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Abstract

Reinforced concrete (RC) structures placed in chloride-contaminated environments are subjected to deterioration processes that affect their performance, serviceability and safety. Chloride ingress leads to corrosion initiation and its interaction with service loading could reduce its operational life. Chloride ingress and corrosion propagation are highly influenced by weather conditions in the surrounding environment including climate change. Therefore, both structural design and maintenance should be adapted to these new environmental conditions. This study focuses on the assessment of the costs and benefits of climate adaptation strategies for existing RC structures subjected to chloride ingress and climate change. We studied RC structures built at different periods under different construction standards in France. The cost-effectiveness of adaptation measures was measured in terms of the Benefit-to-Cost Ratio (BCR) and the probability that BCR exceeds unity – i.e., Pr(BCR>1). The results of the paper could provide practical advice to policy makers to improve the management of existing RC structures under a changing climate by discussing the influence of the following factors on the mean BCR and Pr(BCR>1): specific exposure conditions, climate change scenarios, risk reduction due to the implementation of adaptation strategies, type of structural component, years of construction and adaptation, discount rates and damage costs.

Keywords: reliability, climate change, adaptation, Benefit-to-Cost Ratio, chloride ingress, reinforced concrete, existing structures

1 INTRODUCTION

1.1 Background

Environmental and operational actions inevitably affect the performance, serviceability and safety of reinforced concrete (RC) structures. Among these actions, chloride ingress into concrete leads to: corrosion initiation, loss of cross-sectional area of reinforcing steel, cover cracking and spalling, and loss of concrete-steel bond. Therefore, the interaction of these deterioration processes with service loading could reduce significantly the operational life of RC structures. Experimental evidence and numerical studies indicate that chloride ingress and corrosion propagation are highly influenced by weather conditions in the surrounding environment (Bastidas-Arteaga et al., 2011; Breysse et al., 2014; Saetta et al., 1993). Until recently all corrosion research assumed constant average climatic conditions for the development of deterioration models. However, even under an optimistic scenario where CO_2 emissions are abated, the average temperature could increase by 2°C in 2100 (IPCC, 2007,

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2013). Therefore, a potentially important factor for asset management is the possible influence of climate change. Since structures have been designed against past climates, structural design and maintenance should be adapted to these new environmental conditions.

Depending on precise exposure conditions, the effect of climate change on initiation or propagation of corrosion could have a detrimental effect on maintenance costs and remaining life. The annual cost of corrosion worldwide is estimated to exceed $1.8 trillion, which translates to 3% to 4% of the Gross Domestic Product (GDP) of industrialised countries (Schmitt, 2009). Since the direct and indirect costs of corrosion are immense, a climate-change induced acceleration of the corrosion process by only a few percent can result in increased maintenance and repair costs of hundreds of billions of dollars annually.

Various studies have been devoted to the assessment on climate change effects on the durability of RC structures. Bastidas-Arteaga et al. (2010) proposed a stochastic approach to study the influence of global warming on chloride ingress for RC structures. They found that chloride ingress could induce reductions of the corrosion initiation stage varying from 2% to 18%. Concerning corrosion propagation until failure, Bastidas-Arteaga et al. (2013) found that climate change could reduce the time to failure by up to 31% for RC structures subjected to chloride ingress. Recent work also focused on the assessment of climate change on the durability of concrete structures in specific locations. Stewart et al. (2011) found that the temporal and spatial effects of a changing climate can increase current predictions of carbonation-induced damage risks by more than 16% which means that one in six structures will experience additional and costly corrosion damage by 2100 in Australia and presumably elsewhere. Wang et al. (2012) studied the impact of climate change on corrosion-induced damage in Australia. They proposed a probabilistic approach to assess corrosion damage taking into account the influence of climate change on areas characterised by different geographical conditions. Talukdar et al. (2012) estimated the effects of climate change on RC carbonation in Canadian cities (Toronto and Vancouver). They found potential increases in carbonation depths over 100 years of approximately 45%. However, this work did not consider the uncertainties related to climate, materials and models. (L. Peng & Stewart, 2014; Lizhengli Peng & Stewart, 2014) assessed spatially distributed and time-dependent corrosion damage risks for cities in Australia and China. They found that a changing climate might increase mean carbonation depths by up to 45% and cause an additional 7–20% of carbonation-induced damage by 2100. De Larrard et al. (2014) used a comprehensive carbonation model to estimate the effect of climate change on corrosion initiation risks due to carbonation for several cities in France. They report that climate change consequences are largely dependent on the weather of each city. This finding justifies the implementation of different adaptation measures for specific locations.

Other studies have focused on climate adaptation of deteriorating RC structures. Stewart & Peng (2010) used a simplified carbonation model and global IPCC (2007) CO_2 concentration and temperature change data to assess the cost-effectiveness of increasing design cover as an adaptation measure. This preliminary analysis found that increasing design cover may not be cost-effective. Nevertheless, the results were based on an oversimplified carbonation model and the authors highlighted that the reported preliminary results cannot be generalised and that further research is needed to better characterise the cost-effectiveness of adaptation strategies. Stewart et al. (2012) considered the effect of climate adaptation strategies including increases in cover thickness, improved quality of concrete, and coatings and barriers on damage risks. It was found that increases in design cover ameliorate the RC durability under a changing climate. However, such a study does not include cost-benefit assessment of climate adaptation strategies. Bastidas-Arteaga & Stewart (2013) performed a probabilistic cost-benefit analysis of two climate change adaptation strategies for new RC structures exposed to chloride ingress. The results indicated that the cost-effectiveness of a given adaptation strategy will depend mainly on the type of structural component, exposure conditions and climate change scenarios.

1.2 Aims and Scope

Within this context, this paper addresses mainly two challenges:
- The extension of the framework for probabilistic cost-benefit analysis proposed by Bastidas-

Arteaga & Stewart (2013, 2015) for existing structures.

- The application of the methodology to existing structures located in specific locations in France and therefore exposed to particular environmental conditions.

These challenges require the consideration of time-dependent damage risks and costs that differ as a function of the construction time and particular geographical conditions. Time-dependent damage risks depend mainly on exposure (climate change projections) and technical considerations (design standards at the time of construction, properties of the construction and repair materials, inspection and maintenance strategies, implementation (or not) of standards updating, etc.). The assessment of cost-effectiveness of adaptation strategies is mainly related to economic aspects (discount rate, repair and adaptation costs, etc.) and the level of use of structures (remaining lifetime, assessment time, adaptation times, etc.). This study includes many of the above-mentioned aspects.

The present paper focuses on the assessment of the costs and benefits of one type of climate adaptation measure (increase in design cover) for existing RC structures in France. The cost-effectiveness is measured in terms of Benefit-to-Cost Ratio (BCR) equal to benefits divided by the cost. This study considers a probabilistic approach that enables a risk-based economic assessment of climate adaptation strategies. The stochastic analysis also allows the probability that BCR exceeds unity Pr(BCR>1) to be calculated. In this case, while mean BCR can be high, there may be a likelihood of BCR less than one (net loss) that also provides useful risk-averse information for decision-makers. Other decision metrics can be used, such as maximising net present value (net benefit), or minimising life-cycle costs. While the formulations may differ, the decision outcomes are identical, and BCR is selected because it seems that government and policy makers are familiar with this metric. Although the paper focuses only on one type of adaptation strategy and specific locations, it provides a methodology that could be extended to study other adaptation strategies or deterioration processes for existing structures worldwide.

Section 2 describes the main considerations for climate change modelling based on the recommendations of the IPCC as well as the climate projections for the coastal French cities considered in this study (Saint-Nazaire and Marseille). The deterioration models used for the probabilistic assessment of chloride-induced severe cracking or spalling under climate change are provided in Section 3. To generalise the results, this work focuses on a serviceability limit state in which the cost-effectiveness of adaptation measures is evaluated in terms of its effect on the time to corrosion damage of the concrete cover. Section 4 describes the repair and adaptation strategies as well as the considerations for time-dependent damage risk assessment by taking into account the evolution in time of design standards in France. Section 5 presents the proposed framework for BCR analysis that is used to estimate the cost-effectiveness of adaptation strategies for existing structures. Section 5 also details the costs (damage and adaptation) and discusses the discount rates that are used in the illustrative example (Section 6).

2 CLIMATE CHANGE EFFECTS AND MODELLING

The IPCC Fifth Assessment Report (AR5) uses Representative Concentration Pathways (RCPs) 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). These RCPs include a mitigation scenario leading to a low forcing level (RCP 2.6), two medium stabilisation scenarios (RCP 4.5/RCP 6) and one high baseline emission scenario (RCP 8.5) (Moss et al., 2010). This study covers RCP 8.5 and RCP 4.5 scenarios representing high and medium emission scenarios, respectively. Figure 1 describes the IPCC projection of CO_2 concentrations from 1900 for RCP 8.5 and RCP 4.5 CO_2 stabilisation scenarios (Clarke et al., 2007; 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). So it is increasingly likely that 'business as usual' CO_2 concentrations will reach 1,000 ppm by the end of this century. This concentration is related to the RCP 8.5 scenario (Figure 1).

The economic assessment of adaptation measures is widely influenced by time-dependent changes in environmental parameters (temperature, relative humidity (RH)) that are site-specific. This work focuses on the study of the cost-effectiveness of climate change adaptation strategies for two port

locations in France: Saint-Nazaire and Marseille (Figure 2). These cities correspond to different types of climate. Saint-Nazaire is close to the Atlantic Ocean in the Northern part of the country and it has a temperate oceanic climate. Marseille is placed in the South-East of France (Mediterranean coast) and it has a Mediterranean climate that is rather hot and dry.

The overall impact of climate change on the future weather of the selected locations was estimated by using data computed by the French general circulation model CNRM-CM5. Figure 3 presents the yearly projections of temperature and RH for Saint-Nazaire and Marseille for the selected climate change scenarios (RCP 4.5 and RCP 8.5) since 2005. By comparing climate (before 2005) in both cities, it is noted that Marseille is hotter and dryer than Saint-Nazaire. Climate change projections mainly predict temperature increase without significant changes in RH for both cities until the end of this century. By comparing mean temperatures over the periods (2001-2010) and (2091-2100) for both places, it was found that climate change could increase temperature by approximately 1.5ºC and 3.5ºC for RCP 4.5 and RCP 8.5 scenarios, respectively. Although climate change effects are relatively similar for both cities, the kinematics of chloride ingress and corrosion propagation is affected by the climate specific to each location. Comprehensive deterioration models are therefore useful tools for estimating the effects of specific climate conditions on RC durability.

Climate projections are subjected to considerable uncertainty, and dependent on CO_2 emission scenarios and accuracy of general circulation models (GCM). A comprehensive model of weather (RH and temperature) should be integrated with deterioration models to assess the effects of a changing climate. Given the difficulties of integrating a GCM with deterioration models, a simplified approach for modelling climate is considered in this study. It accounts for (i) influence of climate change, (ii) seasonal variations, and (iii) random nature of weather within a season. The formulation of this model is detailed in Bastidas-Arteaga et al. (2011, 2013).

3 DETERIORATION MODELLING

Deterioration modelling allows estimating the effects of chloride ingress with regard to serviceability or ultimate limit states. Ultimate limit states are highly dependent on both, geometrical characteristics (cross-sectional dimensions, span length, etc.) and loading (dead, live, seismic, etc.). Therefore, to generalise the results, this work focuses on a serviceability limit state in which the cost-effectiveness of adaptation measures is evaluated in terms of its effect on the time to corrosion damage of the concrete cover (severe cracking or spalling). Corrosion-induced cover cracking and damage occurs on the concrete surface above and parallel to the rebars. The time to corrosion damage, (severe cracking or spalling), T_{sp} is thus obtained as the sum of 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_{sev}, time for crack to develop from crack initiation to a limit crack width, w_{lim}) − i.e., $T_{sp}=T_i+T_{1st}+T_{sev}$. After corrosion initiation, the kinematics of T_{1st} and T_{sev} is controlled by corrosion propagation.

3.1 Corrosion Initiation

The time to corrosion initiation, T_i, is estimated by comparing the chloride concentration at the cover depth, c_t, with a threshold concentration for corrosion initiation C_{th}. The adopted chloride ingress model considers the interaction between three physical processes: *chloride ingress*, *moisture diffusion* and *heat transfer* (Bastidas-Arteaga et al., 2011). These processes are modelled by a set of partial differential equations with the following general form:

$$\zeta \frac{\partial \psi}{\partial t} = \underbrace{\text{div}\,J}_{\text{diffusion}} + \underbrace{\text{div}\,J'}_{\text{convection}} \tag{1}$$

where t is the time and the values of ψ, ζ, J, J' for each physical process are detailed in Table 1.

For chloride ingress, C_{fc} is the concentration of free chlorides, h is the relative humidity and D_c^* and D_h^* represent the apparent chloride and humidity diffusion coefficients that depend on: concrete

age, chloride binding, water content, and temperature and RH inside concrete (Bastidas-Arteaga et al., 2011).

$$D_c^* = \frac{D_{c,ref} f_1(T) f_2(t) f_3(h)}{1 + (1/w_e)(\partial C_{bc}/\partial C_{fc})} \tag{2}$$

$$D_h^* = \frac{D_{h,ref} g_1(h) g_2(T) g_3(t_e)}{1 + (1/w_e)(\partial C_{bc}/\partial C_{fc})} \tag{3}$$

where $D_{c,ref}$ and $D_{h,ref}$ are reference diffusion coefficients measured to standard conditions (Saetta et al., 1993), w_e is the evaporable water content, and f_i and g_i are correction functions to account for the effects of temperature, relative humidity, ageing and degree of hydration of concrete. These functions are detailed in Bastidas-Arteaga (2010). The term $\partial C_{bc}/\partial C_{fc}$ represents the binding capacity of the cementitious system that relates the free and bound chlorides concentration at equilibrium. A Langmuir isotherm is used in this work. The constants of the isotherm are $\alpha_L = 0.1185$ and $\beta_L = 0.09$.

For moisture diffusion, the humidity diffusion coefficient D_h is estimated by accounting for the influence of the parameters presented in Eq. (3). The term $\partial w_e/\partial h$ (Table 1) represents the moisture capacity which relates the amount of free water, w_e, and the pore relative humidity, h. For a given temperature this relationship has been determined experimentally by adsorption isotherms. According to the Brunauer-Skalny-Bodor (BSB) model, the adsorption isotherm depends on temperature, water/cement ratio, w/c, and the degree of the hydration attained in the concrete, t_e. This work adopts the BSB model to represent the moisture capacity.

Finally, for heat transfer (Table 1) ρ_c is the concrete density, c_q is the concrete specific heat capacity, λ is the thermal conductivity of concrete, and T is the temperature inside the concrete after time t.

The flow of chlorides into concrete is estimated by solving simultaneously the system of equations described by Eq. (1) and Table 1. The numerical approach used to solve the coupled system of partial differential equations combines a finite element formulation with finite difference to estimate the spatial and temporal variation of C_{fc}, h and T. The environmental inputs of the model are outside temperature, RH and chloride concentration. The heat transfer equation is firstly solved to estimate the temperature distribution inside the concrete. This temperature distribution is after used to determine the moisture (RH) distribution by solving the moisture diffusion equation. Finally, temperature and RH distributions are considered to determine the chloride profiles inside concrete and then to evaluate corrosion initiation risks.

3.2 Crack Initiation and Propagation

Chloride-induced corrosion is characterised by pitting corrosion with a time-variant corrosion rate $i_{corr}(t)$. Given the complexity of the corrosion process, $i_{corr}(t)$ depends on many factors such as concrete pH and availability of oxygen and water in the corrosion cell. For instance, the optimum relative humidity for corrosion is 70-80%. This study considers the following time-variant corrosion rate model that takes into account the effect of temperature changes (Duracrete, 2000a, 2000b):

$$i_{corr}(t) = i_{corr,20}\left[1 + K_c\left(T(t) - 20\right)\right] \tag{4}$$

where $i_{corr,20}$ is the corrosion rate at 20 °C, $T(t)$ is the temperature at time t (in °C) and K_c is a factor that depends on the value of $T(t)$. For instance, $K_c = 0.025$ if $T(t) < 20$ºC or $K_c = 0.073$ if $T(t) > 20$ºC.

The time to crack initiation, T_{1st}, is estimated based on the model by El Maaddawy & Soudki (2007):

$$T_{1st} = \left[\frac{7117.5(d_0 + 2\delta_0)(1 + v + \psi)}{i_{corr}(t)E_c}\right]\left[\frac{2c_t f_{ct}}{d_0} + \frac{2\delta_0 E_c}{(d_0 + 2\delta_0)(1 + v + \psi)}\right] \tag{5}$$

where d_0 is the diameter of the steel bar (mm), δ_0 is the thickness of porous zone around the bar in ($\delta_0=10\times10^3$ mm), v is the Poisson's ratio ($v=0.18$), c_t is the cover thickness (mm), E_c is the concrete elastic modulus (MPa), $i_{corr}(t)$ is the corrosion rate in μA/cm^2, f_{ct} is the concrete tensile strength (MPa), and $\psi = (d_0 + 2\delta_0)^2 / (2c_t(c_t + d_0 + 2\delta_0))$.

The time to severe cracking, T_{ser} is the time when concrete cover crack reaches a critical limit width w_{lim} (mm) Mullard & Stewart (2011):

$$T_{ser} = k_R \frac{w_{lim} - 0.05}{k_{cr} 0.0008 e^{-1.7\psi_{cp}}} \left(\frac{0.0114}{i_{corr}(t)} \right) \tag{6}$$

where k_{cr} is a confinement factor ($k_{cr}=1$ for a bar in an internal location – i.e., not an edge or corner bar), $i_{corr}(t)$ is the corrosion rate in μA/cm^2, ψ_{cp} is a cover cracking parameter ($\psi_{cp} =c_t/(d_0 f_{ct})$, $0.1< \psi_{cp} <1$) and k_R is a scale factor between natural and accelerated corrosion tests ($0.25 < k_R <1$):

$$k_R = 0.95 \left[\exp\left(-\frac{0.3 i_{exp}}{i_{corr}(t)} \right) - \frac{i_{exp}}{2500 i_{corr}(t)} + 0.3 \right] \tag{7}$$

where i_{exp} is the accelerated corrosion rate of 100 μA/cm^2.

Small values of limit crack (0.1 to 0.3mm) related mainly with durability or aesthetic limit states are commonly used for new structures (Andrade et al., 1993). For existing structures, it is recommended to carry out maintenance and repair measures when $w_{lim} > 0.5$mm for main structural members (Eurocode 2 - (European standard, 2004)). The definition of how 'excessive' the crack width is depends on individual conditions and asset owner policies. This study considers concrete structures as 'severely cracked' when $w_{lim} = 1.0$ mm.

The times of crack initiation and propagation depend on the corrosion rate. Therefore, Eq. (4) is used herein to account for the time-dependency of these times on corrosion rate including climate change effects. Concrete strength is time-variant, and the time-dependent increase in concrete compressive strength after one year using the ACI method is $f_c=1.162 f_c(28)$ where $f_c(28)$ is the 28 day compressive strength (Stewart, 1996). Time-dependent gains in strength beyond one year are not considered in the present analysis.

4 REPAIR AND ADAPTATION STRATEGIES FOR EXISTING STRUCTURES

The repair strategy is often defined by taking into account various criteria such as: remaining lifetime, extent of damage, costs, etc. Probabilistic modelling of deterioration can be used to: (i) determine the remaining lifetime, (ii) calculate the extent of damage and (iii) optimise maintenance costs (Bastidas-Arteaga et al., 2011; Bastidas-Arteaga & Schoefs, 2012). However, for existing structures, probabilistic modelling should also account for the evolution in time of design standards, construction technologies, repair materials, etc. Under a changing climate, the repair strategy could be combined with a climate adaptation strategy to account for these new environmental conditions. Since adaptation strategies could lead to additional costs, a comprehensive framework should: (i) compare the cost-effectiveness of adaptation strategies with respect to existing repair techniques, and/or (ii) optimise the effectiveness of adaptation strategies. This section presents the repair strategy and the evolution in time of durability design standards in France that are considered in the assessment of damage risks of existing structures. Section 5 will describe the framework proposed for the economic assessment of climate adaptation strategies for existing structures.

4.1 Repair Strategy and Damage Risks

The cumulative distribution function for the time of first damage in the period [0, t] for original concrete is:

$$p_s(0,t) = \Pr\left[t \geq T_{sp}\right] \tag{8}$$

where T_{sp} is the time when concrete cover severely cracks (reaches limit crack width w_{lim}), and where the asset owner can specify the limit crack width as the criterion for repair.

A patch repair is the most common technique to repair corrosion damage in RC structures – e.g., (BRE, 2003; Canisius & Waleed, 2004). For a patch repair, the concrete cover is typically removed to approximately 25 mm past the steel bars (which are then cleaned of corrosion products) and a repair material is installed. The maintenance strategy assumes that (Stewart, 2001):

- concrete is inspected at time intervals of Δt;
- patch repair is carried out immediately after corrosion damage has been discovered at time of ith inspection $i\Delta t$;
- patch repair includes an updating to the design cover at the repair time (increase of concrete cover according to current construction standards);
- repair could improve durability performance of the structure when there is an updating of cover requirements before repair (i.e., design specifications at repair time suggest an increase of concrete cover with respect to the original design specification). However, for simplicity, it is supposed that the repair material has the same durability performance than the construction material;
- damage limit state exceedance results in entire RC surface being repaired;
- 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 (repair) incidents is $n_{max} = T_t/\Delta t$, where T_t is the service life.

In addition, the time-dependent damage risks of the repaired material will not be the same as the original material $p_s(0, t)$ due to changed temperature and RH 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 (Bastidas-Arteaga & Stewart, 2013, 2015; Stewart et al., 2014):

$$p_{s,i}(i\Delta t,t) = \Pr\left[t \geq T_{sp,i}\right] \tag{9}$$

where $T_{sp,i}$ is the time to severe cracking when new concrete is exposed to the environment for the first time after repair.

4.2 Evolution of Design Standards in France and Adaptation Strategies

Construction standards provide design recommendations to account, in a simplified way, for uncertainties related to material properties, models, loading, geometry, etc. They consider different mechanical (bending, shear, etc.) or durability (chloride ingress, carbonation, etc.) solicitations for specific conditions (i.e., seismic zones, wind maps, etc.).

Concerning chloride ingress, design standards should include recommendations about the minimal concrete cover, cement content, use of admixtures, use of stainless steel, etc. However, the Eurocode 2 (European standard, 2004) and the European Norm 206 (EN-206, 2000) include mainly recommendations concerning maximum w/c ratio, concrete cover and the compressive strength. The French National Annex (NF EN-206, 2004) adds more recommendations about use of admixtures, the minimum cement content and the kind of cement.

Advances in understanding the behaviour of materials and physical phenomena and practical experience imply evolution of construction standards. In the case of RC structures subjected to chloride ingress, Figure 4 shows the evolution in time of the minimum concrete cover recommended by design standards. These standards were published as circulars of the official diary in France. The first circular on 20 October 1906, recommended a minimum concrete cover varying between 15 and 20 mm for the main reinforcement without distinction of the kind of exposure. In 1934, a new circular recommended 35 mm cover for structures close to the sea and 20 mm for structures in land. Concrete cover was increased to 40 mm and 50 mm in 1964 and 1992, respectively for structures close to the sea. Finally, the Eurocode 2 (European standard, 2004), which is mandatory after 2010 in France,

suggests concrete covers for different exposure conditions. For example, there are three exposure zones for marine structures: atmospheric XS1, submerged XS2 and splash and tidal XS3. Figure 4 plots the concrete cover for a XS3 exposure (55 mm). Figure 4 also includes the considered adaptation strategies that are implemented after the adaptation year t_{adapt}:

- Adaptation 1: increase existing (Eurocode) design covers by Δc_{adapt} =5 mm for XS3 exposure, or
- Adaptation 2: increase existing (Eurocode) design covers by Δc_{adapt} =10 mm for XS3 exposure.

5 COST BENEFIT ANALYSIS OF EXISTING STRUCTURES

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. In the present study, costs and benefits of existing structures are measured from the time of the cost benefit assessment, t_{assess}. For example, if a decision-maker is making a decision in t_{assess} =2013 about predicted costs and benefits of adaptation measures for a structure built in $t_{construct}$ =1970, then damage costs incurred prior to t_{assess} =2013 are not considered in the benefit-to-cost ratio as the decision maker is only concerned with costs and benefits that arise after 2013. If an existing structure is built in the calendar year $t_{construct}$, and if it is assumed that corrosion damage is always detected when the structure is inspected, then the expected damage cost measured from year of assessment t_{assess} to end of service life, is $E_{damage}(T_t)$ and is the product of probability of corrosion damage and damage costs, i.e.,

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

(10)

with $i\Delta t > t_{assess} - t_{construct}$, $t_{construct} < t_{assess}$

where T_t is the service life (typically T_t=100 years), Δt is the time between inspections, n is the number of damage incidents, i is the number of inspections, $p_{s,n}(t)$ the probability of the nth damage incidence before time t, r is the discount rate and 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 example, an asset owner should be able to quantify the unit repair cost (€/m^2), and if the area of damage is known then repair cost can be estimated.

Eq. (10) can be generalised for costs arising from multiple limit states, such as flexural failure, shear failure, etc. Corrosion damage (severe corrosion-induced cracking) is considered herein as the most influential mode of failure for the estimation of benefits. Eq. (10) can be re-expressed as:

$$E_{damage}\left(T_t\right) = \sum_{i=1}^{T_t/\Delta t}\Delta P_{s,i}\frac{C_{damage}}{\left(1+r\right)^{t_{construct}+i\Delta t - t_{assess}}} \text{ with } i\Delta t > t_{assess} - t_{construct}, \ t_{construct} < t_{assess}$$

(11)

where $\Delta P_{s,i}$ is the probability of damage incident between the $(i-1)^{th}$ and i^{th} inspections which is a function of time since last repair which is turn is affected by damage risks for original and repaired concrete $p_s(0,t)$ and $p_{si}(i\Delta t,t)$, respectively. The repaired concrete may have the same durability design specifications as the original concrete, or may be repaired to a higher standard (e.g., increased concrete cover). The risk reduction caused by an adaptation measure is thus:

$$\Delta R\left(T_t\right) = \frac{E_{damage\text{-}BAU}\left(T_t\right) - E_{damage\text{-}adaptation}\left(T_t\right)}{E_{damage\text{-}BAU}\left(T_t\right)}$$

(12)

where $E_{damage\text{-}BAU}(T_t)$ and $E_{damage\text{-}adaptation}(T_t)$ are the cumulative expected damage cost (economic risk) for no adaptation measures (business as usual BAU or existing practice) and adaptation measures, respectively. If an adaptation measure is cost-effective then $E_{damage\text{-}adaptation}(T_t)$ will be significantly lower than $E_{damage\text{-}BAU}(T_t)$ resulting in high risk reduction $\Delta R(T_t)$. In other words, $\Delta R(T_t)$ represents the proportional reduction in expected repair costs due to an adaptation measure.

The cost of adaptation, in this case, additional repair costs associated with increased cover, will occur at the same time as the damage (repair) costs are incurred. It follows that the expected cost of adaptation is directly proportional to damage costs

$$E_{adapt}(T_t) = \sum_{i=1}^{T_t/\Delta t} \Delta P_{s,i} \frac{C_{adapt}}{(1+r)^{t_{construct}+i\Delta t-t_{assess}}} \text{ with } i\Delta t > t_{adapt} - t_{construct}, \ t_{construct} < t_{assess} \qquad (13)$$

where C_{adapt} is the cost of adaptation measures that reduces risk by ΔR and t_{adapt} is the adaptation year (Figure 4).

The 'benefit' of an adaptation measure is the reduction in damages associated with the adaptation strategy, and the 'cost' is the cost of the adaptation strategy. The benefit-to-cost ratio $BCR(T_t)$ is:

$$BCR(T_t) = \frac{E_{damage-BAU}(T_t)\Delta R(T_t)}{E_{adapt}(T_t)} \qquad (14)$$

Clearly, an adaptation measure that results in a benefit-to-cost ratio exceeding unity is a cost-effective adaptation measure. Since costs and benefits are time-dependent then it follows that the benefit-to-cost ratio is time-dependent. Thus, an adaptation measure may not be cost-effective in the short-term, due to high adaptation cost for example, but the benefits may accrue over time resulting in improved cost-effectiveness in the longer-term. Note that the additional cost of repair for concrete with a higher cover is treated herein as an adaptation cost, and not as a reduced benefit.

The analysis assumes that many input variables are random variables (see Table 6) and so the output of the analysis (BCR) is also variable. This allows the mean BCR and the probability that an adaptation measure is cost-effective Pr(BCR>1) to be calculated. These criteria are used to evaluate the cost-effectiveness of adaptation strategies. Monte-Carlo simulation analysis is used as the computational tool to propagate uncertainties through the cost-benefit analysis, although analytical methods could also be used – e.g., (Stewart & Melchers, 1997).

For all 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, costs are expressed in 2013 euros. Note however, that Eqns. (12-14) show that BCR is not dependent on the monetary units, but it is a function of the ratio of damage that is related to adaptation costs. It is assumed that design and inspection costs are similar for different adaptation measures and so are not needed for this comparative analysis. Hence, adaptation strategies will only affect the expected damage costs. As we are concerned about outdoor exposures then the external RC structural elements of interest are slabs, beams and columns. Corrosion damage is assumed to occur on one (exposed) face of a slab and beam, and all faces of a column.

5.1 Cost of Damage (C_{damage})

The cost of repair or replacement and associated user losses, etc. are considerable and for some structures user losses are often much greater than direct repair, replacement and maintenance costs. Val & 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. The estimated cost for concrete patch repair using ordinary Portland cement is 286€/m^2 (BRE, 2003; Mullard & Stewart, 2012; Yunovivh et al., 2001). 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. However, for bridges closure of one lane for a four lane bridge can cause user delay costs of 39,650€ per day (Yunovivh et al., 2001). To allow for a minor user disruption cost (Bastidas-Arteaga & Stewart, 2013, 2015) assumed that the total failure cost was C_{damage}=325€/m^2 (approximately \$450-500/m^2 in United States dollars).

This study considers costs estimated for the repair of RC slabs and beams of the Agri-foodstuffs terminal of the Nantes Saint-Nazaire Port (Srifi, 2012). Table 2 summarises the costs of damage that include costs of damage reports, preparation of the building site, repair (removal and reconstruction of the cover) and operating losses. These costs were computed taking into account recent maintenance operations for three repair alternatives:

- *Repair 1*: it is a preventive repair strategy in which the structure is repaired before corrosion

initiation. Chloride-contaminated concrete cover is repaired by removing 4 cm of material for slabs and beams. Corroded bars are not replaced.

- *Repair 2*: it 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. Cracked/chloride-contaminated concrete cover is repaired by removing about 6 cm of material for slabs and beams. Corroded bars are not replaced.

- *Repair 3*: it is a corrective repair strategy in which repair takes place after severe concrete cracking where the loss of cross-sectional area of rebars is significant. Cracked/chloride-contaminated concrete cover is repaired by removing about 6 cm of material for slabs and beams. Corroded bars are replaced.

It is noted from Table 2 that most of the costs are related to concrete removal and repair operations, followed by the costs of studies and preparation of the building site and operation losses. For the alternative 3, the replacement of corroded steel requires a longer repair time and so increasing operational losses.

5.2 Cost of Adaptation (C_{adapt})

The baseline case for construction cost per unit volume (C_{cv}) including forms, concrete, reinforcement, finishing and labour is approximately 490-850€/m^3, 910-1010€/m^3 and 780-1560€/m^3 for RC slabs (4.6-7.6 m span), RC beams (3.0-7.6 m span) and RC columns (300 mm × 300 mm to 900 mm × 900 mm), respectively (RSMeans, 2012). These values will therefore be used to estimate the costs of the two adaptation strategies.

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 extra volume of concrete needed. Since C_{damage} units are €/m^2 of surface area, but C_{cv} is given as per unit volume, then 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 dimension such as slab depth or beam or column width (D). Table 3 describes the data and the relationships used to evaluate the adaptation costs. Identical formulations apply for RC square and circular columns where D is the column width or diameter.

Based on the information given in Table 3, Table 4 provides the adaptation costs for various structural elements (per mm of extra cover). This table also presents the adaptation costs for 5 and 10 mm increase in extra cover. Clearly, adaptation costs are higher for a square column where damage can occur on all four faces and then cover should be increased on all faces.

5.3 Discount Rates

There is some uncertainty about the level of discount rate, particularly for climate change economic assessments (e.g. (Dasgupta, 2008)). France used a discount rate of 8% to evaluate public investments from 1985 to 2005. However, following the 2005 Lebègue Report (Lebègue et al., 2005), the 'Commissariat Général au Plan' has recommended a 4% discount rate for short term investments and a lower discount rate of 2% for cash flows occurring after more than 30 years (Gollier, 2012). These discount rates were revised in 2013 by the 'Commissariat Général à la Strategie et à la Prospective' recommending 2.5% and 1.5% discount rates for short term (lifetime lower than 70 years) and long term investments, respectively (Quinet, 2013). Quinet (2013) also recommends carrying out a sensitivity analysis with a 4.5% discount rate to compare new and old approaches. Countries and institutions worldwide use other discount rates. The European Commission recommends a 5% discount rate (Harrison, 2010). Infrastructure Australia recommends discount rates of 4%, 7% and 10% for infrastructure projects (IA, 2008). Other discount rates vary from 3% (Germany) to over 10% (World Bank) (Harrison, 2010).

Discount rates are generally assumed constant with time. However, this may not be appropriate when considering intergenerational effects often associated with climate change policy decisions (e.g. (Boardman et al., 2011)). Projects with significant effects beyond 30-50 years are considered intergenerational, and so for example, the British Treasury recommends the following time-declining discount rates (HM Treasury, 2003): 3.5% (0-30 years), 3.0% (31-75 years), 2.5% (76-125 years),

2.0% (126-200 years), 1.5% (201-300 years), and 1.0% (300+ years). However, there is some controversy about time-declining discount rates (e.g. (Viscusi, 2007)), and the Australian Office of Best Practice and Regulation (OBPR) states that 'there is no consensus about how to value impacts on future generations' and 'Rather than use an arbitrarily lower discount rate, the OBPR suggests that the effects on future generations be considered explicitly' (OBPR, 2010). Nonetheless, the 2006 U.K. Stern Review adopted a discount rate of 1.4% (Stern, 2006), and the Australian Garnaut Review adopted discount rates of 1.35% and 2.65% (Garnaut, 2008). These relatively low discount rates were selected to not underestimate climate impacts on future generations. However, others suggest higher discount rates when assessing economic impacts of climate change (e.g, (Nordhaus, 2007)).

The above quantification of discount rates relates mainly to public-sector investments in infrastructure. Private investments in infrastructure, such as the owners of a port, power station, airport, etc., tend to include a risk premium which leads to a higher discount rate (BTRE, 2005). The marginal rate of return of private investments is suggested as one method to derive discount rates for private investments (e.g. (Boardman et al., 2011)). According to Boardman et al. (2011), the best 'proxy' for marginal rate of return of private investments is the before-tax rate of return on corporate bonds - or approximately 4.5%.

In the present paper, 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 representative of values used to consider intergenerational and climate change effects.

6 ILLUSTRATIVE EXAMPLE

6.1 Problem Description

This example illustrates the probabilistic assessment of the cost-effectiveness of adaptation strategies for existing RC structures placed in two coastal cities in France (Saint-Nazaire and Marseille) under a splash and tidal exposure. Table 5 describes the design cover according to the recommendations of French standards for structures built between 1970 and 2010 (Figure 4). For structures built in 2010, the cover was selected for a splash and tidal zone (XS3 exposure (European standard, 2004)) and a rebar diameter of d_0=16 mm. It is also assumed that if severe concrete cracking is detected when there is an evolution of the construction standard, the concrete cover is increased according to standard recommendations at the time of repair. For example, a 1970 structure damaged in 2015 is repaired to the 2015 specified design cover. This study does not focus on a specific structure. Therefore, there is no information about the characteristics of the concrete used to build the existing structures. Besides, there is no information about the repair materials that will be used in the future. For simplicity, this paper assumes that the same concrete was used for the construction and repair of all structures during their lifetime. This concrete has a characteristic compressive strength, f'_{ck}= 35 MPa, as recommended by the Eurocode 2 for XS3 exposure (European standard, 2004).

Other assumptions are summarised as follows:
- the structural lifetime is T_t=100 years,
- the limit crack width for repair is w_{lim}=1 mm,
- the environmental chloride concentration, C_{env}, corresponds to a XS3 exposure,
- all structural components will be subjected to the same environmental conditions,
- the chloride ingress is one-dimensional,
- the time of adaptation will vary between 2020 and 10 years before the end of the structural lifetime (Table 5),
- the adaptation strategy consists of increasing concrete cover by 5 or 10 mm with respect to standard recommendations,
- the time for assessment of costs is t_{assess} = 2013,
- the costs of damage are: C_{damage}=263.2€/m^2, C_{damage}=323€/m^2, and C_{damage}=353.4€/m^2 (Table 2).
- the adaptation costs are defined according to Table 4, and
- the discount rates are 2, 4 and 8%.

The probabilistic models used to estimate damage probabilities ($p_{s,i}$) are given in Table 6. It is assumed that all the random variables are statistically independent. Monte Carlo simulations were used for the assessment of damage probabilities and the propagation of uncertainties throughout the cost-benefit analysis. For more details of the deterioration models used herein see Bastidas-Arteaga et al. (2011, 2013).

6.2 Results

6.2.1 *Damage probabilities and climate change effects*

Figure 5a depicts the time-dependent probability of severe cracking for the RCP 4.5 climate change scenario and structures built under different construction standards in Saint-Nazaire and Marseille. Although the concrete properties and cover are the same for both cities, probabilities of severe cracking are larger for structures built in Saint-Nazaire. As indicated in Section 2, RH is about 8% larger in Saint-Nazaire than Marseille by increasing water content in the capillary pores and chloride diffusion rate. This higher RH will therefore shorten the time to corrosion initiation by increasing the probability of severe cracking. It is also observed the effects of the evolution of construction standards that, for a given time after construction, decrease the probabilities of damage. These results indicate that although increasing design cover is an effective protection for reducing damage induced by chloride ingress, the effectiveness of this measure depends on specific exposure conditions.

For Marseille, climate change projections show increases of temperature of 1.8 and 3.5ºC by 2100 with respect to year 2000 levels for the RCP 4.5 and RCP 8.5 climate change scenarios, respectively. Figure 5b shows the effects of these climate change scenarios on the probability of severe cracking. As expected, probabilities of severe cracking are larger for RCP 8.5 exposure because higher temperatures accelerate chloride ingress and corrosion propagation (Bastidas-Arteaga et al., 2011). The effects of the RCP 8.5 scenario are larger for recent structures because the differences in temperatures between both climate change scenarios announced by general circulation models increase after 2050 (Figure 3). As a consequence, for structures built in 1970 the difference of temperature between both climate change scenarios is about 0.7 ºC (in 2070) whereas for recent structures (built in 2010), this difference is 1.7ºC (in 2110). From an engineering point of view, RCP 8.5 and RCP 4.5 climate change scenarios could be interpreted respectively as upper of lower bounds for carrying out sensitivity studies. This point is illustrated later in the assessment of the cost-effectiveness of adaptation measures.

6.2.2 *Cost-effectiveness of damage adaptation strategies*

This section illustrates the probabilistic assessment of the cost-effectiveness of adaptation strategies for existing RC structures. These results were computed for a discount rate r =4% and a damage cost C_{damage}=323€/m^2. According to Table 2, this damage cost corresponds to a medium damage extent that does not require replacement of corroded steel. Sensitivity studies about the effects of discount rate and damage costs will be presented in sections 6.2.3 and 6.2.4, respectively.

Monte Carlo simulations were used for estimating the risk reduction due to the implementation of adaptation measures, ΔR. Figure 6 shows the effects of the two adaptation strategies (Δc_{adapt}=5mm and Δc_{adapt}=10mm) on the probability of ΔR for structures built in Saint-Nazaire in 2010 under the RCP 4.5 scenario and three adaptation times. The most likely risk reduction corresponds to the case where there is no risk reduction for the adaptation strategies (ΔR = 0%). A zero risk reduction arises because: (i) there is no repair during the structural lifetime, or (ii) the repair schedule is the same for existing and adaptation maintenance strategies. It is also noted that likelihood of ΔR reduces when the adaptation time is larger. For example, the remaining structural lifetime is half of the total lifetime for t_{adapt}=2060 and, therefore, the reduction of the number of repairs due to the implementation of adaptation measures will be lower than the cases where t_{adapt} is smaller. In addition, discount effects reduce the benefit of future repairs by decreasing ΔR. It is also observed in Figure 6 that the risk reduction is higher when Δc_{adapt} =10 mm, which is almost double that when Δc_{adapt} =5 mm.

The probabilistic analysis was also used to study the effect of the construction year on the mean ΔR for structures built between 1970 and 2010 and various adaptation years (Table 7). It is noted that in all studied cases the mean ΔR is larger when the adaptation is implemented sooner rather than later,

and Δc_{adapt}=10mm. Concerning the effect of the construction year, it is observed that the mean ΔR deceases for older structures. This is expected because for a structure lifetime of 100 years, the remaining lifetime of older structures is shorter (even if t_{adapt}=2020) and consequently there is not enough time to make the investment of the adaptation strategy profitable. The mean ΔR presented in Table 7 indicates that increasing cover depth to 10 mm is more effective to reduce risks. However, these results cannot be used directly for estimating the cost-effectiveness of adaptation measures because they do not include adaptation costs. The adaptation cost will be considered in the following BCR analysis.

Figure 7 presents the mean BCR for several structural components in Marseille and Saint-Nazaire under the RCP 4.5 scenario. These results were obtained for structures built in 2010 and t_{adapt} = 2020. Time of adaptation is taken as 2020 to represent the shortest practical time for a national standard (e.g., Eurocode 2 (European standard, 2004)) to consider changes to existing specification, recommend changes, and have changes implemented in future standards. The overall behaviour indicates that the mean of BCR is highly dependent on both the location and the type of structural component. The mean BCR is lower for Marseille and small structural components. For most structural components in Marseille the mean BCR is lower than 1 indicating that the adaptation strategies are not cost-effective. This means that, for the studied material, current design cover (55 mm) is cost-efficient for Marseille. On the opposite, adaptation strategies are cost-effective for all components in Saint-Nazaire. Thus, recommendations of current standards and adaptation measures could be more or less adapted to local climate conditions. It is also noted that the mean BCR decreases for small structural components for which the adaptation cost is relatively higher (e.g., Table 4). For both locations, increasing extra cover by 10 mm is less cost-effective than a 5 mm increase in cover. Even if risk reduction ΔR is larger for Δc_{adapt}=10 mm (Table 7), the costs associated to this adaptation strategy are larger by reducing the mean BCR. Thus, it is possible to conclude that the mean BCR for the whole structure could be maximised by performing different actions for individual components: (i) optimising the extra cover, (ii) considering different types of adaptation strategies, and/or (iii) doing nothing.

The mean BCR is also influenced by the year of construction. To explain this relationship, Figure 8 depicts the mean BCR as a function of the construction year for slabs built in Saint-Nazaire and t_{adapt}=2020. It is noted that the BCR is lower than one for older structures and increases for recent ones. A BCR<1 implies that the adaptation measure is not cost-effective for old structures and that the existing standards recommendations are most cost-effective during the structural lifetime. The increment of BCR is due, on the one hand, to the increase of concrete cover recommended by the standards and/or considered by the adaptation measures. This means that larger concrete cover is more effective for this kind of exposure. On the other hand, larger BCR values are also related to the increase of climate change effects on deterioration rates that justify the implementation of adaptation measures. As mentioned in previous results, the adaptation strategy Δc_{adapt}=10 mm is less cost-effective for all adaptation years. The following results will therefore focus on the repair strategy Δc_{adapt}=5 mm. Figure 8 also indicates the effects of the climate change scenario. It is noted that higher mean BCR are expected for the RCP 8.5 scenario that announces more important changes with respect to actual climate. The differences between results for both scenarios are larger for recent structures because they will be exposed to the largest climate variations that are more pronounced after 2050 (Figure 3). These climate variations will induce more corrosion damage by increasing the cost-effectiveness of adaptation strategies.

Figure 9 shows the effect of the time of adaptation on the mean BCR and Pr(BCR>1) for slabs (D=300mm) in Saint-Nazaire, Δc_{adapt}=5 mm and the RCP 4.5 scenario. It is noted that both the mean BCR and the Pr(BCR>1) decrease when the adaptation year is close to the end of the structural lifetime. Of interest is that the Pr(BCR>1) reaches only slightly above 60% when mean BCR exceeds 4. This illustrates the high variability of damage risks caused by uncertainties of climate projections, and variabilities of material, dimensional and deterioration parameters. These results could be used by an owner/stakeholder to evaluate the benefits and the risks of implementing adaptation strategies at various years. For example, it is observed that mean BCR and Pr(BCR>1) are small for older structures and therefore the owner/stakeholder could prioritise investments in adaptation measures for recent structures. These curves could be also used to evaluate the impact of the adaptation year. For example, for structures built in 2000, if the owner/stakeholder decides to postpone the adaptation

actions until 2040 the mean BCR is about 2.5 which is still interesting. However, the Pr(BCR>1) for this adaptation time is less than 0.3 indicating that the risks of having no benefits are larger.

6.2.3 *Effect of Discount Rate*

Figure 10 describes the influence of discount rates (r) on both the mean BCR and the Pr(BCR>1) for slabs (D=300mm) built in several years in Saint-Nazaire and t_{adapt}=2020. It is observed that the mean BCR and Pr(BCR>1) are very sensitive to r and both parameters are larger for small discount rates. This is explained by the fact that small discount rates imply that future costs are larger at present cost by increasing the cost-effectiveness of adaptation measures for repairs close to the end of the structural lifetime. As discussed in Section 5.3, various governments recommend lower discount rates of about 2% for long-term investments. The probabilistic BCR analysis therefore shows that the adaptation strategies are more cost-effective according to these recommendations. The Pr(BCR>1) could be also used to determine a period for a 'break-even probability' implying a given level of risk. For instance, a break-even probability Pr(BCR>1)=50% implies risks of having 50% benefits or losses. It is noted that this level of risk (Pr(BCR>1)=50%) is reached when the age of the structure varies between 1993 and 1998 for all considered discount rates.

6.2.4 *Effect of Damage Costs*

Damage costs are structure and site specific and depend on many factors such as: the extent of damage, material properties, structural configuration, environmental aggressiveness, climate change effects, inspection interval, time without repair, etc. Previous results focused on a damage cost (C_{damage} = 323€/m^2) that represented a medium extent of damage. This section presents a sensitivity study for lower and higher values of C_{damage} related to lower and higher extents of damage (Table 2). Figure 11 shows the influence of C_{damage} on the mean BCR and probability of BCR for slabs (D=300mm) in Saint-Nazaire, r = 4%, and t_{adapt}=2020. It is observed that the mean BCR increases for larger damage costs that are directly related to deeply concrete removal (Repair 2) and the replacement of corroded rebars (Repair 3). This is expected because the adaptation cost remains constant for the same damage level whereas E$_{damage-BAU}$ depends directly on C_{damage} (Eq. (14)). This means that the adaptation measures are more cost-effective for larger extent and consequences of damage. Concerning the effects of C_{damage} on Pr(BCR>1), it was found that for this case the Pr(BCR>1) is not affected by a variation of C_{damage}. The Pr(BCR>1) for the three values of C_{damage} as a function of the construction year, corresponds to the case r = 4% presented in Figure 10. To explain this, Figure 11 also presents the cumulative probability of BCR for the three values of C_{damage} and $t_{construct}$ =1990. It is observed that C_{damage} influences the mean BCR but does not affect the Pr(BCR>1). For this example, a larger C_{damage} increases the mean BCR but the risks of a good investment are not affected by this parameter. However, Pr(BCR>1) depends on C_{damage} when the mean BCR is close to 1.

7 CONCLUSIONS

The kinematics of chloride ingress and corrosion propagation mechanisms is highly influenced by the surrounding environmental conditions including climate change. Therefore, it is possible to expect that climate change will accelerate or decelerate these deterioration processes depending on specific exposure and environmental conditions. This paper focused on climate adaptation for existing RC structures built at different years (and therefore under different durability standards), and subjected to two different types of climate in France. The adaptation strategies consisted of increasing the cover recommended in the standards by 5 or 10 mm.

The assessment of the mean BCR and the Pr(BCR>1) indicated that although increasing design cover is an effective protection for reducing damage induced by chloride ingress, the cost-effectiveness of this measure depends on specific exposure conditions. An adaptation strategy consisting of increasing design cover by 5 mm was cost-effective for Saint-Nazaire but it was not cost-effective for Marseille. The probabilistic analysis also indicated that the mean BCR and Pr(BCR>1) are dependent on type of structural element, age of construction, adaptation time, damage costs and discount rates. The overall results indicate that the probabilistic framework is well suited to

assessing the impact of climate change on RC corrosion damage, and assessing the cost-effectiveness of climate adaptation strategies. Future work will consider other adaptation strategies such as improved concrete quality and coatings.

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Table 1. Correspondence between Eq. (1) and the governing differential equations.

Physical Process	ψ	ζ	J	J'
Chloride ingress	C_{fc}	1	$D_c^* \vec{\nabla} C_{fc}$	$C_{fc} D_h^* \vec{\nabla} h$
Moisture diffusion	h	$\partial w_e / \partial h$	$D_h^* \vec{\nabla} h$	0
Heat transfer	T	$\rho_c c_q$	$\lambda \vec{\nabla} T$	0

Table 2. Costs of damage of the Agri-foodstuffs terminal of the Nantes Saint-Nazaire Port (Srifi, 2012).

Item	Repair 1 (€/m²)	Repair 2 (€/m²)	Repair 3 (€/m²)
Damage reports, site installation, scaffolding	65.0	76.1	76.3
Concrete removal	62.7	71.8	71.8
Repair[a]	111.6	134.3	135.0
Operating loss	12.5	29.4	58.9
Other costs	11.4	11.4	11.4
Total	263.2	323.0	353.4

[a]Repair includes only direct costs associated to concrete cover rebuilding and replacement of corroded bars for the repair alternative 3

Table 3. Data and relationships for the assessment of adaptation costs.

	Slabs	Beams	Columns
D (mm)	100 to 300	200 to 800	300 to 900
C_{cv} (€/m³)	490 to 850	980	780 to 1560
C_c (€/m²)	$C_{cv} \times D$ (m)	$C_{cv} \times D$ (m)	$C_{cv} \times D^2/4D$ (m)
C_{adapt}[a] (€/m²)	$C_c \times 1/D$ (mm)	$C_c \times 1/D$ (mm)	$C_c \times 4/D$ (mm)

[a]Per mm of extra cover

Table 4. Adaptation costs for various structural elements.

Structural element	D (mm)	C_{adapt}[a] (€/m²)	Δc_{adapt} =5 mm (€/m²)	Δc_{adapt} = 10 mm (€/m²)
Slabs - small	100	0.85	4.23	8.45
Slabs - large	300	0.49	2.44	4.88
Beams	200 to 800	0.98	4.88	9.75
Sq. columns - small	300	1.56	7.80	15.60
Sq. columns - large	600	1.01	5.01	9.75

[a]Per mm of extra cover

Table 5. Design cover and adaptation years for structures built between 1970 and 2010.

Construction Year	End of lifetime	Standard	Design cover (mm)	Adaptation years
1970	2070	Circular 14/11/1964	40	[2020:10:2060]
1980	2080	Circular 14/11/1964	40	[2020:10:2070]
1990	2090	BAEL 91	50	[2020:10:2080]
2000	2100	BAEL 91	50	[2020:10:2090]
2010	2110	Eurocode 2	55	[2020:10:2100]

Table 6. Probabilistic models of the random variables.

Variable	Units	Distribution	Mean	COV	Reference
Reference chloride diffusion coefficient, $D_{c,ref}$	m²/s	log-normal	3×10^{-11}	0.20	(Duracrete, 2000a; Saetta et al., 1993; Val & Trapper, 2008)
Environmental chloride concentration, C_{env}	kg/m³	log-normal	7.35	0.20	(Duracrete, 2000a; Vu & Stewart, 2000)
Concentration threshold for corrosion initiation, C_{th}	wt% cem.	normal[a]	0.5	0.20	(Bastidas-Arteaga & Schoefs, 2012; Duracrete, 2000a)
Cover thickness, c_t	mm	normal[b]	Table 5	0.25	(European standard, 2004; Val & Stewart, 2003)
Reference humidity diffusion coefficient, $D_{h,ref}$	m²/s	log-normal	3×10^{-10}	0.20	(Saetta et al., 1993; Val & Trapper, 2008)
Thermal conductivity of concrete, λ	W/(m°C)	beta on [1.4;3.6]	2.5	0.20	(Neville, 1981)
Concrete specific heat capacity, c_q	J/(kg°C)	beta on [840;1170]	1000	0.10	(Neville, 1981)
Density of concrete, ρ_c	kg/m³	normal[a]	2400	0.04	(JCSS (Joint committee of structural safety), 2001)
Pitting factor, α	-	gumbel	5.65	0.22	(Val & Stewart, 2003)
Reference corrosion rate, $i_{corr,20}$	μA/cm²	log-normal	6.035	0.57	(Duracrete, 1998)
28 day concrete compressive strength, $f_c(28)$	MPa	normal[a]	$1.3(f'_{ck})$	0.18	(Pham, 1985)
Concrete tensile strength, f_{ct}	MPa	normal[a]	$0.53(f_c)^{0.5}$	0.13	(Mirza et al., 1979)
Concrete elastic modulus, E_c	MPa	normal[a]	$4600(f_c)^{0.5}$	0.12	(Mirza et al., 1979)

[a]truncated at 0, [b]truncated at 10mm

Table 7. Effect of the construction year on the mean ΔR for structures built in Saint-Nazaire under the RCP 4.5 scenario.

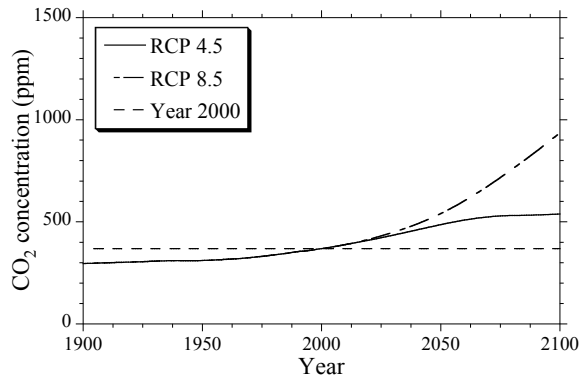| Adaptation year	$t_{costruct}$ = 1970		$t_{costruct}$ = 1990		$t_{costruct}$ = 2010	
	Δc_{adapt}=5 mm	Δc_{adapt}=10 mm	Δc_{adapt}=5 mm	Δc_{adapt}=10 mm	Δc_{adapt}=5 mm	Δc_{adapt}=10 mm
2020	0.4%	0.7%	2.3%	4.2%	3.1%	5.7%
2030	0.1%	0.2%	1.5%	2.6%	3.0%	5.5%
2040	0.0%	0.0%	0.8%	1.3%	2.5%	4.6%
2050	0.0%	0.0%	0.4%	0.6%	1.9%	3.3%
2060	0.0%	0.0%	0.1%	0.2%	1.1%	1.9%
2070	-	-	0.0%	0.0%	0.5%	0.9%
2080	-	-	0.0%	0.0%	0.2%	0.2%
2090	-	-	-	-	0.0%	0.0%

Figure 1. Projected annual average CO_2 concentration for RCP emission scenarios.

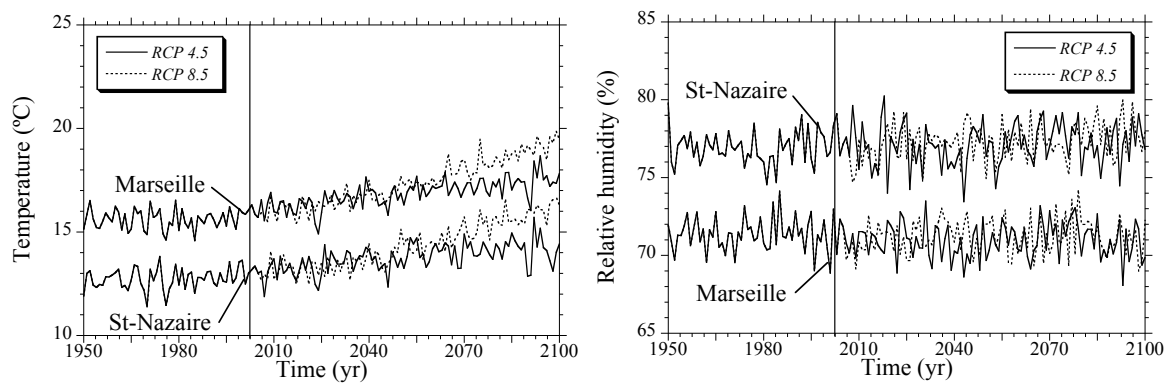Figure 2. Studied locations in France.

Figure 3. Yearly temperature and relative humidity projections for Saint-Nazaire and Marseille.
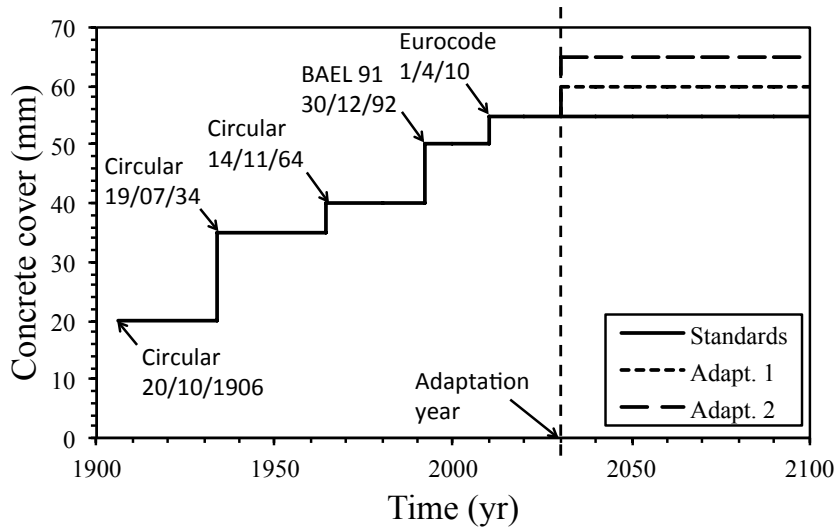
Figure 4. Recommended concrete cover and adaptation measures for RC structures subjected to chloride ingress in a splash and tidal zone.
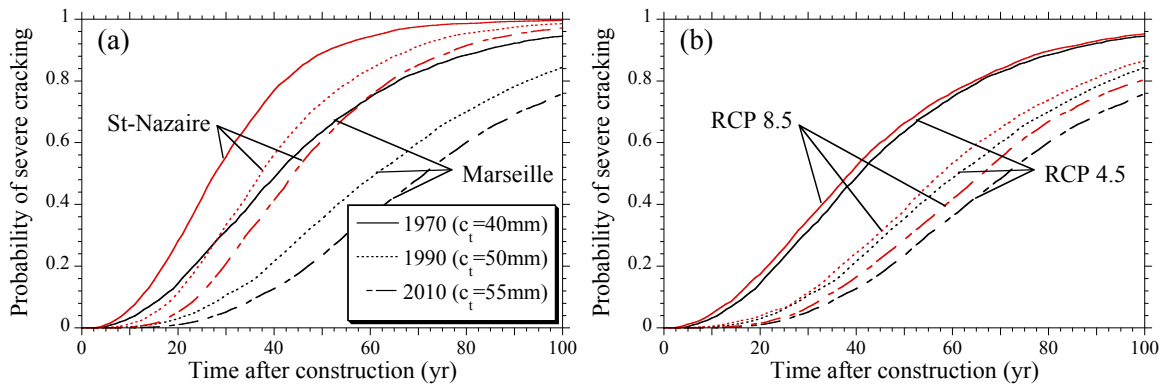

Figure 5. Probability of severe cracking for: (a) structures built in Saint-Nazaire and Marseille under the RCP 4.5 scenario, and (b) structures built in Marseille under RCP 4.5 and RCP 8.5 scenarios.
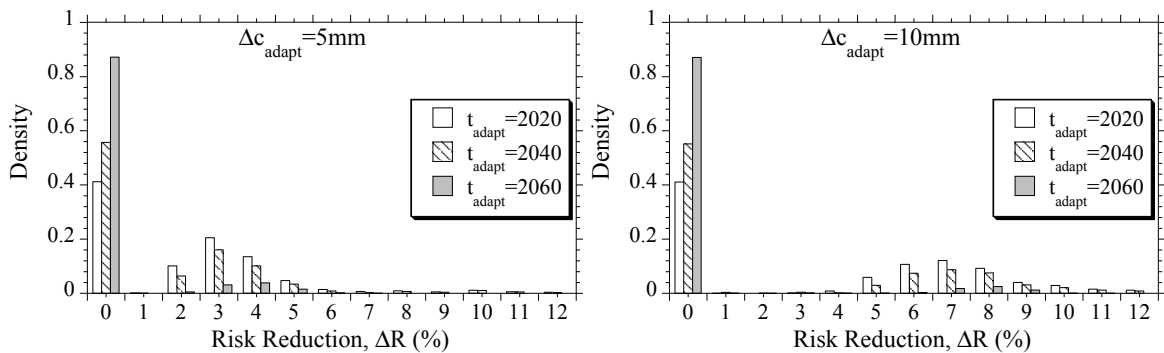
Figure 6. Probability histograms of risk reduction for structures built in Saint-Nazaire in 2010 under the RCP 4.5 scenario.
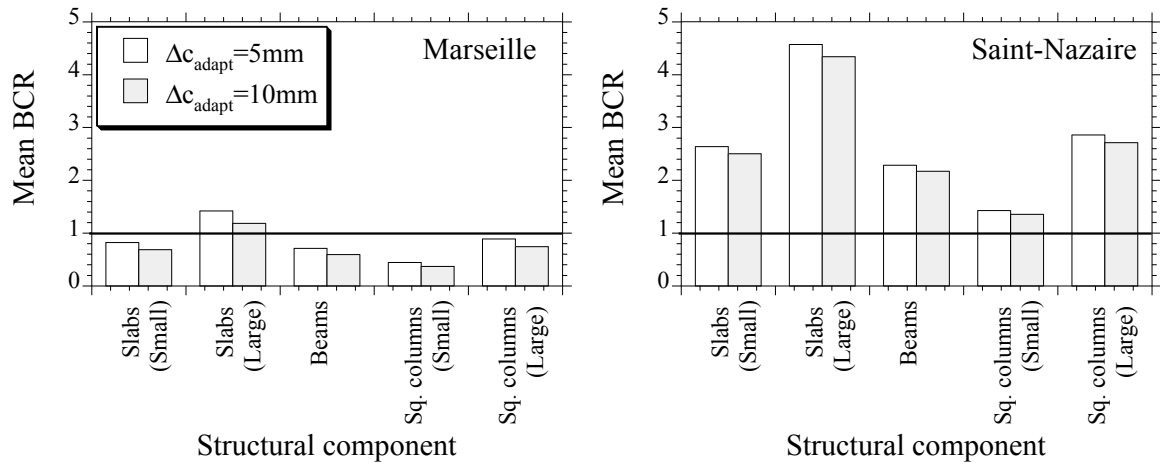
Figure 7. Mean BCR for structural components in Marseille and Saint-Nazaire under a RCP 4.5 climate change scenario, for structures built in 2010 and t_{adapt} = 2020.
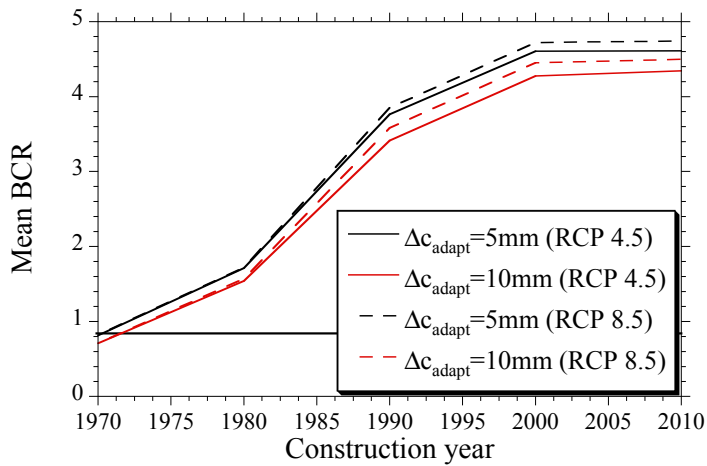
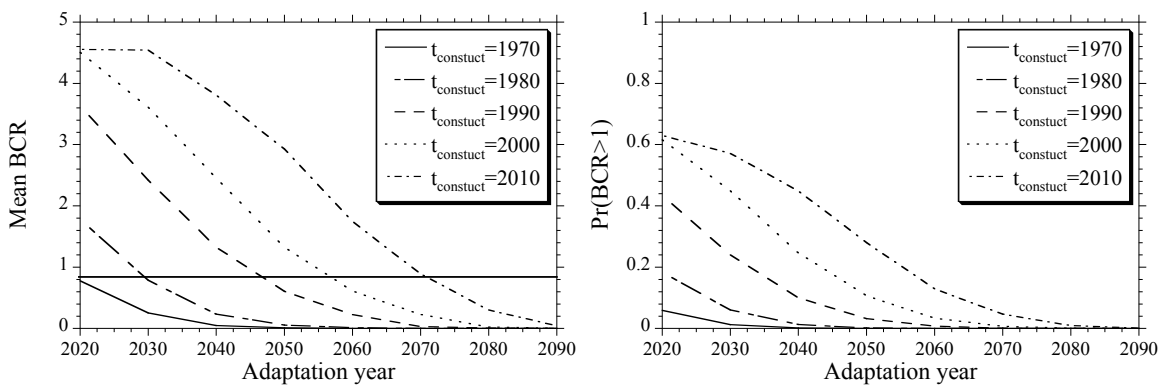Figure 8. Mean BCR for slabs (D=300mm) built in several years in Saint-Nazaire and t_{adapt}=2020.

Figure 9. Effect of the adaptation year on the mean BCR and Pr(BCR>1) for slabs (D=300mm) built at several years in Saint-Nazaire, RCP 4.5 scenario and Δc_{adapt}=5 mm.
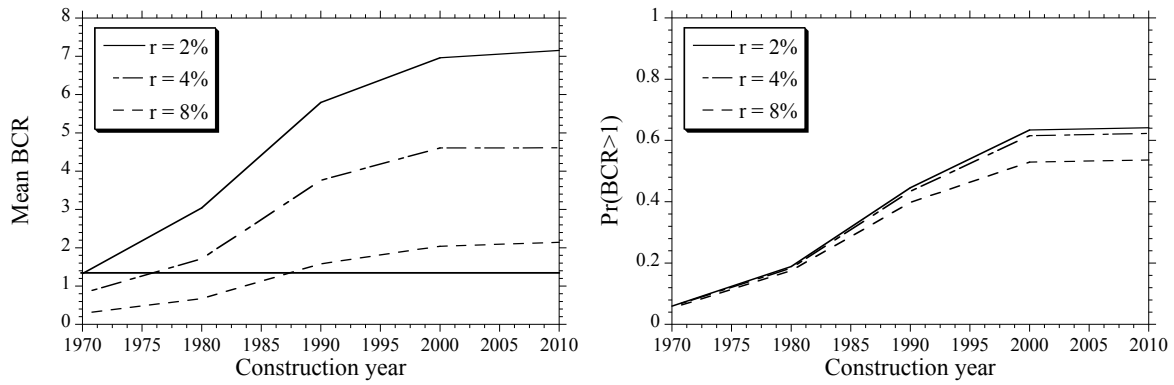
Figure 10. Effect of discount rate on the mean BCR and the Pr(BCR>1) for slabs (D=300mm) built in several years in Saint-Nazaire and t_{adapt}=2020.
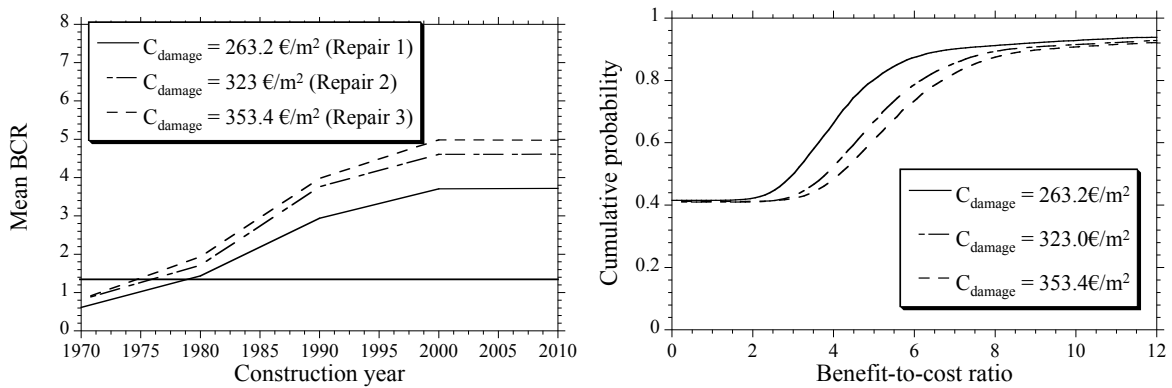

Figure 11. Effect of damage costs on the mean BCR and the cumulative probability of BCR for slabs (D=300mm) built in several years in Saint-Nazaire, r=4%, and t_{adapt}=2020.