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## Cost-effective adaptation to address climate change impacts

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## Chapter 22

### Title: Cost-effective adaptation to address climate change impacts

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#### Abstract

Concrete is the predominant construction material for buildings, bridges, wharves, and other infrastructure worldwide. A potentially important factor for asset management is the possible influence of climate change. This may alter the environment to which infrastructure is exposed, and in turn may alter the factors known to affect the corrosion of reinforcing steel, including atmospheric CO<sub>2</sub> concentration, temperature, humidity, ocean acidification, airborne pollutants, etc. This chapter poses the problem of adaptation of deteriorating RC structures. It describes a framework for evaluating the cost-effectiveness of adaptation measures that accounts for deterioration models, probabilistic methods and cost-benefit analysis. The methodology is illustrated with an example focusing on cost-effective adaptation of existing RC structures placed in the coastal French cities of Saint-Nazaire and Marseille.

**Key words:** Reinforced concrete, adaptation, cost-benefit analysis, corrosion, reliability

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## 22.1. Introduction

### 22.1.1. Background

Concrete is the predominant construction material for buildings, bridges, wharves, and other infrastructure in Europe, Australia and elsewhere. A potentially important factor for asset management is the possible influence of climate change. This may alter the environment to which infrastructure is exposed, and in turn may alter the factors known to affect the corrosion of reinforcing steel, including atmospheric CO<sub>2</sub> concentration, temperature, humidity, ocean acidification, airborne pollutants, etc.

A rise in temperature will increase the rate of infiltration of deleterious substances (increased material diffusivity) and increase the corrosion rate of steel. Changes in relative humidity levels may also increase the rate of infiltration of deleterious substances (Nguyen et al. 2017; Stewart et al. 2011). Typically these parameters must be considered as random variables or stochastic processes, and their statistical characteristics will gradually change with time. An appropriate framework for dealing with this, and other climate adaptation problem, is structural reliability and risk-based decision analysis considering updating with inspection data (Stewart and Deng 2015; Tran et al. 2016a; b).

Several studies have used probabilistic or deterministic methods to assess climate change effects on durability of RC structures placed in several countries (Bastidas-Arteaga et al. 2010, 2013; de Larrard et al. 2014; Stewart et al. 2011; Talukdar et al. 2012; Wang et al. 2012). These researches found that the temporal and spatial effects of a changing climate can increase current chloride ingress or carbonation-induced damage risks and justify the need for comprehensive adaptation. Thus, Stewart & Peng (2010) used a simplified carbonation model and global IPCC (2007) CO<sub>2</sub> 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, and later work in this area proved inconclusive (Peng and Stewart 2015).

### 22.1.2. Aims and Scope

Within this context, the main objectives of this chapter are:

- to propose a framework for probabilistic cost-benefit analysis of adaptation measures to ameliorate chloride-induced corrosion in new and existing structures,
- to apply the methodology to structures subjected to various environmental conditions and/or located in specific locations in France.

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.

Section 22.2 poses the problem of adaptation of deteriorating RC structures. Section 22.3 describes the proposed framework for evaluating the cost-effectiveness of adaptation measures. It accounts for deterioration models, probabilistic methods and cost-benefit

analysis. Section 22.4 presents the repair and adaptation strategy that will be used in the illustrative example (Section 22.5). The example focuses on adaptation of existing RC structures placed in the coastal French cities of Saint-Nazaire and Marseille.

## 22.2. Adaptation of RC structures

Climate adaptation in terms of the enhancement of adaptive capacity can be done by developing new technologies and materials to counter the impact of increasing corrosion risk under a changing climate. From a practical point of view, three questions about the problem of adaptation of structures emerge:

- which measure could be used to adapt?,
- when is the best time to implement adaptation measures?, and
- how to measure the cost and effectiveness of adaptation measures?.

### 22.2.1. Adaptation Measures

There is a wide range of existing and ‘low-tech’ options that can enhance the durability of concrete structures and these can be applied to reduce the adverse affects of climate change. The design options generally include the selection of cover, concrete mix, surface coating barriers, extraction, and cathodic protection. In addition to reducing environmental exposure as much as possible, practical adaptation solutions in a new design may come from increasing cover and strength grade, or any approaches that reduce material diffusion coefficient without compromising the reliability and serviceability of concrete. Adaptation measures for new and existing concrete structures may include:

- Surface treatments,
- Realkalization,
- Extra design cover,
- Increase concrete durability,
- Stainless or galvanised steel reinforcement,
- Corrosion inhibitors,
- Cathodic protection, and
- Replace existing cover with new concrete.

### 22.2.2. Adaptation Time

The time of adaptation is highly variable and dependent on extent and location of corrosion damage. Some adaptation strategies could be applied at time of construction (coatings/surface treatments, reinforcement), and others at time of corrosion initiation (realkalization, chloride extraction). Clearly, it is preferable to use adaptation strategies that are implemented during design and construction rather than in-service (e.g., when corrosion damage occurs) as the latter will be much more costly in terms of direct costs and inconvenience/user delays and other indirect costs (Stewart et al. 2012).

### 22.2.3. Cost-Effectiveness of Adaptation Strategies

Adaptation strategies will have varying degrees of effectiveness and cost. Some will require regular maintenance over the life of the structure, such as surface treatments, which will increase their lifecycle cost. Given that there are many millions of new and existing concrete infrastructure in many countries the cost of adaptation can be immense. For this reason, a risk-based approach is needed to assess the optimal level, if any, of adaptation measures. This includes the cost, location, timing and extent of adaptation measures.

An increase in cover thickness can increase the time of carbonation and chloride ingress to reach concrete reinforcement and in turn delay carbonation and chloride-induced corrosion. It is therefore one of the most obvious and simplest adaptation options in the design of concrete infrastructure under a changing climate to maintain structural durability and serviceability.

While the change of cover is considered as the most straightforward design approach to reduce the impact of changing climate, other options may also include the selection or design of concrete materials to reduce the diffusion coefficient of deleterious substances –i.e. slow the ingress of those substances, and hence delay corrosion of concrete reinforcement. In practice, selection of a higher strength grade of concrete is one approach to reduce the diffusion coefficient, in addition to enhancing its mechanical properties.

Stewart and Peng (2010) have conducted a preliminary lifecycle cost assessment to assess the cost-effectiveness of increasing design cover as an adaptation measure to mitigate the effects of carbonation of concrete. The lifecycle cost analysis considered 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<sub>2</sub> emission scenarios. The preliminary analysis found that lifecycle costs for the current situation (‘do nothing’ – use existing covers) are lower than lifecycle 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. However, a more detailed assessment of risks, costs, benefits of adaptation measures and environmental impact may reveal a different conclusion. Further work considered the effects of spatially variable corrosion damage, and the costs and benefits of adaptation measures (Peng and Stewart 2014, 2015). What is important is the need to estimate time-dependent changes in damage risks, the effectiveness and cost of one or more adaptation measures, costs of repairing damage, and other criteria needed to assess the optimal level of adaptation measures both now and into the future.

22.3. Proposed Adaptation Framework

The main practical problem concerning adaptation of structures lies in evaluating its costs and effectiveness. Figure 22.1 shows the proposed framework for dealing with this problem that combines:

- general methods that could be applied to any structure (deterioration models, stochastic approaches and cost-benefit analysis), with
- information specific to each structure (climate change predictions and structural characteristics).

The following sections focus on describing the general methods that are employed in the proposed framework. Information specific to each structure will be detailed in the numerical application (Section 22.5).

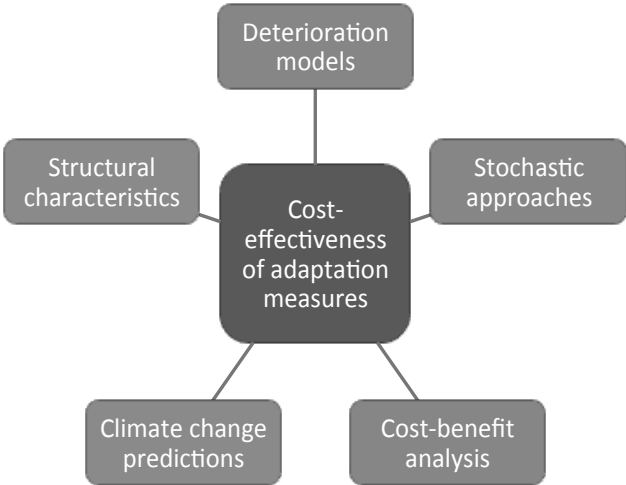


Figure 22.1 – Proposed framework for determining the cost-effectiveness of adaptation measures

### 22.3.1. Deterioration Models

As mentioned in (Bastidas-Arteaga et al. 2010; de Larrard et al. 2014; Stewart et al. 2014), the assessment of climate change consequences on RC durability requires deterioration models that account for the influence of surrounding environmental actions (temperature, relative humidity, rain, etc.). Table 22.1 summarises the characteristics of the models considered in this study. We will focus on modelling deterioration since construction (or repair) until severe concrete cracking. Thus, the whole lifetime will be divided into three stages (Table 22.1) that are modelled based on different principles. For the stage of corrosion immunity (i.e., before corrosion initiation), we use a finite element model that accounts for chloride ingress into concrete by diffusion and convection (Bastidas-Arteaga et al. 2011; Saetta et al. 1993). This model takes into account the influence of temperature and relative humidity variations on the chloride penetration process. The stage of active corrosion is modelled based on electrochemical principles and considers only the effects of temperature (Duracrete 2000). The stage of corrosion until severe concrete cracking is assessed by two models. The former concerns the time since corrosion initiation until the appearance of a first crack on the concrete surface (El Maaddawy and Soudki 2007). The second represents the time until reaching a threshold crack width  $w_{lim}$  (Mullard and Stewart 2011). Both models combine mechanical and electrochemical principles. The corrosion propagation models considered in the second stage that are influenced by temperature variations drive both models. Although corrosion propagation is widely influenced by relative humidity, nowadays there is not a model that accounts for this factor in a comprehensive manner.

**Table 22.1** – Main characteristics of deterioration models considered in this study

| Deterioration stages            | Physical phenomena             | Influencing environmental factors |
|---------------------------------|--------------------------------|-----------------------------------|
| Corrosion immunity              | Diffusion and convection       | Temperature, relative humidity    |
| Active corrosion                | Electrochemistry               | Temperature                       |
| Corrosion until severe cracking | Electrochemistry and mechanics | Temperature                       |

### 22.3.2. Stochastic Assessment of Time-Dependent Damage Risks

The cumulative distribution function for the time of first damage (corrosion initiation, severe concrete cracking, failure)  $T_{dam}$  in the period  $[0, t]$  for original concrete is:

$$p_s(0, t) = \Pr[t \geq T_{dam}] \quad [22.1]$$

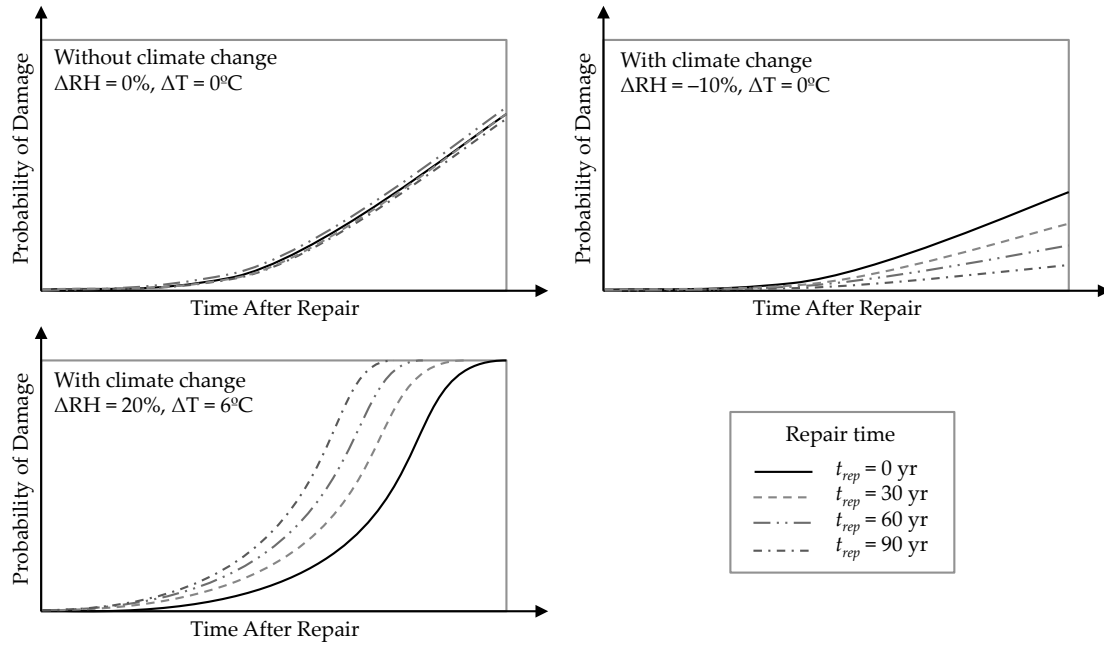
where depending on the asset maintenance policy the owner can specify the limit criteria for repair: threshold chloride content, limit crack width, etc. However, after repair, the time-dependent damage risks of the repaired structure will not be the same as the original one  $p_s(0, t)$  due to changed temperature and RH at the time of repair. Hence, the damage risk for repaired structures 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 and Stewart 2013, 2015; Nguyen et al. 2017; Stewart et al. 2014):

$$p_{s,i}(i\Delta t, t) = \Pr[t \geq T_{dam,i}] \quad [22.2]$$

where  $T_{dam,i}$  is the time to damage when the repaired structure is exposed to the environment for the first time after repair and  $\Delta t$  is the inspection length.

Figure 22.2 presents a conceptual description of the time-dependent probability of damage for a structure subjected to chloride-induced deterioration under various climate change scenarios. The climate change scenarios are simply represented as relative changes with respect to current climate conditions of a given location after 100 years. The overall trend indicates that the deterioration processes increase probability of damage with time. Figure 22.2 clearly shows that the rate of damage risk is highly dependent on climate change effects. If there is no change in climate ( $\Delta RH=0\%$  and  $\Delta T=0^\circ C$ ), the probability of damage increases with time and remains constant irrespective of time of repair. However, if climate change reduces the environmental relative humidity, i.e.  $\Delta RH= -10\%$  in 100 years, the chloride

ingress mechanism slows down, and consequently, the probability of severe cracking decreases. In this case, climate change has a ‘positive effect’ on RC durability by reducing corrosion damage risk. An opposite behaviour is observed when climate change increases the temperature and relative humidity ( $\Delta RH=20\%$  and  $\Delta T=6^\circ\text{C}$ ). In this case, the probability of damage increases if the structure is new or repaired at time  $t_{rep}$ . Therefore, the time-dependency of damage risks should be considered for a comprehensive cost-benefit analysis of adaptation measures.



**Figure 22.2** – Conceptual description of the probability of damage for various climate change scenarios and repair times

### 22.3.3. Cost-Benefit Analysis

#### 22.3.3.1. General Framework

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. The proposed cost-benefit framework could be applied to both new and existing structures. Costs and benefits are measured from the time of the cost benefit assessment,  $t_{assess}$ . For example, for an existing structure, 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. For a new structure  $t_{assess} = t_{construct}$ . If a 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.,

$$E_{damage}(T_t) = \sum_{n=1}^{T_t/\Delta t} \sum_{i=n}^{T_t/\Delta t} [p_{s,n}(i\Delta t) - p_{s,n}(i\Delta t - \Delta t)] \frac{C_{damage}}{(1+r)^{t_{construct} + i\Delta t - t_{assess}}} \quad [22.3]$$

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

where  $T_t$  is the service life (typically  $T_t=100$  years),  $\Delta t$  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  $n^{\text{th}}$  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<sup>2</sup>), and if the area of damage is known then repair cost can be estimated.

Eq. [22.3] 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. [22.3] can be re-expressed as:

$$E_{\text{damage}}(T_t) = \sum_{i=1}^{T_t/\Delta t} \Delta P_{s,i} \frac{C_{\text{damage}}}{(1+r)^{t_{\text{construct}} + i\Delta t - t_{\text{assess}}}} \quad [22.4]$$

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

where  $\Delta P_{s,i}$  is the probability of damage incident between the  $(i-1)^{\text{th}}$  and  $i^{\text{th}}$  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:

$$\Delta R(T_t) = \frac{E_{\text{damage-BAU}}(T_t) - E_{\text{damage-adaptation}}(T_t)}{E_{\text{damage-BAU}}(T_t)} \quad [22.5]$$

where  $E_{\text{damage-BAU}}(T_t)$  and  $E_{\text{damage-adaptation}}(T_t)$  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  $E_{\text{damage-adaptation}}(T_t)$  will be significantly lower than  $E_{\text{damage-BAU}}(T_t)$  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

$$E_{\text{adapt}}(T_t) = \sum_{i=1}^{T_t/\Delta t} \Delta P_{s,i} \frac{C_{\text{adapt}}}{(1+r)^{t_{\text{construct}} + i\Delta t - t_{\text{assess}}}} \quad \text{with } i\Delta t > t_{\text{adapt}} - t_{\text{construct}}, \quad t_{\text{construct}} < t_{\text{assess}} \quad [22.6]$$

where  $C_{\text{adapt}}$  is the cost of adaptation measures that reduces risk by  $\Delta R$  and  $t_{\text{adapt}}$  is the adaptation year.

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  $\text{BCR}(T_t)$  is:

$$\text{BCR}(T_t) = \frac{E_{\text{damage-BAU}}(T_t) \Delta R(T_t)}{E_{\text{adapt}}(T_t)} \quad [22.7]$$

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 (such as increasing cover on repaired concrete) is treated herein as an adaptation cost, and not as a reduced benefit.

Since the proposed adaptation framework considers uncertainties (Section 22.3.2), the output of the analysis (BCR) is also variable. This allows the mean BCR and the probability that an adaptation measure is cost-effective  $\text{Pr}(\text{BCR} > 1)$  to be calculated. These criteria are used to evaluate the cost-effectiveness of adaptation strategies. Monte-Carlo simulation analysis will be used as the computational tool to propagate uncertainties through the cost-benefit analysis, although analytical methods could also be used – e.g., (Stewart and 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 chapter, costs are expressed in 2013 euros. Note however, that Eqns. ([22.3] to [22.7]) 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.

#### 22.3.3.2. 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 this chapter, 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.

## 22.4. Considered Repair and Adaptation Measures

### 22.4.1. Definition

For illustrative purposes, this study considers patch repair as the most common technique to repair corrosion damage in RC structures – e.g., (BRE 2003; Canisius and 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  $\Delta t$ ;
- patch repair is carried out immediately after corrosion damage has been discovered at time of  $i^{\text{th}}$  inspection at time  $i\Delta t$ ;
- damage limit state exceedance 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  $n_{max} = T_t/\Delta t$ , where  $T_t$  is the total lifetime.

We consider increase in design cover as the climate adaptation measure aimed at reducing the impact of chloride-induced corrosion damage. Increasing the strength grade of concrete, which reduces the diffusion rate, is another possible climate adaptation measure. For more details of this adaptation measure see Bastidas-Arteaga and Stewart (2015, 2016) and Peng and Stewart (2015).

### 22.4.2. Repair and Adaptation Costs

#### 22.4.2.1. Cost of Damage ( $C_{\text{damage}}$ )

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<sup>2</sup> (BRE 2003; Mullard and 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 and Stewart 2013, 2015) assumed that the total failure cost was  $C_{\text{damage}}=325€/m^2$  (approximately \$450-500/m<sup>2</sup> in United States dollars).

On a more detailed cost analysis, (Bastidas-Arteaga and Stewart 2016) considered costs estimated for the repair of RC slabs and beams of the Agri-foodstuffs terminal of the Nantes Saint-Nazaire Port (Srifi 2012). Table 22.2 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.

**Table 22.2** – Repair strategies and corresponding costs of damage of the Agri-foodstuffs terminal at the Port of Nantes Saint-Nazaire

| Description of repair strategy | Type of cost in €/m2          |                  |        |                |             | Total |
|--------------------------------|-------------------------------|------------------|--------|----------------|-------------|-------|
|                                | Reports and site installation | Concrete removal | Repair | Operating loss | Other costs |       |
|                                |                               |                  |        |                |             |       |

|  |      |      |       |      |      |       |
|--|------|------|-------|------|------|-------|
| <i>Repair 1</i> : preventive maintenance where the structure is repaired before corrosion initiation. Actions: 4 cm concrete removal without replacement of corroded bars.   | 65   | 62.7 | 111.6 | 12.5 | 11.4 | 263.2 |
| <i>Repair 2</i> : corrective maintenance where repair takes place after severe concrete cracking but the loss of cross-sectional area of rebars is not significant. Actions: 6 cm concrete removal without replacement of corroded bars.   | 76.1 | 71.8 | 134.3 | 29.4 | 11.4 | 323   |
| <i>Repair 3</i> : corrective maintenance where repair takes place after severe concrete cracking and therefore the loss of cross-sectional area of rebars is significant. Actions: 6 cm concrete removal and replacement of corroded bars. | 76.3 | 71.8 | 135   | 58.9 | 11.4 | 353.4 |

It is noted from Table 22.2 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.

#### 22.4.2.2. Cost for the Adaptation Strategy

The baseline case for construction cost per unit volume ( $C_{cv}$ ) including forms, concrete, reinforcement, finishing and labour is approximately 490-850€/m<sup>3</sup>, 910-1010€/m<sup>3</sup> and 780-1560€/m<sup>3</sup> 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 (RSMMeans 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  $\Delta c_{adapt}$  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<sup>2</sup> 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 22.3 provides the adaptation costs for various structural elements (per mm of extra cover). The relationships used to evaluate the adaptation costs are provided in (Bastidas-Arteaga and Stewart 2016). This table also presents the adaptation costs for  $\Delta c_{adapt} = 5$  and 10 mm increase in extra cover.

**Table 22.3** – Adaptation costs for slabs

| Structural element | D (mm) | $\Delta c_{adapt}$ 5 mm (€/m <sup>2</sup> ) | $\Delta c_{adapt}$ 10 mm (€/m <sup>2</sup> ) |
|--------------------|--------|---|--|
| Slabs              | 100    | 4.23  | 8.45   |
| Slabs              | 300    | 2.44  | 4.88   |

### 22.5. Illustrative Example: Adaptation of Existing RC Structures

#### 22.5.1. Problem Description

##### 22.5.1.1. Concrete Cover and Exposure

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. Since this application deals with existing structures, it is important to determine which were the standards at the construction time. There was an evolution in time of the minimum concrete cover recommended by French design standards for maritime exposure. 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. For the XS3 exposure the recommended concrete cover is 55 mm.

This example considers two adaptation strategies implemented at a given adaptation time (e.g.,  $t_{adapt}=2030$ ) defined as follows:

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

It is 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 (40 mm design cover) damaged in 2015 is repaired to the 2015 specified design cover of 55 mm.

This study does not focus on a specific structure. Therefore, there is not 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 example 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,
- rebar diameter is  $d_0 = 16$  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,
- 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:  $C_{damage} = 263.2\text{€}/\text{m}^2$ ,  $C_{damage} = 323\text{€}/\text{m}^2$ , and  $C_{damage} = 353.4\text{€}/\text{m}^2$  for Repairs 1, 2 and 3 (Table 22.2).
- the adaptation costs are defined according to Table 22.3, and
- the discount rates are 2, 4 and 8%.

The probabilistic models used to estimate damage probabilities ( $p_{s,i}$ ) are given in Table 22.4.  $p_{s,i}$  (Eq. [22.2]) corresponds herein to the probability of severe concrete cracking until reaching the limit crack width  $w_{lim}$ . This serviceability limit state allows us to generalise the results. 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) and Nguyen et al. (2017).

**Table 22.4** – Probabilistic models of the random variables

| Variable  | Units                  | Distribution        | Mean                | COV  | Reference  |
|---|------------------------|---------------------|---------------------|------|--|
| Reference chloride diffusion coefficient, $D_{c,ref}$ | $\text{m}^2/\text{s}$  | log-normal          | $3 \times 10^{-11}$ | 0.20 | (Duracrete, 2000a; Saetta et al., 1993; Val & Trapper, 2008) |
| Environmental chloride concentration, $C_{env}$       | $\text{kg}/\text{m}^3$ | log-normal          | 7.35                | 0.20 | (Duracrete 2000; Vu and Stewart 2000)                        |
| Concentration threshold for                           | wt% cem.               | normal <sup>a</sup> | 0.5                 | 0.20 | (Bastidas-Arteaga and  |

|   |                           |                     |                     |      |  |
|---|---------------------------|---------------------|---------------------|------|--|
| corrosion initiation, $C_{th}$                        |                           |                     |                     |      | Schoefs 2012; Duracrete 2000)                      |
| Cover thickness, $c_t$                                | mm                        | normal <sup>b</sup> | 55                  | 0.25 | (European standard, 2004; Val & Stewart, 2003)     |
| Reference humidity diffusion coefficient, $D_{h,ref}$ | m <sup>2</sup> /s         | log-normal          | $3 \times 10^{-10}$ | 0.20 | (Saetta et al., 1993; Val & Trapper, 2008)         |
| Thermal conductivity of concrete, $\lambda$           | 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, $\rho_c$                         | kg/m <sup>3</sup>         | normal <sup>a</sup> | 2400                | 0.04 | (JCSS (Joint committee of structural safety) 2001) |
| Reference corrosion rate, $i_{corr,20}$               | $\mu\text{A}/\text{cm}^2$ | log-normal          | 6.035               | 0.57 | (Duracrete 1998)                                   |
| 28 day concrete compressive strength, $f_c(28)$       | MPa                       | normal <sup>a</sup> | $1.3(f'_{ck})$      | 0.18 | (Pham 1985)  |
| Concrete tensile strength, $f_{ct}$                   | MPa                       | normal <sup>a</sup> | $0.53(f_c)^{0.5}$   | 0.13 | (Mirza et al. 1979)                                |
| Concrete elastic modulus, $E_c$                       | MPa                       | normal <sup>a</sup> | $4600(f_c)^{0.5}$   | 0.12 | (Mirza et al. 1979)                                |

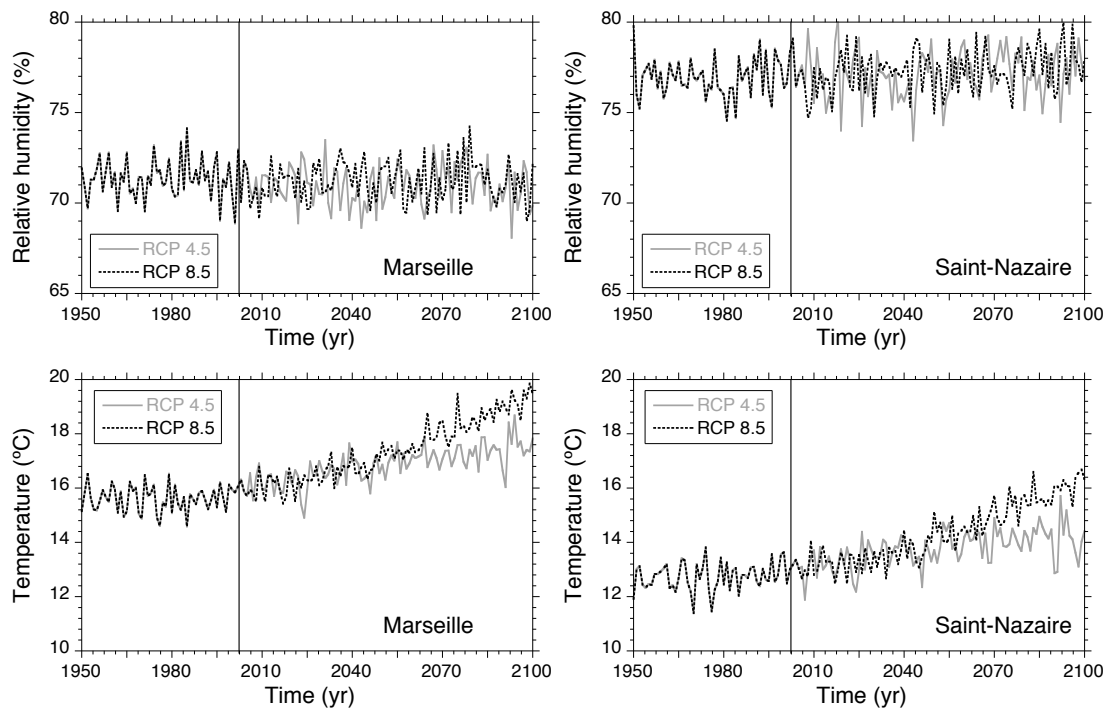
<sup>a</sup>truncated at 0, <sup>b</sup>truncated at 10mm

### 22.5.1.2. Climate change scenarios

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. 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 22.3 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 1.5°C and 3.5°C 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<sub>2</sub> 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 (Teodorescu et al. 2017). 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).

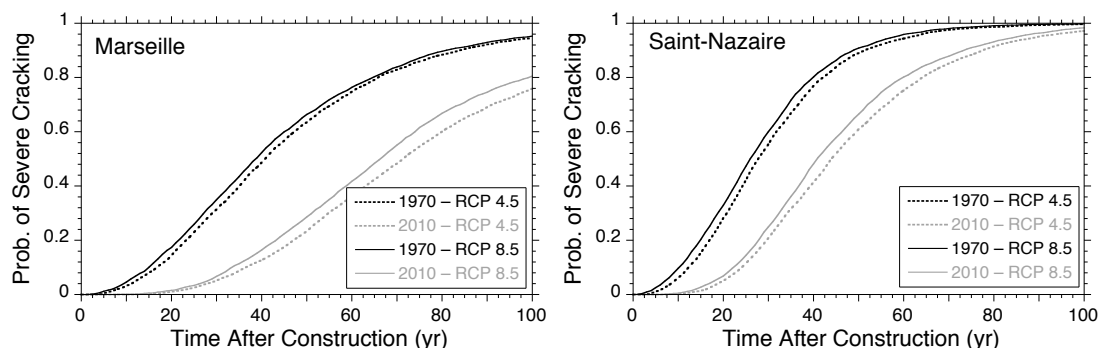


**Figure 22.3** – Yearly temperature and relative humidity projections for Saint-Nazaire and Marseille for RCP 4.5 and 8.5 climate change scenarios

## 22.5.2. Results

### 22.5.2.1. Damage probabilities and climate change effects

Figure 22.4a 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 22.5.1.2, 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 (in 1990 and 2010) 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.



**Figure 22.4** – Probability of severe cracking for structures built in Saint-Nazaire and Marseille under the RCP 4.5 and RCP 8.5 scenarios

For both places, climate change projections show increases of temperature with respect to year 2000 levels for the RCP 4.5 and RCP 8.5 climate change scenarios, respectively. The

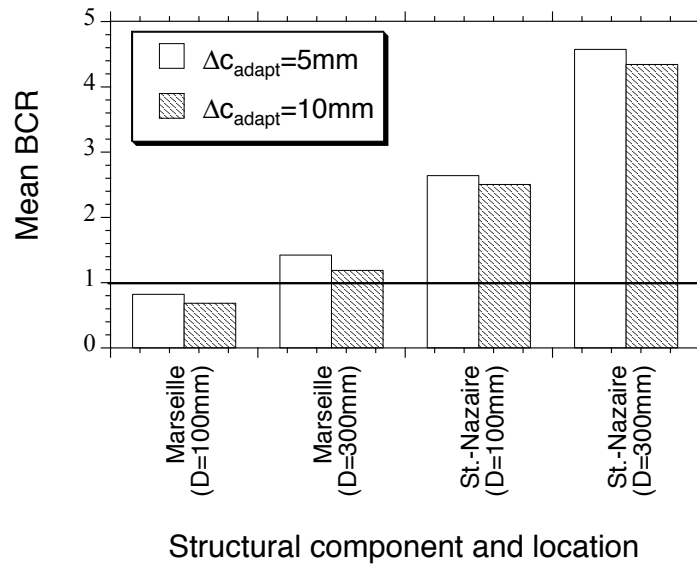
effects of these climate change scenarios on the probability of severe cracking in also illustrated in Figure 22.4. As expected, probabilities of severe cracking are larger for RCP 8.5 exposure because higher temperatures accelerate chloride ingress and corrosion propagation (Bastidas-Arteaga et al. 2011; Nguyen et al. 2017). 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 22.3). As a consequence, for structures built in 1970 the difference of temperature between both climate change scenarios is about 0.7 °C (in 2070) whereas for recent structures (built in 2010), this difference is 1.7°C (in 2110). From an engineering point of view, RCP 8.5 and RCP 4.5 climate change scenarios could be interpreted respectively as upper and lower bounds for carrying out sensitivity studies. This point is illustrated later in the assessment of the cost-effectiveness of adaptation measures.

#### 22.5.2.2. 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  $C_{damage}=323\text{€}/\text{m}^2$  (Repair 2) According to Table 22.2, this damage cost corresponds to a medium damage extent that does not require replacement of corroded steel. A sensitivity study about the effects of discount rate will be presented in section 22.5.2.3.

Figure 22.5 presents the mean BCR for RC slabs in Marseille and Saint-Nazaire under the RCP 4.5 scenario. These results were obtained for slabs 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 slab size. The mean BCR is lower for Marseille and small slabs. For slabs with  $D=100$  mm 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 slab sizes in Saint-Nazaire. Thus, recommendations of current standards and adaptation measures could be more or less adapted to local climate conditions. For both locations, increasing extra cover by 10 mm is less cost-effective than a 5 mm increase in cover. Even if risk reduction  $\Delta R$  should be higher for  $\Delta c_{adapt}=10$  mm, 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) business as usual (or do nothing).





**Figure 22.5** – Mean BCR for RC slabs in Marseille and Saint-Nazaire under a RCP 4.5 climate change scenario, for structures built in 2010 and  $t_{adapt} = 2020$

The mean BCR is also influenced by the year of construction. To explain this relationship, Table 22.5 provides 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  $\Delta c_{adapt}=10$  mm is less cost-effective for all adaptation years. The following results will therefore focus on the repair strategy  $\Delta c_{adapt}=5$  mm. Table 22.5 also gives 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 22.3). These climate variations will induce more corrosion damage by increasing the cost-effectiveness of adaptation strategies.

**Table 22.5** – Mean BCR for slabs ( $D=300mm$ ) built in several years in Saint-Nazaire and  $t_{adapt}=2020$

| Construction year | RCP 4.5                |                         | RCP 8.5                |                         |
|-------------------|------------------------|-------------------------|------------------------|-------------------------|
|                   | $\Delta c_{adapt}=5mm$ | $\Delta c_{adapt}=10mm$ | $\Delta c_{adapt}=5mm$ | $\Delta c_{adapt}=10mm$ |
| 1970              | 0.81                   | 0.71                    | 0.82                   | 0.71                    |
| 1990              | 3.76                   | 3.41                    | 3.85                   | 3.58                    |
| 2010              | 4.57                   | 4.34                    | 4.74                   | 4.5                     |

The effect of the time of adaptation on the mean BCR and  $Pr(BCR > 1)$  for slabs ( $D=300mm$ ) in Saint-Nazaire,  $\Delta c_{adapt}=5$  mm and the RCP 4.5 scenario is shown in Table 22.6. 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  $\text{Pr}(\text{BCR}>1)$  are small for older structures and therefore the owner/stakeholder could prioritise investments in adaptation measures for recent structures. These results could be also used to evaluate the impact of the adaptation year. For example, for structures built in 1990, if the owner/stakeholder decides to postpone the adaptation actions until 2040 the mean BCR is about 1.4 which is still interesting. However, the  $\text{Pr}(\text{BCR}>1)$  for this adaptation time is less than 11% indicating that the risks of having no benefits are larger.

**Table 22.6** – Mean BCR and  $\text{Pr}(\text{BCR}>1)$  (within brackets) for various adaptation years (slabs ( $D=300\text{mm}$ ) built at several years in Saint-Nazaire, RCP 4.5 scenario and  $\Delta c_{\text{adapt}}=5$  mm)

| Construction year | Adaptation year |              |              |             |
|-------------------|-----------------|--------------|--------------|-------------|
|                   | 2020            | 2040         | 2060         | 2080        |
| 1970              | 0.83 (6.1%)     | 0.05 (0.2%)  | 0 (0.0%)     | 0 (0.0%)    |
| 1990              | 3.64 (43.5%)    | 1.38 (10.2%) | 0.2 (0.7%)   | 0 (0.0%)    |
| 2010              | 4.63 (59.0%)    | 3.85 (44.7%) | 1.69 (13.1%) | 0.29 (0.9%) |

### 22.5.2.3. Effect of Discount Rate

Table 22.7 describes the influence of discount rates ( $r$ ) on both the mean BCR and the  $\text{Pr}(\text{BCR}>1)$  for slabs ( $D=300\text{mm}$ ) built in several years in Saint-Nazaire and  $t_{\text{adapt}}=2020$ . It is observed that the mean BCR and  $\text{Pr}(\text{BCR}>1)$  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 22.3.3.2, various governments recommend lower discount rates of about 2% for long-term investments. The BCR analysis therefore shows that the adaptation strategies are more cost-effective according to these recommendations.

**Table 22.7** – Mean BCR and  $\text{Pr}(\text{BCR}>1)$  (within brackets) for various discount rates and construction years (slabs ( $D=300\text{mm}$ ) built at several years in Saint-Nazaire, RCP 4.5 scenario and  $\Delta c_{\text{adapt}}=5$  mm)

| Construction Year | $r=2\%$      | $r=4\%$      | $r=8\%$      |
|-------------------|--------------|--------------|--------------|
| 1970              | 1.32 (5.9%)  | 0.81 (5.9%)  | 0.29 (5.4%)  |
| 1990              | 5.8 (44.6%)  | 3.76 (43.4%) | 1.58 (39.8%) |
| 2010              | 7.15 (64.2%) | 4.61 (62.3%) | 2.15 (53.6%) |

## 22.6. Conclusions

The kinematics of the chloride ingress and corrosion propagation mechanisms is highly influenced by the surrounding environmental conditions including climate change that could accelerate or decelerate these processes depending on specific exposure and environmental conditions. Therefore, the results presented in this chapter highlight that a comprehensive probabilistic cost-benefit analysis of adaptation strategies should combine general methods (deterioration models able to account for climate variations, stochastic methods to propagate uncertainties and cost-benefit analysis) with factors specific to each structure (structural characteristics and climate change predictions). The numerical results 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  $\text{Pr}(\text{BCR}>1)$  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  $\text{Pr}(\text{BCR}>1)$  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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