltered slabs are reduced significantly.

The RCP 8.5, RCP 4.5 and the Year 2010 emission scenarios have a significant effect on carbonation depths, and carbonation depths for RCP 8.5 and RCP 4.5 emission scenarios can be increased by up to 5.5 mm by 2100. For example, for buildings in Sydney, the RCP 8.5 emission scenario increases carbonation depth by approximately 35% when compared to the reference Year 2010 CO_2 emissions, whereas the RCP 4.5 increases carbonation depths by only 14%. The COV of carbonation depth increases from 0.05 in 2010 to 0.08 by 2100.

Carbonation depth for RC structures in Sydney is very close to those in Canberra. Because both cities have similar temperature and RH projections, as well as the same durability design requirements. Further, the mean carbonation depth for buildings and bridges in Australia are expected to be the same (see Figure 5-14). For example, the mean carbonation depth for RC buildings in Sydney is identical as those for bridges. The only difference of design requirement between RC bridges and buildings in Australia is concrete cover, however concrete cover thickness will not influence the carbonation process.



Figure 5-14. Mean carbonation depth of building slabs and bridge decks in Sydney and Canberra, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.

Carbonation depth increases with time, when the carbonation depth reaches the surface of reinforcing bar then corrosion is initiated. Therefore, the extent of corrosion initiation is mainly determined by carbonation depth and concrete cover. Figure 5-15 and Figure 5-16 present the mean extent of corrosion initiation for RC building slabs and bridge decks in Sydney and Canberra, including sheltered and unsheltered members, cast in-situ and precast members. It can easily seen that the mean extent of corrosion initiation of sheltered structural members is significantly higher than those for unsheltered members, due to the higher carbonation depth of sheltered members. It is interesting to note that the mean extent of corrosion initiation is slightly higher for precast slabs than cast in-situ ones. This is due to the relatively lower mean concrete cover of precast members (see Table 4-9). However, bridge decks show no significant difference between precast and cast in-situ methods. The elements that are corrosion initiated for RCP 8.5 emission scenario can be twice as much for the reference Year 2010 emission scenario (see Figure 5-15 and Figure 5-16). In Sydney and Canberra, less than 5% of the sheltered concrete surface of bridge decks will start corrosion by the end of this century; while the extent is about 12% for sheltered RC building slabs (see Figure 5-15 and Figure 5-16).



Figure 5-15. Mean extent of corrosion initiation of building slabs in Sydney and Canberra, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.



Figure 5-16. Mean extent of corrosion initiation of bridge decks in Sydney and Canberra, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.

After corrosion is initiated, the corrosion process is dominated by the corrosion rate. Figure 5-17 and Figure 5-18 present the mean extent of corrosion initiation for RC building slabs and bridge decks in Sydney and Canberra, including sheltered and unsheltered members, cast in-situ and precast members. Even though the sheltered members are predicted to have significantly more extent of corrosion initiation than unsheltered ones, the corrosion damages are estimated to be similar. Therefore, regardless of sheltered or unsheltered exposures and cast in-situ or precast construction methods, for RC building slabs in Sydney it is likely that about 1% of RC surfaces will be damaged by 2100 (see Figure 5-17 a-d). Less than 0.5% of the concrete surface will be damaged for RC bridge decks in Sydney and Canberra, for RCP 8.5 emission scenario by 2100, respectively (see Figure 5-17 and Figure 5-18). The extent of corrosion damage for RC structures in Canberra is slightly lower than Sydney's, due to Sydney's higher temperature and humidity. Precast members have less corrosion damage extent than cast in-situ ones, due to better quality control (lower standard deviation, see Table 4-9). The differences between results of RCP 8.5 and Year 2010 emission scenario for unsheltered members are about twice as much for sheltered members, these mean that climate change can have larger impacts on unsheltered members than sheltered ones. Overall, the extents of corrosion initiation and corrosion damage for RC building slabs and bridge decks in Sydney and Canberra are quite low.

The probability of severe corrosion damage for any repair threshold can be represented by $Pr(d_{crack}(t) \ge X_{repair})$. A review of the literature shows that the repair threshold (X_{repair}) ranges from 0.5 - 5% for patch repair, and 12 - 20% for complete replacement (e.g. Mullard & Stewart, 2012). If repair threshold is set as 5%, then the mean time to first repair is obviously later than 2100 for all RC buildings and bridges in Sydney and Canberra. Selecting a more stringent repair may happen at 2090, 2092 and 2094 for cast in-situ sheltered building slabs, cast in-situ unsheltered slabs and precast sheltered slabs in Sydney under RCP 8.5, respectively (see Figure 5-17 a-c). It is interesting to note that, all structural type shown in Figure 5-17 and Figure 5-18 do not need repair before 2100 if under the reference 2010 emission scenario. These indicate that climate change may cause severe corrosion damages to RC structures that normally last longer. The mean time to first repair of other structural types in Sydney and all structures in Canberra are later than 2100.



Figure 5-17. Mean extent of corrosion damage of building slabs in Sydney and Canberra, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.



Figure 5-18. Mean extent of corrosion damage of bridge decks in Sydney and Canberra, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.

Figure 5-19 shows the histograms of the extent of surface corrosion damage for cast in-situ sheltered RC slabs in Sydney at 2040, 2070 and 2100, under RCP 4.5 with interval of bins of 0.7%. It can be seen that $f_{d_{crack}}(d_{crack},t)$ is highly non-Gaussian. The likelihood of no or little damage is significantly high, but it will decrease with time.

After 30 years (2040), the likelihood of no or little damage is almost 100% for cast insitu sheltered RC slabs under RCP 4.5, and the probability can still be as high as 60% after 90 years (2100). The histograms of the extent of surface corrosion damage for other slabs and other emission scenarios show similar trends.

Similar conclusions can be found from Figure 5-20. Figure 5-20 shows probability contours for the extent of surface corrosion damage for cast in-situ sheltered building slabs in Sydney, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios. Each probability contour represents the probability that at least x% of the concrete surface is damaged (given by Eq. 4-40). Figure 5-20 shows the extent of surface corrosion damage for cast in-situ sheltered RC slabs in Sydney is very low, that only about 5.2% of the total area will be corrosion damaged by 2100 under the reference Year 2010 emission scenario, even for 0.05 probability contours. These indicate that there is 95% confidence that the extent of surface corrosion damage will be less than 5.2% of the total area of cast in-situ sheltered RC slabs in Sydney by 2100. RCP 4.5 and RCP 8.5 only increase the damage extent by 0.7% and 1.6%, respectively.

The probability contours may also be used to predict the time to first repair. For example, there is a 5% chance that the first repair for cast in-situ sheltered slabs in Sydney under RCP 8.5 emission scenario can occur as soon as 2060 and 2085 for repair thresholds of 1% and 5% respectively (see Figure 5-20).



Figure 5-19. Simulation histogram of the extent of surface corrosion damage for cast in-situ sheltered RC building slabs in Sydney in 2040, 2070 and 2100, under RCP 4.5 emission scenario.



Figure 5-20. Probability contours for the extent of surface corrosion damage for cast in-situ sheltered building slabs in Sydney, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.

5.6 Applications: Chinese RC building slabs and beams and bridge decks

5.6.1 Structural configuration for Chinese cases

The climate in three Chinese cities, Kunming, Xiamen and Jinan, range from temperate, hot and humid climate to cold and dry climate, and therefore the results for these three cities will include a range of possible impacts of climate change on RC structures. On the other hand, RC structures in different locations or under different situations may use other exposure classifications according to durability design requirements. For example, RC bridges in Xiamen are under exposure class II due to coastal environments. The exposure classifications for RC buildings and bridges in each city are presented in Table 5-5. Detailed descriptions of each exposure class are presented in Table 5-1, Table 5-2, Table 5-3 and Table 5-4.

	Kunming	Xiamen	Jinan	Sydney	Canberra
Sheltered:					
Buildings	Ι	Ι	Ι	A2, B1, B2, C	A2
Bridges	Ι	II	Ι	A, B1, B2, C	А
Unsheltered:					
Buildings	IIa	IIa	IIb	A2, B1, B2, C	A2
Bridges	Ι	II	Ι	A, B1, B2, C	А

Table 5-5: Exposure classifications for RC buildings and bridges in five cities.

Other than differences in exposure classifications, Chinese durability design specifications define different requirements for various structural types (see Table 5-3 and Table 5-4). Unlike Australian specifications, the RC slabs and beams in a building are under different design requirements. Therefore, except for RC building slabs and bridge decks, RC building beams will also be analysed for three Chinese cities. The geometry of RC building slabs and bridge decks are assumed to be 6 m \times 6 m and 10 m \times 10 m, respectively. As discussed in Section 5.4.2, a RC beam as Figure 5-12 shows can be discretised into one-dimensional random field. Therefore, the bottom face of a RC beam with size of 0.2 m \times 0.5 m \times 6 m can be modelled as Figure 5-21 shows. Durability design requirements for Chinese RC structures are presented in Table 5-3 and Table 5-4.



Figure 5-21. RC beam for spatial analysis.

Unless otherwise noted, the parameters used in the analysis of the RC structures in China are those shown below (for a full description of variables and statistical parameters, refer to Chapter 4):

- Discretised element size $\Delta = 0.5$ m
- Scale of fluctuation for concrete cover $\theta = 2.0$ m
- Scale of fluctuation for concrete compressive strength $\theta = 1.0$ m
- Scale of fluctuation for CO_2 diffusion coefficient $\theta = 2.0$ m
- Scale of fluctuation for concrete binding capacity $\theta = 2.0 \text{ m}$
- Size of limit crack width $w_{lim} = 1.0 \text{ mm}$

5.6.2 Results for Chinese RC building slabs, beams and bridge decks

Figure 5-22 shows the mean carbonation depth for sheltered and unsheltered RC building slabs and beams and bridge decks in three Chinese cities under all emission scenarios. Like the results for Australian cases, sheltered structural members will suffer more carbonation than unsheltered ones, and climate change has a significant effect on carbonation depths, for RCP 8.5 emission scenario increase the depth by up to 9 mm by 2100.

Different construction methods (cast in-situ and precast) or type of structural members (building slabs and beams) in a building or bridge in China will result in the same mean carbonation depth. The mean carbonation depth for bridges in China is generally less than those for buildings (see Figure 5-22). This is because the enhanced durability design requirement for bridges compared to buildings in China. The mean

carbonation depths for sheltered members in bridges and buildings in Jinan are 24.1mm and 28.6mm under RCP 8.5 by 2100, respectively, which are the largest among the three Chinese cities. This is due to the fact that lower RH in Jinan will favour the carbonation process that increases f_{RH} . Buildings in Kunming have the lowest mean carbonation depth, because temperatures in Kunming are lower than Xiamen (see Figure 5-2). Bridges in Xiamen have significantly lower carbonation depths than those in the other two cities. Because Xiamen is a coastal city, RC bridges in coastal areas have higher durability design specifications (exposure class II), which will result in reduced rates of carbonation.



Figure 5-22. Mean carbonation depth of buildings and bridges in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.

Figure 5-23, Figure 5-24 and Figure 5-25 present the extent of corrosion initiation for RC building slabs and beams, and bridge decks in the three Chinese cities, including sheltered and unsheltered members, cast in-situ and precast members. It can easily be found out that the extent of corrosion initiation of sheltered structures members is significantly higher than those for unsheltered members. For building slabs and bridge decks, the cast in-situ method will result in more elements to be corrosion initiated than the precast method. Building beams show no significant differences between these two construction methods, because statistical properties for concrete cover and strength are the same for both cast in-situ and precast beams. RC building slabs and beams in Jinan have the highest extent of corrosion initiation, followed by those in Xiamen and Kunming. However, RC bridge decks in Xiamen have the lowest extent of corrosion initiation due to higher durability design requirements. Building slabs are predicted to have a larger extent of corrosion initiation than building beams and bridge decks with more than 80% of cast in-situ sheltered slab surfaces may at risk to start corrosion by 2100.





Figure 5-23. Mean extent of corrosion initiation of building slabs in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.



Figure 5-24. Mean extent of corrosion initiation of building beams in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.





Figure 5-25. Mean extent of corrosion initiation of bridge decks in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.

Figure 5-26, Figure 5-27 and Figure 5-28 present the mean corrosion damage of building slabs and beams and bridge decks in three Chinese cities under all emission scenarios. There is unlikely to be any significant corrosion damage for the first 30–40 years of service life, but the extent of corrosion damage then increases to 7–37% for RC buildings in three Chinese cities for the RCP 8.5 emission scenario by 2100 (see Figure 5-26 and Figure 5-27). In these cases, damage areas will increase by an additional 2–13% for structural members in buildings for the worst case emission scenario (RCP 8.5) compared with that for the (reference) best mitigation scenario. In practical terms, this is equivalent to expecting that an additional 2–13% of the concrete surface will be damaged and in need of maintenance or repair by 2100.

RC buildings in three Chinese cities show a high level of damage (see Figure 5-26 and Figure 5-27). Corrosion damages for RC buildings in three Chinese cities are very much different. Due to the hot and humid climate in Xiamen, RC buildings will suffer the most severe corrosion damage among the three cities, followed by the temperate and dry city Kunming, and then the cold and dry city Jinan. On the other hand, the extent of surface corrosion damage for RC slabs in Kunming is reduced significantly

from 16% for sheltered and cast in-situ slabs (Figure 5-26 a), to less than 5% for unsheltered or precast slabs (Figure 5-26 b, c). RC building slabs in Xiamen and Jinan show similar trends. These are due to higher durability requirements for unsheltered members (exposure IIa) than sheltered members (exposure I) in China, and that concrete cover of precast slabs in China has a higher mean and a less COV than cast in-situ slabs. Therefore, the corrosion damage risks for precast unsheltered RC slabs in Kunming, Xiamen and Jinan are negligible (Figure 5-26 d, h and l). Generally, RC beams are expected to have less corrosion damage than cast in-situ slabs but more severe corrosion damage than precast slabs (see Figure 5-27).

RC bridges in three Chinese cities show much less corrosion damage than RC buildings (see Figure 5-28). Unlike RC buildings, RC bridges in Xiamen have the lowest extent of corrosion damage due to enhanced durability design requirements for RC bridges in the coastal area (exposure classification II instead of classification I). The corrosion damage extent for RC bridges in Jinan shows an unusual phenomenon in that unsheltered members have similar corrosion damage extent as those for sheltered ones (see Figure 5-28 i-1). This might be because the RH in Jinan is too low that rainfall can favour the corrosion process instead of slowing it down.

The mean time to first repair can also be derived from Figure 5-26, Figure 5-27 and Figure 5-28. Take Kunming, for instrance. If the threshold of repair is 5%, the first repair may happen in 2072 and 2081 for cast in-situ building sheltered slabs and sheltered beams under RCP 8.5, respectively (see Figure 5-26 a, Figure 5-27 a). Other types of slabs and beams and all bridge decks in Kunming will not need repair before 2100. However, for the more stringent repair threshold ($X_{repair}=1\%$), the first repair may occur as soon as 42 years (2052) after construction for cast in-situ sheltered building slabs (see Figure 5-26 a). All structural members in buildings and bridges in Kunming require repair before 2100, except for precast sheltered and unsheltered bridge decks, and precast unsheltered slabs (see Figure 5-28 c-d, Figure 5-26 d). The mean time to first repair can be estimated for RC structures in the other two Chinese cities by applying a similar analysis to the one presented above.



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Figure 5-26. Mean extent of corrosion damage of building slabs in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.



Figure 5-27. Mean extent of corrosion damage of building beams in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.





Figure 5-28. Mean extent of corrosion damage of bridge decks in Kunming, Xiamen and Jinan, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.

The corrosion loss of the reinforcement diameter is quite modest due to the long time consuming process for corrosion initiation and low corrosion rates associated with carbonation. For example, the mean additional corrosion losses for building slabs, with RCP 8.5 emission scenario and exposure I by 2100 (when compared to corrosion losses for the reference scenario) are 0.06 mm, 0.12 mm and 0.07 mm for Kunming, Xiamen and Jinan, respectively. These are the highest corrosion losses for any exposure or structural member. For a 12 mm diameter reinforcing bar in a slab, they represent an additional 1.0%, 2.0% and 1.2% loss of the cross-sectional area.

Figure 5-29 shows the histograms of extent of surface corrosion damage for cast insitu sheltered RC slabs in Kunming in 2040, 2070 and 2100, under RCP 4.5 with interval of bins of 0.7%. Similar to the results of Sydney (see Figure 5-19), $f_{d_{crack}}(d_{crack},t)$ is highly non-Gaussian. The likelihood of no or little damage is high, but after 90 years (2100), the probability of no or little damage is about 22%. The histograms of the extent of surface corrosion damage for other structural members and other emission scenarios show similar trends.
Results from the spatial time-dependent reliability analysis can be shown as probability contours as shown in Figure 5-30. Each probability contour represents the probability that at least x% of the concrete surface is damaged (given by Eq. 4-40). Figure 5-30 shows the results of cast in-situ sheltered slabs in Kunming, and little difference between the results for emission scenarios RCP 8.5 and RCP 4.5 can be found. However, compared to the reference Year 2010 emission scenario, RCP 8.5 and RCP 4.5 scenarios can cause the highest possible (0.05 probability contour) extent of surface corrosion damage, being an increase of 9.7-16.3% for cast in-situ sheltered RC slabs in Kunming. Likewise, for the 50% probability contour, RCP 8.5 and RCP 4.5 can lead to a 3.6-5.2% increase in the extent of surface corrosion damage for cast in-situ sheltered RC slabs in Kunming.

The probability contours may also be used to predict the time to first repair. For example, for $X_{repair}=5\%$ there is a 50% probability that the first repair will occur between 2050 and 2091 (probability of occurrence between 0.05 and 0.55), for cast in-situ sheltered slabs in Kunming under RCP 8.5 emission scenarios (see Figure 5-30). Selecting a more stringent repair threshold ($X_{repair}=1\%$) will shorten repair actions by at least 6 years. There is a 5% chance that the first repair for cast in-situ sheltered slabs in Kunming and RCP 8.5 emission scenario can occur as soon as 2045 and 2050 for repair thresholds of 1% and 5%, respectively.



Figure 5-29. Simulation histogram of the extent of surface corrosion damage for cast in-situ sheltered RC building slabs in Kunming at 2040, 2070 and 2100, under RCP 4.5 emission scenario.



Figure 5-30. Probability contours for the extent of surface corrosion damage for cast in-situ sheltered building slabs in Kunming, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios.

5.7 Comparison between Australian and Chinese RC structures

There are various factors that can influence the corrosion damage of RC structures, including climate conditions, construction methods, durability design requirements and construction qualities, etc. Therefore, in order to compare Australian and Chinese RC structures, two cities with similar climate conditions have been chosen. As discussed in Section 5.2, the temperature and RH projections for Sydney and Kunming are alike. Hence, these two cities are representatives of each country for comparison purposes. On the other hand, as presented in Sections 5.5 and 5.6, cast insitu sheltered building slabs are assessed to have the highest corrosion damage extent, so only the results and discussion of this type will be shown later.

Carbonation depth for RC structures in Australia is generally lower than those in China, because carbonation depth for RC slabs in Sydney is about 5 mm less than those in Kunming; even though both cities have similar temperature and RH (see Figure 5-14 and Figure 5-22). These are mainly due to lower w/c ratio and higher cement content requirements in Australian standards, which lead to reduced CO_2 diffusion coefficient.

Figure 5-17 and Figure 5-26 present the mean extent of surface corrosion damage for RC slabs in Sydney and Kunming for all emission scenarios. It can be found that RC slabs in Kunming experience more severe corrosion damage than RC slabs in Sydney. For instance, the extent of surface corrosion damage for cast in-situ sheltered RC slabs in Kunming (Figure 5-26 a) is about ten times higher than those in Sydney (Figure 5-17 a). Accordingly, the mean time to first repair for cast in-situ sheltered RC slabs in Sydney is significantly later than those in Kunming.

Figure 5-19 and Figure 5-29 show the histograms of the extent of surface corrosion damage for cast in-situ sheltered RC slabs in Sydney and Kunming for RCP 4.5 in 2040, 2070 and 2100. Both Figures show highly non-Gaussian distributed $f_{d_{crack}}(d_{crack},t)$. The likelihood of no or little damage is significantly high, but it will decrease with time. After 90 years (2100), the probability of significant damage remains small.

Probability contours for the extent of surface corrosion damage for cast in-situ sheltered building slabs in Sydney and Kunming, under RCP 8.5, RCP 4.5 and Year 2010 emission scenarios are shown in Figure 5-20 and Figure 5-30, respectively. Each probability contour represents the probability that at least x% of the concrete surface is damaged. Figure 5-20 and Figure 5-30 show that RCP 8.5 and RCP 4.5 scenarios can cause the highest possible (0.05 probability contour) extent of surface corrosion damage, being an increase of 0.7-1.6% and 9.7-16.3% for cast in-situ sheltered RC slabs in Sydney and Kunming, respectively. The probability for cast in-situ sheltered RC slabs in Kunming to have a large extent of corrosion damages is significantly higher than those in Sydney.

For $X_{repair}=5\%$, there is a 50% probability that the first repair will occur between 2050 and 2091 (probability of occurrence between 0.05 and 0.55) for cast in-situ sheltered slabs in Kunming under RCP 8.5 emission scenarios (see Figure 5-30). However, the probability of no repair being needed for cast in-situ sheltered slabs in Sydney under RCP 8.5 emission scenarios can be as high as 90% (see Figure 5-20). For RC slabs in Sydney, the 5% probability of the first repair will occur at significantly longer times than Kunming. For example, there is a 5% chance that the first repair for cast in-situ sheltered slabs in Sydney under RCP 8.5 emission scenario can occur as soon as 2060 and 2085 for repair thresholds of 1% and 5%, respectively (see Figure 5-20). While for cast in-situ sheltered slabs in Kunming under RCP 8.5 emission scenario, the 5th percentile of time of first repair are 2045 and 2050 for repair thresholds of 1% and 5%, respectively (see Figure 5-30).

5.8 Accuracy of carbonation depth model

Generally, the corrosion process takes several decades, so it is impractical to verify predictions by waiting for a specimen to corrode in the natural environment. However, it may be instructive to assess carbonation depth in existing structures and compare them with predictions using the models described as Eq. 3-1. Niu (2003) has reported the carbonation depth of RC structures in China, as well as the locations, temperature, RH and age. Therefore, a comparison is made between the observations and the predictions made by the modified carbonation depth model given by Eq. 3-1. Due to the lack of historical climate data provided by Niu (2003) for each location, CO_2 concentration is considered as a time-dependent parameter by using historical

global CO₂ observations (Dlugokencky & Tans, 2013), but temperature and RH are assumed at year 2000 values. Except for the data provided by Niu (2003), the other input data are based on the standardised requirements from Table 4-9 and Table 5-3, such as water/cement ratio, diffusion coefficient, etc. The comparison is presented in Figure 5-31. The mean model error which is defined as actual (observed) value divided by predicted value is 1.08. Clearly, the predicted carbonation depth underestimates observed values. More investigations are needed, but Figure 5-31 shows that the predicted carbonation depths are within 10% of actual values measured from real structures.



Figure 5-31. Comparison between observation and predictions and the model error.

5.9 Sensitivity analysis

A sensitivity analysis is conducted in two stages to investigate the effects of random variables on the results, including (i) climate input including CO_2 concentration, temperature and RH; and (ii) structural properties comprising concrete cover, bar diameter, concrete strength, water/cement ratio and cement content. For comparative purposes, the reference parameters in the model are for a sheltered cast in-situ slab in a RC slab in Kunming, under RCP 8.5 emission scenario and exposure I. All the variables are changed 10% separately, and their effects on carbonation depth and extent of corrosion initiation and damage are calculated. Note that the CO_2 diffusion coefficient and age factor are dependent variables of the water/cement ratio.

Figure 5-32 shows the effects of climate input change on carbonation depth and the extent of corrosion initiation and damage. The influences of climate variables on carbonation depth and corrosion initiation are very similar and therefore a 10% change of in RH will have the largest influence while temperature and CO_2 concentration show almost identical impacts. It is a little tricky for the corrosion damage, and a 10% rise in temperature will result in the highest level of corrosion damage. These may due to the fact that optimum RH conditions for carbonation and corrosion processes are different since relatively dry conditions would favour carbonation while wetter environments may encourage corrosion. Therefore, RH within a certain range will cause the worse carbonation induced corrosion damage, and both dryer and wetter conditions will hinder the corrosion process. Higher temperatures can accelerate the carbonation and corrosion processes while increased CO_2 concentration only affects the carbonation process. However, this assumes the CO_2 concentration, temperature and RH are independent, when in reality they are inter-related.

Figure 5-33 shows that concrete quality parameters have different effects on carbonation depth and extents of corrosion initiation and damage. Carbonation will be influenced by w/c ratio and cement content only. Similarly, corrosion initiation is affected by these two parameters, as well as concrete cover thickness. For the corrosion process, all the concrete quality parameters will impact it. As expected, concrete cover is the most sensitive parameter for the likelihood and extent of

186

corrosion initiation and corrosion damage. Concrete strength and cement content have the lowest effects on the extent of corrosion damage. Likewise, all these concrete quality parameters may be related with each other.



Figure 5-32. Effects of climate input change on carbonation depth and extent of corrosion initiation and damage.



Figure 5-33. Effects of concrete quality change on carbonation depth and the extent of corrosion initiation and damage.

5.10 Conclusions

The results presented in this chapter have demonstrated the applications of the spatial time-dependent reliability analysis developed in this study. The sensitivity of the model to key input parameters was investigated, and the carbonation induced corrosion deterioration of RC buildings and bridges in two Australian cities and three Chinese cities under a changing climate were analysed to illustrate the application of the model. Three carbon emission scenarios are used to represent high, moderate and reference (current) emission scenarios for future climate projections. Durability design requirements of these two countries, various climate conditions, microclimate conditions and the exposure environment, as well as different construction methods, are taken into account in this analysis. The spatial random fields of concrete cover, concrete strength, CO_2 diffusion coefficient and binding capacity are covered whilst other material and corrosion parameters are treated as dependent spatial or random variables. Time-dependent CO_2 , temperature and RH effects on carbonation depth and corrosion rates are also included.

Sydney in Australia and Kunming in China have similar climate conditions, so RC structures in these two cities are used for comparison. It was discovered that a changing climate can result in the extent of damage increasing by up to 0.5% and 6% for RC infrastructure in Sydney and Kunming, respectively. The time of the first repair for cast in-situ sheltered building slabs in Kunming can occur as soon as 2045 and 2050 for repair thresholds of 1% and 5%, respectively. Both climate variables and concrete quality factors can influence the carbonation induced corrosion process significantly, and the results were most sensitive to concrete cover. RC bridges will suffer less corrosion damage than RC buildings because of their enhanced durability design requirements. Further, precast members have less likelihood and extent of deterioration than cast in-situ members, due to their better construction quality control. Therefore, the improvement of durability design specifications and construction quality can decrease the likelihood and extent of corrosion damage and therefore delay the timing of maintenance. The likelihood and extent of corrosion damage in Chinese structures are more severe than Australian structures. RC structures located in hot or temperate climate areas in China were most susceptible to climate change, and these structures may merit appropriate and cost-effective adaptation measures.

5.11 References

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Chapter 6 : Climate adaptation and maintenance strategies and cost assessments

6.1 Introduction

As discussed in Chapter 5, climate change impacts on carbonation induced corrosion damage are significant so some adaptation strategies are needed to improve the performance of RC structures under a changing climate. It is not easy to determine whether an adaptation strategy is worth taking, because there are number of issues that need addressing, including estimation of damage risks, effectiveness and expenditure of adaptation and maintenance methods, cost of damage losses, and decision-making criterion, etc. A cost-benefit analysis of climate adaptation strategies will be applied to help make decisions.

Generally speaking, if the 'benefit' of a climate adaptation strategy is higher than the 'cost' of it, and then it is worth taking. In this case, the 'benefit' of a climate adaptation strategy is the reduced damage losses which include direct losses and indirect losses; and the 'cost' is the expenditure of executing the climate adaptation strategy. To accurately assess the 'benefit' and 'cost' is the tricky part. In this Chapter, climate adaptation strategies that are available will be described and discussed; expenditure of climate adaptation strategies and cost of damages will be assessed; a repair strategy will be proposed; and then the decision making criterion will be presented. Costs of adaptation strategies and damages for RC buildings and bridges will be different. However, because the mean corrosion damage extent for RC bridges is very low compared to RC buildings (e.g. the mean d_{crack} is 1.45% and 0.47% for cast in-situ sheltered building slabs and bridge decks in Sydney under RCP 8.5, respectively), therefore, only results for RC buildings will be presented in Chapters 6 and 7.

6.2 Adaptation methods

For concrete structures under a changing climate, the climate adaptation in terms of the enhancement of adaptive capacity can be done by developing new technologies and materials to counter the impact of increasing corrosion risk. However, a wide range of methods can be used to enhance the durability of RC structures, and these can be applied to reduce the adverse effects of a changing climate. The design options generally include a selection of cover, cement and mix, surface coating barriers, extraction, and cathodic protection. In addition to reducing environmental exposure as much as possible, adaptation solutions in a new design may come from increasing cover or strength grade, or any approaches that reduce the material diffusion coefficient without compromising the reliability and serviceability of concrete. Repair measures for existing structures will be very different from adaptation strategies at the design stage. However, only adaptation strategies will be analysed herein.

6.2.1 Increase in concrete cover thickness

The increase in cover thickness can increase the time of carbonation ingress to reach concrete reinforcement and in turn delay carbonation induced corrosion. According to Peng and Stewart (2014), carbonation induced corrosion damage is most sensitive to concrete cover thickness. It is, therefore, one of the most obvious and simplest adaptation options in the design of concrete infrastructure under a changing climate to maintain structural durability and serviceability. Moreover, increased concrete cover can provide thermal insulation, which protects the reinforcement bars from fire, and gives reinforcing bars sufficient embedding to enable them to be stressed without slipping. However, increasing cover thickness may increase the cost of forms, concrete, reinforcement, finishing and labour, as well as the structure's deadweight.

As shown in Section 5.3, according to the durability design requirements, the minimum concrete cover for RC slabs of Chinese buildings is required to be 20 to 25 mm for indoor and outdoor exposure conditions (exposure class I and IIa in Table 5-3), respectively. In Australian, concrete structures in inland cities that are designed for exposure A1 and A2 (20–30 mm design cover, see Table 5-1) are more vulnerable to carbonation under the impact of changing climate. Stewart et al. (2012) estimated how the cover should be changed to meet the climate change challenge (i.e. keep the corrosion damage probability at the same level as those for no climate change scenario), and how this should be carried out in relation to exposures A1 and A2 for RC structures in Australia. They found that for the A1FI emission scenario and exposures A1 and A2 the required cover increase of nine urban centres is 5–6 mm and 4–7 mm, respectively.

6.2.2 Increasing strength grade

Except for increasing concrete cover, other options may also be included in the selection or design of concrete materials to reduce the diffusion coefficient of deleterious substances – i.e. slow the ingress of those substances, and hence delay corrosion of concrete reinforcement. One example would be the use of low water/cement ratio and the increase of cement content. Rather than giving details on the mix of cement or water/cement ratio, selection of a higher strength grade of concrete is one approach to reducing the diffusion coefficient, in addition to enhancing its mechanical properties, and therefore reduces concrete may cause difficulties in curing and introduce the potential of brittle failure which is violent and unexpected failure of RC structures. Further, increase in cement content may lead to more CO_2 emissions since producing cement is one of the biggest sources of CO_2 emission (Worrell et al., 2001).

For concrete produced with fixed cement and aggregate types and grading, strength of concrete is directly related to the water/cement ratio and cement content. Thus, concrete with lower w/c ratio and higher cement content could be considered as concrete with a better strength grade. The water/cement ratio and cement content are assumed to be the values defined in durability design specifications for each concrete strength grade correspondingly herein. For example, for Australian exposure classification A2 (see Table 5-1), the concrete strength grade is 25 MPa, and the minimum cement content and maximum water/cement ratio are 320 kg/m³ and 0.56, respectively; if 32 MPa strength concrete (as specified for B1 in Table 5-1) is applied, then the cement content and water/cement ratio will be 320 kg/m³ and 0.50, respectively. Stewart et al. (2012) reported that the use of higher strength grade of concrete will reduce the corrosion damage risk from 29.1% to 13.3% by 2100.

6.2.3 Special concrete and corrosion inhibitors

Any measure to reduce the diffusion coefficient of carbon dioxide or inhibit the corrosion rate will increase the adaptive capacity of concrete structures against a changing climate. Special concrete which use different aggregates or various cementitious materials may reduce CO_2 emissions, such as ground granulated blast furnace cement (ggbs), fly ash, and silica fume, and may also reduce the transport of

deleterious agents in concrete due to a reduced diffusion coefficient thus increasing the durability of concrete materials. However, some researchers claim that fly ash and ggbs may increase the rate of carbonation (Neville, 2008).

Using corrosion inhibitors could be one solution. Corrosion inhibitors are typically chemicals used to slow or stop corrosion in RC structures. Corrosion inhibitors can be added to the fresh concrete mix in new structures or applied to existing structures where the corrosion inhibitor must migrate through the hardened concrete. There are three main ways in which corrosion inhibitors work to resist the corrosion process (BRE, 2003): i. reducing the anodic reaction; ii. reducing the cathodic reaction or; iii. reducing both the anodic and cathodic reactions.

Papadakis (2000) tested all supplementary cementing materials (SCM; silica fume, low- and high-calcium fly ash) effects on carbonation, and found that for all SCM tested, the carbonation depth decreases when aggregate replacement by SCM increases, but increases when cement replacement by SCM increases. Otsuki et al. (2003) observed that carbonation depths of recycled aggregate concrete are slightly higher than those of normal aggregate concrete.

Limited data is available for long term performance of most types of corrosion inhibitors or special concrete. On the other hand, both commercially available corrosion inhibitors and special concrete frequently consist of a blend of different compounds and as such, the mechanism by which they work can be difficult to define. Further, there is difficulty in evaluating and quantifying their effectiveness (Bertolini et al., 2013), and more research is needed in this area.

6.2.4 Concrete surface coatings

Protective coatings can be applied to the concrete after concrete curing to increase resistance to corrosion deterioration. The surface coatings slow the ingress of corrosion initiators such as carbon dioxide and reduce the penetration of water that drives the corrosion process. Coatings are typically applied to the concrete surface using a brush or spray to a thickness of between $100 - 300 \mu m$ (El-Reedy, 2008).

Acrylic-based surface coatings can reduce carbonation depths by 10–65% (Ho & Harrison, 1990), whereas Swamy et al. (1998) reported a 60–83% reduction in carbonation depth after 2.5 years of testing. Moreno et al. (2007) found an 85% reduction in carbonation depth after 64 days for acrylic and 'good quality' vinyl-acrylic coatings, but that most vinyl-acrylic coatings had smaller reductions in carbonation depth. If the treatment is uniformly applied with the minimum number of defects then the reduction in carbonation depth can be considerable. Basheer et al. (1997) reported that a small reduction in corrosion rate may be achieved by reducing the moisture to fuel corrosion by the use of surface treatments.

However, a comprehensive model and little quantitative data is available on the effectiveness of surface coatings in reducing carbonation although some manufacturers provide estimates on the equivalent cover thickness or the possible delay (in years) of corrosion initiation that is possible through the use of their products. More importantly, surface coatings usually last for one or two decades (Almusallam et al., 2003), so there will be no significant effects on improving RC structures durability in the long term.

6.2.5 Other adaptation measures

There are other adaptation measures that may be suitable to ameliorate the effects of a changing climate. This includes stainless steel and a galvanised reinforcement, concrete-polymer composites, and cathodic protection. Their cost and effectiveness vary. For example, stainless steel bars are about six to nine times more expensive than carbon steel reinforcing bars, but a life-cycle cost analysis of deteriorating structures found that they might be cost-effective in some circumstances (Val & Stewart, 2003).

Austenitic and duplex stainless steel can offer high corrosion resistance. The benefits of using stainless steel reinforcement include: cover requirements can be reduced to 30 mm; allowable crack width to be as large as 0.3mm; very low corrosion rates (at least 1000 times lower than for carbon steel reinforcement); and, low maintenance requirements over the whole service life. Generally, the cost of austenitic stainless steel is about 8 -11 times more than those of unalloyed steels, and duplex is about 12 times as much. Typical applications of stainless steel reinforcement are structures that are exposed to very aggressive environments. Only the 'intelligent' use of stainless

steel, (i.e. a combination of stainless steel with traditional carbon steel) in locations exposed to very corrosive environments may be a cost-effective option when considering different rehabilitation methods. As presented in Bertolini's work (Bertolini & Pedeferri, 2011), for structures subjected only to carbonation, unless an extremely long service life is required (i.e. higher than 100 years), prevention of steel corrosion can be achieved more economically through a proper design of the concrete mix or the concrete cover. On the other hand, there is little quantitative information about the performance of galvanised steel in carbonated concrete, and the galvanising is not long-lasting. Sistonen et al. (2008) found that the corrosion rate of galvanised steel did not change in carbonated concrete when compared to carbon steel.

Concrete-polymer composites are materials made when part or all of the cement hydrate binders of conventional mortar or concrete is replaced with polymers; increasing the strength of the cement hydrate binder with polymers to improve concrete workability and durability (Yoshihiko, 1997). Concrete-polymer composites have many advantages, such as rapid curing, high tensile, flexural and compressive strengths, excellent adhesion to most surfaces, low permeability to water and aggressive solutions, good resistance to chemicals and corrosion, and little weight. They can be costly, however, and there are some safety issues involving the use of volatile, combustible and toxic ingredients. As reported in Yoshihiko (1997), it is found that concrete-polymer composites have a marked resistance to carbonation.

6.2.6 Summary

A range of adaptation techniques and durability improvement systems for RC structures subject to carbonation induced corrosion have been discussed. Many of these techniques have been used successfully to prolong the service life of RC infrastructure, but detailed data on their effectiveness in a range of environments is difficult to obtain. Moreover, almost every adaptation measure has other co-benefits in addition to corrosion resistance, but as well as some disadvantages. Therefore, increased concrete cover and increased concrete strength grade are used as adaptation strategies for RC structures subject to carbonation induced corrosion under a changing climate. Four adaptation strategies are defined based on current durability design requirements as following:

• Adaptation strategy A1: increase concrete cover by 5 mm;

- Adaptation strategy A2: increase concrete cover by 10 mm;
- Adaptation strategy A3: increase concrete strength by one grade (Australia: 25MPa → 32MPa; China: C20 → C25);
- Adaptation strategy A4: increase concrete strength by two grades (Australia: 25MPa → 40MPa; China: C20 → C30).

6.3 Maintenance strategies

The maintenance strategies used in the management of deteriorating RC structures can have a significant impact on lifetime performance. A maintenance strategy can be defined as an infrastructure management plan, the aim of which is to keep structures in a serviceable state, and the key aspects have been discussed in detail in Chapter 2. These include:

- Inspection interval
- Damage limit state
- Maintenance technique

These key aspects will be integrated into a spatial time-dependent reliability analysis and thus provide a quantitative insight into the cost of the deterioration of RC structures. As discussed in Section 2.8.6, the aim of this study is to study the impact of climate adaptation strategies, so the maintenance strategy will be simplified for comparison purposes only.

6.3.1 Inspection interval

Regular monitoring of RC infrastructure is a critical part of any maintenance strategy (Ahmed et al., 2007; Broomfield, 2007). The inspection interval is usually based on the controlling authority's policy or the prior experience of the asset owner/operator. Elements of a RC surface that are more structurally significant (such as bridge piers) may require more regular inspection than a non-structural facade and the inspection interval may be shortened as the asset ages and the probability of damage becomes higher.

A longer inspection interval will save on inspection costs but can mean that when detected, the damage to the structure has exceeded the pre-defined repair threshold and hence results in a more costly repair area. Further, at longer inspection intervals,

there is the risk of the corrosion damage reducing the structural safety of the RC structure, and this must be carefully considered when defining the inspection interval. The inspection interval will be treated as a time-invariant parameter of the maintenance strategy, i.e., the timing of inspections will be constant over the life of the structure. It is possible to increase the frequency of inspections as the likelihood of damage increases and thus save on inspection costs in the early stages of the service life of a RC structure.

The cost of inspections, however, is minimal when compared to the cost of construction or repair actions, and thus this will have a negligible impact on the total life-cycle cost. Further, it is general practice to conduct frequent visual observations on RC infrastructure and as such, it is considered that a regular inspection interval of between one and five years is most appropriate.

In this study, RC buildings are inspected at time intervals of Δt . For a RC building, such as a residential apartment which people can easily inspect and people use it often, any corrosion damage can be noticed quickly, therefore the routine inspection interval Δt is assumed as 1 year (Sommer et al., 1993; Vejbroer, 1980; Nowak & Absi, 1987).

6.3.2 Damage limit state

The damage limit state, X_{repair} (also referred to as the repair threshold in Section 4.5.3), is usually over a range of practical values from 0.5 % to 5 % for patch repairs and values of 12 % and 20 % for complete rehabilitative overlay (Fitch et al., 1995; Englund et al., 1999; Directoraat-Gerneraal-Rijkswaterstaat, 2000; Li, 2004; Stewart, 2006).

Patch repair is assumed to be carried out immediately if the corrosion damage extent has been discovered to exceed the repair threshold X_{repair} at the time of ith inspection at time i Δt . In other words, all observed corrosion damaged areas will be repaired immediately after inspection. In this study, the minimum repair area is 0.5 m × 0.5 m for RC slabs and beams. Therefore, X_{repair} is 0.7% and 8.3% for RC slabs and beams, respectively. Corrosion damage is defined when cover cracking is larger than the limit crack width 1.0 mm.

6.3.3 Maintenance techniques

Patch repairs are the most common type of repair for corrosion damaged RC structures (BRE, 2003; Canisius & Waleed, 2004). It is a corrective repair strategy in which repair takes place after severe concrete cracking, but the loss of the cross-sectional area of rebars is not significant. As described in Chapter 2, the patch repair process involves the mechanical removal of the damaged concrete (typically to approximately 25 mm beyond the rebar), the cleaning and treatment of the corroded steel and reinstatement of the concrete cover with a suitable material. The use of patch repairs in a maintenance strategy can be modelled very effectively using a spatial time-dependent reliability analysis as both the likelihood and extent of damage can be predicted. Therefore, the patch repair maintenance technique is defined herein, and this will be integrated into the cost-benefit analysis of an adaptation strategy.

It is assumed that (i) severe corrosion induced cracking is always detected when the structure is visually inspected; (ii) only the damaged area will be repaired; the remaining of the RC surface area will continue to deteriorate; (iii) repair will not improve durability performance of the repaired structures; i.e. cover and concrete quality are the same as the original design specification; and (iv) damage will not reoccur for the repaired area during the remaining service life of the structure. This last assumption is slightly non-conservative for estimating corrosion damage losses, however, previous chapters show less than 5% and 0.2% probability that damage may reoccur once repaired for RC structures in China and Australia, respectively.

6.4 Costs of adaptation (C_{adapt})

Adaptation strategies, such as increasing concrete cover and upgrading concrete strength, are conducted at the design stage. Therefore, additional construction costs will be generated due to adaptation strategies. Further, all these costs are country, site and structure specific. Therefore, in this section, the estimation of costs of adaptation strategies (C_{adapt}) is investigated based on the average level of local construction cost in terms of price per unit area or volume.

This section presents the costs related to the adaptation measures of RC structures subject to carbonation induced corrosion under a changing climate. Note that all the costs are expressed in terms of the 2014 value in US dollars unless noted otherwise.

6.4.1 C_{adapt} for increasing design cover (A1, A2)

Because construction prices for Australian RC structures are rarely seen, Australian prices for construction are represented by those for the U.S. since both countries are developed countries, even though Australian prices might be slightly higher than those for the U.S. The baseline case for U.S. construction cost per unit volume (C_{cv}), including forms, concrete, reinforcement, finishing and labour, is approximately \$780-1350/m³ and \$1450-1610/m³ for RC slabs (4.6-7.6 m span) and RC beams (3.0-7.6 m span), respectively (RSMeans, 2012).

Jiangsu Construction Bureau (2004) published a similar document as RSMeams which states that the construction costs per unit volume (C_{cv}) are approximately \$185/m³, \$193/m³ and \$215/m³ for 100 mm RC slabs, 300 mm RC slabs and RC beams, respectively, in Jiangsu Province, China. These values will, therefore, be used to estimate the costs of the two adaptation strategies A1 (increase cover by 5 mm) and A2 (increase cover by 10 mm).

It is assumed that an increase in design cover ΔC_{adapt} would increase the cost of forms, concrete, reinforcement, finishing and labour by an amount proportional to the additional volume of concrete needed. Since all costs units are $/m^2$ of surface area, but C_{cv} is given as per unit volume, then the cost of construction (C_C) and C_{adapt} should be converted to cost per surface area exposed to deterioration, and so is corrected for structural member dimensions such as slab depth or beam width (D). Table 6-1 describes the data and the relationships used to evaluate the adaptation costs. Table 6-1 and Table 6-2 provide the adaptation costs for various structural elements (per mm of extra cover) in Australia and China. Table 6-2 also presents the adaptation costs for a 5 and 10 mm increase in additional cover.

Table 6-1. Data and relationships for	r the assessment of ada	ptation costs of A1 and A2
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	Slabs	Beams	
D (mm)	100 to 300	200 to 800	
$C_{cv}(\$/m^3)$			
Australia	780 to 1350	1450	
China	185 to 193	215	
$C_{c} (\$/m^{2})$	$C_{cv} \times D$ (m)	$C_{cv} \times D$ (m)	
C_{adapt}^{a} (\$/m ²)	$C_c \times 1/D$ (mm)	$C_c \times 1/D$ (mm)	
^a Don many of autro action			

^aPer mm of extra cover

Structural element	<i>D</i> (mm)	C_{adapt}^{a} (/m ²)	A1: cover +5 mm $(/m^2)$	A2: $cover + 10 \text{ mm}$ (/m ²)
Australia				
Slabs – small	100	\$1.35	\$6.75	\$13.5
Slabs – large	300	\$0.78	\$3.9	\$7.8
Beams	800	\$1.45	\$7.25	\$14.5
China				
Slabs – small	100	\$0.185	\$0.925	\$1.85
Slabs – large	300	\$0.193	\$0.965	\$1.93
Beams	800	\$0.215	\$1.075	\$2.15

Table 6-2. Adaptation costs of A1 and A2 for various structural elements.

^aPer mm of extra cover

6.4.2 C_{adapt} for increasing strength grade (A3, A4)

The cost of normal weight ready mix concrete using Ordinary Portland Cement including aggregates, sand, cement and water (but excluding additives and treatments) delivered on-site increases from $137/m^3$ to $302/m^3$ for 14 MPa and 55 MPa concrete, respectively (RSMeans, 2012). Jiangsu Construction Bureau (2004) reported prices for commercial concrete C20, C25 and C30 are $55/m^3$, $58/m^3$ and $61/m^3$, respectively. C_{adapt-v} increases for higher strength grades are estimated from these data.

Similarly, for the adaptation strategies A3 and A4, all costs are converted to cost per surface area exposed to deterioration. Table 6-3 presents the additional costs of increasing strength grades $C_{adapt-v}$ (in $/m^3$) for RC structures in Australia and China. The relationship used to compute the adaptation costs for several structural elements as a function of the structural dimension D is $C_{adapt} = C_{adapt-v} \times D$. Using this relationship, Table 6-3 also shows the adaptation costs for various structural elements. Contrary to the adaptation strategies A1 and A2, adaptation costs decrease for smaller structural elements.

Table 6-3: Adaptation costs of A3 and A4 for various structural elements.

Structural elements	D (mm)		A3: strength +1 grade	A4: strength +2 grade
Australia		$C_{adapt-v}$ (/m ³)	\$8	\$35
Slabs – small	100		\$0.8	\$3.5
Slabs – large	300	C_{adapt} (/m ²)	\$2.4	\$10.5
Beams	800	1	\$6.4	\$28
China		$C_{adapt-v}$ (/m ³)	\$3	\$6
Slabs – small	100	·	\$0.3	\$0.6
Slabs – large	300	C_{adapt} (/m ²)	\$0.9	\$1.8
Beams	800	4	\$2.4	\$4.8

6.5 Costs of damage (C_{damage})

The cost of maintaining RC infrastructure in corrosive environments can be considerable. For example, Zhang and Mailvaganam (2006) report that the money paid for rehabilitating corrosion affected RC structures now takes 50% of the total construction expense. The expenditure on concrete repairs in the UK was assessed to exceed one billion pounds in 2003(BRE, 2003), whilst in Japan, the maintenance and repair budget is expected to double by 2015 (Nasu et al., 2004). The costs of severe corrosion damage of a RC structure can be expressed by the cost of repair or rehabilitation (maintenance) of corrosion induced damage during its service life. Further, the cost associated with user delay due to the structure being 'offline' should be taken into account when assessing the cost of maintenance actions and these costs are not insignificant (Li, 2004; Tominaga et al., 2004). In order to fully predict the actual cost of a structure over its service life, all expenditure associated with its operation must be included.

Cost of damage (C_{damage}) includes direct costs of repair of corrosion damage (C_{repair}) and indirect costs caused by user delays, etc. Likewise, the estimation of corrosion damages (C_{damage}) are investigated based on the average level of local construction cost in terms of price per unit area or volume.

This section presents the costs related to the maintenance of RC structures subject to carbonation induced corrosion under a changing climate. Note that all the costs are expressed in terms of the 2014 value in US dollars unless noted otherwise.

6.5.1 Repair costs (C_{repair})

A corrective repair strategy is a repair that takes place after severe concrete cracking, but the loss of cross-sectional area of bars is not significant. Patch repair is defined in Section 6.3.3. The cost of repairs for RC structures will be based on the method of repair and the extent of repaired area. The use of random fields allows the extent of corrosion damage to be predicted at a given point in time and so now we must define the expected costs of various repair techniques so that they may be included in the reliability based cost-benefit analysis presented herein. Table 6-4 shows the expected costs of patch repair. The cost data is taken from two primary sources (Yunovich et al., 2001; BRE, 2003). Mullard (2010) estimated the cost for concrete patch repair using

ordinary Portland cement in RC bridges to be $480/m^2$ in Australia. Bastidas-Arteaga and Stewart (2014) estimated costs for the repair of RC slabs and beams of the Agrifoodstuffs terminal of the Nantes Saint-Nazaire Port (Srifi, 2012). The repair method is a corrective repair strategy in which repair takes place after severe concrete cracking, but the loss of cross-sectional area of rebars is not significant. Damaged concrete cover is repaired by removing about 6 cm of material for slabs and beams. Corroded bars are not replaced. The C_{repair} of this kind of repair method in France is estimated to be $360/m^2$, and it is assumed to be the same for Australian RC buildings. According to Department of Housing and urban-rural development of Jiangsu Province (2009), the cost of repair of concrete buildings is about $96/m^2$.

Repair technique	Unit Cost (\$/m ²)	Fixed Cost	Reference
Bituminous concrete patch	168	N/A	Yunovich et al. (2001)
Portland cement patch	740	N/A	Yunovich et al. (2001)
Complete overlay using LMC	330	N/A	Yunovich et al. (2001)
Concrete patch (up to 5m ²)	132-395	10% of unit cost	BRE (2003)
Concrete patch (> $5m^2$)	52-390	10% of unit cost	BRE (2003)

Table 6-4: Expected costs of patch repair.

The total cost of a repair can be calculated based on the unit cost, the area to be repaired and any fixed costs associated with a particular repair technique. Corrosion damage is assumed to occur on one (exposed) face of the slab and beam.

The cost data presented indicates a high variability of repair costs and these values can vary with geographic regions and current economic circumstances. Further, sitespecific access costs and asset owner restrictions must be considered. For example, a repair on an internal wall would likely involve significantly more associated costs than an external wall.

6.5.2 User delay costs

Most existing studies consider direct losses related to structural damages. Indirect losses such as user delay costs, etc. can be considerable. User delay costs (often simply referred to as user costs) include all costs associated with user disruption due to the RC structure operating at a reduced level of service. Based on the type of structure and its use, these costs can be significant. As an example, for a RC concrete bridge deck that carries a high volume of traffic, the user costs associated with a

repair action can be substantially higher than the materials and labour costs for the repair (Yunovich et al., 2001). The user delay costs of highway bridges in the United States was reported by Corotis and Gransberg (2005) to be as high as \$14,300 per lane mile/ day.

Val and Stewart (2003) assumed that the cost of RC bridge deck replacement is double the construction cost based on cost data for removal and replacement costs. However, this is likely to over-estimate the repair costs for most corrosion damage of RC buildings. Bastidas-Arteaga & Stewart (2015b) estimates that the total costs is about twice as direct cost of repair for the repair of RC slabs and beams of the Agrifoodstuffs terminal in France.

User losses and other user disruption costs are, however, site and structure specific, and they depend on which structural members are to be repaired as well. For many RC structures, such costs will be minimised if the RC element to be repaired is an external or sheltered structural member such as walls, columns or facade panels. In this study, the user delay costs are assumed to be the same as the total direct costs.

6.6 Summary

Typical adaptation techniques for carbonation induced corrosion damaged RC structures have been discussed. Estimates have been provided for both the expected service life and the cost of the various adaptation techniques and repair costs for corrosion damage. These data will be used within a spatial time-dependent reliability analysis to model the performance of adaptation strategies. Note that all these costs are country, site and structure specific. A full description of the analysis conducted herein, including the integration of the adaptation strategy and cost-benefit analysis is provided in Chapter 7.

According to costs listed in Section 6.4 and 6.5, the price for a 100 mm RC slab, a 300 mm RC slab and a RC beam for adaptation strategies A1, A2, A3 and A4 and the cost of corrosion damage in Australia and China are presented in Table 6-5. All the values are in 2014 US dollars (US\$/m²). Prediction of costs in the future is beyond the scope of this research, and as such all costs are assumed unchanged.

Costs	Structural	D	A1:	A2:	A3:	A4:
	element	(mm)) cover + 5 mm	cover + 10 mm	strength + 1 grade	strength + 2 grade
Australia						
C_{adapt}	Slabs – small	100	\$6.75	\$13.5	\$0.8	\$3.5
	Slabs – large	300	\$3.9	\$7.8	\$2.4	\$10.5
(/111)	Beams	800	\$7.25	\$14.5	\$6.4	\$28
C_{damage} (/m ²)	\$720					
$C_{adapt}/ \ C_{damage}$	Slabs – small	100	0.0094	0.0188	0.0011	0.0049
	Slabs – large	300	0.0054	0.0108	0.0033	0.0146
	Beams	800	0.0101	0.0201	0.0089	0.0389
China						
C_{adapt} (/m ²)	Slabs – small	100	\$0.925	\$1.85	\$0.3	\$0.6
	Slabs – large	300	\$0.965	\$1.93	\$0.9	\$1.8
	Beams	800	\$1.075	\$2.15	\$2.4	\$4.8
C_{damage} (/m ²)	\$192					
$C_{adapt}/ \ C_{damage}$	Slabs – small	100	0.0048	0.0096	0.0016	0.0031
	Slabs – large	300	0.0050	0.0101	0.0047	0.0094
	Beams	800	0.0056	0.0112	0.0125	0.0250

Table 6-5: Costs of adaptation strategies A1 to A4 and damage for RC structural elements.

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Chapter 7 : Cost-benefit analysis of climate adaptation strategies

7.1 Introduction

As discussed in Chapters 2, 5 and 6, climate change can cause additional corrosion damage on RC structures, and these can result in significant direct and indirect losses. Therefore, adaptation strategies are needed to improve the performance of RC structures during their service span. However, adaptation strategies demand extra investment on the construction process, so a cost-benefit analysis of the economic efficiency of them is needed for policy makers to make decisions. The cost-benefit analysis includes the estimation of damage risks and consequent costs, as well as the cost of adaptation strategies.

Based on results of spatial time-dependent reliability analysis for climate change impacts on RC structures in Chapter 5, sheltered cast in-situ slabs of RC buildings in a hot or temperate climate zone may have the highest likelihood and extent of carbonation induced corrosion damage, and these structures may merit appropriate and cost-effective adaptation measures. In order to make a comparison of the economic efficiency of different adaptation strategies, it is necessary to define the cost of RC structures from construction through to the end of their functional life. These costs can be incurred at different times throughout the life of the structure, and it is common to give the total cost in terms of the present value. The cost-effectiveness of an adaptation strategy is measured in terms of Net Present Value (NPV) equal to benefits minus the cost. This study considers a spatial time-dependent reliability analysis that enables a risk-based economic assessment of climate adaptation strategies. The stochastic analysis also allows the probability that NPV exceeds zero (Pr(NPV>0)) to be calculated. In this case, while mean NPV can be high, there may be a likelihood of NPV being less than zero that also provides useful risk-averse information for decision makers. Sheltered cast in-situ slabs and beams in RC buildings in two Australian cities and three Chinese cities will be used as an application of these cost-benefit analyses. Although this study focuses only on two types of adaptation strategies and specific locations, it provides a methodology that

could be extended to study other adaptation strategies or deterioration processes for RC structures worldwide.

7.2 Life cycle cost modelling

The range of costs associated with the design, construction and usage of RC infrastructure were discussed in Chapter 2. Stewart (2006) proposed that the total life cycle cost can be described as:

$$LCC(T) = C_D + C_C + C_{QA} + C_{IN}(T) + E_{damage}(T)$$
 7-1

where C_D is the design cost, C_C is the construction cost (materials and labour), C_{QA} is the cost of quality assurance/control, $C_{IN}(T)$ is the cost of inspections and $E_{damage}(T)$ is the expected cost of repair or rehabilitation (maintenance) of corrosion-induced damage during service life T.

The expected cost of maintenance can be described as a present value by the following equation (Val & Stewart, 2003):

$$E_{damage}(T) = \sum_{i=1}^{T/\Delta t} \Delta P_{f,i} \frac{C_{damage}}{(1+r)^{i\Delta t}}$$
7-2

where T is the service life of the structure (years), Δt is the inspection interval, $\Delta P_{f,i}$ is the probability that the extent of damage exceeds the repair threshold between the (i-1)th and ith inspections, C_{damage} is the cost of damage and r is the discount rate. C_{damage} will consist of the maintenance costs and user delay costs (defined in Section 6.4.3) for the corresponding maintenance strategy. For example, an asset owner should be able to quantify the unit repair cost ($/m^2$), and if the area of damage is known, then the repair cost can be estimated. At the time of the first repair, $\Delta P_{f,i}$ can be expressed in terms of the repair threshold (X_{repair}) and the extent of damage (d_{crack}) as:

$$\Delta P_{f,i} = \Pr\left(d_{crack}\left(t\right) \ge X_{repair} \left| d_{crack}\left(t - \Delta t\right) < X_{repair}\right)\right.$$
7-3

The probabilities used in the estimation of the expected maintenance costs are calculated using Monte Carlo Simulation techniques which are described in Section 4.5.

Therefore, the life cycle costs of keeping "business as usual" and the adaptation strategy can be expressed as Eq. 7-4 and Eq. 7-5, respectively.

$$LCC_{BAU}(T) = C_D + C_C + C_{QA} + C_{IN}(T) + E_{damage-BAU}(T)$$
 7-4

$$LCC_{adaptation}(\mathbf{T}) = C_D + C_C + C_{QA} + C_{adapt} + C_{IN}(\mathbf{T}) + \mathbf{E}_{damage-adaptation}(\mathbf{T})$$
7-5

where C_{adapt} is defined as the extra costs to take adaptation measures for a unit area ($/m^2$), which can be found from Chapter 6. The adaptation strategies are taken at the design stage, so the costs are already present values.

It has been discussed that visual inspection is the primary method of inspection for RC structures (see Chapters 2 and 6). A regular visual inspection is generally part of the asset owner's or the local authority's policy for maintaining infrastructure and therefore (ignoring the use of automatic sensing equipment) there is not a large scope for variation in the inspection regime. Further, inspection costs for visual inspections are typically relatively small when compared to repair costs and as such it is assumed herein that the cost of inspection will be essentially equal for all analysed cases and can be omitted from the life-cycle cost analysis. It should also be noted that to isolate the effect of adaptation strategies; the inspection interval is assumed constant for all alternatives defined herein. Similarly, because C_{adapt} has included all extra costs related to adaptation strategies, the initial costs for design, construction and quality assurance/control can be assumed to be identical for business as usual and different adaptation strategies.

The aim of the current study is to investigate the effect of adaptation strategies on the life cycle cost of RC structures. As such, initial costs of design, construction, quality assurance and inspection costs will be assumed to be equal for all cases analysed. The economic performance is then governed primarily by the expected damage costs which are affected by adaptation strategies and the extra costs to conduct adaptation strategies. In these terms, the life-cycle cost for keeping business as usual and adaptation strategy can be simplified as:

$$LCC_{BAU}(\mathbf{T}) = \mathbf{E}_{damage-BAU}(\mathbf{T})$$
 7-6

 $LCC_{adaptation}(T) = C_{adapt} + E_{damage-adaptation}(T)$

7-7

7.3 Decision rules

To decide whether an adaptation strategy is more cost effective than keeping business as usual, a straightforward method is to compare the LCC of each adaptation strategy with the LCC of keeping business as usual, as long as LCC_{BAU} is higher than $LCC_{adaptation}$, then this adaptation strategy is cost effective. However, Net Present Value (NPV) will be used as a decision metric. Other decision metrics can be used, such as maximising benefit-cost ratio or minimising LCC. While the formulations may differ, the decision outcomes are identical, and NPV is selected because it seems that government and policy makers are familiar with this metric.

7.3.1 Net Present Value

Generally, the NPV is used to make a decision. If the NPV of benefits is positive then, the project should be adopted. However, there might be some restrictions in using net present value, i.e. the impact of budget constraints; complementarity among projects; the interaction of budget constraints and project timing choice; and comparison of projects with different lengths of life (COA, 2006). Other decision rules may apply, but should be used with caution, such as the internal rate of return, the benefit-cost ratio (BCR); and the payback period (COA, 2006). For example, the BCR is a useful measure when there are so many proposals, and there is not enough resources available to undertake them all, even if they all have high NPVs. Empirically, the projects with the highest BCRs should be chosen to ensure maximum use of money in terms of generating benefits, but it does not displace the objective of maximising net present value. However, in more general cases, the BCR is biased towards small projects and must be used cautiously. In this chapter, the net present value will be discussed in detail.

The NPV of a proposal can be calculated as:

$$NPV = \sum_{t=0}^{t} B(t) - \sum_{t=0}^{t} C(t)$$
7-8

where B(t) is the benefit at time t expressed in present value, and C(t) is the cost at time t expressed in present value. Discount rate will be used to calculate the present value. Subject to budget constraints and other considerations, and assuming that there are no alternative projects under consideration, a project may be accepted if the sum of present values of benefits exceeds the sum of present values of costs; that is, where its NPV exceeds zero. If properly calculated, the NPV is a relatively objective method of determining the benefits resulting from the project, and the NPVs of several projects can be compared. Generally, when several alternatives have positive net present values, then the one with the maximum net present value should be chosen (Lind, 1995).

A project with a higher NPV ranks ahead of the alternative, if all the proposals are the same in all other respects. Otherwise, a higher NPV is not determinative, because other qualitative aspects may be important. For example, one proposal may have greater intangible net benefits or indirect benefits (co-benefit), and then a negative NPV does not rule out the proposal.

7.3.2 Net Present Value of adaptation strategies

The 'benefit' of an adaptation measure is the reduction in damages associated with the adaptation strategy, and the 'cost' is the extra costs to take the adaptation strategy:

$$B(t) = E_{damage-BAU}(t) - E_{damage-adaptation}(t) + \Delta B$$

$$7-9$$

$$C(t) = C_{adapt}(t)$$

$$7-10$$

The parameter ΔB is introduced as the cobenefit of adaptation strategies such as reduced losses to other hazards except for carbonation induced corrosion damages, etc. Costs of adaptation, cost of maintenance, timing of repair and discount rates need to be included in any risk analysis. The co-benefit of adaptation (ΔB) may include thermal insulation provided by increased concrete cover, or reduced usage of concrete and reinforcing bars due to upgraded concrete strength, etc. For instance, flexural capacity of a RC beam will increase if concrete strength is upgraded. These indicate that concrete or reinforcement bars usage can be reduced to some extent that will also meet the capacity requirement if concrete strength is upgraded, and therefore the ratio C_{adapt}/C_{damage} will decrease accordingly. However, co-benefit of adaptation can be negative values, such as increased structure's deadweight due to concrete cover increasement, or increased CO₂ emission because of higher consumption of cement content in higher strength concrete. As such, co-benefit of adaptation can include various aspects and are difficult to quantify, so ΔB is assumed as zero. Two criteria will be used to assess the cost-effectiveness of adaptation strategies: (i) expected NPV and (ii) Pr(NPV>0). The Monte-Carlo simulation analysis is used as a computational tool to work out uncertainties through the cost-benefit analysis. All costs are discounted to present values. Clearly, an adaptation measure that results in an NPV larger than zero is a cost-effective adaptation measure. NPV is time-dependent because costs and benefits are time-dependent. Thus, an adaptation measure may not be cost-effective in the short-term due to the high cost, but be cost-effective in the longer-term for the benefits may increase over time.

The spatial time-dependent reliability analysis assumes that many input variables are random variables or spatial variables, so the output of the analysis (NPV) is also variable. This allows the each percentile of NPV to be calculated, as well as the probability that an adaptation measure is cost-effective at time T denoted herein as Pr(NPV(T)>0).

7.4 Discount rate

Choosing a discount rate is critical for the cost-benefit analysis and has crucial implications for resource allocations. Intergenerational projects such as those proposed to combat climate change and environmental degradation are typical examples of the equity issue when discounting benefits and costs. The social discount rate reflects a society's relative valuation of today's welfare versus well-being in the future. A higher discount rate implies greater risks for the benefits of the project to be reaped and makes it less likely the project will be funded. A small change of the discount rate can influence enormously the benefits far into the future, so it is critical to be as accurate as possible when choosing a discount rate to apply.

Discount rates can be quite variable due to many economic, social and political factors. Developed nations use a lower rate (3-7%) than developing nations (8-15%). For example, discount rates used by various government agencies are: Australia 7%, U.S. 3-7%, China around 8%, Norway 3.5%, Finland 6%, India 12%, Philippines 15%, UK 3.5-6%, Sweden 4%, German 3% to over 10% and European Commission 5% (Val & Stewart, 2003; Zhuang, 2007; Harrison, 2010). Nevertheless, Maddocks et al. (2011) reported that a significantly lower discount rate (compared with 7%) may be appropriate. This statement is given due to the long life of infrastructure and the
possible impact of climate change on future generations. Nonetheless, the 2006 U.K. Stern Review adopted a discount rate of 1.4% (Stern, 2006), and the Australian Garnaut Review used discount rates of 1.35% and 2.65% (Garnaut, 2008). These relatively small discount rates were selected to not underestimate climate impacts on future generations. However, others suggest higher discount rates when assessing the economic efficiency of projects with long periods, such as economic effects of climate change (e.g., (Nordhaus, 2007; Baumol, 1968)).

Discount rates are usually assumed as constant with time. However, to consider intergenerational effects of projects beyond 30-50 years, this may not be appropriate when considering climate change policy decisions (Boardman, 2006). Many researchers report that it is potentially socially efficient to discount far distant cash flows using a decreasing discount rate (e.g. Gollier, 2002; Weitzman, 1994, 1998, 2001; Henderson & Langford, 1998; Cropper & Laibson, 1998). The British Treasury recommends the following of time-declining discount rates (Treasury, 2003): 3.5% (0 to 30 years), 3.0% (31 to 75 years), 2.5% (76 to 125 years), 2.0% (126 to 200 years), 1.5% (201 to 300 years), and 1.0% (over 300 years). However, there is some controversy about time-declining discount rates (Viscusi, 2006). The Australian OPBR suggests that before the effects on future generations can be considered explicitly; an arbitrarily lower discount rate should not be used (OBPR, 2010).

The above quantification of discount rates relates mainly to the social discount rate, such as public-sector investments in infrastructure. Private investments in infrastructure, such as owners of the power station, port and airport, etc., tend to include a risk premium which leads to a higher discount rate (BTRE, 2005). According to Boardman et al. (2011), discount rates for private investment can be decided based on the marginal rate of return that is approximately 4.5%.

In the present study, discount rates of 2%, 4% and 8% are considered. These discount rates represent the range of discount rates in several countries and the lower (2%) discount rate is also used to consider intergenerational and climate change effects.

7.5 Risk reduction

The expected damage costs (economic risk) for business as usual (existing practice) and adaptation measures can then be defined as $E_{damage-BAU}(t)$ and $E_{damage-adaptation}(t)$, respectively. The risk reduction caused by an adaptation measure is thus:

$$\Delta R(t) = \frac{E_{damage-BAU}(t) - E_{damage-adaptation}(t)}{E_{damage-BAU}(t)}$$
7-11

where $E_{damage-BAU}(t)$ and $E_{damage-adaptation}(t)$ are the expected damage costs for business as usual (existing practice) and adaptation measures, respectively. If an adaptation measure is effective then, $E_{damage-adaptation}(t)$ will be significantly lower than $E_{damage-BAU}(t)$ resulting in a high risk reduction $\Delta R(t)$. In other words, $\Delta R(t)$ represents the proportional reduction in damage losses due to an adaptation measure. Eq. 7-9 can be rewritten as:

$$B(t) = E_{damage-BAU}(t) \times \Delta R(t) + \Delta B$$
7-12

where ΔB is assumed as zero.

7.6 Break-even analysis

The likelihood and extent of corrosion damage, cost of damage, and adaptation costs are subject to considerable uncertainty, and are country, site and structure specific. For this reason, calculations of corrosion risks, costs, and benefits will be imprecise. Hence, a break-even analysis may be useful to estimate the minimum risk reduction and the maximum C_{adapt}/C_{damage} required to make the benefit of adaptation strategies to equal to their cost, such that there is 50% probability that benefits equal costs—i.e., mean(NPV) >0 (Stewart & Deng, 2015). In other words, if the risk reduction is lower than break-even value or the ratio of C_{adapt}/C_{damage} exceeds the predicted break-even value, then adaptation is not cost-effective. Decision makers can then judge whether an adaptation strategy meets these break-even values. The break-even values, then, are the output of the cost-benefit analysis, and it can be useful information to the decision maker, based on expert advice about the anticipated ratio, to decide whether or not an adaptation measure is cost effective. Therefore, a breakeven cost-benefit analysis is used herein. Break-even analysis will be applied to find out the minimum risk reduction $\Delta R(t)$ and the maximum C_{adapt}/C_{damage} required to make an adaptation strategy to be cost-effective. The results of the break-even analysis will be shown in Section 7.9.

7.7 Applications: Australian RC building slabs and beams

A cost benefit analysis of adaptation strategies based on a spatial time-dependent reliability analysis can be applied to any specific cases based on their own structural information, such as dimensions, concrete quality, climate conditions, cost of damage, cost of adaptation strategy and discount rate, etc. For Australian cases, the durability design requirements for BAU and four adaptation strategies are described in Table 7-1. Statistics of structural dimensions, concrete quality, and spatial variables for Australian structures are listed in Table 4-9. Cost data for Australian cases are presented in Table 7-2.

adaptation strategies.									
	Cover (n	nm) f' _c (MPa)	w/c	$C_e (kg/m^3)$					
BAU	30	25	0.56	320					
Adaptation A1	35	25	0.56	320					
Adaptation A2	40	25	0.56	320					

32

40

0.50

0.46

320

370

Table 7-1: Durability design requirements of Australian RC buildings for BAU and fouradaptation strategies.

 Table 7-2: Costs of adaptation strategies A1 to A4 and damage for RC structural elements in Australia.

			A 1.	A 2.	A 2.	A 4.
Costs	Structural element	<i>D</i> (mm)	+5 mm	+ 10 mm	+ 1 grade	+2 grade
Australia						
	Slabs – small	100	6.75	13.5	0.8	3.5
C_{adapt} (\$/m ²)	Slabs – large	300	3.9	7.8	2.4	10.5
1	Beams	800	7.25	14.5	6.4	28
C_{damage} (\$/m ²)	720					
	Slabs – small	100	0.0094	0.0188	0.0011	0.0049
Cadapt / Cdamage	Slabs – large	300	0.0054	0.0108	0.0033	0.0146
	Beams	800	0.0101	0.0201	0.0089	0.0389

Other assumptions are summarised as follows:

Adaptation A3

Adaptation A4

• the structural lifetime is 2015-2100 and service life T=85 years,

30

30

• sheltered cast in-situ RC slabs (D=100 mm and D=300 mm), and RC beams are studied (as described in Chapter 5),

• the discount rate is 4%, and all costs are in 2014 US dollars.

RC beams and slabs in Australia use identical durability design requirements, so they are estimated to have the same likelihood and extent of corrosion damage. Figure 7-1 shows the effects of adaptation strategies A1 to A4 on the time-dependent mean extent of corrosion damage of RC buildings in Sydney and Canberra under RCP 8.5 scenarios. It can be easily found out that all these four adaptation strategies can reduce the mean extent of corrosion damage significantly. Increasing concrete cover by 5 mm and 10 mm will decrease the corrosion damage area more than increasing the concrete strength by 1 grade and 2 grades, respectively. However, whether these adaptation strategies are cost effective or not depends on the cost-benefit analysis. Because the mean extent of corrosion damage of BAU for RC buildings in Canberra is lower than those for Sydney, the reduced mean extent of corrosion damage due to adaptation strategies is lower accordingly. Figure 7-2 shows the effects of these climate change scenarios on the mean extent of corrosion damage of RC buildings in Sydney for BAU and four adaptation strategies. As expected, the mean extent of corrosion damage is larger under RCP 8.5 exposure. The differences between results of RCP 8.5 and RCP 4.5 are larger for adaptation strategies A3 and A4 (increase in concrete grades) compared to adaptation strategies A1 and A2 (increase in concrete cover), which indicate that the impacts of adaptation strategies A3 and A4 are more reliant on climate conditions.



Figure 7-1: Mean extent of surface corrosion damage of BAU and four adaptation strategies for RC buildings in Sydney and Canberra under RCP 8.5.



Figure 7-2: Mean extent of surface corrosion damage of BAU and four adaptation strategies for RC buildings in Sydney under RCP 8.5 and RCP 4.5.

Figure 7-3 shows the mean damage cost E_{damage} of BAU and four adaptation strategies for RC buildings in Sydney under RCP 8.5. Compared to Figure 7-1 (a), the expected damage costs are found to be closely related to the mean extent of corrosion damage. On the other hand, even though adaptation strategies can significantly reduce the expected damage costs by 50 - 90% compared to those for BAU, the reduced costs $0.4/m^2 - 0.7/m^2$ are still lower than adaptation costs (see Table 7-2), which indicate that these adaptation strategies are not cost effective.



Figure 7-3: Expected damage costs (\$/m²) of BAU and four adaptation strategies for RC buildings in Sydney under RCP 8.5.

The mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs (D=100 mm and D=300 mm) and RC beams in Sydney and Canberra under RCP 8.5, RCP 4.5 and Year 2015 are given in Table 7-3. The mean NPV(T) and Pr(NPV(T)>0) for Australian cities Sydney and Canberra are too low, so that it is highly unlikely for all the adaptation strategies to be cost effective. The mean NPV(T) and Pr(NPV(T)>0) for RC buildings in Sydney are slightly higher than for Canberra. These results are expected because the predictions of the likelihood and extent of corrosion damage for BAU are very low, and then the reduced damage costs due to adaptation strategies will be low accordingly. Therefore, the higher the likelihood and extent of corrosion damage, the more chances for adaptation strategies to be cost effective. In other words, it is highly likely that climate adaptation strategies will not be cost effective for those RC structural components having a small extent of corrosion damage.

			slab 100 n	nm	5	slab 300 n	nm	beam				
		RCP 8.5	RCP 4.5	Year 2015	RCP 8.5	RCP 4.5	Year 2015	RCP 8.5	RCP 4.5	Year 2015		
					mear	n NPV(T)	$(\$/m^2)$					
	A1	-6.2	-6.3	-6.4	-3.4	-3.4	-3.5	-6.7	-6.8	-6.9		
	A2	-12.8	-12.9	-13.0	-7.1	-7.2	-7.3	-13.8	-13.9	-14.0		
	A3	-0.4	-0.4	-0.5	-2.0	-2.0	-2.1	-6.0	-6.0	-6.1		
Sudmari	A4	-2.9	-2.9	-3.0	-9.9	-9.9	-10.0	-27.4	-27.4	-27.5		
Sydney					Р	r(NPV(T))>0)					
	A1	0.4%	0.3%	0.2%	2.0%	1.9%	1.3%	0.3%	0.2%	0.1%		
	A2	0.1%	0.1%	0.1%	0.9%	0.8%	0.4%	0.1%	0.1%	0.0%		
	A3	15.2%	14.4%	13.1%	3.0%	3.2%	2.9%	0.2%	0.2%	0.2%		
	A4	4.6%	4.4%	3.4%	0.3%	0.4%	0.2%	0.0%	0.0%	0.0%		
	mean NPV(T) $(\$/m^2)$											
	A1	-6.4	-6.4	-6.5	-3.6	-3.6	-3.7	-6.9	-6.9	-7.0		
	A2	-13.0	-13.1	-13.2	-7.3	-7.4	-7.5	-14.0	-14.1	-14.2		
	A3	-0.6	-0.6	-0.6	-2.2	-2.2	-2.2	-6.2	-6.2	-6.2		
Combonno	A4	-3.1	-3.1	-3.2	-10.1	-10.1	-10.2	-27.6	-27.6	-27.7		
Canberra					Р	r(NPV(T))>0)					
	A1	0.1%	0.2%	0.1%	1.2%	1.1%	0.6%	0.1%	0.1%	0.1%		
	A2	0.0%	0.1%	0.0%	0.4%	0.4%	0.2%	0.0%	0.1%	0.0%		
	A3	10.4%	10.4%	9.2%	1.7%	2.1%	1.8%	0.1%	0.1%	0.0%		
	A4	2.7%	2.7%	2.1%	0.2%	0.2%	0.1%	0.0%	0.0%	0.0%		

Table 7-3: Mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs (D=100 mm and D=300 mm) and RC beams in two Australian cities under RCP 8.5, RCP 4.5 and Year 2015.

Peng and Stewart (2015) presented cost-benefit analysis of adaptation strategies using updated cost data of Australian RC structures and other structural dimensions, as well as consideration of co-benefit of adaptation strategies.

7.8 Applications: Chinese RC building slabs and beams

RC buildings in the Chinese cities of Kunming, Xiamen and Jinan were selected to illustrate a spatial time-dependent reliability analysis based CBA of adaptation strategies. The durability design requirements of Chinese RC buildings for BAU and four adaptation strategies are listed in Table 7-4. Statistics of structural dimensions, concrete quality, and spatial variables are listed in Table 4-9. Cost data for Chinese cases are presented in Table 7-5. Note that Chinese durability design standards have different requirements for RC slabs and beams. Therefore, the likelihood and extent of corrosion damage and corresponding damage costs for RC slabs and beams in China

should be different (find out more details in Chapter 5). Other assumptions are summarised as follows:

- the structural lifetime is 2015-2100 and service life T=85 years,
- sheltered cast in-situ RC slabs (D=100 mm and D=300 mm), and RC beams are studied (as described in Chapter 5),

• the adaptation strategy consists of increasing concrete cover by 5 or 10 mm and increasing concrete strength by one or two grades with respect to standard recommendations,

• the discount rate is 4%, and all costs are expressed in 2014 US dollars.

Table 7-4: Durability design requirements of Chinese RC buildings for BAU and four adaptationstrategies.

	Cover for slabs (mm)	Cover for beams (mm)	f' _c (MPa)	w/c	$C_e (kg/m^3)$
BAU	20	25	C20	0.6	225
Adaptation A1	25	30	C20	0.6	225
Adaptation A2	30	35	C20	0.6	225
Adaptation A3	20	25	C25	0.55	250
Adaptation A4	20	25	C30	0.5	275

Table 7-5: Costs of adaptation strategies A1 to A4 and damage for RC structural elements in
China.

Costs	Structural element	D(mm)	A1:	A2:	A3:	A4:
COStS	Structural ciciliciti	D (mm)	+ 5 mm	+ 10 mm	+ 1 grade	+ 2 grade
China						
	Slabs – small	100	0.925	1.85	0.3	0.6
C_{adapt} (\$/m ²)	Slabs – large	300	0.965	1.93	0.9	1.8
I	Beams	800	1.075	2.15	2.4	4.8
C_{damage} ($/m^2$)	192					
	Slabs – small	100	0.0048	0.0096	0.0016	0.0031
Cadapt / Cdamage	Slabs – large	300	0.0050	0.0101	0.0047	0.0094
- 0	Beams	800	0.0056	0.0112	0.0125	0.0250

Figure 7-4 shows effects of adaptation strategies A1 to A4 on the mean extent of corrosion damage of RC slabs and beams in Kunming, Xiamen and Jinan under RCP 8.5 scenario. These four adaptation strategies can bring about a noticeable reduction of the mean extent of corrosion damage. For RC slabs, adaptation strategies A1 (cover +5 mm) and A2 (cover +10 mm) are more effective in decreasing the mean extent of corrosion damage than adaptation strategies A3 (strength +1 grade) and A4 (strength +2 grades), respectively. However, A2 (cover +10 mm) shows similar effectiveness as adaptation strategy A4 (strength +2 grades) on RC beams. These are because concrete



Figure 7-4: Mean extent of surface corrosion damage of BAU and four adaptation strategies for RC slabs and beams in Kunming, Xiamen and Jinan under RCP 8.5.

Figure 7-5 shows the effects of these climate change scenarios on the mean extent of corrosion damage of RC slabs and beams in Xiamen for BAU and four adaptation strategies. Similar to Australian cases, the mean extent of corrosion damage is larger under RCP 8.5 exposure. However, the differences between results of RCP 8.5 and RCP 4.5 are about the same level for BAU and four adaptation strategies.



Figure 7-5: Mean extent of surface corrosion damage of BAU and four adaptation strategies for RC slabs and beams in Xiamen under RCP 8.5 and RCP 4.5.

Figure 7-6 shows the expected damage costs of BAU and four adaptation strategies for RC slabs and beams in Xiamen under the RCP 8.5 scenario. The mean cumulative damage cost for RC slabs in Xiamen of BAU under RCP 8.5 is $6.9/m^2$ over the period 2015-2100. The BAU corrosion cost for all of China can be estimated as: population × per capita living space (m²) × ratio of population in temperate and cold climate zone × mean corrosion damage cost per m². China is estimated to have a population of 1.36 billion, and half of the population are living in urban areas. Further, Chinese per capita living space is 37.1 m² for urban areas. Assume that people in urban area all live in RC structures. China is divided into three climatic zones, i.e. severe cold zone, cold zone and temperate zone (see (Peng & Stewart, 2014a; GB50010-2010, 2010)). Severe cold zone is not considered because RC structures in this area are usually subject to chloride induced corrosion for de-icing salt. About 80% of Chinese population are distributed in temperate and cold climate zones. Another assumption is that a quarter of RC structures in temperate and cold zones deteriorate at the same rate as RC structures in Xiamen and Jinan, respectively; the remaining 50% are similar to RC structures in Kunming. Moreover, the mean corrosion damage of a whole structure is assumed to be one third of cast in-situ sheltered slabs (more detailed results for precast and unsheltered structural members are presented in (Peng & Stewart, 2014a, b). Therefore, the BAU damage cost in China is estimated to be \$21.2 billion under the RCP 8.5 scenario (1.36 billion \times 50% \times 80% \times 37.1m² \times ($6.9/m^2$ (Xiamen) × 25%+ $0.9/m^2$ (Jinan) × 25%+ $2.4/m^2$ (Kunming) × 50%)×1/3). For a reference scenario Year 2015 (assume no climate change), the damage cost is \$14.8 billion. So climate change is likely to cause up to an additional \$6.4 billion for maintenance of carbonation-induced corrosion damage of RC buildings in China. This estimation has not included factories, public infrastructure, warehouses and business buildings, etc. So the additional costs could be even higher. A more accurate assessment could be made if precise data were available. As such, some climate adaptation strategies may be necessary for RC buildings in China if they are proven to be cost effective.

Figure 7-7 shows the simulation histograms of NPV of the four adaptation strategies for RC slabs (D=100 mm) in Xiamen under the RCP 8.5 emission scenario with intervals of bins of 0.5 m^2 . It is observed in Figure 7-7 that the NPV is more likely to be higher for A2 (cover +10 mm) and A3 (strength +2 grades) compared with A1 (cover +5 mm) and A3 (strength +1 grade), respectively. The histogram of NPV for each adaptation strategy is significantly different. However, what they have in common is that they are highly non-Gaussian distribution and the probability decreases for higher NPV. These indicate that the highest mean NPV may not be sufficient for an adaptation strategy to be optimal, a high Pr(NPV>0) is also very important. Therefore, both the mean NPV and Pr(NPV>0) will be discussed in the following sections.



Figure 7-6: Mean E_{damage} (\$/m²) of BAU and four adaptation strategies for RC slabs and beams in Xiamen under RCP 8.5.





Figure 7-7: Simulation histogram of NPV of the four adaptation strategies for RC slabs (D=100 mm) in Xiamen under the RCP 8.5 emission scenario.

Figure 7-8 shows time-dependent mean NPV and Pr(NPV>0) of four adaptation strategies for RC building slabs (D =100 mm and D =300 mm) and beams in Xiamen under the RCP 8.5. It is noted that the mean NPV will increase with time, which indicates that the adaptation strategies are likely to be cost effective in the long term due to their high cost in the short term. However, in this case, the mean NPVs of RC beams for A3 (strength +1 grade) and A4 (strength +2 grades) at 2100 are still lower than zero, indicating that the adaptation strategies are not cost-effective. The timedependent mean NPV and Pr(NPV>0) of each adaptation strategy for various structural components show very different trends. For example, for mean NPV of RC slabs (D = 100 mm, see Figure 7-8 a), the NPV of A4 is the highest, follow by A1, A2 and A3 at 2065, but the sequence will change to A4>A2>A1>A3 at 2100. Of interest is that the Pr(NPV>0) shows another story (see Figure 7-8 b). The probability of adaptation strategy A4 (strength +2 grades) to be cost effective is still the highest, but the sequence becomes A4>A3>A1>A2 and A4>A1>A3>A2 at 2065 and 2100, respectively. However, for both criteria, adaptation strategy A4 (strength + 2 grades) is always the best alternative. But for RC slabs (D = 300 mm, see Figure 7-8 c and d), adaptation strategy A2 (cover +10 mm) has the highest mean NPV, but adaptation strategy A1 (cover +5 mm) has the highest Pr(NPV>0) at 2100. In practice, these results indicate that adaptation strategy A2 (cover +10 mm) is likely to be more cost effective but it also has the higher risk of losing money, while adaptation strategy A1 (cover +5 mm) may be less cost effective, but there is with more confidence that it will not lose money. The differences between the results of mean NPV and Pr(NPV>0) are due to the high variability of damage risks caused by uncertainties of climate projections, and variability of material, dimensional and deterioration parameters. The variability of NPV can be expressed as a histogram plot, and it will be described in detail in Section 7.9. These results of mean NPV and Pr(NPV>0) could be used by an owner/stakeholder to evaluate the cost effectiveness and the risks of implementing adaptation strategies at various years.



Figure 7-8: Time-dependent mean NPV and Pr(NPV>0) of four adaptation strategies for RC building slabs (D=100 mm and D=300 mm) and beams in Xiamen under the RCP 8.5.

The mean NPV and the Pr(NPV>0) of adaptation strategies A1 to A4 for RC slabs (D=100 mm and D=300 mm) and RC beams in Kunming, Xiamen and Jinan under RCP 8.5, RCP 4.5 and Year 2015 scenarios at the end of service life T are given in Table 7-6. The overall behaviour indicates that the mean of NPV is highly dependent on the location. The mean NPV is lower for RC building slabs and beams in Jinan, and none of the NPVs are positive indicating that the adaptation strategies are not cost-effective. This means that, for the studied material, the current design cover (20 mm for slabs, 25 mm for beams) or strength (C20) is cost-efficient for Jinan compared to four adaptation strategies. On the opposite side, adaptation strategies are

cost-effective for RC slabs in Xiamen. Thus, recommendations for current standards and adaptation measures could be more or less adapted to local climate conditions.

On the other hand, the mean NPV reduces for lower emission scenarios, for example, the mean NPV is lower for Year 2015 compared to those under RCP 8.5. These are because the 'benefits' for lower emission scenarios are also lower while the 'cost' of adaptations are the same. Therefore, adaptation strategies are more likely to be costeffective if more severe climate conditions are expected. Larger NPV values are related to an increase of climate change effects on deterioration rates that justify the implementation of adaptation measures. For RC slabs in Kunming and RC beams in Xiamen, the decision of whether an adaptation strategy should be taken or not relies on the accuracy of projections of future climate conditions, because for some cases, mean NPV is positive for a high emission scenario but not for a low emission one. However, for RC slabs in Xiamen, RC slabs and beams in Jinan, the decision is relatively easy to make, for the mean NPV is either positive or negative for all emission scenarios. Generally, when several alternatives have positive net present values, then the one with the maximum net present value should be chosen, further, Pr(NPV>0) based on reliability analysis could provide more evidence in the decision making process.

As discussed in Section 7.7, the higher the likelihood and extent of corrosion damage the more chances for adaptation strategies to be cost effective. Applications of Chinese cases showed the same conclusion. As such, structural components that predicted large corrosion damage areas are merit appropriate adaptation strategies. Thus, it is possible to conclude that the mean NPV or Pr(NPV>0) for the whole structure could be maximised by performing different actions for individual components: (i) optimising the extra cover, (ii) optimising the concrete strength, (iii) considering different types of adaptation strategies, and/or (iv) doing nothing.

		5	slab 100 r	nm	S	lab 300 n	nm		beam			
		RCP 8.5	RCP 4.5	Year 2015	RCP 8.5	RCP 4.5	Year 2015	RCP 8.5	RCP 4.5	Year 2015		
					mear	n NPV(T)	$(\$/m^2)$					
	A1	0.4	0.3	-0.1	0.3	0.2	-0.1	-0.3	-0.4	-0.7		
	A2	0.0	-0.2	-0.7	-0.1	-0.3	-0.7	-1.1	-1.2	-1.5		
	A3	0.2	0.2	0.1	-0.4	-0.4	-0.5	-2.0	-2.0	-2.1		
V.	A4	1.1	1.0	0.6	-0.1	-0.2	-0.6	-3.7	-3.8	-4.1		
Kunming	5				Р	r(NPV(T))>0)					
	A1	42.4%	40.8%	30.6%	41.4%	39.9%	29.7%	24.8%	22.1%	15.1%		
	A2	33.5%	31.6%	22.0%	32.2%	30.3%	21.0%	18.5%	16.5%	10.6%		
	A3	42.3%	44.1%	35.3%	20.0%	20.0%	14.7%	3.4%	3.1%	1.9%		
	A4	55.0%	56.3%	45.6%	31.6%	31.0%	22.6%	7.0%	6.1%	3.7%		
					mear	n NPV(T)	(\$/m ²)					
	A1	2.6	2.4	1.7	2.6	2.4	1.7	1.0	0.8	0.2		
	A2	3.2	3.0	1.9	3.2	2.9	1.8	0.9	0.6	-0.2		
	A3	1.1	1.1	1.0	0.5	0.5	0.4	-1.3	-1.3	-1.6		
Xiamen	A4	4.0	3.8	3.1	2.8	2.6	1.9	-1.7	-1.9	-2.7		
Alamen		Pr(NPV(T)>0)										
	A1	78.2%	77.6%	70.3%	77.6%	76.9%	69.5%	54.1%	50.0%	38.6%		
	A2	70.7%	70.1%	61.5%	69.8%	69.1%	60.1%	45.9%	42.0%	31.1%		
	A3	75.7%	75.4%	72.3%	52.6%	53.2%	48.4%	15.9%	14.8%	10.5%		
	A4	85.6%	85.1%	80.9%	67.5%	67.1%	60.8%	24.1%	22.6%	15.8%		
					mear	n NPV(T)	$(%/m^2)$					
	A1	-0.5	-0.6	-0.6	-0.5	-0.6	-0.7	-0.8	-0.8	-0.9		
	A2	-1.2	-1.3	-1.4	-1.3	-1.4	-1.5	-1.7	-1.8	-1.9		
	A3	-0.2	-0.2	-0.2	-0.8	-0.8	-0.8	-2.3	-2.3	-2.3		
Iinan	A4	-0.1	-0.1	-0.2	-1.3	-1.3	-1.4	-4.4	-4.4	-4.5		
Jinan					Р	r(NPV(T))>0)					
	A1	15.7%	12.8%	10.7%	15.2%	12.4%	10.4%	10.3%	8.2%	5.7%		
	A2	11.2%	8.9%	7.4%	10.5%	8.5%	7.0%	6.9%	5.3%	3.5%		
	A3	12.2%	11.9%	11.8%	4.1%	3.6%	3.4%	0.6%	0.5%	0.4%		
	A4	22.7%	20.9%	18.8%	9.2%	8.0%	7.3%	1.7%	1.4%	1.2%		

Table 7-6: Mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs (D=100 mm and D=300 mm) and RC beams in three Chinese cities under RCP 8.5, RCP 4.5 and Year 2015.

7.9 Break-even analysis of ΔR and C_{adapt}/C_{damage} ratio

Assuming the $E_{damage - BAU}$ of a RC building component is known, and then the minimum risk reduction ΔR required to make an adaptation strategy to be cost-effective can be solved by the following:

$$NPV = B(t) - C(t) = E_{damage-BAU}(t) \times \Delta R(t) - C_{adapt} \ge 0$$

$$\Delta R(t) \ge \frac{C_{adapt}}{E_{damage-BAU}(t)}$$

7-13

Eq. 7-13 is derived from Eq. 7-8 to 7-12.

As discussed in Section 7.8, it is not necessary for the adaptation strategy with a high mean NPV to have a high probability of being cost effective, and this is mainly due to high uncertainties of climate projections and variability of material, dimensional and deterioration parameters. Monte Carlo simulations were used for estimating the risk reduction due to the implementation of adaptation measures, ΔR . Figure 7-9 shows the effects of the four adaptation strategies on the histogram of ΔR for RC slabs in Xiamen under the RCP 8.5 scenario with interval of bins of 1.4%. It is obvious that the histogram of ΔR of each adaptation strategy is very different. The most likely risk reduction corresponds to the case where there is no risk reduction for the adaptation strategies ($\Delta R = 0\%$) or complete risk reduction ($\Delta R = 100\%$). A zero or one hundred percent risk reduction arises because: (i) there is no repair during the structural lifetime for BAU or adaptation maintenance strategies, or (ii) the repair schedule is the same for BAU and adaptation maintenance strategies. These results are expected because the probability of no corrosion damage during structure's service life is very high (see Figure 5-29). It is observed in Figure 7-9 that the risk reduction is more likely to be higher for A2 (cover +10 mm) and A4 (strength +2 grades) compared with A1 (cover +5 mm) and A3 (strength +1 grade), respectively.





Figure 7-9: Effects of the four adaptation strategies on the histogram of ΔR for RC slabs (D=100 mm) in Xiamen under the RCP 8.5 scenario.

On the other hand, E_{damage - BAU} is a variable as well, and C_{adapt} changes for different adaptation strategies and structural components. Table 7-7 is generated to show the ΔR that is required to make Pr(NPV>0) > X% for RC slabs and beams in Xiamen under RCP 8.5, RCP 4.5 and Year 2015 scenarios. All the cost data needed to generate Table 7-7 are presented in Table 6-5, and discount rate is 4%. It can be observed that the more confident that the adaptation strategy is of being cost effective, the higher ΔR value is required. Further, the more severe the climate projection, the lower ΔR value is needed. In other words, the worse the climate becomes, the more likely that the adaptation strategy will be cost effective. In some cases, ΔR needs to be higher than the one needed to make the adaptation strategy be cost-effective. In practice, this is impossible to happen. Therefore, the adaptation strategy is not suggested to be taken if that high confidence is needed. However, Table 7-7 can be used to estimate the Pr(NPV>0) if ΔR of an adaptation strategy for a certain RC structural components is known. For example, if ΔR of adaptation strategy A1 (cover + 5 mm) for a RC slab (D = 100 mm) in Xiamen is 20%, then the decision makers can assess that the probability of implementing the adaptation strategy is cost effective will be higher than 50% if future climate is tracking RCP 8.5 and RCP 4.5 emission scenarios. If assuming there is no climate change (Year 2015 scenario), then Pr(NPV>0) = 40% for $\Delta R = 20\%$.

Tables like Table 7-7 can be generated for other locations, structural type, structural dimensions, durability design requirements, microclimate conditions, adaptation strategies, climate projections, discount rate and cost data, etc., based on spatial time-dependent reliability analysis of structures deterioration. Decision makers can make their own tables to figure out quickly the Pr(NPV>0) of alternatives by applying the data that suits their criteria.

slab 100	X (%)	95	90	80	70	60	50	40	30	20	10	5
	A1	>100%	>100%	66.2%	38.0%	25.8%	18.2%	13.5%	10.4%	7.9%	5.8%	4.6%
	A2	>100%	>100%	>100%	76.0%	51.6%	36.4%	27.1%	20.9%	15.9%	11.7%	9.3%
KCF 0.3	A3	>100%	67.9%	22.1%	12.7%	8.6%	6.1%	4.5%	3.5%	2.6%	1.9%	1.5%
	A4	>100%	>100%	42.7%	24.6%	16.7%	11.7%	8.7%	6.7%	5.1%	3.8%	3.0%
	A1	>100%	>100%	70.8%	39.9%	26.8%	19.4%	14.4%	11.2%	8.6%	6.2%	4.9%
	A2	>100%	>100%	>100%	79.8%	53.6%	38.7%	28.9%	22.3%	17.2%	12.4%	9.8%
KCF 4.3	A3	>100%	73.2%	23.6%	13.3%	8.9%	6.5%	4.8%	3.7%	2.9%	2.1%	1.6%
	A4	>100%	>100%	45.7%	25.8%	17.3%	12.5%	9.3%	7.2%	5.6%	4.0%	3.2%
	A1	>100%	>100%	>100%	58.1%	37.8%	27.0%	20.0%	15.0%	11.0%	7.8%	6.1%
Voor 2015	A2	>100%	>100%	>100%	>100%	75.6%	54.1%	39.9%	29.9%	22.1%	15.6%	12.1%
1 eai 2013	A3	>100%	>100%	37.9%	19.4%	12.6%	9.0%	6.7%	5.0%	3.7%	2.6%	2.0%
	A4	>100%	>100%	73.4%	37.5%	24.4%	17.5%	12.9%	9.7%	7.1%	5.0%	3.9%
slab 300	X (%)	95	90	80	70	60	50	40	30	20	10	5
	A1	>100%	>100%	68.9%	39.6%	26.9%	18.9%	14.1%	10.9%	8.3%	6.1%	4.8%
PCP 8 5	A2	>100%	>100%	>100%	80.0%	54.3%	38.3%	28.5%	22.0%	16.7%	12.3%	9.8%
KCI 0.5	A3	>100%	>100%	64.8%	37.2%	25.3%	17.8%	13.3%	10.2%	7.8%	5.7%	4.5%
	A4	>100%	>100%	>100%	74.4%	50.5%	35.6%	26.5%	20.5%	15.5%	11.4%	9.1%
	A1	>100%	>100%	73.8%	41.6%	27.9%	20.2%	15.0%	11.6%	9.0%	6.5%	5.1%
PCP 4 5	A2	>100%	>100%	>100%	83.9%	56.4%	40.7%	30.4%	23.5%	18.1%	13.1%	10.3%
KCI 4.5	A3	>100%	>100%	69.4%	39.1%	26.2%	19.0%	14.1%	10.9%	8.4%	6.1%	4.8%
	A4	>100%	>100%	>100%	78.1%	52.5%	37.9%	28.3%	21.9%	16.8%	12.2%	9.6%
	A1	>100%	>100%	>100%	60.5%	39.4%	28.2%	20.8%	15.6%	11.5%	8.1%	6.3%
Voor 2015	A2	>100%	>100%	>100%	>100%	79.5%	56.9%	42.0%	31.5%	23.2%	16.4%	12.8%
10ai 2013	A3	>100%	>100%	>100%	56.8%	37.0%	26.5%	19.6%	14.7%	10.8%	7.6%	5.9%
	A4	>100%	>100%	>100%	>100%	74.0%	52.9%	39.1%	29.3%	21.6%	15.3%	11.9%
beam	X (%)	95	90	80	70	60	50	40	30	20	10	5
	A1	>100%	>100%	77.2%	44.3%	30.1%	21.2%	15.8%	12.2%	9.3%	6.8%	5.4%
RCP 8 5	A2	>100%	>100%	>100%	88.7%	60.2%	42.4%	31.6%	24.4%	18.5%	13.6%	10.8%
Ref 0.5	A3	>100%	>100%	>100%	99.0%	67.2%	47.4%	35.3%	27.2%	20.7%	15.2%	12.1%
	A4	>100%	>100%	>100%	>100%	>100%	94.7%	70.6%	54.4%	41.3%	30.4%	24.2%
	A1	>100%	>100%	82.6%	46.5%	31.3%	22.6%	16.8%	13.0%	10.0%	7.3%	5.7%
RCP 4 5	A2	>100%	>100%	>100%	93.1%	62.5%	45.2%	33.7%	26.0%	20.1%	14.5%	11.4%
KCI 4.5	A3	>100%	>100%	>100%	>100%	69.8%	50.4%	37.6%	29.1%	22.4%	16.2%	12.7%
	A4	>100%	>100%	>100%	>100%	>100%	>100%	75.2%	58.1%	44.8%	32.4%	25.5%
	A1	>100%	>100%	>100%	67.7%	44.1%	31.5%	23.3%	17.5%	12.9%	9.1%	7.1%
Vear 2015	A2	>100%	>100%	>100%	>100%	88.1%	63.1%	46.6%	34.9%	25.8%	18.2%	14.1%
Year 2015	A3	>100%	>100%	>100%	>100%	98.4%	70.4%	52.0%	39.0%	28.8%	20.3%	15.8%
	A4	>100%	>100%	>100%	>100%	>100%	>100%	>100%	77.9%	57.5%	40.6%	31.6%

Table 7-7: ΔR that required to make Pr(NPV>0) larger than X% for RC slabs in Xiamen under RCP 8.5, RCP 4.5 and Year 2015 scenarios.

Nevertheless, as discussed in Section 6.4, all cost data for damage and adaptation strategies are country, site and structure specific. If cost data changes from one structure to another, instead of working out tables like Table 7-7 every time costs change, another approach can be used. To take account of various adaptation costs, the ratio C_{adapt}/C_{damage} can be used as an indicator. Table 7-8 presents the probability that NPV(T)>0 of adaptation strategies A1 to A4 when C_{adapt}/C_{damage} equals to some certain values for a RC building slabs and beams in Xiamen under three emission scenarios. The discount rate is 4%. Table 7-8 is based on spatial time-dependent reliability analysis of structures deterioration. Hence RC slabs and beams in China should generate different results. It is obvious that, the lower the ratio C_{adapt}/C_{damage} is, the more likely for the adaptation strategy to be cost-effective. For a specific project, decision makers can easily make a rough estimation of Pr(NPV(T)>0) for a proposed C_{adapt}/C_{damage} based on this kind of tables. For example, the ratios C_{adapt}/C_{damage} of adaptation strategies A1 to A4 for RC slabs (D = 100 mm) are 0.0048, 0.0096, 0.0016 and 0.0031, respectively. Then decision makers can estimate the Pr(NPV(T)>0) for RCP 8.5 scenario according to corresponding area in Table 7-8 are around 78%, 70%, 75% and 85% for adaptation strategies A1 to A4, respectively. To compare the exact value, see Table 7-9.

		RCP 8.5				RCP 4	.5			Year 2015			
	C_{adapt}/C_{damage}	A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4
	0.0005	93.9%	94.3%	87.4%	94.1%	94.0%	94.4%	87.6%	94.3%	91.6%	92.2%	85.3%	92.2%
	0.001	91.7%	92.5%	81.8%	92.1%	91.8%	92.7%	81.4%	92.4%	88.6%	90.0%	78.4%	89.9%
	0.002	88.2%	89.8%	72.0%	89.1%	87.7%	89.5%	72.1%	88.8%	83.2%	85.8%	68.4%	85.7%
alah	0.005	77.6%	81.9%	51.0%	79.5%	76.9%	81.5%	51.3%	79.3%	69.5%	75.0%	46.4%	74.3%
siao	0.01	62.7%	70.0%	28.5%	66.1%	61.3%	69.3%	28.2%	65.8%	51.2%	60.4%	23.5%	59.0%
	0.02	38.8%	51.1%	6.9%	45.9%	36.3%	49.1%	6.9%	44.9%	25.6%	37.8%	5.6%	36.4%
	0.05	4.6%	15.7%	0.0%	12.8%	3.8%	13.5%	0.0%	11.7%	1.9%	7.8%	0.0%	7.8%
	0.1	0.0%	0.7%	0.0%	0.6%	0.0%	0.7%	0.0%	0.6%	0.0%	0.3%	0.0%	0.4%
	0.0005	67.4%	67.5%	66.0%	67.6%	63.3%	63.5%	62.2%	63.6%	52.8%	53.1%	52.4%	53.4%
	0.001	67.3%	67.5%	63.1%	67.6%	63.2%	63.5%	59.9%	63.5%	52.7%	53.1%	50.4%	53.3%
	0.002	66.7%	67.5%	58.2%	67.3%	62.7%	63.4%	55.1%	63.3%	52.3%	53.0%	46.7%	53.2%
haam	0.005	55.3%	58.6%	40.8%	58.2%	51.4%	54.6%	38.8%	54.4%	40.4%	43.5%	31.2%	43.7%
Dealli	0.01	40.6%	48.0%	21.9%	47.6%	36.9%	44.4%	20.8%	44.2%	26.6%	33.3%	15.1%	33.9%
	0.02	19.3%	30.1%	5.4%	30.2%	16.7%	27.8%	5.1%	28.3%	10.0%	18.2%	3.4%	20.2%
	0.05	1.0%	7.0%	0.0%	8.0%	0.9%	5.3%	0.0%	6.7%	0.4%	3.0%	0.0%	4.5%
	0.1	0.0%	0.2%	0.0%	0.5%	0.0%	0.2%	0.0%	0.5%	0.0%	0.1%	0.0%	0.4%

Table 7-8: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various C_{adapt}/C_{damage} for RC building slabs and beams in Xiamen under RCP 8.5, RCP 4.5 and Year 2015 scenarios.

RCP 8.5						RCP 4.5				Year 2015			
	A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4	
slab 100	78.2%	70.7%	75.7%	85.6%	77.6%	70.1%	75.4%	85.1%	70.3%	61.5%	72.3%	80.9%	
slab 300	77.6%	69.8%	52.6%	67.5%	76.9%	69.1%	53.2%	67.1%	69.5%	60.1%	48.4%	60.8%	
beam	54.1%	45.9%	15.9%	24.1%	50.0%	42.0%	14.8%	22.6%	38.6%	31.1%	10.5%	15.8%	

Table 7-9: Pr(NPV(T)>0) of adaptation strategies A1 to A4 using C_{adapt}/C_{damage} from Table 6-5 for RC building slabs and beams in Xiamen under RCP 8.5, RCP 4.5 and Year 2015 scenarios.

The likelihood and extent of corrosion damage, cost of damage, and adaptation costs are subject to considerable uncertainty and are country, site and structure specific. For this reason, calculations of corrosion risks, costs, and benefits will be imprecise. Hence, a break-even analysis may be useful to estimate the maximum adaptation costs required to make the benefit of an adaptation strategy equal to its cost, such that there is 50% probability that benefits equal costs—i.e., Pr(NPV>0)=50% (Stewart & Deng, 2015). In other words, if adaptation costs exceed the predicted break-even value, then adaptation strategy meets these break-even values. The break-even values, then, are the output of the cost-benefit analysis, and it can be useful information to the decision maker, based on expert advice about the anticipated costs, to decide whether or not an adaptation measure is cost effective. Therefore, break-even analysis will be applied to find out the maximum adaptation costs are normalised by the ratio C_{adapt}/C_{damage} to reflect uncertainty in C_{adapt} and C_{damage} .

Table 7-10 shows the maximum C_{adapt}/C_{damage} ratio of four adaptation strategies to ensure that Pr(NPV(T)>0)>50% for RC building slabs and beams in Kunming, Xiamen and Jinan under all emission scenarios. It can be found that break-even values for RC slabs in Jinan and RC beams in Kunming and Jinan are zero which indicates that adaptation strategies will not be cost effective for those structures no matter how low the costs of adaptation strategies are. Therefore, for those structures with breakeven values equal to zero, the BAU is the most cost effective alternative even under climate change. On the other hand, if break-even values are not zero, adaptation strategies might be cost effective. For example, break-even values of four adaptation strategies for RC slabs in Xiamen under all emission scenarios are larger than the ratio C_{adapt}/C_{damage} listed in Table 7-5, except for adaptation strategy A3 under the Year 2015 emission scenario with a break-even value equals to 0.45 which is less than the ratio C_{adapt}/C_{damage} equals to 0.47 for a deep slab (D=300). It suggests that if cost of adaptation strategy A3 can be reduced by 5% and other costs kept the same as Table 7-5 shows, then any of the four adaption strategies are going to be cost effective for RC slabs in Xiamen even if no climate change is expected. Results in Table 7-6 indicate similar conclusions.

Table 7-7, Table 7-8 and Table 7-10 can be generated for other locations, structural type, structural dimensions, durability design requirements, microclimate conditions, adaptation strategies, climate projections, discount rate and cost data, etc., based on spatial time-dependent reliability analysis of structures deterioration. Tables of the probability that NPV(T)>0 of adaptation strategies A1 to A4 when C_{adapt}/C_{damage} equals to some certain values for a RC building slabs and beams in all five cities under three emission scenarios are presented in Appendix A. Table 7-10 can be generated for other criteria other than 50% and is a straight forward measure for decision makers to quickly determine if an adaptation strategy is cost effective or not, based on their own estimates of C_{adapt} and C_{damage} .

			Slab			Beam	
		RCP 8.5	RCP 4.5	Year 2015	RCP 8.5	RCP 4.5	Year 2015
	A1	0.0033	0.0032	0.0017	0.0000	0.0000	0.0000
Kunming	A2	0.0044	0.0043	0.0022	0.0000	0.0000	0.0000
(temperate &dry)	A3	0.0010	0.0012	0.0007	0.0000	0.0000	0.0000
	A4	0.0041	0.0042	0.0024	0.0000	0.0000	0.0000
	A1	0.0148	0.0141	0.0103	0.0070	0.0056	0.0031
Xiamen	A2	0.0206	0.0195	0.0140	0.0090	0.0076	0.0033
(hot & humid)	A3	0.0052	0.0052	0.0045	0.0033	0.0031	0.0011
	A4	0.0178	0.0172	0.0133	0.0088	0.0075	0.0033
	A1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Jinan	A2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
(cold & dry)	A3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	A4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Table 7-10: The Maximum C_{adapt}/C_{damage} ratio of four adaptation strategies to make Pr(NPV(T)>0)>50% for RC slabs and beams in Xiamen under all emission scenarios.

7.10 Sensitivity analysis of the discount rate

Figure 7-10 describes the influence of discount rates (r) on both the mean NPV and the Pr(NPV>0) of adaptation strategy A1 (cover + 5 mm) for RC slabs (D = 100 mm) in Xiamen under RCP 8.5. It is observed that the mean NPV and the Pr(NPV>0) are very sensitive to the discount rate and both the mean NPV and the Pr(NPV>0) are larger for small discount rates. This can be explained by the fact that repairs usually happen at the end of the structural lifetime; on the other hand, future costs can be converted to higher present value due to low discount rates. As discussed in Section 7.4, various governments recommend lower discount rates of about 2% for long-term investments. The results, therefore, show that the adaptation strategies are more cost-effective according to these recommendations.



Figure 7-10: Influence of discount rates (r) on both the mean NPV and the Pr(NPV>0) of adaptation strategy A1 (cover + 5 mm) for RC slabs (D = 100 mm) in Xiamen under RCP 8.5.

Table 7-11 shows the impacts of the discount rate on the mean NPV and the Pr(NPV>0) of adaptation strategies A1 to A4 at the end of service life T for RC slabs (D=100 mm and D=300 mm) and RC beams in Xiamen under RCP 8.5, RCP 4.5 and Year 2015. Because the discount rate will influence the present value of future costs significantly, while in these cases, only maintenance will happen in the future, and the maintenance strategy for each alternatives are assumed the same, so the discount rate will not change the sequence of the mean NPV and the Pr(NPV>0) of adaptation strategies. However, if various maintenance strategies are considered including using different inspection intervals, repair thresholds and repair methods, etc., then discount rates are likely to alter the order of cost effectiveness of alternatives (Mullard, 2010). Tables of impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs (D=100 mm and D=300 mm) and RC beams in all five cities under three emission scenarios based on spatial time-dependent reliability analysis of structures deterioration are presented in Appendix B.

		RCP 8.5					RCI	P 4.5		Year 2015			
		A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4
					М	ean NP	V (\$/m	2)					
	slab 100	9.0	12.6	3.3	11.7	8.4	11.7	3.3	11.3	6.5	8.6	3.0	9.2
r=2%	slab 300	8.9	12.5	2.7	10.5	8.4	11.6	2.7	10.1	6.4	8.5	2.4	8.0
	beam	4.8	6.7	0.7	4.0	4.1	5.6	0.5	3.2	2.7	3.3	-0.1	1.0
	slab 100	2.6	3.2	1.1	4.0	2.4	3.0	1.1	3.8	1.7	1.9	1.0	3.1
r=4%	slab 300	2.6	3.2	0.5	2.8	2.4	2.9	0.5	2.6	1.7	1.8	0.4	1.9
	beam	1.0	0.9	-1.3	-1.7	0.8	0.6	-1.3	-1.9	0.2	-0.2	-1.6	-2.7
	slab 100	-0.3	-1.0	0.0	0.3	-0.3	-1.0	0.0	0.3	-0.4	-1.2	0.0	0.1
r=8%	slab 300	-0.3	-1.0	-0.6	-0.9	-0.4	-1.1	-0.6	-0.9	-0.5	-1.3	-0.6	-1.1
	beam	-0.7	-1.6	-2.2	-4.2	-0.8	-1.7	-2.2	-4.3	-0.8	-1.8	-2.2	-4.4
						Pr(NP	V>0)						
	slab 100	91.3%	89.2%	88.1%	93.1%	91.3%	88.9%	88.3%	93.3%	87.9%	84.4%	86.0%	91.0%
r=2%	slab 300	91.2%	89.0%	78.2%	88.1%	91.2%	88.5%	78.1%	87.7%	87.7%	83.9%	74.9%	84.3%
	beam	65.7%	66.9%	45.4%	54.7%	61.9%	62.8%	43.0%	50.7%	51.5%	52.4%	35.9%	39.6%
	slab 100	78.2%	70.7%	75.7%	85.6%	77.6%	70.1%	75.4%	85.1%	70.3%	61.5%	72.3%	80.9%
r=4%	slab 300	77.6%	69.8%	52.6%	67.5%	76.9%	69.1%	53.2%	67.1%	69.5%	60.1%	48.4%	60.8%
	beam	54.1%	45.9%	15.9%	24.1%	50.0%	42.0%	14.8%	22.6%	38.6%	31.1%	10.5%	15.8%
	slab 100	23.4%	14.5%	31.3%	42.5%	22.2%	13.4%	30.4%	42.0%	16.3%	9.4%	26.5%	35.4%
r=8%	slab 300	22.2%	13.4%	8.4%	15.2%	21.2%	12.5%	8.3%	14.3%	15.4%	8.7%	6.9%	11.1%
	beam	9.5%	6.3%	0.6%	1.5%	8.6%	5.2%	0.4%	1.6%	5.7%	3.5%	0.5%	1.2%

Table 7-11: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs (D=100 mm and D=300 mm) and RC beams in Xiamen under RCP 8.5, RCP 4.5 and Year 2015.

7.11 Consideration of co-benefit

The co-benefit (Δ B) for adaptation strategies can be considerable. For adaptation strategies A1 and A2, increase concrete cover can result in higher dead load, therefore, higher reinforcement ratio might be needed. In general, co-benefit is highly likely to be negative values for adaptation strategies A1 and A2. However, an increase in concrete strength can reduce the effective depth of RC structural members whilst still maintaining flexure and shear capacities. Cost data presented in Table 6-5 show that a 5 mm concrete cover reduction can make a difference in deciding the cost effectiveness of adaptation strategies A3 and A4.

According to calculations, the reduction of depth is related to original design of reinforcement ratio and depth. Generally, the higher value of reinforcement ratio and depth are for original design, the more reduction of depth is expected. For example, assume the reinforcement ratio is 1% for an 800 mm deep RC beam, and then reduced depth can be as high as 15 mm and 25 mm for adaptation A3 and A4, respectively. These could result in co-benefit to be as high as \$3.225 and \$5.375 for adaptation A3 and A4 in Chinese structures, and \$21.75 and \$36.25 for adaptation A3 and A4 in Australian structures. If co-benefit values listed above for adaptation strategies A3 and A4 are added to the mean NPV results in Table 7-3 and Table 7-6 then mean NPV for RC beams in all three Chinese cities and two Australian cities would be positive values under all emission scenarios. On the other hand, if the reinforcement ratio 0.4% is assumed for a 100 mm deep RC slab then only 1 mm and 2 mm reduction in depth are expected for adaptation A3 and A4, respectively. In practice, structural dimension of RC structures normally change by 5 mm for the convenience of construction. So design outcomes do no changes for these cases.

A precise calculation of co-benefit will depend on the structural configuration under consideration. Nonetheless, there is likely to be a strong co-benefit in terms of increased flexural and shear capacity, as well as increased member stiffness, resulting from a design with higher concrete grade. If this is factored into the cost-benefit decision process, then adaptation strategies A3 and A4 may well be cost-effective. This is an area for further work before revisions to existing RC design practices (such as specified in AS3600 and GB50010) can be recommended.

7.12 Conclusion

This chapter focused on climate adaptation strategies for RC structures built in 2015 in five cities from two countries (and therefore under different durability standards), and were subjected to two different climate scenarios projections. The adaptation strategies A1 to A4 consist of increasing the concrete cover recommended in the standards by 5 or 10 mm, and increasing the concrete strength by one or two grades. The spatial time-dependent reliability analysis is used to estimate the likelihood and extent of corrosion damage and corresponding damage cost for RC slabs (D =100 mm and D = 300 mm) and beams. The risk reduction and NPV of each adaptation strategy can then be calculated. Break-even analysis is then used to assess the minimum risk reduction or the maximum C_{adapt}/C_{damage} ratio required to make an adaptation strategy to be cost-effective, and the results can help decision makers quickly assess the probability of some proposed adaptation strategies being cost effective.

The assessment of the mean NPV and Pr(NPV>0) indicated that the cost-effectiveness of an adaptation strategy depends on various factors including locations, structural type, structural dimensions, durability design requirements, microclimate conditions, adaptation strategies, climate projections, discount rate and cost data, etc. Structural components that are predicted to have a large likelihood and extent of corrosion damage need merit appropriate adaptation strategies, and there is a higher possibility that adaptation strategies applied on these structural components will be cost effective. Further, the more severe future climate condition than expected or the lower C_{adapt}/C_{damage} ratio can lead to higher chances for the adaptation strategy to be costeffective. According to current information, keeping business as usual for RC buildings in Sydney and Canberra, as well as Jinan and Kunming, are likely to be more cost-effective than those four adaptation strategies under a changing climate. However, for RC buildings in warm and humid areas in China, such as Xiamen, climate adaptation strategies are highly recommended, for the chance that the adaptation strategy will be cost effective is up to 85%. The discount rate or co-benefit can be critical when deciding whether the adaptation strategy is cost effective or not. The overall results indicate that the reliability framework is well suited to assessing the cost-effectiveness of climate adaptation strategies. Moreover, the framework can easily adapt to updates or adjustments of information. The results and analysis can

greatly assist designers and asset owner or operators in improving and optimising the management of RC structures in corrosive environments.

7.13 References

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Chapter 8 : Conclusions and recommendations for future work

8.1 Summary and conclusions

It was established in Chapter 1 that the durability of RC structures subject to corrosion damage is an important issue facing designers and asset owner/operators, especially considering the accelerated deterioration rate caused by a changing climate. Attention must be focused on both the design of new infrastructure and the management and maintenance of RC structures. This thesis has addressed the need to provide predictive information on the performance of RC structures subject to carbonation induced corrosion damage under a changing climate and gives decision makers a useful tool to aid in the selection and optimisation of climate adaptation strategies for new RC structures.

The literature review (refer Chapter 2) introduced and discussed the key variables that influence the carbonation and corrosion induced cracking in RC structures and these include climate conditions such as CO₂ concentration, temperature and RH as well as properties of RC structures, i.e. concrete cover, concrete strength, w/c ratio, cement content, reinforcing bar diameter and corrosion rate. An improved carbonation depth model and corrosion rate model was introduced that considered time-dependent climate conditions, and the properties of RC structures was also taken into account in the modelling of corrosion processes. It was found that the influence of climate change was significant, and several correction factors were proposed to account for the changing climate. These modified models integrated with cracking initiation and crack propagation models can provide a fast and robust estimation of the time from corrosion initiation to severe cracking (crack widths of 1.0 mm) and is thus ideally suited for use within a reliability based stochastic analysis where a large number of computations are required.

A range of concrete adaptation measures (e.g., concrete surface treatment, corrosion inhibitors, etc.) were discussed and data from the literature was used to quantify the influence of these measures on the processes of corrosion initiation and corrosion propagation. The climate adaptation of corrosion damaged RC structures can significantly influence performance, particularly for long design lives. Four primary climate adaptation strategies were investigated:

- •Adaptation strategy A1: increase concrete cover by 5 mm;
- •Adaptation strategy A2: increase concrete cover by 10 mm;
- •Adaptation strategy A3: increase concrete strength by one grade;
- •Adaptation strategy A4: increase concrete strength by two grades;

The costs associated with these adaptation strategies and the costs of corrosion damage, including direct costs and user delays were investigated and the expected damage costs of business as usual and four adaptation strategies were estimated, so that their economic performance could be assessed and cost-benefit analysis could be conducted. In this study, all costs are assumed unchanged.

Improved models for corrosion initiation and corrosion rate were integrated with the existing model for crack initiation and crack propagation and, along with the inclusion of spatially variable corrosion damage and adaptation strategies, a spatial time-dependent reliability analysis was undertaken. The analysis accounted for the uncertainty of the model input variables, and the spatial variability of concrete dimensions and properties was modelled through the use of stationary random fields. The application of the spatial time-dependent reliability analysis was illustrated by analysing RC slabs and beams in two Australian cities and three Chinese cities of business as usual and four adaptation strategies under two climate scenarios and the results presented in terms of:

- the likelihood and extent of corrosion damage;
- the mean NPV and Pr(NPV>0) of adaptation strategies;
- the break-even analysis of ΔR and C_{adapt}/C_{damage} ratio.

As discussed in a comprehensive literature review, (refer Chapter 2), the models used in the spatial time-dependent reliability analysis presented herein are the most appropriate from a range of models found in the literature. The corrosion initiation model and corrosion rate model improved herein included the effects of a changing climate and time-dependent corrosion process which are critical to predict the longterm performance of RC structures accurately. The models selected for crack initiation and crack propagation are based on theoretic and accelerated corrosion experiments, respectively. Both models include the key influencing variables of corrosion rate, cover, bar diameter and concrete strength, etc. All these models are not overly computationally expensive, as this is prohibitive for stochastic analyses, and to date, the more complex models (finite element models, etc.) have not been proven to increase the accuracy of predictions. The framework developed in this thesis, however, can be modified to incorporate future model developments and thus represents a useful tool for the prediction of the likelihood and extent of corrosion damage and for the cost-effectiveness of climate adaptation strategies for RC structures subject to corrosion damage under a changing climate.

It was discovered that a changing climate can result in the extent of damage to increase by up to 0.5% and 6% for RC infrastructure in Sydney and Kunming, respectively. If we keep business as usual, RC structures located in hot or temperate climate areas in China, such as Xiamen and Kunming, were most susceptible to climate change. Corresponding cost-benefit analysis of climate adaptation strategies indicates that the cost-effectiveness of an adaptation strategy depends on various factors including locations, structural type, structural dimensions, durability design requirements, microclimate conditions, adaptation strategies, climate projections, discount rate and cost data, etc. Structural components that are predicted to have a large extent of corrosion damage will have a higher possibility that adaptation strategies applied on them will be cost effective. Further, the more severe future climate conditions are expected, or the lower C_{adapt}/C_{damage} ratio, can lead to higher chances for the adaptation strategy to be cost-effective. The discount rate or cobenefit can be critical when deciding whether the adaptation strategy is cost effective or not. As expected, for RC buildings in warm and humid areas in China, such as Xiamen, climate adaptation strategies are highly recommended, for the chance that adaptation strategy will be cost effective is up to 85%.

8.2 Recommendations for future work

The spatial time-dependent reliability model developed in the thesis makes use of improved and existing models to predict the likelihood and extent of corrosion damage in RC structures. The overall model is an integrated collection of individual models for, among others, corrosion initiation, corrosion propagation, crack initiation, crack propagation, spatial variability, adaptation strategies, maintenance performance and cost-benefit analysis. There exists a number of areas for further work in the improvement and validation of these models, and these are discussed in this section.

The corrosion initiation model is based on Fick's first law of diffusion and assumes carbonation process in one dimension. For RC structures with many edges and corners such as columns, the use of a 2-dimensional carbonation model could be considered. Concrete properties such as aggregate size or the inclusion of admixtures are not considered in the available models, and these have the potential to affect both the rate of CO_2 diffusion and the concrete binding capacity. Further, the influence of shrinkage or flexural cracking on the rate of carbonation is not accounted for, and it is likely that these phenomena could influence the timing of corrosion initiation. As the time to corrosion initiation is significantly longer than the time to reach a limit crack width. Moreover, for those RC structures located near coast or extreme cold area, carbonation as well as chloride may interact and causing more severe corrosion damage. Therefore, more work is required in this area. This could include more fieldwork to validate existing models for different concrete types and research to develop an improved carbonation model (which should also be validated through field data).

The corrosion propagation model used in this thesis (Breysse et al., 2014) included temperature and RH effects based on reference conditions, an improved model that considers such things as the concrete electrical resistivity, the influence of concrete properties/admixtures etc. would provide a more robust estimation of the corrosion rate. The crack initiation model (El Maaddawy & Soudki, 2007) shows reasonable agreement with experimental data and although its accuracy could be improved by further research, the time from corrosion initiation to crack initiation is very short and therefore the accuracy of the crack initiation model does not significantly affect the overall accuracy of the analysis (Stewart & Mullard, 2007). The crack propagation model used herein is an empirical model based on the accelerated corrosion testing program. The crack propagation model is limited by the bounds of the experimental investigation and therefore more work in this area to validate the influence of the key variables is required, particularly bar diameter where only two bar sizes were investigated. Similarly, the confinement correction factor and the rate of loading correction factor are based on experimental investigations and more data to better characterise these factors is needed.

The use of random fields to model the spatial variability of concrete dimensions and properties is well established (Englund & Sorensen, 1998; Faber & Rostram, 2001; Sterritt et al., 2001; Stewart, 2006; Mullard & Stewart, 2009). The characterisation of the spatial parameters (i.e., scale of fluctuation, correlation function, discretised element size), however, is based on limited data and the collection of field data can improve the confidence in the estimation of these parameters. For example, there are numerous methods to calculate the scale of fluctuation based on field observations (Vanmarcke, 1983; O'Connor & Kenshel, 2013; Kenshel, 2009) and more field data is required to improve and validate the current estimations for the scale of fluctuation of, amongst others, the CO₂ diffusion coefficient, binding capacity, concrete cover and the concrete strength. Data from Zhu et al. (2001) was used to characterise the non-stationary mean of the random field for concrete strength in the analysis of a RC column. So the inclusion of a non-stationary mean could influence the likelihood and extent of corrosion damage, and needs further investigation. More field data is required to capture and quantify the stationarity of the mean for the various spatial parameters.

Two types of adaptation strategies, including increasing concrete cover and concrete strength grade, were analysed in detail in this thesis. However, other kinds of adaptation strategy could be considered if more accurate information about their influences on corrosion initiation and corrosion rate is available. Further, a thorough and accurate estimation of co-benefit of adaptation strategies is needed.

Only one simplified maintenance strategy was modelled in this thesis for comparison purpose. However, there are various common techniques used in practice for RC structures (Bertolini et al., 2004; HB84, 2006; Ahmed et al., 2007; Broomfield, 2007). Moreover, using different inspection intervals, repair threshold and repair extent, etc. may change the expected damage costs significantly. This could be another approach to minimise the life cycle costs and better manage long time performance of RC structures, except for conducting climate adaptation strategies. Further, there are some assumptions of maintenance strategy used in this thesis that may need some improvement. For example, it is assumed that the damage may not re-occur after repair during the remaining service life of the structure. This could be a nonconservative assumption, even though the likelihood of this assumption to change
current results is very low as a large proportion of structural components may need no repair at all during their service life.

The cost data for adaptation strategies, maintenance and user delay, were taken from published data. The cost may vary in different regions and with the use of different products. Some other factors such as the discount rate and co-benefit, etc. may change as well, and as such, the framework used in this thesis can be adapted to include any updates or adjustments of data.

Along with general model development and refinement as described above, some specific areas for future work include:

• The improvement of corrosion models. The influence of shrinkage and flexural cracking on corrosion damage can be investigated using accelerated carbonation tests on the cracks at the concrete surface, particularly flexural cracks. Further, laboratory testing could be conducted on corrosion initiated RC specimens, both with and without flexural cracking to assess the influence on the corrosion rate. Similarly, longterm monitoring of real structures could assess the influence of flexural cracking on carbonation depth. The influence of concrete strength could have been related more to the permeability of the concrete (as permeability is typically a function of strength). Thus, investigation into the influence of concrete permeability on corrosion damage is recommended, and this could be achieved through an accelerated corrosion testing program. Other factors, such as the space of reinforcing bars, and the transverse reinforcement effects on crack propagation models could also be studied by these tests. • The investigation of more comprehensive adaptation/maintenance strategies. More investigation is required into the influence of adaptation measures or repair durability improvements (such as concrete surface treatments and corrosion inhibitors) on corrosion initiation and propagation of RC structures. This can be achieved through both experimental works and by data collection from real structures. Concrete surface treatments can be assessed by monitoring the carbonation depth in structures, and the influence of corrosion inhibitors can be assessed by monitoring both the carbonation depth at the time of corrosion initiation and the corrosion rate. Further, maintenance strategies include various inspection intervals, repair measures, repair threshold and repair extent, as well as multiple repairs, climate change effects on repaired structures and a thorough and accurate estimation of co-benefit of adaptation strategies could be

investigated to minimise life cycle costs and better manage long-time performance of RC structures.

• Further development of the spatial time-dependent reliability model. The integrated analysis developed herein could be expanded to include complete structural system assessments. For example, a RC building or bridge could be analysed with multiple components (e.g. abutment walls, columns, crossheads, girders, etc.), each with a unique adaptation or maintenance strategy. The framework of the model could also be amended to analyse other forms of RC deterioration (such as chloride penetration) and even for other materials. For example, the spatial time-dependent reliability model could be adapted to predict the likelihood and extent of corrosion damage for steel structures. Moreover, corrosion damage impacts on structural strength reduction could need some further investigation. Except for new structures, the durability of existing structures could be another challenging topic to look at.

As discussed, the spatial time-dependent reliability model developed in this thesis uses current best practice models to predict the likelihood and extent of corrosion damage and to optimise maintenance strategies for RC structures. Areas for further research have been highlighted above, and any new work can be integrated into the existing framework of the model.

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Appendix A: Pr(NPV(T)>0) for various Cadapt/Cdamage

As described in Section 7.9, cost-effectiveness of adaptation strategies can be influenced by various factors, such as locations, structural type, structural dimensions, durability design requirements, microclimate conditions, adaptation strategies, climate projections, discount rate and cost data, etc. Tables of the probability that NPV(T)>0 of adaptation strategies A1 to A4 when C_{adapt}/C_{damage} equals to some certain values for RC building slabs and beams in all five cities under three emission scenarios based on spatial time-dependent reliability analysis of structures deterioration are presented herein.

Sheltered and cast in-situ slabs and beams in Sydney, Canberra, Kunming, Xiamen and Jinan are analysed. Costs of damage and adaptation strategies are assumed to be the same values as listed in Table 6-5. The discount rate is 4%.

		RCF	P 8.5	
C_{adapt}/C_{damage}	A1	A2	A3	A4
0.0005	30.4%	31.6%	26.5%	31.0%
0.001	21.8%	24.2%	16.7%	23.3%
0.002	11.9%	15.2%	7.4%	14.0%
0.005	2.4%	5.0%	1.2%	4.4%
0.01	0.3%	1.2%	0.1%	0.9%
0.02	0.0%	0.1%	0.0%	0.1%
0.05	0.0%	0.0%	0.0%	0.0%
0.1	0.0%	0.0%	0.0%	0.0%
C_{adapt}/C_{damage}		RCF	° 4.5	
0.0005	27.4%	28.6%	24.5%	28.2%
0.001	19.1%	21.6%	15.6%	21.1%
0.002	10.5%	13.6%	7.1%	13.0%
0.005	2.3%	4.5%	1.3%	4.3%
0.01	0.2%	1.1%	0.2%	1.0%
0.02	0.0%	0.1%	0.0%	0.1%
0.05	0.0%	0.0%	0.0%	0.0%
0.1	0.0%	0.0%	0.0%	0.0%
C_{adapt}/C_{damage}		Year	2015	
0.0005	23.5%	24.8%	21.8%	24.6%
0.001	16.0%	18.1%	14.2%	17.8%
0.002	8.4%	10.9%	6.8%	10.7%
0.005	1.6%	3.3%	1.1%	3.2%
0.01	0.2%	0.6%	0.1%	0.6%
0.02	0.0%	0.0%	0.0%	0.0%
0.05	0.0%	0.0%	0.0%	0.0%
0.1	0.0%	0.0%	0.0%	0.0%

Table A-1: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various C_{adapt}/C_{damage} for RC building slabs and beams in Sydney under RCP 8.5, RCP 4.5 and Year 2015 scenarios.

		RCF	9 8.5	
C_{adapt}/C_{damage}	A1	A2	A3	A4
0.0005	22.2%	23.4%	19.1%	22.9%
0.001	15.3%	17.3%	11.4%	16.6%
0.002	7.4%	10.1%	4.7%	9.5%
0.005	1.4%	2.9%	0.7%	2.6%
0.01	0.1%	0.5%	0.0%	0.5%
0.02	0.0%	0.0%	0.0%	0.0%
0.05	0.0%	0.0%	0.0%	0.0%
0.1	0.0%	0.0%	0.0%	0.0%
C_{adapt}/C_{damage}		RCF	9 4.5	
0.0005	20.3%	21.6%	18.6%	21.3%
0.001	13.7%	15.8%	11.3%	15.6%
0.002	6.6%	9.0%	4.9%	8.7%
0.005	1.3%	2.7%	0.8%	2.6%
0.01	0.1%	0.5%	0.0%	0.6%
0.02	0.0%	0.1%	0.0%	0.0%
0.05	0.0%	0.0%	0.0%	0.0%
0.1	0.0%	0.0%	0.0%	0.0%
C_{adapt}/C_{damage}		Year	2015	
0.0005	17.4%	18.5%	16.5%	18.3%
0.001	11.1%	12.8%	9.9%	12.6%
0.002	5.5%	7.4%	4.6%	7.4%
0.005	0.8%	2.0%	0.6%	2.0%
0.01	0.1%	0.3%	0.0%	0.3%
0.02	0.0%	0.0%	0.0%	0.0%
0.05	0.0%	0.0%	0.0%	0.0%
0.1	0.0%	0.0%	0.0%	0.0%

Table A-2: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various C_{adapt}/C_{damage} for RC building slabs and beams in Canberra under RCP 8.5, RCP 4.5 and Year 2015 scenarios.

			RCI	P 8.5			RCI	P 4.5		Year 2015			
	C_{adapt}/C_{damage}	A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4
	0.0005	72.0%	73.3%	60.3%	73.2%	72.9%	74.2%	63.6%	74.4%	62.5%	63.9%	54.2%	64.2%
	0.001	66.3%	68.9%	50.3%	68.7%	67.3%	69.6%	53.1%	69.7%	56.5%	59.1%	43.9%	59.7%
	0.002	57.8%	61.6%	38.0%	61.2%	58.7%	62.6%	39.3%	62.7%	47.5%	51.4%	31.2%	52.4%
	0.005	41.4%	47.8%	18.8%	46.2%	39.9%	46.7%	18.6%	46.5%	29.7%	36.2%	13.7%	36.9%
siad	0.01	24.6%	32.4%	6.2%	30.2%	22.3%	30.5%	6.2%	29.3%	14.7%	21.2%	4.3%	21.4%
	0.02	8.8%	15.7%	0.6%	13.9%	7.0%	13.2%	0.6%	12.7%	4.0%	8.4%	0.4%	8.7%
	0.05	0.2%	1.9%	0.0%	1.5%	0.2%	1.1%	0.0%	1.1%	0.0%	0.5%	0.0%	0.6%
	0.1	0.0%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
	0.0005	37.9%	38.4%	36.4%	38.8%	35.2%	35.7%	34.8%	36.1%	25.5%	26.2%	25.5%	26.6%
	0.001	37.7%	38.3%	32.9%	38.7%	34.9%	35.6%	32.3%	36.1%	25.3%	26.1%	23.5%	26.6%
	0.002	36.8%	38.1%	28.8%	38.4%	34.1%	35.4%	28.0%	35.9%	24.7%	25.9%	20.3%	26.5%
1	0.005	25.8%	29.4%	15.6%	29.8%	23.3%	27.0%	14.9%	27.9%	16.0%	19.0%	10.3%	20.0%
beam	0.01	14.4%	20.5%	5.6%	20.8%	11.9%	18.3%	5.2%	19.3%	7.5%	12.2%	3.2%	13.5%
	0.02	4.1%	9.0%	0.7%	9.6%	3.0%	7.3%	0.6%	8.8%	1.7%	4.2%	0.4%	5.7%
	0.05	0.1%	0.7%	0.0%	1.2%	0.0%	0.6%	0.0%	1.1%	0.0%	0.3%	0.0%	0.7%
	0.1	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

Table A-3: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various C_{adapt}/C_{damage} for RC building slabs and beams in Kunming under RCP 8.5, RCP 4.5 and Year 2015 scenarios.

			RCI	P 8.5			RCI	P 4.5		Year 2015				
	C_{adapt}/C_{damage}	A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4	
	0.0005	93.9%	94.3%	87.4%	94.1%	94.0%	94.4%	87.6%	94.3%	91.6%	92.2%	85.3%	92.2%	
	0.001	91.7%	92.5%	81.8%	92.1%	91.8%	92.7%	81.4%	92.4%	88.6%	90.0%	78.4%	89.9%	
	0.002	88.2%	89.8%	72.0%	89.1%	87.7%	89.5%	72.1%	88.8%	83.2%	85.8%	68.4%	85.7%	
	0.005	77.6%	81.9%	51.0%	79.5%	76.9%	81.5%	51.3%	79.3%	69.5%	75.0%	46.4%	74.3%	
slab	0.01	62.7%	70.0%	28.5%	66.1%	61.3%	69.3%	28.2%	65.8%	51.2%	60.4%	23.5%	59.0%	
	0.02	38.8%	51.1%	6.9%	45.9%	36.3%	49.1%	6.9%	44.9%	25.6%	37.8%	5.6%	36.4%	
	0.05	4.6%	15.7%	0.0%	12.8%	3.8%	13.5%	0.0%	11.7%	1.9%	7.8%	0.0%	7.8%	
	0.1	0.0%	0.7%	0.0%	0.6%	0.0%	0.7%	0.0%	0.6%	0.0%	0.3%	0.0%	0.4%	
	0.0005	67.4%	67.5%	66.0%	67.6%	63.3%	63.5%	62.2%	63.6%	52.8%	53.1%	52.4%	53.4%	
	0.001	67.3%	67.5%	63.1%	67.6%	63.2%	63.5%	59.9%	63.5%	52.7%	53.1%	50.4%	53.3%	
	0.002	66.7%	67.5%	58.2%	67.3%	62.7%	63.4%	55.1%	63.3%	52.3%	53.0%	46.7%	53.2%	
1	0.005	55.3%	58.6%	40.8%	58.2%	51.4%	54.6%	38.8%	54.4%	40.4%	43.5%	31.2%	43.7%	
beam	0.01	40.6%	48.0%	21.9%	47.6%	36.9%	44.4%	20.8%	44.2%	26.6%	33.3%	15.1%	33.9%	
	0.02	19.3%	30.1%	5.4%	30.2%	16.7%	27.8%	5.1%	28.3%	10.0%	18.2%	3.4%	20.2%	
	0.05	1.0%	7.0%	0.0%	8.0%	0.9%	5.3%	0.0%	6.7%	0.4%	3.0%	0.0%	4.5%	
	0.1	0.0%	0.2%	0.0%	0.5%	0.0%	0.2%	0.0%	0.5%	0.0%	0.1%	0.0%	0.4%	

 Table A-4: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various C_{adapt}/C_{damage} for RC building slabs and beams in Xiamen under RCP 8.5, RCP 4.5 and Year 2015 scenarios.

		RCP 8.5					RCI	9 4.5		Year 2015			
	C_{adapt}/C_{damage}	A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4
	0.0005	38.1%	39.4%	24.4%	39.1%	35.3%	36.7%	24.2%	36.8%	31.1%	32.4%	22.9%	32.5%
	0.001	33.3%	35.5%	17.1%	34.9%	30.2%	32.3%	16.8%	32.5%	26.3%	28.2%	16.3%	28.3%
	0.002	26.2%	29.6%	10.2%	28.0%	23.5%	26.5%	9.9%	26.1%	20.4%	23.0%	9.9%	23.3%
alah	0.005	15.2%	19.1%	3.8%	16.7%	12.4%	16.1%	3.1%	14.8%	10.4%	13.6%	3.1%	13.8%
slab	0.01	7.3%	10.7%	0.7%	8.6%	5.6%	8.6%	0.8%	7.4%	4.4%	7.1%	0.7%	6.7%
	0.02	2.0%	4.2%	0.1%	3.0%	1.3%	2.9%	0.1%	2.2%	0.8%	2.4%	0.1%	2.1%
	0.05	0.1%	0.4%	0.0%	0.2%	0.0%	0.2%	0.0%	0.2%	0.0%	0.1%	0.0%	0.1%
	0.1	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
	0.0005	19.2%	19.5%	16.3%	19.8%	16.1%	16.6%	14.6%	16.9%	12.4%	12.8%	11.4%	13.2%
	0.001	18.9%	19.4%	13.4%	19.5%	15.8%	16.5%	12.4%	16.8%	12.3%	12.8%	9.7%	13.1%
	0.002	17.9%	19.3%	11.0%	19.1%	15.2%	16.3%	9.8%	16.7%	11.7%	12.6%	7.7%	13.0%
1	0.005	11.0%	13.6%	4.4%	13.3%	8.9%	11.0%	3.8%	11.5%	6.3%	8.1%	3.1%	8.7%
beam	0.01	4.9%	8.0%	1.3%	7.8%	3.5%	6.1%	0.9%	6.6%	2.3%	4.1%	0.9%	4.7%
	0.02	1.1%	3.0%	0.1%	2.8%	0.7%	2.0%	0.1%	2.4%	0.4%	1.3%	0.1%	1.7%
	0.05	0.0%	0.2%	0.0%	0.3%	0.0%	0.1%	0.0%	0.1%	0.0%	0.1%	0.0%	0.1%
	0.1	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

Table A-5: Pr(NPV(T)>0) of adaptation strategies A1 to A4 with various C_{adapt}/C_{damage} for RC building slabs and beams in Jinan under RCP 8.5, RCP 4.5 and Year 2015 scenarios.

Appendix B: Discount rate impacts on mean NPV(T) and the Pr(NPV(T)>0)

As described in Section 7.10, discount rate will influence the present value of future costs significantly. Tables of impacts of the discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs (D=100 mm and D=300 mm) and RC beams in all five cities under three emission scenarios based on spatial time-dependent reliability analysis of structures deterioration are presented herein.

Sheltered and cast in-situ slabs and beams in Sydney, Canberra, Kunming, Xiamen and Jinan are analysed. Costs of damage and adaptation strategies are assumed to be the values listed in Table 6-5. The discount rate equals to 2%, 4% and 8%, respectively.

Table B-1: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs and RC beams in Sydney under RCP 8.5, RCP 4.5 and Year 2015.

			RC	P 8.5			RC	P 4.5		Year 2015			
		A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4
					Μ	ean NP	V (\$/m	1 ²)					
	slab 100	-5.1	-11.3	0.3	-1.5	-5.3	-11.5	0.3	-1.6	-5.6	-11.9	0.2	-1.9
r=2%	slab 300	-2.2	-5.6	-1.3	-8.5	-2.4	-5.8	-1.3	-8.6	-2.7	-6.2	-1.4	-8.9
	beam	-5.6	-12.3	-5.3	-26.0	-5.8	-12.5	-5.3	-26.1	-6.1	-12.9	-5.4	-26.4
	slab 100	-6.2	-12.8	-0.4	-2.9	-6.3	-12.9	-0.4	-2.9	-6.4	-13.0	-0.5	-3.0
r=4%	slab 300	-3.4	-7.1	-2.0	-9.9	-3.4	-7.2	-2.0	-9.9	-3.5	-7.3	-2.1	-10.0
	beam	-6.7	-13.8	-6.0	-27.4	-6.8	-13.9	-6.0	-27.4	-6.9	-14.0	-6.1	-27.5
	slab 100	-6.7	-13.4	-0.7	-3.4	-6.7	-13.4	-0.7	-3.4	-6.7	-13.4	-0.8	-3.4
r=8%	slab 300	-3.8	-7.7	-2.3	-10.4	-3.8	-7.7	-2.3	-10.4	-3.9	-7.7	-2.4	-10.4
	beam	-7.2	-14.4	-6.3	-27.9	-7.2	-14.4	-6.3	-27.9	-7.2	-14.4	-6.4	-27.9
						Pr(NP	V>0)						
	slab 100	7.3%	3.3%	33.9%	19.9%	6.2%	3.0%	31.5%	17.8%	4.8%	1.9%	27.8%	15.0%
r=2%	slab 300	16.0%	9.5%	17.0%	4.8%	14.0%	8.2%	15.7%	4.6%	11.1%	6.4%	14.6%	3.4%
	beam	6.3%	2.7%	3.5%	0.2%	5.2%	2.5%	3.6%	0.2%	4.0%	1.6%	3.4%	0.1%
	slab 100	0.4%	0.1%	15.2%	4.6%	0.3%	0.1%	14.4%	4.4%	0.2%	0.1%	13.1%	3.4%
r=4%	slab 300	2.0%	0.9%	3.0%	0.3%	1.9%	0.8%	3.2%	0.4%	1.3%	0.4%	2.9%	0.2%
	beam	0.3%	0.1%	0.2%	0.0%	0.2%	0.1%	0.2%	0.0%	0.1%	0.0%	0.2%	0.0%
	slab 100	0.0%	0.0%	1.0%	0.1%	0.0%	0.0%	1.1%	0.1%	0.0%	0.0%	0.8%	0.1%
r=8%	slab 300	0.0%	0.0%	0.1%	0.0%	0.0%	0.0%	0.1%	0.0%	0.0%	0.0%	0.1%	0.0%
	beam	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

Table B-2: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation
strategies A1 to A4 for RC slabs and RC beams in Canberra under RCP 8.5, RCP 4.5 and Year
2015.

			RC	P 8.5			RC	CP 4.5		Year 2015				
		A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4	
					М	ean NI	PV (\$/r	n ²)						
	slab 100	-5.6	-12.0	0.0	-2.1	-5.7	-12.1	0.0	-2.1	-5.9	-12.4	-0.1	-2.4	
r=2%	slab 300	-2.8	-6.3	-1.6	-9.1	-2.9	-6.4	-1.6	-9.1	-3.1	-6.7	-1.7	-9.4	
	beam	-6.1	-13.0	-5.6	-26.6	-6.2	-13.1	-5.6	-26.6	-6.4	-13.4	-5.7	-26.9	
	slab 100	-6.4	-13.0	-0.6	-3.1	-6.4	-13.1	-0.6	-3.1	-6.5	-13.2	-0.6	-3.2	
r=4%	slab 300	-3.6	-7.3	-2.2	-10.1	-3.6	-7.4	-2.2	-10.1	-3.7	-7.5	-2.2	-10.2	
	beam	-6.9	-14.0	-6.2	-27.6	-6.9	-14.1	-6.2	-27.6	-7.0	-14.2	-6.2	-27.7	
	slab 100	-6.7	-13.4	-0.8	-3.5	-6.7	-13.5	-0.8	-3.5	-6.7	-13.5	-0.8	-3.5	
r=8%	slab 300	-3.9	-7.7	-2.4	-10.5	-3.9	-7.8	-2.4	-10.5	-3.9	-7.8	-2.4	-10.5	
	beam	-7.2	-14.4	-6.4	-28.0	-7.2	-14.5	-6.4	-28.0	-7.2	-14.5	-6.4	-28.0	
						Pr(NI	PV>0)							
	slab 100	4.4%	1.9%	25.2%	14.0%	3.9%	1.8%	24.2%	13.0%	2.9%	1.1%	21.5%	10.7%	
r=2%	slab 300	11.1%	6.1%	12.0%	3.0%	9.8%	5.6%	11.7%	3.0%	7.9%	4.2%	10.6%	2.2%	
	beam	3.8%	1.7%	2.2%	0.1%	3.3%	1.5%	2.6%	0.2%	2.5%	0.9%	2.3%	0.0%	
	slab 100	0.1%	0.0%	10.4%	2.7%	0.2%	0.1%	10.4%	2.7%	0.1%	0.0%	9.2%	2.1%	
r=4%	slab 300	1.2%	0.4%	1.7%	0.2%	1.1%	0.4%	2.1%	0.2%	0.6%	0.2%	1.8%	0.1%	
	beam	0.1%	0.0%	0.1%	0.0%	0.1%	0.1%	0.1%	0.0%	0.1%	0.0%	0.0%	0.0%	
	slab 100	0.0%	0.0%	0.5%	0.0%	0.0%	0.0%	0.5%	0.0%	0.0%	0.0%	0.4%	0.0%	
r=8%	slab 300	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	
	beam	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	

		RCP 8.5 RCP 4.5								Year 2015					
		A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4		
					М	ean NP	V (\$/m ²	²)							
	slab 100	3.3	4.1	1.2	4.6	2.9	3.5	1.3	4.4	1.8	1.9	0.9	3.2		
r=2%	slab 300	3.2	4.0	0.6	3.4	2.8	3.4	0.7	3.2	1.8	1.9	0.3	2.0		
	beam	1.3	1.3	-1.2	-1.2	1.0	0.8	-1.2	-1.6	0.3	-0.2	-1.6	-2.6		
	slab 100	0.4	0.0	0.2	1.1	0.3	-0.2	0.2	1.0	-0.1	-0.7	0.1	0.6		
r=4%	slab 300	0.3	-0.1	-0.4	-0.1	0.2	-0.3	-0.4	-0.2	-0.1	-0.7	-0.5	-0.6		
	beam	-0.3	-1.1	-2.0	-3.7	-0.4	-1.2	-2.0	-3.8	-0.7	-1.5	-2.1	-4.1		
	slab 100	-0.8	-1.6	-0.2	-0.4	-0.8	-1.6	-0.2	-0.4	-0.8	-1.7	-0.2	-0.4		
r=8%	slab 300	-0.8	-1.7	-0.8	-1.6	-0.8	-1.7	-0.8	-1.6	-0.9	-1.8	-0.8	-1.6		
	beam	-1.0	-2.0	-2.3	-4.6	-1.0	-2.0	-2.3	-4.6	-1.0	-2.1	-2.4	-4.7		
						Pr(NP	V>0)								
	slab 100	65.5%	60.4%	61.9%	70.9%	66.3%	61.3%	64.8%	72.1%	55.3%	49.7%	55.6%	62.1%		
r=2%	slab 300	65.2%	59.7%	46.2%	59.8%	66.0%	60.4%	48.0%	61.3%	54.9%	48.9%	39.5%	50.7%		
	beam	35.8%	37.3%	20.8%	26.6%	32.9%	34.6%	20.0%	24.7%	23.8%	25.1%	14.3%	17.2%		
	slab 100	42.4%	33.5%	42.3%	55.0%	40.8%	31.6%	44.1%	56.3%	30.6%	22.0%	35.3%	45.6%		
r=4%	slab 300	41.4%	32.2%	20.0%	31.6%	39.9%	30.3%	20.0%	31.0%	29.7%	21.0%	14.7%	22.6%		
	beam	24.8%	18.5%	3.4%	7.0%	22.1%	16.5%	3.1%	6.1%	15.1%	10.6%	1.9%	3.7%		
	slab 100	3.6%	1.6%	7.0%	11.7%	2.9%	1.2%	7.2%	11.4%	2.0%	0.7%	5.2%	8.1%		
r=8%	slab 300	3.3%	1.4%	0.8%	2.0%	2.7%	1.1%	0.7%	1.6%	1.8%	0.6%	0.5%	1.2%		
	beam	1.4%	0.7%	0.0%	0.2%	1.2%	0.5%	0.0%	0.1%	0.7%	0.4%	0.0%	0.1%		

Table B-3: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs and RC beams in Kunming under RCP 8.5, RCP 4.5 and Year 2015.

			RCF	P 8.5			RCI	P 4.5		Year 2015			
		A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4
					Μ	ean NP	V (\$/m ²	2)					
	slab 100	9.0	12.6	3.3	11.7	8.4	11.7	3.3	11.3	6.5	8.6	3.0	9.2
r=2%	slab 300	8.9	12.5	2.7	10.5	8.4	11.6	2.7	10.1	6.4	8.5	2.4	8.0
	beam	4.8	6.7	0.7	4.0	4.1	5.6	0.5	3.2	2.7	3.3	-0.1	1.0
	slab 100	2.6	3.2	1.1	4.0	2.4	3.0	1.1	3.8	1.7	1.9	1.0	3.1
r=4%	slab 300	2.6	3.2	0.5	2.8	2.4	2.9	0.5	2.6	1.7	1.8	0.4	1.9
	beam	1.0	0.9	-1.3	-1.7	0.8	0.6	-1.3	-1.9	0.2	-0.2	-1.6	-2.7
	slab 100	-0.3	-1.0	0.0	0.3	-0.3	-1.0	0.0	0.3	-0.4	-1.2	0.0	0.1
r=8%	slab 300	-0.3	-1.0	-0.6	-0.9	-0.4	-1.1	-0.6	-0.9	-0.5	-1.3	-0.6	-1.1
	beam	-0.7	-1.6	-2.2	-4.2	-0.8	-1.7	-2.2	-4.3	-0.8	-1.8	-2.2	-4.4
						Pr(NP	V>0)						
	slab 100	91.3%	89.2%	88.1%	93.1%	91.3%	88.9%	88.3%	93.3%	87.9%	84.4%	86.0%	91.0%
r=2%	slab 300	91.2%	89.0%	78.2%	88.1%	91.2%	88.5%	78.1%	87.7%	87.7%	83.9%	74.9%	84.3%
	beam	65.7%	66.9%	45.4%	54.7%	61.9%	62.8%	43.0%	50.7%	51.5%	52.4%	35.9%	39.6%
	slab 100	78.2%	70.7%	75.7%	85.6%	77.6%	70.1%	75.4%	85.1%	70.3%	61.5%	72.3%	80.9%
r=4%	slab 300	77.6%	69.8%	52.6%	67.5%	76.9%	69.1%	53.2%	67.1%	69.5%	60.1%	48.4%	60.8%
	beam	54.1%	45.9%	15.9%	24.1%	50.0%	42.0%	14.8%	22.6%	38.6%	31.1%	10.5%	15.8%
	slab 100	23.4%	14.5%	31.3%	42.5%	22.2%	13.4%	30.4%	42.0%	16.3%	9.4%	26.5%	35.4%
r=8%	slab 300	22.2%	13.4%	8.4%	15.2%	21.2%	12.5%	8.3%	14.3%	15.4%	8.7%	6.9%	11.1%
	beam	9.5%	6.3%	0.6%	1.5%	8.6%	5.2%	0.4%	1.6%	5.7%	3.5%	0.5%	1.2%

Table B-4: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs and RC beams in Xiamen under RCP 8.5, RCP 4.5 and Year 2015.

			RCI	P 8.5			RCI	P 4.5		Year 2015				
		A1	A2	A3	A4	A1	A2	A3	A4	A1	A2	A3	A4	
					М	ean NP	V (\$/m ²	2)						
	slab 100	0.7	0.4	0.1	1.2	0.4	0.0	0.1	1.0	0.2	-0.4	0.1	0.8	
r=2%	slab 300	0.7	0.4	-0.5	0.0	0.3	-0.1	-0.5	-0.2	0.1	-0.4	-0.5	-0.4	
	beam	0.0	-0.6	-2.0	-3.4	-0.3	-1.0	-2.0	-3.6	-0.5	-1.3	-2.1	-3.9	
	slab 100	-0.5	-1.2	-0.2	-0.1	-0.6	-1.3	-0.2	-0.1	-0.6	-1.4	-0.2	-0.2	
r=4%	slab 300	-0.5	-1.3	-0.8	-1.3	-0.6	-1.4	-0.8	-1.3	-0.7	-1.5	-0.8	-1.4	
	beam	-0.8	-1.7	-2.3	-4.4	-0.8	-1.8	-2.3	-4.4	-0.9	-1.9	-2.3	-4.5	
	slab 100	-0.9	-1.8	-0.3	-0.5	-0.9	-1.8	-0.3	-0.5	-0.9	-1.8	-0.3	-0.6	
r=8%	slab 300	-0.9	-1.9	-0.9	-1.7	-0.9	-1.9	-0.9	-1.7	-0.9	-1.9	-0.9	-1.8	
	beam	-1.0	-2.1	-2.4	-4.8	-1.1	-2.1	-2.4	-4.8	-1.1	-2.1	-2.4	-4.8	
						Pr(NP	V>0)							
	slab 100	32.7%	29.1%	25.9%	37.0%	29.5%	25.7%	25.6%	34.4%	25.6%	22.2%	24.0%	30.3%	
r=2%	slab 300	32.6%	28.4%	14.9%	27.2%	29.4%	25.0%	14.5%	25.1%	25.4%	21.7%	14.2%	22.3%	
	beam	17.3%	18.7%	8.3%	11.2%	14.6%	15.9%	7.5%	9.8%	11.2%	12.3%	6.0%	7.4%	
	slab 100	15.7%	11.2%	12.2%	22.7%	12.8%	8.9%	11.9%	20.9%	10.7%	7.4%	11.8%	18.8%	
r=4%	slab 300	15.2%	10.5%	4.1%	9.2%	12.4%	8.5%	3.6%	8.0%	10.4%	7.0%	3.4%	7.3%	
	beam	10.3%	6.9%	0.6%	1.7%	8.2%	5.3%	0.5%	1.4%	5.7%	3.5%	0.4%	1.2%	
	slab 100	0.6%	0.3%	0.9%	2.2%	0.4%	0.2%	0.9%	1.9%	0.3%	0.1%	0.8%	1.7%	
r=8%	slab 300	0.5%	0.3%	0.1%	0.3%	0.4%	0.1%	0.1%	0.2%	0.3%	0.1%	0.1%	0.2%	
	beam	0.2%	0.1%	0.0%	0.0%	0.1%	0.0%	0.0%	0.0%	0.1%	0.1%	0.0%	0.0%	

Table B-5: Impacts of discount rate on the mean NPV(T) and the Pr(NPV(T)>0) of adaptation strategies A1 to A4 for RC slabs and RC beams in Jinan under RCP 8.5, RCP 4.5 and Year 2015.