Life-Cycle Cost Assessment of Climate Change Adaptation Measures to Minimise Carbonation-Induced Corrosion Risks

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ABSTRACT

The paper describes a reliability-based approach that predicts the probability of corrosion initiation and damage (severe cracking) for RC structures subjected to corrosion resulting from concrete carbonation when atmospheric CO₂ concentration and temperature increases with time over the next 100 years based on the latest IPCC report for climate change. Increasing design cover is a suggested climate change adaptation strategy. A life-cycle cost analysis is then conducted that considers costs associated with extra design cover and expected maintenance/repairs for typical RC structures and elements over the next 100 years considering several IPCC atmospheric CO₂ emission scenarios. If the proposed increases in design cover produce a minimum life-cycle cost then increasing design cover will be a cost-effective measure to mitigate the effects of carbonation-induced corrosion damage. It was found that life-cycle costs for the current situation (‘do nothing’ – use existing covers) are lower than life-cycle costs for proposed increases in design cover. This suggests that although enhanced greenhouse conditions will lead to increased carbonation-induced corrosion of RC structures it may not be cost-effective to increase design covers.

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INTRODUCTION

Global warming and climate change studies show that greenhouse gas emissions (atmospheric CO₂) may more than double this century, causing temperature rises of up to 4.0°C. An increase in atmospheric CO₂ concentrations will increase carbonation depths, and an increase in temperature will increase diffusion coefficient and corrosion rates, and so the likelihood and extent of carbonation-induced corrosion is expected to increase. This increase in carbonation and corrosion rate may cause corrosion damage that will need to be repaired or shorten the service life of reinforced concrete (RC) structures or elements, such as building facade panels, floor slabs and beam, bridges, etc. On the other hand, increasing the design cover, improved concrete quality, surface coatings, realkalization and other adaptation strategies for new and existing RC structures may help offset the effects of global warming.

While much work has progressed on the time-dependent structural reliability of deteriorating structures (mostly chloride-induced corrosion and fatigue), less effort has been directed to the probabilistic modelling of carbonation-induced cover cracking and structural collapse for RC structures. This is understandable as current levels of atmospheric CO₂ of about 380 ppm will, in many cases, not cause significant carbonation–induced corrosion (Stewart and Rosowsky 1998). Sudret (2008) developed spatial reliability models to predict the likelihood and extent of corrosion damage induced by carbonation, but this work assumed a constant (time-invariant) CO₂ concentration. However, climate change and global warming studies predict that the level of atmospheric CO₂ may increase to over 1000 ppm by the year 2100. As a consequence of this, carbonation may become a more critical durability issue for concrete structures in urban environments. Moreover, Stewart et al (2002) found that the ambient CO₂ concentration attributable to a typical urban environment is approximately 5-10 % higher than CO₂ concentrations in a rural environment. The concentration of CO₂ in urban environments are influenced by combustion of fossil fuels from traffic, domestic heating, power generation, etc, and CO₂ concentrations are often higher closer to ground level. Carbonation depths were then calculated for RC structures assuming a climate change prediction of up to 450 ppm for service lives of up to 100 years (Stewart et al 2002). The probabilistic analysis showed that variability in carbonation depths can he high due to uncertainty and variability of environmental and material properties. A model that predicts the effect of climate change on carbonation has been developed by Yoon et al (2007), but this model tends to overestimate carbonation depths. Peng and Stewart (2008) used the latest CO₂ concentration data provided by the fourth assessment report of 2007 Intergovernmental Panel Climate Change (IPCC) and the Yoon carbonation model to predict the likelihood and extent of carbonation-induced cover cracking and safety loss to RC and prestressed concrete beams in flexure and shear.

When developing reliability-based durability design specifications the fib Model Code for Service Life Design (fib 2006) adopted a CO₂ concentration of approximately 500 ppm based on a linear increase of 1.5 ppm/yr over 100 years. However, this work only considered corrosion initiation as the limit state, and the Intergovernmental Panel on Climate Change (IPCC) predict CO₂ concentrations of up to 1000 ppm by 2100. The present paper describes a reliability-based approach that predicts the probabilities of corrosion initiation and corrosion damage (severe cracking) for RC structures subjected to corrosion resulting from concrete carbonation when the atmospheric CO₂ concentration and temperature increases with time over the next 100 years based on the latest Intergovernmental Panel on Climate Change (IPCC) report for climate change. The carbonation model developed by Yoon et al (2007) is modified to better characterise the effect of time-dependent growth of CO₂ concentration on carbonation depth. In the present paper, ‘corrosion damage’ refers to corrosion-induced cover cracking with crack width exceeding 1.0 mm. Increases in design cover to offset the effects of increasing CO₂ concentrations are suggested for sheltered and unsheltered outdoor exposures. A life-cycle cost analysis is then
conducted that considers costs associated with extra design cover and expected maintenance/repairs for typical RC structures and elements over the next 100 years considering several IPCC atmospheric CO\textsubscript{2} emission scenarios. If the proposed increases in design cover produce a minimum life-cycle cost then increasing design cover will be a cost-effective climate change adaptation measure to mitigate the effects of carbonation-induced corrosion damage.

For a sustainable future a life cycle analysis needs to consider environmental impacts of infrastructure design, construction and maintenance. A useful metric for such purposes is embodied energy, for example, the embodied energy for in-situ concrete is approximately 0.7-1.1 GJ/t whereas for structural steel it is 23-35 GJ/t (Collings 2006). However, embodied energies are difficult to estimate for concrete maintenance, user delay costs, etc. Nonetheless, Collings (2006) concludes that the environmental burden of a bridge is approximately proportional to its cost. Hence, the present paper will focus on life-cycle costs as a metric for energy requirements when assessing which design or repair options lead to a more sustainable future.

**SPATIAL AND TEMPORAL VARIABILITY OF CO\textsubscript{2} CONCENTRATION AND TEMPERATURE**

It has been recognised for some time that CO\textsubscript{2} concentrations are subject to temporal and spatial variability. Since the Industrial Revolution, CO\textsubscript{2} concentration in the atmospheric layer has been steadily increasing so that the global atmospheric concentration of CO\textsubscript{2} in 2000 is approximately 365 ppm (Meehl et al. 2007). The 2007 Intergovernmental Panel on Climate Change (IPCC) reported future CO\textsubscript{2} concentrations for six emission scenarios (Meehl et al. 2007). The two emission scenarios considered herein are:

(i) **A1FI** – This scenario describes a globalised future world of rapid economic growth and the rapid introduction of new and more efficient technologies with an emphasis on fossil intensive energy consumption. This is the worst case emissions scenario.

(ii) **B1** – This scenario describes a globalised future world similar to the A1 family but with a rapid change in economic structures toward a service and information economy with reductions in material intensity and the introduction of clean and resource-efficient technologies. This is the lowest emissions scenario.

Both these scenarios assume that there are no controls/regulations to mitigate CO\textsubscript{2} emissions. The annual CO\textsubscript{2} concentration growth-rate is 1.9 ppm per year since 2000, and so the ‘best mitigation’ scenario after 2010 would be that the CO\textsubscript{2} concentration is kept stable at 2010 levels (386 ppm) due to reduction and stabilisation of CO\textsubscript{2} emissions. The fourth Assessment IPCC Report 2007 (AR4) was used for CO\textsubscript{2} emission scenarios for the next century (Meehl et al. 2007) which provided information about mean CO\textsubscript{2} concentrations as well as confidence bounds of mean ± one standard deviation (\(\sigma\)). The confidence bounds take into account the predictive model inaccuracies and inherent variabilities of CO\textsubscript{2} emissions and predicting their effect on atmospheric CO\textsubscript{2} concentrations. The mean and confidence bounds for A1FI and B1 emission scenarios as well as the best mitigation scenario are shown in Figure 1.

![Figure 1](image_url)

The standard deviations are approximated as

\[
\sigma_{\text{upper}}(t) = X_{\text{upper}}(t - 2000) \quad \sigma_{\text{lower}}(t) = X_{\text{lower}}(t - 2000) \quad 2001 \leq t \leq 2100
\]

where \(\sigma_{\text{upper}}(t)\) and \(\sigma_{\text{lower}}(t)\) are the upper and lower bound standard deviations, respectively, and \(t\) is the time from 2001 to 2100. For A1FI emissions scenario, \(X_{\text{upper}}\) and \(X_{\text{lower}}\) are 2.34 and 0.96, respectively. For B1 emissions scenario, \(X_{\text{upper}}\) and \(X_{\text{lower}}\) are 0.78 and 0.39, respectively.
A number of studies have shown elevated CO2 levels in urban environments due to combustion of fossil fuels from traffic, domestic heating, power generation, pollutions, etc. Stewart et al. (2002) recorded CO2 concentrations of up to 575 ppm in Brno (Czech Republic) which were 5-10% higher than rural CO2 concentrations, with CO2 concentrations higher near street level, see Figure 2. George et al. (2007) found that CO2 concentrations in an urban site (Baltimore) were on average 16% higher than a rural site, 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 CO2 concentration near an urban centre (Phoenix) of 19 ppm. As most infrastructure is located in urban environments, then atmospheric CO2 concentrations shown in Figure 1 will be increased by a factor $k_{urban}$. In this paper $k_{urban}$ is assumed normally distributed with mean of 1.15 and Coefficient of Variation (COV) of 0.10 for emission scenarios A1FI and B1. The factor $k_{urban}$ is not applied to the best mitigation scenario as this scenario defines the best possible CO2 outcome which would include minimal elevated CO2 levels in urban environments.

The IPCC report (Meehl et al. 2007) also predicts increases in average global temperature as a result of increased CO2 concentrations; namely, mean increases of 4.0 $^\circ$C and 1.8 $^\circ$C, for A1FI and B1 emission scenarios by 2100, respectively. The increase in temperature over time is approximately linear, and the variability is normally distributed with COV=0.3.

PROBABILITY OF CORROSION INITIATION AND DAMAGE

Time to Corrosion Initiation

The time to corrosion initiation (carbonation) depends on many parameters: concrete quality, concrete cover, relative humidity, ambient carbon dioxide concentration and others. The impact of carbonation has been studied by many researchers and various mathematical models have been developed with the purpose of predicting carbonation depths (for review see e.g., Duracrete 1998, Stewart et al 2002). Corrosion initiates at the time ($T_i$) when carbonation reaches the surface of the reinforcing bar.

The carbonation depth model proposed by Yoon et al. (2007) considers a wide range of influencing parameters and so the carbonation depth ($x_c$ in cm) is predicted from Yoon et al. (2007), but corrected to allow for modelling uncertainties and $k_{urban}$ as

$$x_c(t) = \sqrt{\frac{2D_{CO2}(t)}{a}ME_{CO2}(t)k_{urban}C_{CO2}(t)t\left(\frac{t}{t-1999}\right)^{n_d}} \quad t \geq 2000$$

$$D_{CO2}(t) = D_1(t-1999)^{n_d} \quad a = 0.75C_eC_{CaO}\alpha_M \frac{M_{CO2}}{M_{CaO}}$$

where $ME_{CO2}(t)$ is the time-dependent model error for CO2 concentration with mean equal to one and variability obtained from Eqn. (1); $C_{CO2}(t)$ is the time-dependent mass concentration of ambient CO2 (10$^{-3}$kg/m$^3$) obtained from Figure 1 using the conversion factor 1 ppm = 0.0019×10$^{-3}$ kg/m$^3$; $k_{urban}$ is a factor to account for increased CO2 levels in urban environments; $D_{CO2}(t)$ is CO2 diffusion coefficient in concrete; $D_1$ is the mean CO2 diffusion coefficient after one year equal to 0.65, 1.24 and 2.23 for water-cement ratio (w/c) of 0.45, 0.5 and 0.55 respectively; $n_d$ is the mean age factor equal to 0.218, 0.235 and 0.240 for w/c of 0.45, 0.5 and 0.55 respectively; $C_e$ is cement content (kg/m$^3$) equal to 411, 370, and 336 for w/c of 0.45, 0.5 and 0.55 respectively; $C_{CaO} = 0.60; \alpha_M = 0.78, 0.82 and 0.84$ for w/c of 0.45, 0.5 and 0.55 respectively; $M_{CaO} = 56$ g/mol
and \( M_{\text{CO}_2} = 44 \text{ g/mol} \); \( t_0 \) is the reference period (1 year) and \( n_m \) is the age factor for microclimatic conditions (\( n_m=0 \) for sheltered outdoor, \( n_m=0.12 \) for unsheltered outdoor). Yoon et al (2007) provided estimates of maximum (95\(^{\text{th}}\) percentile) values for \( D_1 \) and \( n_d \). The standard deviation for \( D_1 \) is approximately 0.15, and COV for \( n_d \) is approximately 0.12 for all w/c ratios. These statistics represent model error (or accuracy). Note that the diffusion coefficient is consistent with other studies and so is appropriate for 'good quality concrete' (Sanjuan and del Olmo 2001).

The effect of temperature on diffusion coefficient is modelled using the Arrhenius Law (e.g., Duracrete 2000, Yoon et al 2007), where the time-dependent change in diffusion coefficient when compared to a temperature of 20 \(^{\circ}\)C is:

\[
f_T(t) = e^{\frac{E}{R} \left( \frac{1}{293} - \frac{1}{273+T(t)} \right)}
\]

(4)

where \( T(t) \) is the temperature at time \( t \) (\(^{\circ}\)C), \( E \) is the activation energy of the diffusion process (40 \( \text{kJ/mol} \)) and \( R \) is the gas constant (8.314x10\(^{-3}\) \( \text{kJ/mol.K} \)). As temperature will be increasing over time then \( D_{\text{CO}_2}(t) \) is averaged over time and so \( T(t) \) is also averaged over time.

While Eqn. (2) was used by Yoon et al (2007) to predict carbonation depths for increases in CO\(_2\) concentrations it needs to be recognised that Eqn. (2) is a point-it-time predictive model - i.e., the carbonation depth at time \( t \) assumes that \( \text{CO}_2(t) \) is constant for all times up to time \( t \). This will overestimate carbonation depth as \( \text{CO}_2 \) concentration will be gradually increasing with time up to the peak value \( \text{CO}_2(t) \). Stewart et al (2002) considered this phenomenon and calculated carbonation depths due to enhanced greenhouse \( \text{CO}_2 \) conditions using the average \( \text{CO}_2 \) concentration over the time period, and not the peak value at time \( t \). As such, Eqn. (2) can be re-written as:

\[
x_c(t) = \sqrt{\frac{2f_T(t)D_{\text{CO}_2}(t)}{a} \int_{2000}^{t} ME_{\text{CO}_2}(t) C_{\text{CO}_2}(t) dt \left( \frac{t_0}{t-1999} \right)^{n_m} \quad t \geq 2000}
\]

(5)

where

\[
f_T(t) = e^{\frac{E}{R} \left( \frac{1}{293} - \frac{1}{273+T_{av}(t)} \right)}
\]

and

\[
T_{av}(t) = \frac{\int_{2000}^{t} T(t) dt}{t-1999}
\]

(6)

To be sure, Eqn. (5) is an approximation, and there is a need for an improved carbonation model that considers the time-dependent effect of \( \text{CO}_2 \) concentration and other parameters such as temperature or humidity.

**Corrosion-Induced Cover Cracking**

The carbonation-induced corrosion rate is variable and highly dependent on exposure conditions and atmospheric situations – see Peng and Stewart (2008) for a review of corrosion rates. In the present study, corrosion rate is assumed lognormally distributed with statistical parameters for a temperature of 20 \(^{\circ}\)C given by Duracrete (1998), see Table 1. An increase in temperature will increase corrosion rate, and the model described by Duracrete (2000) is used:

\[
i_{\text{corr}}(t) = i_{\text{corr-20}} \left[ 1 + K(T(t) - 20) \right]
\]

(7)
where \( i_{\text{corr,20}} \) is the corrosion rate at \( 20^\circ \text{C} \) given in Table 1, and \( K=0.025 \) if \( T(t)<20^\circ \text{C} \) and \( K=0.073 \) if \( T(t)>20^\circ \text{C} \). Duracrete (2000) notes that Eqn. (7) is a close correlation to Arrhenius equation, at least for temperature below \( 20^\circ \text{C} \), but may be conservative for \( T(t)>20^\circ \text{C} \). A \( 2^\circ \text{C} \) temperature increase will increase the diffusion coefficient and corrosion rate by 12% and 15%, respectively.

Corrosion-induced cover cracking occurs on the concrete surface above and parallel to the rebars. The various stages of crack growth can be described in three stages:

(i) Corrosion initiation (\( T_i \));
(ii) Crack initiation (\( T_{1st} \), time to first cracking - hairline crack of 0.05 mm width), and;
(iii) Crack propagation (\( T_{ser} \), time for crack to develop from crack initiation to a limit crack width, \( w \)).

**Time to Crack Initiation**

As there is a porous zone around the steel reinforcing bar the corrosion products must firstly fill this porous zone before the products start to induce internal pressure on the surrounding concrete. Therefore, not all corrosion products contribute to the expansive pressure on the concrete. This approach to crack initiation has been used by El Maaddawy and Souki (2007) and their model is used herein. The thickness of the porous zone (\( \delta_n \)) is typically in the range of 10 - 20 \( \mu \text{m} \) and can be described using a normal distribution with mean equal to 15 \( \mu \text{m} \) and COV of 0.1. It should be noted, that the accuracy of the time to severe cracking is dominated by the accuracy of time to corrosion initiation (\( T_i \)) and the time since crack initiation to reach a limit crack width (\( t_{ser} \)), and so service life predictions are relatively insensitive to the crack initiation model (Stewart and Mullard 2007).

**Time to Severe Cracking**

The time to severe cracking referred to herein is the time when concrete cover cracking reaches a limit crack width of 1 mm. Mullard and Stewart (2009) have modelled rate of crack propagation which enables the time for a crack to develop from crack initiation to a limit crack width (\( T_{ser} \)). The time (after corrosion initiation) for cracking of the concrete surface to reach a crack width of \( w \) mm is:

\[
T_{sp} = T_{1st} + T_{ser} = T_{1st} + k_R \frac{w - 0.05}{k_c M_{E_{crack}} r_{crack}} \left( \frac{0.0114}{i_{\text{corr}}} \right) \quad 0.25 \leq k_R \leq 1, \; k_c \geq 1.0, \; w \leq 1.0 \text{mm} \tag{8}
\]

where

\[
\psi_{cp} = \frac{C}{D f_t} \tag{9}
\]

\[
r_{crack} = 0.0008e^{-1.7\psi_{cp}} \quad 0.1 \leq \psi_{cp} \leq 1.0 \tag{10}
\]

\[
k_R = 0.95 \left[ \exp \left( -\frac{0.3i_{\text{corr,exp}}}{i_{\text{corr}}} \right) - \frac{i_{\text{corr,exp}} + 0.3}{2500i_{\text{corr}}} \right] \quad k_R \geq 0.25 \tag{11}
\]

and where \( T_{1st} \) is the time to crack initiation, \( T_{ser} \) is the time since crack initiation to reach a limit crack width (years), \( i_{\text{corr}} \) is the corrosion current density (\( \mu \text{A/cm}^2 \)) assumed constant with time, \( \psi_{cp} \) is the cover cracking parameter, \( r_{crack} \) is the rate of crack propagation in mm/hr, \( M_{E_{crack}} \) is crack propagation model error, \( w \) is the crack width (mm), \( C \) is concrete cover in mm, \( D \) is reinforcing bar diameter in mm, \( f_t \) is the concrete tensile strength in MPa, \( k_R \) is a rate of loading
correction factor where \( i_{corr(exp)} = 100 \mu \text{A/cm}^2 \) is the accelerated corrosion rate used to derive \( r_{crack} \), and \( k_c \) is the confinement factor that represents an increase in crack propagation due to the lack of concrete confinement around external reinforcing bars. In this study, corrosion rates are assumed to remain constant with time (time-invariant).

If the reinforcing bar is in an internal location then \( k_c = 1 \), but for rebars located at edges and corners of RC structures then \( k_c \) is in the range of 1.2 to 1.4. Although the data is limited, there appears to be a trend where \( k_c \) increases as \( \psi_{cp} \) increases. A statistical analysis of model accuracy to account for variabilities between model prediction and experimental data is essential for stochastic or reliability analyses where statistics for model error are required. Hence, the statistics for model error for \( r_{crack} \) (ME\(_{r_{crack}}\)) are: mean(ME\(_{r_{crack}}\)) = 1.04 and COV(ME\(_{r_{crack}}\)) = 0.09. For more details of this improved cover cracking model see Mullard and Stewart (2009).

### Time-Dependent Reliability Analysis

Corrosion will take place when the carbonation depth reaches the surface of the reinforcing bar, and so the cumulative probability of corrosion initiation at time \( t \) is

\[
p_i(t) = \Pr[C - x_c(t) < 0] \quad (12)
\]

where \( C \) is the concrete cover and \( x_c(t) \) is the carbonation depth obtained from Eqn. (5).

Corrosion damage is defined as the time when concrete cover severely cracks. Therefore, the cumulative probability of corrosion damage at time \( t \) is

\[
p_s(t) = \Pr[t > T_i + T_{sp}] \quad (13)
\]

Monte-Carlo simulation is used as a computational method for the time-dependent reliability analysis. Note that the CO\(_2\) concentration is fully correlated with time.

Spatial effects for geometric and physical parameters known to influence structural reliabilities are not considered for general corrosion as their inclusion will be less important as it is for chloride-induced pitting corrosion (e.g., Stewart 2004). However, inclusion of spatial variability of environment, dimensions and material properties is an area for further research. It has been shown by Peng and Stewart (2008) that enhanced greenhouse conditions have negligible effect on structural reliability for flexure and shear limit states.

### LIFE-CYCLE COST ANALYSIS

If the benefits of each alternative are the same, then the life-cycle cost up to time \( T \), LCC(T), may be represented as

\[
\text{LCC}(T) = C_D + C_C + C_{QA} + C_IN(T) + \sum_{i=1}^{M} p_f_i(T)C_{SF_i} \quad (14)
\]

where \( C_D \) is the design cost, \( C_C \) the construction cost (materials and labour), \( C_{QA} \) the expected cost of quality assurance, \( C_IN(T) \) the cost of inspections, \( M \) the number of independent failure limit states (e.g., flexure, shear, spalling), \( p_f_i(T) \) the cumulative probability of failure for each limit state (i.e., probability that failure will occur anytime up to time \( T \)), and \( C_{SF_i} \) the failure cost (i.e., damages, cost of life, injury, user delay, etc.) associated with the occurrence of each limit
state. Costs and benefits may occur at different times so in order to obtain consistent results it is
necessary for all costs and benefits to be discounted to a present value. Discount rates are
influenced by a number of economic, social and political factors and thus can be quite variable.
For example, discount rates used by various government agencies are: Australia 7%, U.S. 2-3%,
UK Department of Transport, Sweden 4% and Finland 6% (Val and Stewart 2003). Note that a
high discount rate favours a short service life whereas a low discount rate encourages longer
service lives.

Since carbonation-induced corrosion of reinforcing steel has negligible influence on structural
reliability then corrosion damage (severe corrosion-induced cracking) is considered as the most
influential mode of failure for the estimation of life-cycle costs. If it is assumed that corrosion
damage is always detected when the structure is inspected then the life-cycle cost given by Eqn.
(14) may then be re-expressed as

\[
LCC(T) = C_D + C_C + C_{QA} + C_{IN}(T) + E_{SF}(T)
\]

where \(E_{SF}(T)\) is the expected cost of corrosion damage during service life \(T\) and \(C_D, C_C, C_{IN}\) and
\(C_M\) are all present value costs. For RC structures the expected cost of corrosion damage can be
estimated as (Val and Stewart 2003):

\[
E_{SF}(T) = \sum_{i=1}^{T/\Delta t} \Delta P_{i,i} \frac{C_{SF}^{\text{SF}}}{(1 + r)^{iM}}
\]

where \(\Delta t\) is the time between inspections, \(C_{SF}\) the cost associated with the occurrence of corrosion
damage (i.e., repair/replacement, user losses, etc.), \(r\) the discount rate, and \(\Delta P_{i,i}\) is the probability
of a damage incident between the \((i-1)^{\text{th}}\) and \(i^{\text{th}}\) inspections. This probability can be calculated by
the following recursive formula

\[
\Delta P_{i,i} = \left\{ P_i(i\Delta t) - P_{i-1}(i-1\Delta t) \right\} + \sum_{j=1}^{i-1} \Delta P_{i,j} \left\{ P_j(i-j\Delta t) - P_{j-1}(i-j-1\Delta t) \right\}
\]

where \(P_i(t)\) is the cumulative distribution function for the time of first damage – see Eqn. (13).
The number of damage incidents \(n\) depends on the maintenance strategy. The maintenance
strategy assumes that (Stewart 2001):

- repair is carried out immediately after corrosion damage has been discovered;
- limit state exceedance (Eqn. (13)) results in entire RC surface being repaired;
- repair provides no improvement in durability performance of the repaired structure (i.e., it is
  repaired with the same cover and concrete quality as the original design specification);
- damage may re-occur during the remaining service life of the structure, i.e., multiple
  repairs may be needed. The maximum possible number of damage incidents is \(n_{\text{max}} = T/\Delta t\).

RESULTS

The statistical parameters for the reliability and life-cycle cost analyses are shown in Table 2. The
specified concrete compressive strength is 30 MPa and water-cement ratio is 0.5. A concrete
design cover of 30-40 mm is typical for many building structures such as internal structural
members and wall panels for building facades under non-marine environments in many countries
including the U.S. and Australia (i.e., ACI318 2005, AS3600 2001).
Probability of Corrosion Initiation

Figure 3 shows the mean carbonation depth for various emission scenarios and exposures. As expected, an outdoor sheltered exposure has a higher carbonation depths than unsheltered exposures because CO₂ diffusion is hindered by rain. For the worst case scenario (A1FI) the carbonation depths are 45% higher than that for the best mitigation scenario. This shows that future emission scenarios induced by economic development and population growth affects concrete carbonation and thus the onset of corrosion of reinforcing bars. The COV of carbonation depth is approximately 0.11. Figure 4 shows the probability of corrosion initiation (pᵢ) for RC structures with 30 mm cover (Cₙ₉₉) and 16 mm diameter reinforcement for (a) sheltered and (b) unsheltered exposures. As seen in Figure 4, probabilities of corrosion initiation increase as the CO₂ concentration increases. When w/c=0.45 and in an unsheltered exposure then probability of corrosion initiation is negligible irrespective of the emission scenario. If design cover is increased to 40 mm the probabilities of corrosion initiation reduces significantly to no more than 0.02.

Probability of Corrosion Damage

Figure 5 shows the probability of corrosion damage (pₛ) for RC structures with 30 mm cover and 16 mm diameter reinforcement for (a) sheltered and (b) unsheltered exposures. For the first 20-30 years of service life the effect of carbonation is negligible. If w/c=0.45 and in an unsheltered exposure then probability of corrosion damage is negligible irrespective of the emission scenario. The probability of corrosion damage for the worst case scenario (A1FI) is up to 500% higher than that observed for the best mitigation scenario. This indicates the higher CO₂ concentration could lead to a significant likelihood and extent of corrosion damage that may need costly and disruptive repairs during the service life of many concrete structures. A larger bar diameter will result in a corresponding higher likelihood of corrosion damage.

Adaptation Strategy - Increase in Design Cover

Table 3 shows the cumulative probability of corrosion damage in the year 2100 for existing covers under all emission scenarios for Y16 rebars with a w/c of 0.5 and sheltered exposure. In this study, the probability of corrosion damage (pₛ) for the best mitigation scenario is taken as the baseline case. Table 3 then shows proposed covers needed for A1FI and B1 emission scenarios so that their reduced probabilities of corrosion damage match the baseline case. Therefore, it is found that existing design cover of 50 mm or less would need to increase by approximately 6-11 mm and 4-8 mm under A1FI and B1 emission scenarios, respectively. However, if existing design cover exceeds 50 mm then the probability of corrosion damage reaches near zero so there is no need for increases in design cover. Identical increases in design cover are needed to ameliorate corrosion damage for Y27 reinforcement (see Table 4). If w/c ratio is reduced to w/c=0.45 then probability of corrosion damage becomes negligible (<10⁻⁴) for the best scenario, so large increases in cover are needed for A1FI and A1B emission scenarios to reduce damage risks to similarly low values, see Table 5. However, these large increases in cover may not be practical as the intent of adaptation strategies is not to reduce damage risks to negligible values. If the intent of code designs is not to reduce damage risks to negligible values then no extra cover is likely to be warranted if w/c=0.45. Finally, Table 6 shows that damage risks are also negligible for unsheltered exposure, and so for the same reasons cited above, no extra cover may be deemed appropriate. Clearly, there are many combinations of design variables so the intent of the present paper is to propose a methodology for assessing the need to adaptation strategies, and not to assess every combination of design variable.
Life-Cycle Cost Analysis

A life-cycle cost analysis is used to compare the mean life-cycle costs for two adaptation options:

(i) ‘do nothing’ – no change to design cover, and
(ii) increase design cover as proposed in Table 3.

For both adaptation options construction and repair cost data are needed, and such cost data is country, site and structure specific and so it is difficult to make generalisations about these costs. In this paper cost data will be taken from several sources and expressed in 2009 U.S. dollars. As we are concerned about outdoor CO₂ exposure then the external RC structural elements of interest are mainly beams and columns (assuming that slabs are mostly indoor exposures). Corrosion damage is assumed to occur on one (exposed) face of the structural element. The baseline case for construction cost including forms, concrete, reinforcement and finishing is approximately $1,400/m³ and $2,400/m³ for RC beams (8 m span) and columns (400 mm × 400 mm), respectively (RSMeans 2007). For a typical 500 mm deep × 300 mm wide RC beam then \( C_C = \frac{210}{\text{m}} \) length, and \( C_C = \frac{384}{\text{m}} \) for a RC column. It is assumed that an increase in design cover would increase cost of forms, concrete, reinforcement and finishing by an amount proportional to the extra volume of concrete needed. Hence, costs of this durability design specification (or quality - \( C_{QA} \)) are then estimated as \( C_{QA} = 1/500 = 0.002C_C \) per mm cover/m or \( C_{QA} = 1/400 = 0.0025C_C \) per mm cover/m for RC beams and columns, respectively. For a 20 mm increase in cover then \( C_{QA} = 0.04 - 0.05C_C \) which is very similar to the costs reported by Troive and Sundquist (1998). Hence, in the analysis to follow \( C_{QA} = 0.00225C_C \) per mm increase in cover. For the ‘do nothing’ option \( C_{QA} = 0 \).

The cost of repair or replacement and associated user losses, etc. (\( C_{SF} \)) are considerable and for some structures user losses are often much greater than direct repair, replacement and maintenance costs. Val and Stewart (2003) assumed that the cost of RC bridge deck replacement is double the construction cost, i.e., \( C_{SF} = 2C_C \) based on cost data for (i) removal (\( C_C \)) and (ii) replacement (\( C_C \)) costs. However, this is likely to over-estimate the repair costs for most corrosion damage. A patch repair is the most common technique to repair corrosion damage in RC structures (e.g., BRE 2003, Canisius and Waleed 2004). The repair technique involves the removal of damaged concrete and replacement with a suitable repair material. The estimated cost for concrete patch repair is $440/m² (BRE 2003). User losses and other user disruption costs are site and structure specific, but for many RC structures such costs will be minimised if the RC element to be repaired is an external structural member such as walls, columns or facade panels.

To allow for a minor user disruption cost the total failure cost is assumed as \( C_{CF} = 500m^2 \) which is \( C_{SF} = 0.5 \times 500/210 = 1.19C_C \) and \( C_{SF} = 0.4 \times 500/384 = 0.52C_C \) for RC beams and columns, respectively. The life-cycle cost analysis will consider two values of \( C_{SF} \): (i) \( C_{SF} = 0.5C_C \) and (ii) \( C_{SF} = 1.5C_C \) to represent variability of failure costs. It is assumed that design and inspection costs (\( C_D, C_{IN} \)) are similar for different design specifications and so are not needed for this comparative analysis. The discount rate is assumed at \( r = 3\% \) and inspection interval is \( \Delta t = 2 \) years.

Mean life-cycle costs are calculated from Eqn. (15) for A1FI and B1 emission scenarios for a design life of \( T = 100 \) years, for (i) existing covers of 20 mm, 30 mm, 40 mm and 50 mm, and (ii) extra cover as shown in Table 3. Figure 6 shows that in all cases that life-cycle costs are less for the current (‘do nothing’) situation when compared to the adaptation option where design cover is increased. This is not surprising as even with the worst emissions scenario (A1FI) and lowest cover (20 mm) the probability of corrosion damage after 100 years is only 0.1584 (see Table 3). Because the probabilities of corrosion damage are relatively low there is negligible chance of multiple repairs (\( n > 1 \)). Hence, most structures will require no repair of corrosion damage, and so increasing the initial construction cost by up to \( C_{QA} = 0.025C_C \) (11 mm increase in cover) is not cost-effective. However, the increases in life-cycle costs caused by increases in design cover are
in the range of 0.9% to 1.9% - these are small increases in life-cycle costs which suggests that the benefit of not providing extra cover is marginal.

The inspection interval has negligible effect on life-cycle costs. If the discount rate changes to 2% or 4% then life-cycle costs increase (r=2%) or reduce (r=4%) slightly but the percentage difference changes by less than 0.4%. If the cost of providing extra cover is reduced from 0.00225Cc per mm cover to 0.0015Cc per mm cover then life-cycle costs for increase in design cover are 0.0% to 1.0% higher than life-cycle costs for the current (‘do nothing’) option. The percentage difference between the current situation and adaptation option reduces slightly if the cost of repair/replacement increases to $C_{SF} = 2.0Cc$. These sensitivity analyses show that for all reasonable combinations of input variables that the trend of the results is unchanged; namely, life-cycle costs for the current (‘do nothing’) option are lower than life-cycle costs for proposed increases in design cover. This suggests that although enhanced greenhouse conditions will lead to increased carbonation-induced corrosion of RC structures, the probability of corrosion damage remains low and so it is not cost-effective to increase existing covers. As life-cycle costs reflect the energy burden of structures, it is also reasonable to suggest that increases in design cover will lead to increased embodied energy use over the life-cycle of most RC structures. Note that improved concrete quality, surface coatings, realkalization, etc. may prove to be more cost-effective and sustainable climate change adaptation strategies than increasing design cover.

The primary anthropogenic factor associated with climate change is CO2 concentration and temperature, but other climatic variables such as humidity and rainfall are predicted to change. Uncertainty about these and other climatic variables is very large (as is their spatial variability), and this uncertainty increases when the effects of potential CO2 mitigation measures are considered. The effect of humidity and other climatic variables on risks of corrosion damage is clearly an area for further research.

A study of this nature necessitates many assumptions as we are dealing with a long time span where the impact of political and socio-economic issues on greenhouse CO2 emissions is unclear. Furthermore, the effects of climate change are spatial, as well as temporal, so climate adaptation strategies to reduce risks of infrastructure deterioration will be region or site-specific. The present paper provides a decision-support framework to help address these issues, and the results provided are indicative for the structural configurations considered herein. Clearly, further research is needed to better characterise the temporal and spatial impacts of climate change, and the cost-effectiveness of adaptation strategies.

CONCLUSIONS

Global warming and climate change studies show that greenhouse gas emissions (atmospheric CO2) may more than double this century. The paper described a reliability-based approach that predicts the probabilities of corrosion initiation and corrosion damage (severe cracking) and the recommended increase in design cover to offset the effects of increasing CO2 concentrations and associated increase in temperatures. A life-cycle costs analysis considered initial construction costs and the costs of providing extra design cover and expected maintenance/repairs for typical RC structures and elements over the next 100 years considering several IPCC atmospheric CO2 emission scenarios. It was found that life-cycle costs for the current situation (use existing covers) are lower than life-cycle costs for proposed increases in design cover. This suggests that although enhanced greenhouse conditions will lead to increased carbonation-induced corrosion of RC structures it may not be cost-effective to increase design covers.
REFERENCES


BRE (2003), Residual Life Models for Concrete Repair - Assessment of the Concrete Repair Process, Building Research Establishment, UK.


<table>
<thead>
<tr>
<th>Exposure Class</th>
<th>Mean</th>
<th>Standard Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>0.345 μA/cm²</td>
<td>0.259 μA/cm²</td>
</tr>
<tr>
<td>Wet- rarely dry (unsheltered)</td>
<td>0.172 μA/cm²</td>
<td>0.086 μA/cm²</td>
</tr>
<tr>
<td>Moderate humidity (sheltered)</td>
<td>0.431 μA/cm²</td>
<td>0.259 μA/cm²</td>
</tr>
</tbody>
</table>

Table 1. Carbonation Corrosion Rates for Various Exposures (Duracrete 1998), for 20°C.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_{cyl}$ cylinder strength</td>
<td>37.5 MPa</td>
<td>$\sigma=6$ MPa</td>
<td>Lognormal</td>
<td>Stewart (1995)</td>
</tr>
<tr>
<td>$k_w$ ($f'<em>c = k_w f'</em>{cyl}$)</td>
<td>0.87</td>
<td>0.06</td>
<td>Normal</td>
<td>Stewart (1995)</td>
</tr>
<tr>
<td>$M_{ECO2}$</td>
<td>1.0</td>
<td>Eqn. (1)</td>
<td>Normal$^a$</td>
<td>-</td>
</tr>
<tr>
<td>$D_1$</td>
<td>-</td>
<td>$\sigma=0.15$</td>
<td>Normal</td>
<td>Yoon et al (2007)</td>
</tr>
<tr>
<td>$n_d$</td>
<td>-</td>
<td>0.12</td>
<td>Normal</td>
<td>Yoon et al (2007)</td>
</tr>
<tr>
<td>$ME_{(r_{crack})}$</td>
<td>1.04</td>
<td>0.09</td>
<td>Normal</td>
<td>Mullard &amp; Stewart (2009)</td>
</tr>
<tr>
<td>$k_{urban}$</td>
<td>1.15</td>
<td>0.10</td>
<td>Normal$^b$</td>
<td>-</td>
</tr>
<tr>
<td>Cover</td>
<td>$C_{nom}+1.6$ mm</td>
<td>$\sigma=11.1$ mm</td>
<td>Normal$^c$</td>
<td>Mirza &amp; MacGregor (1979)</td>
</tr>
<tr>
<td>$f_i$</td>
<td>$0.53(f'_c)^{0.5}$</td>
<td>0.13</td>
<td>Normal</td>
<td>Mirza et al (1979)</td>
</tr>
<tr>
<td>$E_c$</td>
<td>$4600(f'_c)^{0.5}$</td>
<td>0.12</td>
<td>Normal</td>
<td>Mirza et al (1979)</td>
</tr>
<tr>
<td>$\delta_0$</td>
<td>15 μm</td>
<td>0.1</td>
<td>Normal</td>
<td>-</td>
</tr>
</tbody>
</table>

$^a$: censored at 380 ppm, $^b$: truncated at 1.0, $^c$: truncated at 10 mm (stirrup diameter)

Table 2. Statistical Parameters for Corrosion Parameters, Material Properties and Dimensions.
<table>
<thead>
<tr>
<th>Existing Cover (mm)</th>
<th>( p_s (w=1 \text{ mm}, t=100 \text{ years}) )</th>
<th>Proposed Cover (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A1FI ( p_s )</td>
<td>B1 ( p_s )</td>
</tr>
<tr>
<td>20</td>
<td>0.1584</td>
<td>0.1084</td>
</tr>
<tr>
<td>30</td>
<td>0.0483</td>
<td>0.0301</td>
</tr>
<tr>
<td>40</td>
<td>0.0076</td>
<td>0.0042</td>
</tr>
<tr>
<td>50</td>
<td>0.0005</td>
<td>0.0002</td>
</tr>
<tr>
<td>60</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Note: 0.0 denotes \( p_s < 1 \times 10^{-4} \)

Table 3. Probabilities of Corrosion Damage \((p_s)\) for Existing Covers and Proposed Design Covers for A1FI and B1 Emission Scenarios for Y16 Rebar with a w/c of 0.5 and Sheltered Exposure.

<table>
<thead>
<tr>
<th>Existing Cover (mm)</th>
<th>( p_s (w=1 \text{ mm}, t=100 \text{ years}) )</th>
<th>Proposed Cover (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A1FI ( p_s )</td>
<td>B1 ( p_s )</td>
</tr>
<tr>
<td>20</td>
<td>0.1780</td>
<td>0.1212</td>
</tr>
<tr>
<td>30</td>
<td>0.0561</td>
<td>0.0343</td>
</tr>
<tr>
<td>40</td>
<td>0.0092</td>
<td>0.0049</td>
</tr>
<tr>
<td>50</td>
<td>0.0007</td>
<td>0.0003</td>
</tr>
<tr>
<td>60</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Note: 0.0 denotes \( p_s < 1 \times 10^{-4} \)

Table 4. Probabilities of Corrosion Damage \((p_s)\) for Existing Covers and Proposed Design Covers for A1FI and B1 Emission Scenarios for Y27 Rebar with a w/c of 0.5 and Sheltered Exposure.

<table>
<thead>
<tr>
<th>Existing Cover (mm)</th>
<th>( p_s (w=1 \text{ mm}, t=100 \text{ years}) )</th>
<th>Proposed Cover (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A1FI ( p_s )</td>
<td>B1 ( p_s )</td>
</tr>
<tr>
<td>20</td>
<td>0.0447</td>
<td>0.0182</td>
</tr>
<tr>
<td>30</td>
<td>0.0116</td>
<td>0.0043</td>
</tr>
<tr>
<td>40</td>
<td>0.0015</td>
<td>0.0006</td>
</tr>
<tr>
<td>50</td>
<td>0.0001</td>
<td>0.0</td>
</tr>
<tr>
<td>60</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Note: 0.0 denotes \( p_s < 1 \times 10^{-4} \)

Table 5. Probabilities of Corrosion Damage \((p_s)\) for Existing Covers and Proposed Design Covers for A1FI and B1 Emission Scenarios for Y16 Rebar with a w/c of 0.45 and Sheltered Exposure.
<table>
<thead>
<tr>
<th>Existing Cover (mm)</th>
<th>$p_s$ ($w=1 \text{ mm}, t=100 \text{ years}$)</th>
<th>Proposed Cover (mm)</th>
<th>A1FI</th>
<th>B1</th>
<th>Best Mitigation</th>
<th>A1FI</th>
<th>B1</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0.0095</td>
<td></td>
<td>0.0</td>
<td>46</td>
<td>40</td>
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<tr>
<td>30</td>
<td>0.0023</td>
<td></td>
<td>0.0</td>
<td>46</td>
<td>40</td>
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<tr>
<td>40</td>
<td>0.0003</td>
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<td>0.0</td>
<td>46</td>
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<td>50</td>
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<td>50</td>
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<tr>
<td>60</td>
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<td></td>
<td>0.0</td>
<td>60</td>
<td>60</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: 0.0 denotes $p_s < 1 \times 10^{-4}$

Table 6. Probabilities of Corrosion Damage ($p_s$) for Existing Covers and Proposed Design Covers for A1FI and B1 Emission Scenarios for Y16 Rebar with a w/c of 0.5 and Unsheltered Exposure.
Figure 1. Time-dependent Change of Atmospheric CO$_2$ Concentration (Peng and Stewart 2008).

Figure 2. Hourly CO$_2$ Concentrations Recorded for a Typical Day in Brno (Stewart et al 2002).
Figure 3. Mean Carbonation Depth for w/c=0.50 and Outdoor Sheltered and Unsheltered Exposures.
Figure 4. Probability of Corrosion Initiation for 30 mm Cover, for (a) Outdoor Sheltered and (b) Outdoor Unsheltered Exposures.
Figure 5. Probability of Corrosion Damage for 30 mm Cover, for (a) Outdoor Sheltered and (b) Outdoor Unsheltered Exposures.
Figure 6. Life-Cycle Costs for w/c=0.5 and Y16 reinforcement, for (a) A1FI and (b) B1 Emission Scenarios.