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Multi-objective and probabilistic decision-making approaches to sustainable design and management of highway bridge decks

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The aging and deterioration of highway bridges and the new requirements for sustainable infrastructures and communities require innovative approaches for their management that can achieve an adequate balance between social, economic and environmental sustainability. This paper presents a multi-objective decision-making approach for the sustainable design and management of highway bridge decks, which considers several and conflicting objectives, such as the minimization of owner's costs, users costs, and environmental impacts and uses goal setting and compromise programming to determine the satisficing and compromise solutions that yield the best trade-off between all competing objectives. The proposed approach is based on robust reliability-based mechanistic models of the deterioration and service life of reinforced concrete bridge decks, which include diffusion models for the prediction of chloride ingress into concrete and steel corrosion and thick-walled cylinder models for the prediction of stresses induced by the accumulating corrosion products in the concrete cover. The proposed approach is illustrated on the life cycle design and management of highway bridge decks using normal and high performance concrete. It is shown that the high performance concrete deck alternative is a Pareto optimum, while the normal concrete deck is found to be a dominated solution in terms of life cycle costs and environmental impacts.

Keywords: Highway bridge decks; life cycle design; mechanistic service life models; multiobjective decision-making; sustainable management; uncertainty modelling.

1. Introduction

Highway bridges are critical links in North America's transportation network that should be kept safe, serviceable and functional during their service lives to enable personal mobility and transport of goods to support commerce and economic development. The aging and deterioration of Canada's highway bridges and the limited funds available for their preservation require the

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development of innovative design concepts, monitoring technologies and management strategies that will meet the needs of the present and future generations by ensuring: (i) acceptable safety, serviceability, and robustness against nature and man-made hazards; (ii) adequate levels of services provided by the bridges and overall road network in terms of traffic volume, speed and comfort; (iii) protection of the environment by minimizing the risk of degradation of the air, water and soil from with the construction, use, maintenance and renewal of the bridges and the overall road transportation network; and (iv) minimum life cycle costs or maximum benefits to the owners and users of the bridges. In Canada, reinforced concrete decks of highway bridges are the components that are directly exposed to the effects of the aggressive environment, including deicing salts used during winter, freeze-thaw cycles, as well as the increased traffic volume and truck load. Corrosion-induced deterioration is identified by transportation agencies in Canada as the main cause of deterioration of their highway bridge decks. The average service life of bridge decks vary from 15 years up to 50 years, which are continually extended by using different management strategies that include different combinations of maintenance, repair, rehabilitation, replacement, mitigation and adaptation actions.

The use of sustainability as a driving principle in the management of highway bridges will provide decision-makers with a comprehensive assessment of the impacts of their decisions on three key criteria or spheres of influence for sustainable communities, namely: society, economy and environment, which are referred to as the “three pillars of sustainability”. These three pillars are intrinsically linked and have impacts on decision-making at strategic, tactical and project levels of management. The implementation of sustainability principles in asset management decisions has become a “need or a necessity” rather than a “desire” to curb our consumption and emissions, protect the environment and improve our efforts towards social equity and sustainable communities. This concept has gained momentum and consensus since the publication of the *Brundtland* commission report (WCED 1987), in which sustainable development was first coined and defined as a “*development that meets the needs of current generation without compromising the ability of future generations to meet their own needs.*”

In the conventional approach to bridge management, some aspects of the social and economic pillars are used as the drivers and the focus for asset management at all levels of decision-making. However, the environment pillar and the consequences of management

decisions on the natural environment are rarely if at all considered in the decision-making process. The conventional approach to asset management should integrate environmental protection to ensure its compatibility with the principles of sustainable infrastructure and sustainable communities.

Bridge management can be defined as a systematic process of operating, maintaining, upgrading and expanding physical assets effectively throughout their life cycles as shown in Figure 1. It combines engineering principles with sound business practice and economic theory for resources allocation and utilization. The actual bridge management problem can be defined as a multi-objective optimization problem (Lounis and Cohn 1995; Frangopol et al. 1993; 2005) in which the decision maker is seeking to select the best decisions that achieve the best tradeoffs between different competing objectives, such as maximizing public safety, minimizing environmental impacts, maximizing the levels of services and minimizing the life cycle costs. However, most of the current decision-making approaches and bridge management systems are based on the optimization of a single objective, namely the minimization of life cycle costs. The state-of-art approaches to asset management have focused primarily on three objectives, namely: (i) preserving the physical condition of assets; (ii) ensuring acceptable levels of services (e.g. mobility); and (iii) minimizing the life cycle costs. It is clear that limited or no consideration has been given to assessing the life cycle or “cradle to grave” impacts on the environment and the society that result from the construction, operation, maintenance, failure, and replacement of bridge assets. Furthermore, there is also a need to assess the life cycle impacts on the environment of the service provided by these assets, such as vehicle emissions, impacts on environment due to traffic disruption, etc. The implementation of effective sustainable asset management approaches requires the development of approaches that can predict the performance of bridges and the services they provide and their impacts on society, economy and environment over their life cycles using appropriate performance measures.

The integration of environmental management with asset management will provide decision-makers with reliable and comprehensive measures on the asset performance and the associated risk of failure in terms of public safety, health, security, environmental impact, social equity, economic impact over the life cycle of the assets. Several measures can be used to mitigate the negative impacts of bridges on the environment, such as: (i) designing long life assets; (ii)

controlling the growth of service levels; (iii) dematerializing assets by designing efficient systems that consume less materials by using high performance materials and innovative structural systems; (iv) recycling waste and using recycled products as construction materials; (v) minimizing the disruption of service, which can have serious impacts on the environment; and (vi) implementing effective and pro-active management strategies to minimize the risk of failure of engineering assets.

In this paper, a multiobjective approach for decision-making, which can incorporate all relevant objectives, is proposed to enable the sustainable design and management of bridge decks. The objectives of this paper are twofold:

- (i) Present a multiobjective approach to decision-making based on goal setting and compromise programming to ensure sustainable design and management of highway bridge decks. Multiple and conflicting objectives can be considered, including minimization of owner's costs, users costs, and environmental impacts. ; and
- (ii) Present more robust probabilistic mechanistic models for the prediction of the degradation and service life of reinforced concrete bridges decks that are exposed to deicing salts, which are based on more realistic models of the chloride ingress into concrete, onset of corrosion, corrosion products accumulation and degeneration of stresses in the concrete cover that ultimately lead to internal cracks, surface cracks, spalling and delamination.

The proposed multiobjective decision-making approach and mechanistic service life prediction models enable decision-makers to select so-called Pareto optimal or satisficing strategies for design and management of highway bridge decks, which yield the best balance between the above different and competing objectives, which are direct or indirect measures of social, economic, and environmental sustainability for highway bridges and neighboring communities.

2. Multiobjective decision-making for sustainable design and management of highway bridge decks

Highway bridges and the related network of roads contribute to improving the quality of life of Canadians and the sustainability of neighbouring communities. The failure of these assets can have serious consequences on: (i) Public safety and health as a result of fatalities, injuries, health

problems; (ii) Environment quality as a result of greenhouse gas (GHG) emissions, vehicle emissions, consumption of energy and raw materials, pollution of air, water, and/or soil; and (iii) Economy as a result of disruption of service, including delays in personal mobility and transport of goods by trucks, increased maintenance and users costs, loss or damage to residential areas and commercial activities, etc. Furthermore, climate change can induce additional pressures on highway bridges in terms of increased climatic loads such as increased thermal stresses, wind loads, precipitation, flooding, etc. that can reduce the load bearing capacity of bridges and affect the services provided by the assets. Such climate change-induced impacts can have serious consequences on public safety, public health, mobility, social equity and economic development.

The approaches to maintenance optimization implemented in the current bridge management systems are based on single objective optimization, and more specifically on the minimization of life cycle maintenance costs, which represents the present value of all the costs incurred throughout the life cycle of a bridge structure, including, the costs of design, construction, maintenance, repair, rehabilitation, replacement, demolition and, in some instances, the users costs. An approach to the sustainable management of highway bridges can be formulated as a multi-objective optimization problem as the bridge owner or manager seeks to satisfy implicitly and simultaneously several objectives, such as the minimization of life cycle costs to owners and users, improvement of public safety, security, and serviceability, improvement of bridge functionality, minimization of environmental impacts, and maximization of economic benefits.

The use of sustainability as a driving principle for the design and management of infrastructure systems will provide decision-makers with a comprehensive assessment of the impacts of their decisions on three broad criteria or spheres of influence, namely: society, economy and environment, which are referred to as the “three pillars of sustainability”. These three pillars are intrinsically linked and have impacts on decision-making at strategic, tactical and project levels. Owners and managers of roads and bridges are facing challenges, including the preservation of aging and deteriorating assets, responding to increased demands for service and new infrastructure, and limited funds. Furthermore, new pressures and new emerging challenges, such as aging population, population growth in urban centers, protection of the environment, adapting to climate change, and increasing energy costs require the development of

new innovative solutions for the design and management of highway bridges that can take into account of all these competing requirements. In the conventional approach to asset management, some aspects of the social and economic pillars are used as the drivers and the focus for asset management at all levels of decision-making. Up to now, the state-of-art approaches to asset management have focused primarily on three objectives, namely: (i) preserving the physical condition of assets; (ii) ensuring acceptable levels of services (e.g. mobility); and (iii) minimizing the life cycle costs.

The construction, use, deterioration, maintenance, rehabilitation, and replacement of highway bridges contribute to the greenhouse gas (GHG) emissions in Canada and require the consumption of large amounts of water, non-renewable raw materials and energy, which all contribute to negative footprints on the environment, including climate change. The road transportation sector contributes approximately 18% of the total GHG emissions in Canada (Environment Canada 2008). It also contributes to the rise in concentration of volatile organic compounds (VOC) and nitrogen oxides (NO_x), which in turn contribute to the ground level ozone or smog that is associated with negative impacts on public health.

In existing bridge management approaches, limited consideration has been given to assessing the life cycle impacts on the environment that result from the construction, operation, maintenance, failure, and replacement of highway bridges. Furthermore, there is also a need to assess the life cycle impacts on the environment by the services provided by these assets, such as vehicle emissions, impacts on environment due to traffic growth, congestion, disruption, etc. There is a growing awareness of the importance of protecting the natural environment in an effort to ensure the sustainability of neighbouring communities, as well as to ensure a sustainable development at the local, regional and national levels. The implementation of sustainability principles in the design, operation, maintenance and renewal of engineering assets has become a “need or a necessity” rather than just a “desire” resulting in different efforts at the national, regional and local levels to curb consumption and emissions, protect the environment and improve our efforts towards social equity and social progress. This concept has gained momentum and consensus since the publication of the Brundtland commission report (WCED 1987), in which “sustainable development” was first coined and defined as a “*development that meets the needs of current generation without compromising the ability of future generations to meet their own needs.*” This approach is referred to also as the “*Triple Bottom Line*” abbreviated

TBL or 3BL approach to accounting. The TBL approach is very different from the “*Financial Bottom Line*” accounting approach and reframes the accounting process from a vehicle for maximizing profits to one focused on coordinating and satisfying stakeholder long term interests by taking into account the social and environmental criteria in addition to financial criteria (Elkington 1994; Felio and Lounis 2008).

It is clear from the above discussion that the existing approaches to decision making have serious limitations as they express all criteria in monetary terms and consider only one criterion at a time, e.g. minimization of owner costs and cannot consider non-commensurate criteria related to as environmental quality and economic development. As result, bridge management should be formulated as a multi-objective optimization problem in which optimal or trade-off solutions that achieve a satisfactory balance between several criteria or objectives, such as minimization of life cycle design and maintenance costs, maximization of public safety, minimization of traffic disruptions, and minimization of environmental impacts. Reliable information on the life cycle impacts of construction; deterioration, failure, rehabilitation and replacement of highway bridge decks on society, economy and environment require a priori reliable predictive models of the physical performance of the decks.

Corrosion-induced deterioration from the use of de-icing salts during the winter months is the main cause of deterioration of highway bridge decks in Canada. Given the complexity and uncertainty of the factors affecting the performance of the bridge decks, it is important to develop robust and reliable mechanistic models that can describe the different key stages within the life cycle of the deck, including: (i) chloride ingress into concrete and contamination of the deck; (ii) depassivation of the reinforcing steel and onset of corrosion; (iii) initiation of internal cracks; (iv) crack propagation to the surface; and (v) spalling and delamination. Any of these stages can be defined as limit states in terms of performance, which may require different protection, inspection, maintenance and rehabilitation strategies. Furthermore, it is imperative to model the uncertainty of the governing parameters in order to obtain realistic estimates of the performance of bridge decks in terms of probabilities of corrosion, cracking, spalling and delamination, which are then used to define practical criteria for end of life of the deck, which define the times at which different inspection and preservation strategies are needed to extend the service life of bridge decks. The life cycle of a bridge should not be confused with the service life of an asset. The life cycle can be defined as the time period over which the cost or

environmental impact are assessed, while the service life is the time over which the asset deteriorates and reaches a maximum level of damage or minimum performance level, which is referred to as a limit state, which can indicate unacceptable safety or serviceability of the asset, as shown in Figure 1. The service life can also be defined as the time when the service provided by the highway bridge reaches a minimum acceptable level. Bridge deterioration shown on Figure 1 is due to cumulative damage (e.g. corrosion) and does not include random shock-induced damage.

The knowledge of the service life of a bridge deck is required before undertaking a life cycle cost analysis and an environmental impact assessment. The prediction of the deterioration or service life of an asset requires models that can forecast its future condition taking into account factors that contribute to its degradation as illustrated in Figure 1.

2.1 Optimization Objectives

2.1.1 Minimization of bridge owner's life cycle costs

Owner's (or agency) costs are the costs associated with the design, construction, inspection, routine maintenance, repair, rehabilitation, and replacement of highway bridge decks. They consist primarily of the in-place costs of materials and labour for the different actions performed on a bridge deck within its life cycle. These are tangible costs measurable with monetary values (Hawk 2003). The design cost includes the costs of all studies, environmental and other reviews, and consultant contracts prior to solicitation of construction bids for a new bridge deck or major rehabilitation of an existing bridge deck. The construction costs include the administrative and contract costs of the deck and ancillary facilities such as approach slabs. Unit costs and bills of quantities are the most widely used basis for developing construction costs estimates (Hawk 2003). The inspection, maintenance, repair and rehabilitation costs include:

- Costs of routine (visual) inspections of the bridge deck;
- Costs of periodic or special inspections using non-destructive and semi-destructive evaluation (NDE) methods such as the techniques of half-cell potential, radar, chloride content analysis;

- Costs of routine maintenance, which include the costs of asphalt overlays with or without membranes, crack sealing, minor patch repairs, deck washing, and maintenance of drainage and deck joints;
- Patch repair and rehabilitation costs refer to the costs of major patch repairs and concrete overlays of decks using latex modified concrete, low-slump dense concrete, or microsilica concrete. These costs include the costs of removal of spalled, delaminated and critically chloride contaminated concrete; and
- Replacement cost includes the cost of demolition and removal in addition to costs similar to the costs discussed above, i.e. costs of design and construction.

Costs occurring at different points in time during the life cycle are converted to their equivalent present value by “discounting” as follows:

$$P = \frac{F}{(1+r)^t} \quad (1a)$$

where P is the present value of cost occurring in (t) years, F is the future cost (expressed in constant dollars) and r is the discount rate. The resulting or total cost of a bridge deck alternative is known as the Present Value Life Cycle Cost (PVLCC) and it is used to compare two or more alternative designs. The PVLCC of a bridge deck is given by:

$$PVLCC = C_0 + PV_{MRR A} + PV_{MRR U} - PV_{Res} \quad (1b)$$

$$PV_{MRR(A \text{ or } U)} = \frac{C_1}{(1+r)^{t_1}} + \frac{C_2}{(1+r)^{t_2}} + \frac{C_3}{(1+r)^{t_3}} + \dots \quad (1c)$$

where C_0 = construction cost, $PV_{MRR A}$ = present value of all maintenance, repair, and rehabilitation and repairs (MR&R) costs paid by the owner, $PV_{MRR U}$ = present value of all MR&R cost incurred by users, PV_{Res} = present residual value of the bridge deck at the end of the analysis period, C_x = cost of a single occurrence of a MR&R activity (agency and/or user cost), r = discount rate (assumed to be constant within analysis time) and t_x = time of activity occurrence, t is smaller or equal to the analysis period (life cycle). The costs of all occurrences of all defined activities are summed up. The costs of MR&R activities are expressed in today’s dollar value. The discount rate (r) represents an estimate of the average rate of return on private investment, before taxes and after inflation.

A reliable estimate of the life cycle cost of a highway bridge deck requires a rigorous prediction of its service life. In As stated before, all possible solutions are compared on the same analysis time regardless of their different predicted service lives. The service life of one alternative may be shorter than the analysis time and, for this case, the rehabilitation and/or replacement activities that are scheduled to maintain the bridge deck above the minimum acceptable performance (Fig. 1) are included in the analysis up to the end of the analysis period.

2.1.2 Minimization of bridge users costs

User costs include the travel delay costs, vehicle-operating costs and accident/crash costs (Hawk 2003, Golabi *et al.* 1993, Ehlen and Marshall 1996). Users' costs increase with increasing traffic volume and increasing duration and disruption of the maintenance or rehabilitation activity. The user cost model used is based on the methodology of CALTRANS (Golabi *et al.* 1993).

(a) Minimization of travel delay costs

Traffic delay costs are those costs associated with traffic disruption due to inspection, maintenance, repair, rehabilitation, and replacement of the bridge deck. For bridges carrying a high volume of traffic, the user costs might be substantially larger than agency costs. Travel delay costs are the costs imposed on bridge users due to temporary closures of bridge lane(s) for a MR&R activity that causes traffic congestion. Delay costs are calculated using:

$$TDC = \left(\frac{L}{S_a} - \frac{L}{S_n} \right) t_{MRR} \left[(ADT - ADTT) C_{delay}^{auto} + (ADTT) C_{delay}^{truck} \right] \quad (2)$$

where TDC = travel delay costs, L = average length of affected roadway for both directions, S_a = traffic speed during MRR activity, S_n = normal traffic speed, ADT = average daily traffic (number of vehicle per day, including trucks), $ADTT$ = average daily truck traffic, t_{MRR} = duration of MRR activity and C_{delay} = hourly time value of drivers (automobile or truck).

(b) Minimization of vehicle operating costs

The vehicle operating costs are the accrued costs incurred by vehicle drivers crossing a bridge that is affected by a given MR&R activity and are mainly caused by delay. These costs include accelerated vehicle depreciation, increased vehicle repair costs, increased fuel consumption, and increased tire wear.

$$V_{OC} = \left(\frac{L}{S_a} - \frac{L}{S_n} \right) (ADT) t_{MRR} C_{VOC} \quad (3)$$

where V_{OC} = vehicle operating costs, C_{VOC} = hourly vehicle operating cost.

Delay costs and vehicle operating costs can also be due to poor condition of the driving surface of the bridge deck and not be associated to a particular MR&R activity event. These costs are on-going user costs that start when the bridge deck condition reaches a critical state and usually degrades with time until some repair, rehabilitation or replacement activity takes place.

(c) Minimization of accident costs associated with bridge deck maintenance

Accident costs are adopted as a measure of social performance as these costs are mainly covered by insurance premiums (material damages) and health services paid in large by the public or society. The considered accident costs are those costs incurred due to an increase in the accident rate during maintenance, repair, rehabilitation and replacement (MR&R) activities on bridge decks. These costs include costs related to injuries and damage to property, as well as the value of statistical life (VOSL) for 0.4% of the total number of accidents that result in fatalities (Statistics Canada 2003). Estimation of VOSL can vary from \$160,000 to \$33 million (Dionne and Lanoie, 2004). An average accident cost may be estimated using a VOSL of \$1.6 million as proposed by Transport Canada (1994). Accident costs may be quite high and depend primarily on the traffic volume and the increase in the accident rate induced by the work on the bridge deck. This cost can be estimated as follows (Ehlen and Marshall 1996, Wall and Smith 1998):

$$AC = L(ADT)(A_a - A_n)t_{MRR}C_{ACC} \quad (4)$$

where AC = accident costs, A_a = accident rate during MR&R activities (in number of accidents/million vehicles.km), A_n = normal accident rate (per million vehicles.km) and C_{ACC} = average cost per accident. Statistics collected by the insurance industry and accident rates maintained by the provincial transportation agencies, police departments, and national agencies may be used to extract data on accident rates and costs. Accident costs can be considered as a social burden in itself but may also be more appropriately considered as proxy for other social pressures caused by accidents. These social pressures are less tangible and more difficult to objectively measure (i.e. physical pain and stresses, reduced quality of life, etc.)

2.1.3 Minimization of environmental impacts

The quality of the environment has direct and indirect impacts on the health of Canadians and their social well-being, as well as on the economic development and on the overall sustainability of neighbouring communities. Traffic congestion also contributes to an increase in CO₂ emissions and other toxic substances (NO_x, SO_x, VOC) and finally traffic congestion affects people quality of life as nobody enjoys being delayed in their vehicle for extended period of time or be affected by increased air pollution.

A simplified environmental impact assessment is proposed in this paper where the amounts of CO₂ emissions and waste production are estimated for all MR&R activities occurring during the life cycle of highway bridge decks. For a more rigorous and complete analysis of the environmental impacts of different management alternatives, the initial construction phase should be included and based on a comprehensive life cycle assessment (LCA) of the project where all significant environmental impacts would be identified, which is outside the scope of this paper.

(a) Minimization of CO₂ emissions

Regarding the CO₂ emissions, the production of one ton of cement leads to 0.8-1.3 ton of CO₂ emissions, depending on plant efficiency. The use of supplementary cementing materials (SCMs) such as silica fume, slag and fly ash to replace part of the cement can have two environmental benefits, namely: (i) reduce the CO₂ emission associated with cement production; and (ii) use of industry by-products as construction materials instead of land-filling them. The additional CO₂ emissions that are generated through the transportation by trucks of SCMs that are not locally available can be estimated as 0.022 ton of CO₂ per ton of SCM over a distance of 1000 km (Bouzoubaâ and Fournier 2003). The estimation of CO₂ produced by traffic delayed is based on the assumption that the production of CO₂ during this extra travelling time is similar to the production of CO₂ of an average car engine at idle estimated at 4.11×10^{-3} ton/hour (Government of Canada 2006).

The CO₂ released by the production of reinforcing steel is not accounted as it would typically be the same for different bridge deck management strategies as their design is based on the simplified approach of the Canadian Highway Bridge Design Code (CAN/CSA2006), which requires a steel reinforcement ratio of 0.3% for both top and bottom layers in both transverse and longitudinal directions.

(b) Minimization of waste materials for landfill

The waste produced (or landfill use) for different design and management strategies includes the volume of waste material produced during the replacement of asphalt overlay, patch repairs, and deck replacement.

2.2 Formulation and solution of multiobjective sustainable bridge management problem

Several approaches have been developed to solve multiobjective optimization problems, including multi-attribute utility theory (Von Neumann and Morgenstern 1947; Keeney and Raiffa 1976), weighted sum approach (Zadeh 1963), compromise programming, constraint approach, and sequential optimization (Pareto 1919, 1971; Koski 1984; Lounis and Cohn 1993, 1995; Frangopol et al. 1993; 2005). In this paper, the goal setting and compromise programming approach are used to solve the multiobjective bridge deck management problem.

In multi-objective (or vector) optimization problems, the notion of optimality is not that obvious because of the presence of multiple, incommensurable and conflicting objectives. In general, there is no single optimal (non-dominated or superior) solution that simultaneously yields a minimum (or maximum) for all objective functions. The Pareto optimality concept has been introduced as the solution to multiobjective optimization problems (Koski 1984; Lounis and Cohn 1993). A management strategy \mathbf{x}^* is said to be a Pareto optimum if and only if there exists no management strategy in the feasible set of alternatives that may yield an improvement of some criterion without worsening at least one other criterion. The multiobjective sustainable bridge management optimization problem can be mathematically stated as follows:

$$\text{Find: } \mathbf{x}^* = \text{Optimum} \quad (5a)$$

$$\text{Such that: } \mathbf{f}(\mathbf{x}) = [f_1(\mathbf{x}), f_2(\mathbf{x}) \dots \dots \dots f_n(\mathbf{x})] = \text{minimum} \quad (5b)$$

$$\mathbf{X} \in \Omega \quad (5c)$$

$$\Omega = \{\mathbf{X} \in \mathbf{R}^n \mid \mathbf{g}(\mathbf{x}) \leq 0\} \quad (5d)$$

where \mathbf{f} = vector of n optimization objectives or criteria (e.g. minimization of owner costs, user costs, environmental impacts,); $\mathbf{g}=(g_1,\dots\dots,g_m)$ vector of m constraints; Ω = subset of feasible solutions that satisfy the constraints \mathbf{g} .

The concept of Pareto optimality mentioned above, may be stated mathematically as follows (Pareto 1919; Koski 1984; Lounis and Cohn 1993; Frangopol 1993; 2005):

$$\begin{aligned}
& \mathbf{x}^* \text{ is a Pareto optimum if:} \\
& f_i(\mathbf{x}) \leq f_i(\mathbf{x}^*) \quad \text{for } i=1,2,\dots,m \quad (6a) \\
\text{with} \quad & f_k(\mathbf{x}) < f_k(\mathbf{x}^*) \quad \text{for at least one } k \text{ (one of the } m \text{ objectives)} \quad (6b)
\end{aligned}$$

Figure 2 is a schematic illustration of the conflicting nature of the criteria of minimization of maintenance costs and minimization of normalized probability of failure of bridge deck due to corrosion, where different failure criteria are considered, including onset of corrosion, cracking, spalling or delamination. In general, there are several Pareto optimal solutions (also called non-dominated solutions) for a multi-objective optimization problem as shown in Figure 2. In this figure, dominated solutions are also shown to illustrate the concept of dominance. Once the set of Pareto optima is generated, the “best” solution that achieves the best compromise between all competing objectives is sought. Such a solution is referred to as “satisficing” solution in the multi-objective optimization literature (Koski 1984; Yu 1985; Lounis & Cohn 1993,1995, Frangopol et al. 1993; 2005).

In goal setting and compromise programming, the concept of “*satisficing*” solution is introduced as a solution to a multiobjective optimization problem where decision-makers identify a set \mathbf{S} of satisficing solutions such that, whenever the decision of an outcome is an element of \mathbf{S} , the decision-maker will be satisfied and is assumed to have reached the optimal solution (Yu 1985). According to Simon (1955), decision-makers do not seek the best possible solutions to problems but operate within what he referred to as “bounded rationality”.

Let the satisficing set \mathbf{S} be a set of only one point \mathbf{x}^* , i.e. \mathbf{x}^* is a unique target. When each objective f_i , $i=1,\dots,n$, is characterized by “*less is better*”, one can set $\mathbf{x}^* = (x_1^*, \dots, x_n^*)$, where $x_i^* = \inf\{f_i(\mathbf{x}) \mid \mathbf{x} \in \mathbf{X}\}$. In this case, \mathbf{x}^* is called an ideal (or utopia) point because it is not attainable (Yu 1985). This ideal solution is defined as the solution that yields minimum (or maximum) values for all criteria. Such a solution does not exist, but is introduced in compromise programming as a target or a goal to get close to, although impossible to reach. It should be noted that, individually, the x_i^* may be attainable and can be found as solutions to single objective optimization problems as follows:

$$\text{Find:} \quad \mathbf{x}_i^* = \text{Optimum} \quad (7a)$$

$$\text{Such that:} \quad f_i(\mathbf{x}) = \text{minimum} \quad (7b)$$

$$\mathbf{x} \in \mathbf{X} \quad (7c)$$

$$\mathbf{X} = \{\mathbf{x} \in \mathbf{R}^n \mid \mathbf{g}(\mathbf{x}) \leq 0\} \quad (7d)$$

In general, the above problems can be formulated as nonlinear programming problems (Lounis and Cohn 1993), which can be solved by different algorithms (e.g. projected Lagrangian method).

Now given $x \in X$, the “regret” of using x instead of the ideal point x^* may be approximated by the distance from the set of satisfying solutions to the so-called “ideal solution”. Lounis and Cohn (1993, 1995) proposed a multi-objective optimality or criticality index, “MCI”, as a criterion for ranking competing alternatives and determining the “compromise solution”. The criterion used in compromise programming is the minimization of the deviation from the ideal solution f^* measured by the family of L_p metrics (Yu 1985; Lounis and Cohn 1993, 1995). In this paper, the multi-objective criticality index, “MCI”, is defined as the value of the weighted and normalized deviation from the ideal solution f^* measured by the family of L_p metrics:

$$MCI(x) = \left[\sum_{i=1}^m w_i^p \left| \frac{f_i(x) - \min f_i(x)}{\max f_i(x) - \min f_i(x)} \right|^p \right]^{1/p} \quad (8)$$

This family of L_p metrics is a measure of the closeness of the satisficing solution to the ideal solution. The value of the weighting factors w_i of the optimization criteria f_i ($i=1, \dots, m$) depends primarily on the attitude of the decision-maker towards risk in terms of the different objectives. The choice of “ p ” indicates the importance given to different deviations from the ideal solution. For example, if $p=1$, all deviations from the ideal solution are considered in direct proportion to their magnitudes, which corresponds to a group utility (Duckstein 1984). However, for $p \geq 2$, a greater influence is given to larger deviations from the ideal solution, and L_2 represents the Euclidian metric. For $p=\infty$, the largest deviation is the only one taken into account and is referred to as the Chebyshev metric or minimax criterion, and L_∞ corresponds to a purely individual utility (Duckstein 1984; Koski 1984; Lounis and Cohn 1995). In this paper, the Euclidean metric is used to determine the multi-objective criticality index and corresponding compromise solution.

3. Mechanistic models of service life of RC decks in corrosive environments

3.1 Overview of service life of RC structures subjected to chloride attack

In North America, the extensive deterioration of highway bridge deck is mainly caused by chloride-induced corrosion of the reinforcement. The primary source of chlorides derives from

de-icing salts applied to roadways and bridges during winter. Given this predominant deterioration mechanism, most RC bridge deck failures are due to loss of serviceability and functionality and not loss of strength and collapse. As proposed by Tuutti (1982), the service life of concrete structures built in a corrosive environment is modelled as a two-period process and is expressed as follows:

$$t_s = t_i + t_p \quad (9)$$

The service life (t_s) is defined as the time it takes for the concrete deck to degrade to a condition state that reaches a critical level of deterioration that can be defined as a “limit state”. The corrosion initiation time (t_i) is the period during which chlorides penetrate and contaminate the concrete. The reinforcing steel remains in a passive state for a relatively long period of time. Once the chloride ions reach the reinforcing steel and accumulate beyond the threshold chloride content, the corrosion of steel is initiated. The corrosion propagation (t_p) is the period during which the corrosion products accumulate and induce increasing tensile stresses in the concrete cover. Once the maximum tensile stress reaches the tensile strength of concrete, internal cracks are initiated at the steel/concrete interface as shown in Figure 3(a). After the accumulation of additional corrosion products, the cracks start to propagate. Depending on the design of the deck, the cracks may propagate in three different ways as shown in Figure 3 (Lounis *et al.* 2006a):

- (i) Vertically until they reach the deck surface, thus leading to the formation of longitudinal cracks, as in Figure 3(b);
- (ii) At an angle (about 45°) that lead to the spalling of the cover as in Figure 3(c) and;
- (iii) Horizontally leading to the delamination of the cover as in Figure 3(d).

A new corrosion propagation model that assumes the existence of a porous zone around the steel/concrete interface was proposed in the literature (Molina *et al.* 1993; Allan and Cherry 1992; Liu and Weyers 1998; Lounis *et al.* 2006). This model assumes that a specific quantity of the corrosion products accumulate in the so-called “*porous zone*” around the steel/concrete interface and do not contribute in the stress build-up and damage initiation of the concrete cover (i.e. free expansion zone). The assumption of the porous zone leads to an increase in the corrosion propagation time by an amount Δt_{pz} , which is proportional to the thickness of the porous zone. Liu and Weyers (1998) proposed a model for the free expansion period by

estimating the time needed (Δt_{pz}) for the corrosion products to fill the assumed porous zone around the steel/concrete interface, which depends on the w/c ratio, degree of consolidation and hydration, aggregate sizes, and type of steel. However, very limited research and experimental investigations were carried out to assess the validity and reliability of this model, which is less conservative than the model assuming no porous zone. Concrete covers with low porosity, such as high performance concrete with low w/c ratio or containing supplementary cementing materials (e.g. fly ash, slag or silica fume) have a limited space available for the accumulation of the diffusing corrosion products (Allan and Cherry 1992).

In the AASHTO Guide (1998, 2002), the condition state description of RC deck slabs refers to the evaluation of the percentage of deck area that is spalled or delaminated. The proposed life cycle performance or service life models can predict the time it takes to reach anyone of the following six serviceability limit states:

- (i) critical chloride contamination of the deck;
- (ii) depassivation of reinforcing steel and corrosion initiation;
- (iii) onset of internal cracking at the interface of rebar /concrete cover;
- (iv) propagation of internal cracks and onset of surface cracking;
- (v) onset of spalling; and
- (vi) onset of delamination.

3.2 Diffusion-based model of chloride ingress and corrosion initiation in RC decks

Since concrete is a porous medium, chlorides can penetrate into concrete through a number of mechanisms. First, if the concrete is not fully saturated, the salt solution is absorbed into the unfilled spaces by capillary action. As the chlorides migrate into the concrete, some chlorides are absorbed onto and react with the cement hydrates, which slow down the rate of chloride migration. When the surface dries, the salt remains. With successive cycles, there may be a progressive build up of salt at the surface and in the near surface layer until, after few cycles; a limiting value is reached (Bamforth *et al.* 1994). The time it takes to achieve a stable surface level will, however depend on the severity of exposition. In addition to this early absorption, the chloride ions migrate further into the concrete by diffusion through the pore water, driven by the concentration gradient between the surface and the sub-surface layers. As the chlorides migrate further, so the process of ‘binding’ continues to remove chlorides from the pore water.

In concrete which is permanently submerged and saturated, this diffusion process is the only means of transport after the initial wetting. However, even without drying there may be a build-up of chlorides in the pores water to a level above that of the surrounding saline solution, due to a process defined as chloride “condensation”. This is believed to be due to the physical adsorption of chlorides onto the pore surfaces by the electric double layer in cement and can result in chloride concentrations, which are two to four times those of the solutions to which the concrete is exposed. These are commonly referred to as loosely-bound chlorides.

The main transport mechanisms of chlorides into concrete are diffusion and adsorption. However, adsorption occurs in concrete surface layers that are subjected to wetting and drying cycles, and it only affects the exposed concrete surface down to 10-20 mm (Tuutti 1982). Beyond this adsorption zone, the diffusion process will dominate. Chloride diffusion is a transfer of mass by random motion of free chloride ions in the pore solution, resulting in a net flow from regions of higher to regions of lower concentration. The rate of chloride ingress is proportional to the concentration gradient and the diffusion coefficient of the concrete (Fick’s first law of diffusion). Since in the field, chloride ingress occurs under transient conditions, Fick’s second law of diffusion can be used to predict the time variation of chloride concentration for one-dimensional flow, it can be expressed as a relationship between the diffusion coefficient and the gradients of concentration, by direct analogy with the equation of heat conduction (Crank 1975), as follows:

$$\frac{\partial C_{x,t}}{\partial t} = \frac{\partial}{\partial x} \left(D_c \frac{\partial C_{x,t}}{\partial x} \right) \quad (10)$$

where $C(x,t)$ = chloride concentration at depth x after time t , C_s = chloride concentration at the deck surface, and D_c = coefficient of diffusion of chloride ions into concrete. Under the assumptions of a constant diffusion coefficient, constant surface chloride concentration C_s as the boundary condition, and the initial condition specified as $C=0$ for $x>0$, $t=0$, Crank’s solution yields (Crank 1975):

$$C(x,t) = C_s \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{D_c t}} \right) \right] \quad (11)$$

where $\operatorname{erf}(\cdot)$ is the error or (Gauss error) function. The following assumptions are made in modelling the chloride ingress as a Fickian process:

- Some phenomena (e.g. capillary flow, chloride binding, etc.) are not explicitly considered.

- Crank' solution for a plane sheet assumes dealing with plane isotropic concrete structures, i.e. one dimensional diffusion.
- Diffusion coefficient and surface chloride concentration are assumed time-invariant.
- Initial concentration of chloride in the concrete is negligible.
- Interaction with other ions and with electrical double layer is ignored.

These conditions are not necessarily satisfied, for many reasons. For example, the diffusion coefficient cannot be constant because it is influenced by the following: (i) concrete is not homogenous with some micro-cracks or some shrinkage-induced cracks; (ii) moisture is unlikely to be uniform; (iii) some chlorides are bound by cement hydrates; (iv) D_c is influenced by concentration of chlorides content; and (v) chloride binding changes with time.

Therefore, to minimize the error in assuming time-invariant diffusion coefficient and surface chloride concentration, concrete cores are taken from the concrete structure to determine the "apparent" or "effective" diffusion coefficient and "apparent" surface concentration by the fitting the field data to Fick's 2nd law of diffusion. At a shallow depth from the slab surface, the chloride concentration reaches a maximum value that can be assumed quasi-constant after an initial exposure time (Weyers *et al.* 1994). This value is referred to as the "apparent" surface chloride concentration. It is generally obtained from regression analysis as the best fit of Crank's equation to the chloride profiles obtain from field data.

The rate of chloride diffusion in concrete, usually measured by the chloride diffusion coefficient, depends on many other concrete properties (water-to-cement ratio, temperature, cement type, age, moisture, etc.). An apparent value of the chloride diffusion coefficient can be obtained in the field from cored concrete samples of the investigated deck. A regression analysis is performed to best fit the chloride profiles measured from these cores with Crank's equation. The apparent diffusion coefficient thus obtained implicitly includes the effects of the influencing parameters mentioned above and can vary with time, especially during the first 5-10 years after the deck construction or if the environmental conditions change.

The corrosion initiation period ends when the amount of chlorides at the top reinforcing steel reaches the threshold chloride content (C_{th}). To predict the time of corrosion initiation (t_i), the chloride concentration is set equal to the chloride threshold value C_{th} , and the variable x (the

depth from the surface) is set equal to the effective concrete cover depth of the reinforcing steel, c. Equation (11) becomes:

$$t_i = \frac{c^2}{4D_c \left[\operatorname{erf}^{-1} \left(1 - \frac{C_{th}}{C_s} \right) \right]^2} \quad (12)$$

The determination of the chloride threshold concentration (C_{th}) is a very complex problem as it depends on the type of reinforcement, electrochemical environment in concrete, and testing method and consequently, a considerable variation is expected for this parameter. Recent studies of the chloride threshold values showed that, for different types of reinforcement (e.g. carbon steel, stainless steel, galvanized steel) that normally characterizes the chloride threshold there are some variability, which is related to other external variables, such as w/c ratio, humidity, aggregates, chloride binding etc. Glass and Buenfeld (1997) suggested that a threshold value can be expected to fall into a range of 0.03 to 0.07% by concrete weight. Bamforth *et al.* (1997) reported a variation of C_{th} from 0.06% to 2.2% for carbon steel (by cement weight). The threshold chloride content typically used in North America is 0.6 kg/m³ to 0.9 kg/m³ or 0.17% to 0.26% of cementitious material weight.

3.3 Modelling corrosion propagation time using Faraday's Law

The mass of reinforcing steel m_s that was consumed over time is related to the amount of current I_{cor} that flows through the electrochemical cell (i.e. the anodic and cathodic bars areas and the pore solution through which the ions are conducted). There is a considerable uncertainty associated with the determination of the actual volume increase due to corrosion and the corresponding composition of corrosion products. It depends on several factors, including chloride concentration, oxygen supply, and moisture. The typical oxides that are formed during the corrosion process have the following relative volume ratios compared to the volume of iron (Fe): FeO=1.7; Fe₃O₄=2; Fe₂O₃=2.1; Fe(OH)₂=3.6; Fe(OH)₃=4.0; and Fe(OH)₃·3H₂O=6.15

(Rosenberg et al.1989). The quantitative relationship between the metal reacting m_s and the current is defined by Faraday's law as follows:

$$m_s = \frac{I_{cor} t a}{nF} \quad (13a)$$

where I_{cor} is the corrosion current in amperes (A); t is time in seconds (s), a is the atomic mass of the ion being dissolved (for Fe, $a=55.85\text{g/mol}$), n is the valency of the reaction that depends on the type of corrosion products formed (e.g. $n=2$ for Fe(OH)_2) and F is Faraday's constant $F=96,500$ Coulomb/mol, where a Coulomb is defined as the amount of electricity represented by 1A flowing for 1s (Uhlig 1963). The corrosion rate (r) can be obtained by dividing m_s by the anodic surface area and time, as follows:

$$r = \frac{i_{cor} a}{nF} \quad (13b)$$

where i_{cor} is the corrosion current density (in A/cm^2). This model assumes a constant rate of accumulation of corrosion products (Bažant 1979; Andrade et al. 1993). Liu and Weyers (1998) proposed a time-decreasing model for the rate of accumulation of corrosion products, which is due to the fact that the diffusion of iron ions is inversely proportional to the oxide layer thickness. A more practical unit of measurement of corrosion rate is the rate of penetration that is equivalent to the rate of diameter loss $\Delta \dot{d}$, which is obtained by dividing d by the density of the metal. Hence, for steel, ($n=2$), Equation (13b) becomes:

$$\Delta \dot{d} = 0.023 i_{cor} \quad (13c)$$

From Eq. (13c), the total diameter penetration or diameter loss during the corrosion propagation time is given by:

$$\Delta d = 0.023 i_{cor} t_p = 0.023 i_{cor} (t_s - t_i) \quad (13d)$$

In the next section, the increases in the rebar diameter (Δd) related to different stages of corrosion-induced damage will be determined by modelling the system made of “corroding rebar and surrounding concrete cover” as a thick-walled cylinder, as explained below.

3.4 Thick-walled cylinder model of corrosion-induced damage in RC decks

The accumulation of corrosion products over time generates contact pressures between the rebar and the surrounding concrete, which initiate cracks when the tensile stresses in the concrete cover reaches the tensile strength (f'_t). The mechanical behaviour of the corroding rebar and surrounding concrete system and its stress analysis can be simplified by modelling the rebar-concrete cover system as a thick-walled cylinder, which is subjected to the action of uniformly distributed pressure as illustrated in Figure 4(a), assuming that concrete is an homogeneous elastic material (Bažant 1979). The external radius and internal radius of the thick-walled concrete cylinder are defined by the concrete cover depth (c) and rebar diameter (d) as $r_{ext} = c + d/2$ and $r_{int} = d/2$. The deformation is assumed symmetrical with respect to the cylinder axis and consists of a radial displacement of all points in the wall of the cylinder. Radial compressive stresses (σ_r) and tangential tensile stresses (σ_t) are generated in the concrete cover by the internal pressure (p_i) due to the expansion of the corrosion products, as shown in Figures 4(a) and 4(b). The stress of interest is the maximum tensile tangential stress, which is obtained at the inner surface of the cylinder, i.e. at the steel-concrete interface, as follows:

$$\sigma_t^{\max} = p_i \frac{(c + d/2)^2 + (d/2)^2}{(c + d/2)^2 - (d/2)^2} \quad (14a)$$

The internal pressure (p_{cr}) that induces the first internal crack is easily obtained from Eq. (6a). The maximum radial displacement (u_{cr}) corresponding to the onset of internal cracking is obtained as follows:

$$u_{cr} = \left[1 + \nu - \frac{\nu d^2}{(c + d)^2 + c^2} \right] \frac{f'_t}{E_c} d \quad (14b)$$

where E_c and ν are Young's modulus and poisson's ratio of concrete.

Hence, this model allows the calculation of the increases in the rebar diameter (Δd) related to different stages of corrosion-induced damage by calculating the radial displacements at the inner surface of the cylinder, as follows (assuming that the external pressure induced by external loads on the structure is small or negligible). Once an internal crack has initiated at the steel/concrete interface, it starts to propagate through the concrete cover as the corrosion products accumulate further and induce time-increasing tangential stresses in the outer tensile concrete rings as shown in Figure 4(b). The ultimate tensile load capacity of the cylinder will be reached when the crack propagates through the entire concrete cover. However, concrete behaviour remains still linear elastic in the outer part where the tangential stresses are still below f'_t . The internal pressure p_i acting at the steel/concrete interface is now transferred through the concrete teeth between cracks to the uncracked part of the cylinder. This internal pressure from the corroding bar on the inner area of the internally cracked ring must be reduced in comparison with the pressure on the inner surface of the uncracked ring (Tepfers 1979; Lounis *et al.* 2006). The uncracked ring of the concrete cover is then modelled as a thick-walled cylinder in itself and analyzed using equations (6a) and (6b) for determining the maximum tangential stresses and radial displacements for the appropriate cylinders of interest. The increase in diameter (Δd) due to accumulation of corrosion products (or rust) around the rebar is solved for by calculating the radial displacement at the inner surface of the cylinder, as follows:

$$\Delta d = \frac{p_i}{E_c} \left[1 + \nu + \frac{d^2}{2c(c+d)} \right] d \quad (14c)$$

The corrosion propagation times (t_p), corresponding to the onset of internal cracking, surface cracking, and delamination/spalling, respectively, can be determined as a function of the corresponding increase in rebar diameter as follows:

$$t_p = \frac{\pi d (\Delta d)}{2 S j_r \left[\frac{1}{\rho_r} - \frac{\alpha}{\rho_s} \right]} \quad (14d)$$

where S is the rebar spacing; ρ_r is the density of corrosion products (assumed at 3600 kg/m^3 for $\text{Fe}(\text{OH})_3$); ρ_s is the density of steel (7860 kg/m^3); α is the molecular weight ratio of metal iron to the corrosion product (assumed at 0.52); and j_r is the rust production rate per unit area (Bažant 1979), which is related to the corrosion rate of Eq.(13b), as follows:

$$j_r = \frac{\rho_{cor} d(\Delta d)}{St_p} = \frac{(0.023 \times 10^{-6}) d \rho_{cor} (i_{cor})}{S} \quad (14e)$$

Simplified models for corrosion-induced delamination and spalling of the concrete cover were proposed by Bažant (1979) and Lounis *et al.* (2006). After the propagation of an internal crack through the entire concrete cover, the stresses generated by the accumulation of corrosion products around the reinforcing bar will be sustained until part of the cover spalls off or delaminates. The corresponding propagation times to onset of spalling and onset of delamination are given by the following equations:

$$t_{p(sp)} = \frac{\pi}{2Sj_r \left[\frac{1}{\rho_r} - \frac{\alpha}{\rho_s} \right]} \left[1 + \nu + \frac{d^2}{2c(c+d)} \right] \frac{(2cd + d^2) f_t'}{E_c} \quad (14f)$$

$$t_{p(del)} = \frac{\pi}{2Sj_r \left[\frac{1}{\rho_r} - \frac{\alpha}{\rho_s} \right]} \left[1 + \nu + \frac{d^2}{2c(c+d)} \right] (Sd - d^2) \frac{f_t'}{E_c} \quad (14g)$$

3.5 Formulation of limit states of performance RC decks and uncertainty modelling

RC bridge decks may be considered as “failed” if they exceed a particular state, called a limit state, beyond which its performance or use is impaired. A limit state is the border that separates the desired state from “failed” state. The term “failure” is used, in this paper, in its general sense and refers to the violation of any specified performance requirement of the concrete bridge deck at any stage during its service life. Given the focus on corrosion-induced deterioration of RC decks, the limit states considered in this paper refer to excessive local damage (e.g. corrosion and bar diameter loss, cracking, spalling, delamination, loss of strength), which affect the use, durability, appearance and functionality of bridge decks. Furthermore, the limit states of local damage can become serviceability limit states of excessive deformations and vibrations if the deck stiffness is significantly reduced from extensive corrosion-induced spalling and delamination. At the extreme, excessive corrosion and localized damage can significantly reduce the cross-section areas of the rebars, bond and flexural strength, which will affect the ultimate limit states due to loss of load capacity.

For RC bridge decks built in corrosive environments, six failure criteria or limit states can be defined for predicting their life cycle performance of RC decks, namely:

- (i) Reaching critical levels of chloride contamination of RC decks;
- (ii) Initiation of corrosion of top mat of reinforcing steel;
- (iii) Onset of internal cracking at the steel-concrete interface;
- (iv) Onset of cracking at the deck surface;
- (v) Onset of spalling ; and
- (vi) Onset of delamination.

The limit states are selected as they can correspond also to the triggering of different types of management actions for RC bridge decks, which can include: (i) visual inspection; (ii) non-destructive evaluation, using half-cell potential, ground penetrating radar, etc.; (iii) protective membrane for the deck; (iv) cathodic protection; (v) patch repair; (vi) deck rehabilitation; (viii) bridge posting; and (ix) deck replacement.

Depending on the variable of interest, the corresponding performance function can be formulated, generally, as the difference between a term that is equivalent to a “resistance” and a term that is equivalent to a “load or load effect”. For example, the term “resistance” can be used to refer to the maximum permissible chloride concentration at the steel level, or chloride threshold level C_{th} (which defines the resistance to corrosion of a given reinforcing steel type), or resistance of the concrete cover to corrosion-induced cracking, or spalling, or delamination. Similarly, the term “load effect” can be used to refer to the chloride concentration at the steel level, or the corrosion-induced stresses that lead to cracking, spalling, etc.

As discussed in the above sections, the performance functions that describe the above failure modes or limit states are complex nonlinear functions of several parameters. Furthermore, considerable uncertainty is associated with the mechanisms and the parameters that govern chloride ingress into concrete, corrosion initiation, corrosion rate, accumulation of corrosion products and corrosion-induced deterioration of reinforced concrete decks. The uncertainty in the models and parameters is taken into account by modeling all governing parameters as random variables. These performance functions depend on several variables (e.g. concrete cover, diffusion coefficient, bar diameter, bar spacing, etc.). We define the generalized performance function of an RC bridge deck as follows:

$$g(\mathbf{X}) = g(X_1, X_2, \dots, X_n) \quad (15a)$$

where $\mathbf{X} = (X_1, X_2, \dots, X_n)$ is the vector of the performance variables of the deck, and the function $g(\mathbf{X})$ defines the performance, and consequently the service life of the deck. The time-dependent performance functions of RC decks can also be formulated as follows:

$$\text{For } t < t_i \quad f(X) = t_i - t \quad (15b)$$

$$\text{For } t \geq t_i \quad f(X) = t_i + t_p - t \quad (15c)$$

As discussed in the previous sections, the considerable variability and uncertainty of the parameters that govern the chloride ingress into concrete, corrosion initiation, rate of corrosion and accumulation of corrosion products, cracking, spalling and delamination (e.g. concrete cover, chloride threshold, diffusion coefficient, surface chloride concentration, rate of corrosion) require the use of probabilistic models in which all governing parameters are modelled as random variables to provide reliable and meaningful predictions of the life cycle performance of RC bridge decks. The time-dependent probability of failure $P_f(t)$ is the probability that ($P_f = P[f(X) < 0$]) at a given time. The time-dependent probability of failure, in which failure can be reaching critical chloride concentration, onset of corrosion, onset of cracking, spalling or delamination, can be formulated as follows:

$$P_f(t) = P(t_i < t) \quad (16a)$$

$$P_f(t) = P([t_i + t_p] < t) \quad (16b)$$

The determination of the time-dependent probability of failure is a very complex problem due to the high level of nonlinearity of the performance functions. In this paper, the advanced first-order reliability method is used to determine the probability of failure for the above performance functions. The determination of the probability of failure can be formulated as a nonlinear optimization problem, in which the solution that minimizes the distance from the origin to the failure surface (described by any one of the above performance functions) is sought. This minimum distance was defined as a measure of the reliability index by Hasofer and Lind (1974). Once the reliability index (β) is determined, the probability of failure can be estimated as follows:

$$P_f(t) = 1 - \Phi(\beta) \quad (17)$$

In this paper, an iterative numerical algorithm was developed by Lounis and Daigle (2006) to solve the nonlinear optimization problem and determine the reliability index, which was then validated by Monte Carlo simulation. If the random variables are not normal, then the above approach provides only estimates of the probability of failure (Madsen *et al.* 1986). It should also be pointed out that if the investigated governing parameters exhibit considerable spatial variability within the deck, the use of random fields instead of random variables for their modeling may be required (Stewart 2003; Lounis *et al.* 2006). The use of random fields, however, requires more input data such as the correlation function and scale of fluctuation, and adds considerable analytical and computational complexity to the analysis. If the variability of the parameters can be assumed to be completely random, i.e. no systematic variability and no spatial correlation, then the use of random variables for modelling the above parameters are adequate (Lounis *et al.* 2006).

4. Illustrative Example

4.1 Description of bridge deck

To demonstrate the benefits of the proposed life cycle design and sustainable management approach, a typical reinforced concrete bridge deck is used for the analysis as it is the highest risk element for the bridge network from a viewpoint of deterioration and maintenance costs as a large percentage of funds of departments of transportation are allocated for the maintenance and renewal of bridge decks. Any improvements made in the design and management will lead to significant cost savings to the owners and users of highway bridges.

The bridge element under consideration is a 225-mm thick reinforced concrete (RC) slab deck. The slab has a steel reinforcement ratio of 0.3% for both top and bottom mats in both transverse and longitudinal directions, including the main reinforcement and reinforcement for shrinkage, temperature and distribution, and a concrete cover thickness of 75 mm for the top reinforcement, according to the Canadian Highway Bridge Design Code (CAN | CSA 2006). The concrete deck has a width of 12.57 m and a length of 47 m between expansion joints, and is carried by simply-supported prestressed concrete girders. The bridge superstructure and substructure are not considered in the analysis. Traffic data are given in Table 2 along with a summary of the above information.

In this example, it is considered that the end of service life or serviceability limit state of RC decks in corrosive environments is reached when an estimated 30% of the deck area is distressed. This percentage corresponds to a condition state “4” out of five categories of deck condition with increasing damage (Table 1) as established by the AASHTO Guide (1998, 2002) which is also similar to the criteria used in the bridge management systems of several Canadian provinces and states in the USA.

Two alternative designs and management strategies for the construction, inspection, maintenance, repair, rehabilitation and replacement of RC bridge deck are evaluated and compared in terms of owners costs, users costs, and environmental impacts ; (i) RC deck made with high performance concrete (HPC) containing supplementary cementing materials (SCM) and reinforced with conventional carbon steel and; (ii) RC deck made with normal performance concrete (NPC) and reinforced with conventional carbon steel. SCMs are reclaimed industrial by-products that improve concrete physical performance and extend the service life of RC structures as a result of reduced permeability (Lounis *et al.* 2006b). The HPC deck contains 25% of fly ash and has a 28-day compressive strength of 45 MPa. The NPC deck water/cement ratio of 0.4 is associated to a 28-day compressive strength of 30 MPa. The reinforcement consists of #10M conventional black steel rebars with a yield strength of 400 MPa for both alternatives.

The mean value for cover depth, bar spacing and bar diameter (Table 3) are taken from the construction specifications and variations can be estimated based on the quality control of workmanship. A surface chloride concentration of 6 kg/m^3 is assumed, which is within the range proposed by Weyers (1994) for typical severe conditions found in Canada and Northern USA. For the NPC alternative, a chloride diffusion coefficient of $0.4 \text{ cm}^2/\text{year}$ is estimated based on the work of Pedersen and Arntsen (1998) on a similar concrete mix ($w/c=0.4$) and under temperature conditions that approximate temperature variations experienced in Canada and Northern USA (yearly variation of -15°C to 25°C). A chloride diffusion coefficient of $0.2 \text{ cm}^2/\text{year}$ is used for the HPC deck (Bamforth and Chapman-Andrews 1994). Chloride diffusion coefficients for concrete containing fly ash cover quite a wide range in the literature. The value of $0.2 \text{ cm}^2/\text{year}$ is quite conservative considering that the difference in chloride diffusion coefficients between NPC and HPC with fly ash can be higher than an order of magnitude (Lounis *et al.* 2006b). The coefficient of variations for surface chloride content (25%) and the

coefficients of diffusion (25%) are based on variations observed from measurements at one specific field test site (Lounis et al. 2006; Cusson *et al.* 2008).

A chloride threshold of 0.7 kg/m^3 with a coefficient of variation of 20% is assumed for the conventional steel reinforcement (ACI 222, 2001), above which corrosion can initiate; and a moderate corrosion rate of $0.5 \text{ } \mu\text{A/cm}^2$ is assumed once corrosion has started (Rodriguez *et al.* 1994). It can be argued that these values may differ from actual values depending on the severity and variability of the local exposure conditions; however, the results of this example is comparative (the two alternatives are compared using the same sets of values for surface chloride concentration, chloride threshold, and corrosion rate) and serve to illustrate the life cycle design approach.

4.2 Prediction of time-dependent probability of failure

The proposed models can be used to predict the times at which different limit states are reached. Here, the focus is on the limit state of spalling. The time-dependent probability of failure due to spalling for both deck alternatives is calculated using the models described above and the bridge and service life parameter data given in Table 3. Considering that the bridge deck can be divided into a very large number of small surface areas with same probability ($P\%$) of spalling after a number of years (t), it can be approximated that after (t) years a percentage ($P\%$) of them are spalled. From Figure 5, the end of service life is assumed reached after 22 years for the NPC deck and 40 years for the HPC deck, corresponding to a 30% probability of failure due to spalling in both cases. For this example, spalling is the dominant failure mode due to the deck design and geometry.

4.3 Multiobjective decision-making for design and management of bridge

As discussed in previous sections, the satisficing solution that achieves the best trade-off between the objectives of minimum owner's costs, minimum users costs and minimum environmental impacts. The initial construction costs (in-place costs for the steel and concrete mixes) of the two alternatives are presented in Table 4. Additional construction costs are not specified for the two alternatives as they are considered to be approximately the same. Figure 6 summarizes the timing and costs of the different maintenance, repair and rehabilitation (MR&R) activities of both alternatives. The cost and time data presented in this example are taken from various sources (Lounis and Daigle 2008). The costs of the patch repairs include:

- Cost of removing the contaminated or deteriorated concrete.
- Cost of concrete patching.
- Cost of traffic control when the bridge is partially opened to traffic.

In this example, patch repairs are made when the deck spalling area reaches 10% and 20% as outlined in Figure 5. It is assumed that after 22 years, the damaged NPC deck is replaced with a similar type of deck (i.e. normal concrete with “carbon” or “black” steel reinforcement in both top and bottom mats). The replacement cost includes the initial construction cost and the costs of demolition and disposal that were assumed equal to \$70/m². Since the end of life of the HPC deck is equal to the analysis period, its replacement is not included in the life cycle cost analysis (LCCA). At the end of the analysis period, the HPC alternative will have no residual service life or value whereas the NPC deck will have a four-year residual life and a residual value calculated as 18% (remaining service life over predicted service life) of the replacement cost. For the NPC alternative, the schedules of MR&R activities for the replacement deck (year 22 and after) are similar to those of the initial deck construction (up to year 22).

Users costs considered in this example include the costs incurred by the users of the bridges (lost driver’s time and increased vehicle operating costs) due to traffic delay caused by MR&R activities. The duration of each (MR&R) activity (average for both ways), length of affected road during the activity (average for both ways), and reduced traffic speed during the activity (Table 5) are used to estimate life cycle user costs. These values are considered to be the same for both deck alternatives except for the replacement option that is only affecting the NPC deck. The different unit or hourly user costs are found in Table 6. Reference values for C_{VOC} and C_{delay} can be found in Ehlen and Marshall (1996) and Wall and Smith (1998).

Figure 7 illustrates the results of the life cycle cost analysis (LCCA) that was evaluated using the present value life cycle cost (PVLCC) approach. The difference between the agency and user life cycle costs of the two alternatives favours the use of HPC decks as there is a reduction of 20% and 70% of these costs, respectively. When based only on initial (partial) construction costs, the NPC deck alternative would seem to be a slightly better choice.

In this example, environmental impacts, in terms of CO₂ emissions and waste production are estimated for all MR&R activities occurring during the life cycle of both concrete deck alternatives as outlined in the economic performance analysis as shown in Figure 8 and Figure 9.

In this example, it is found that the CO₂ emissions for the normal concrete deck alternative are almost three times higher than those of the HPC deck alternative (Fig. 8). This difference is mainly due to the lower cement consumption of the HPC mix that uses supplementary cementing materials as a replacement material for a portion of the cement. The shorter service life of the normal concrete deck, which leads to an increase in traffic disruption due to earlier replacement, also accounts for the higher CO₂ emissions of the NPC deck.

A comparison of the waste produced (or landfill use) for the two deck alternatives is shown in Figure 9, which includes the volume of waste material produced during the replacement of asphalt overlay, patch repairs, and deck replacement. Traffic delays are first considered in the analysis of the life cycle economic performance as they result in increased costs for vehicle drivers and could have potential impacts on the economy of a whole region.

The social impacts of traffic delays, measured in time lost by drivers, are generally associated to a reduced quality of life. Average accident costs and the normal accident rate are taken from statistics published by Transport Canada (1994, 2006) and Statistics Canada (2003). Considering the lack of statistical data on accident in work zones, a rule of thumb of three times the normal accident rates is used as suggested by Walls & Smith (1998). Data taken for this example are presented in Table 7.

Figure 10 shows the accident cost per deck area for both deck alternatives, as well as the total time lost by drivers due MR&R activities. As for environmental impacts, the shorter service life of the NPC deck and its required replacement after 22 years, which causes an increase in traffic disruption, greatly affect the social performance of the NPC deck.

From the example above, the following conclusions can be drawn:

- In terms of durability, the HPC deck alternative incorporating SCMs has a service life that is almost twice as long as the service life of normal concrete deck.
- In terms of environmental impacts, the HPC deck alternative yields a reduction of 65% in the CO₂ emissions compared to the normal concrete deck.
- In terms of life cycle costs, the HPC deck alternative is found to be more cost-effective than the normal concrete deck for both owner costs and user costs.
- In terms of users' costs, time lost and accident cost associated to the HPC deck alternative are estimated to be less than half of what was estimated for the NPC deck.

From Table 8, it is seen that the HPC deck alternative selected is a satisficing solution. On the other hand, the normal concrete (NPC) deck alternative is a dominated solution or a non-Pareto optimum as HPC dominates NPC for the three investigated objectives of minimum owner costs; minimum users' costs; and minimum environmental impacts.

Assume now that the decision-maker has serious budgetary constraints and has limited funds to invest. Therefore, the minimization of initial construction costs becomes an important objective to the decision-maker. From Fig.7 and Table 8, it is seen that the initial construction costs are 133 and 146 (\$/m²) for the NPC deck and HPC deck alternatives, respectively. Hence, the NPC solution is not anymore a dominated solution as it is better than the HPC solution in terms of initial costs. Using the multiobjective decision-making approach presented in Section 2, the so-called ideal but non-existent solution is the one associated with the minimum values for all objective functions, i.e. $\mathbf{x}^*=(f_{1min}, f_{2min}, f_{3min}, f_{4min}, f_{5min})=(263, 79, 53, 0.17, 133)$. Assuming equal weights for all objectives and using the Euclidian L2 metric in Eq.8, the multiple criteria indices (MCI) for the HPC and NPC deck solutions are determined and are found equal to 1 and 4, which shows that the HPC deck solution is the best compromise solution even when considering the minimization of the initial construction costs as an objective of the optimization.

4. Summary and conclusions

A multiobjective decision-making approach is proposed for the implementation of sustainability in the design and management of highway bridge decks. This approach provides decision-makers with a practical tool to consider several and conflicting objectives in selecting optimal design and maintenance strategies of highway bridge decks, including the minimization of owners' costs, minimization of users' costs and minimization of environmental impacts.

In this paper, it is shown that the implementation of multiobjective decision-making for bridge management would lead to high performance highway bridge decks with low life cycle owners and users' costs as well as reduced environmental footprints in terms of lower CO₂ emissions and lower waste materials than conventional normal concrete bridge decks. The use of HPC materials results in bridge decks with longer service lives, reduced life cycle costs, and

better environmental and social profiles when compared to conventional normal concrete bridge decks.

The implementation of sustainable design and management practices for highway bridges requires a priori reliable estimation of service life and associated schedules and extents of MR&R activities. Beyond their significant influence on economic performance, they also drive the results of the users' costs (or social impacts) and environmental impacts analysis. For bridges with high level of traffic, the social and environmental "costs" of frequent and/or extended interventions should be taken into consideration to move towards a sustainable approach for the design and management of highway bridges. A strong emphasis on the durability of all components and of the overall structure, as well as a reduced need for extensive repairs, rehabilitation and replacement constitute a major step towards the sustainable management of infrastructures. High-priced technologies, designs or policies aimed to extend service life and/or reduce maintenance needs must be evaluated within a life cycle and multiobjective decision-making framework including economic, environmental and social performance criteria to get a more comprehensive evaluation of the impacts and "true costs" of current decisions on present and future generations.

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Table 1. Description of bridge deck condition states (AASHTO 2002)

| Condition state | Description |
|------------------------|---|
| 1 | The surface of the deck has no patched area and no spall in the deck surface. |
| 2 | The combined distress area (i.e. existing patches, delamination and spalling) of the deck is less than 10%. |
| 3 | The combined distress area of the deck is between 10% and 25%. |
| 4 | The combined distress area of the deck is between 25% and 50%. |
| 5 | The combined distress area of the deck is more than 50%. |

Table 2. General information on highway bridge example.

| | |
|--|---------|
| Bridge width | 12.57 m |
| Bridge length | 47.5 m |
| Deck thickness | 225 mm |
| Isotropic reinforcement percentage for both mats | 0.3% |
| Annual Average Daily Traffic (AADT) | 22000 |
| Annual Average Daily Truck Traffic (AADTT) | 4500 |
| Normal traffic speed (km/hr) | 100 |

Table 3. Service life parameters.

| Parameter | Mean value | COV* (%) |
|---|-------------------|---------------------|
| Concrete cover depth (mm) | 70 | 25 |
| Bar spacing (mm) | 150 | 5 |
| Bar diameter (mm) | 9.5 | - |
| Surface chloride content (kg/m ³) | 6 | 25 |
| Chloride (apparent) coefficient of diffusion (cm ² /year) - NPC | 0.40 | 25 |
| | 0.20 | 25 |
| Chloride (apparent) coefficient of diffusion (cm ² /year) – HPC | | 20 |
| | 0.70 | |
| Threshold chloride content (kg/m ³) | | |
| Corrosion rate (μA/cm ²) | 0.5 | 20 |

* COV = coefficient of variation

Table 4. In-place costs of materials.

| Material | Unit Cost |
|---------------------------------|--------------------------|
| Normal concrete (NPC) | 460 (\$/m ³) |
| High performance concrete (HPC) | 520 (\$/m ³) |
| Conventional (“black”) steel | 1.80 (\$/kg) |

Table 5. Data for estimation of users costs.

| Activities | Duration (days) | Length affected (km) | Traffic speed during MR&R (km/hr) |
|------------------------|----------------------------|-------------------------------------|--|
| Routine inspection | 0.35 | 0.1 | 80 |
| Detailed inspection | 0.5 | 0.5 | 50 |
| Asphalt overlay | 1.5 | 1 | 40 |
| Patch repair | 2.5 | 1 | 30 |
| Replacement | 15 | 1 | 30 |

Table 6. Average unit user costs.

| | |
|--------------------------------|------|
| Value of driver's time (\$/hr) | |
| Car | 12 |
| Truck | 20 |
| Vehicle operating cost (\$/hr) | |
| Car | 8.85 |
| Truck | 8.85 |

Table 7. Accident rates and cost.

| | |
|--------------------------------------|--------|
| Normal accident rate | 2.1 |
| (per million vehicle * km) | |
| Accident rate during MR&R activities | 6.0 |
| (per million vehicle * km) | |
| Average accident cost (\$) | 33,000 |

Table 8. Values of objective functions for HPC and NPC deck solutions

| Objective function value (f_i) | HPC deck | NPC deck |
|--|-----------------|-----------------|
| Service life (years)* | 40 | 22 |
| Owner's life cycle costs (\$/m ²)- f_1 | 263 | 327 |
| Users' life cycle costs (\$/m ²) – f_2 | 79 | 265 |
| CO ₂ emissions (kg/ m ²) – f_3 | 53 | 151 |
| Waste materials for land filling (m ³ /m ²) f_4 | 0.17 | 0.48 |
| Initial costs (\$/m ²) – f_5 | 146 | 133 |
| MCI | 1 | 4 |

**Service life is assumed as the time the probability of failure due to spalling reaches 30%.*

Figure 1. Schematic description of life cycle performance of bridge decks.

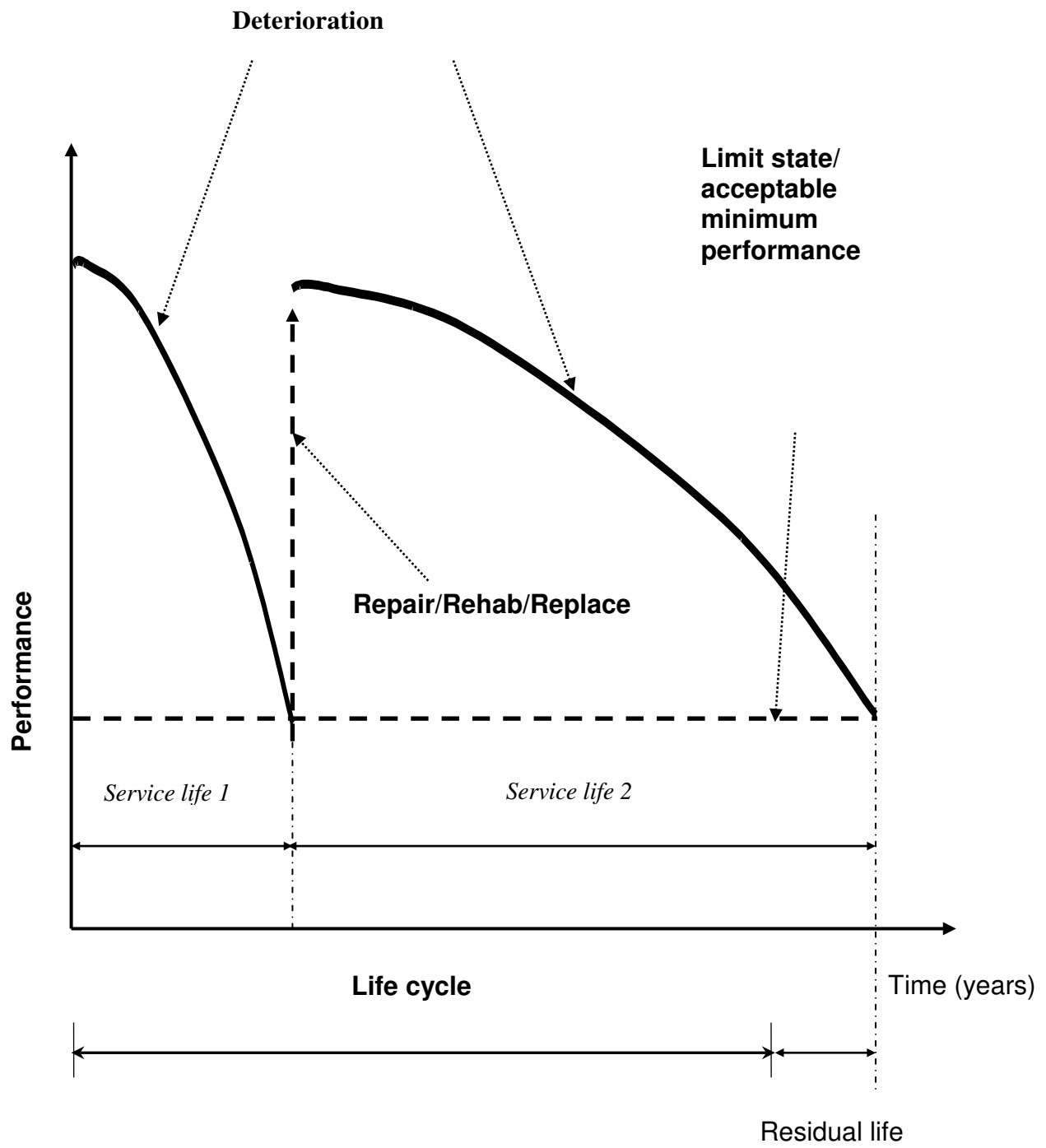


Figure 2. Schematic description of conflicting optimization objectives and Pareto optima

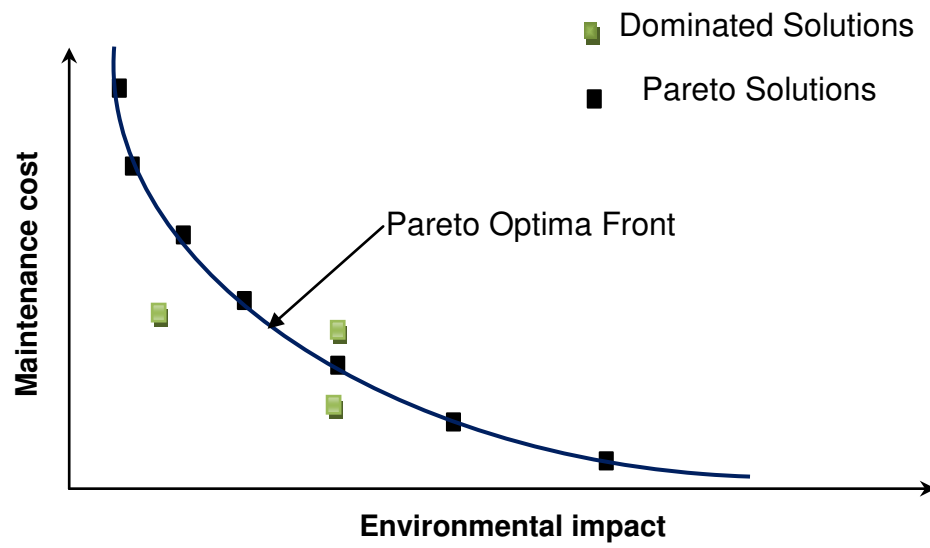
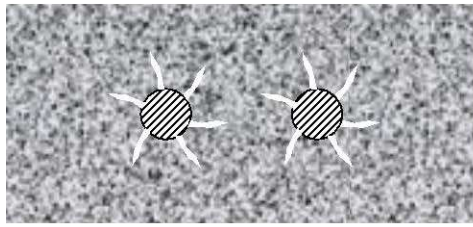
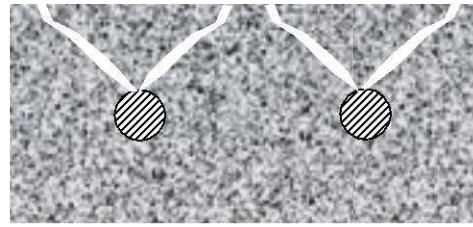


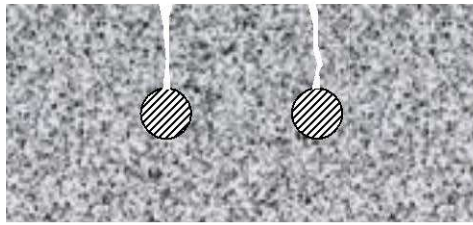
Figure 3. Corrosion-induced failure modes of RC bridge deck slabs.



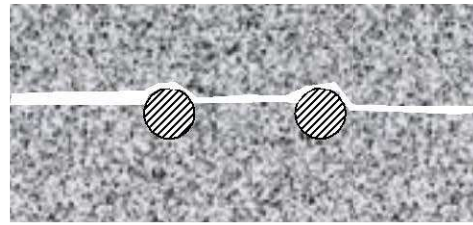
a) Internal cracking



c) Spalling



b) Surface cracking



d) Delamination

Figure 4. Thick-walled cylinder model of corroding RC deck: (a) Corrosion-generated tensile stresses and internal cracks; (b) Propagation of internal cracks in partially-cracked thick-walled cylinder

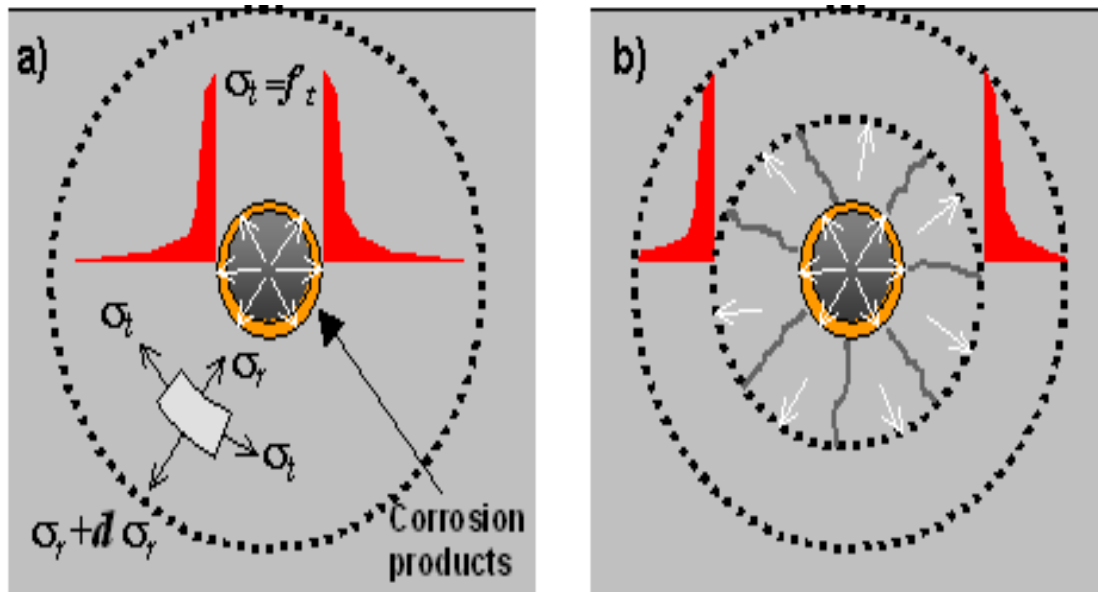


Figure 5. Time-dependent probability of failure of RC deck due to concrete cover spalling

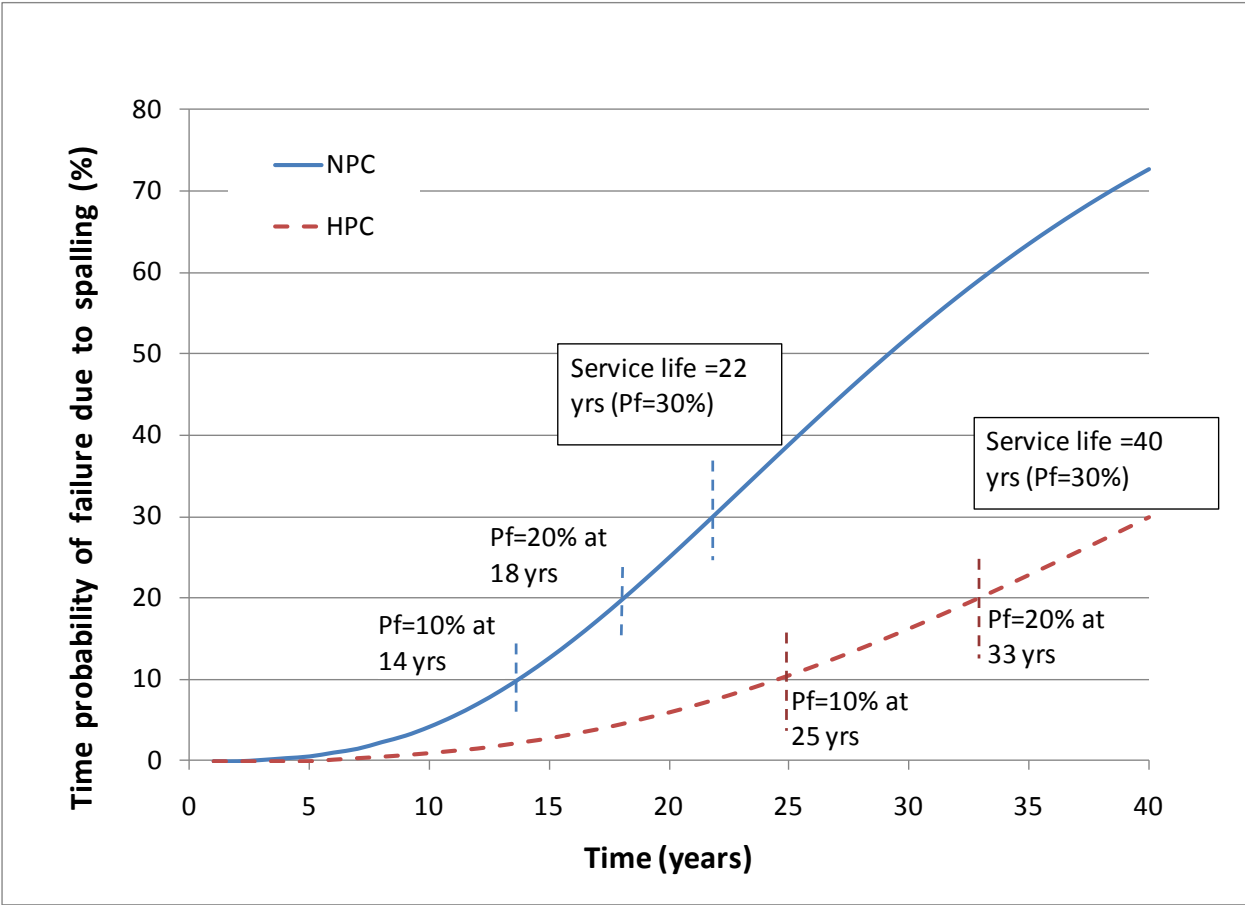


Figure 6. Unit costs and schedules of MRR activities for NPC and HPC decks over life cycles.

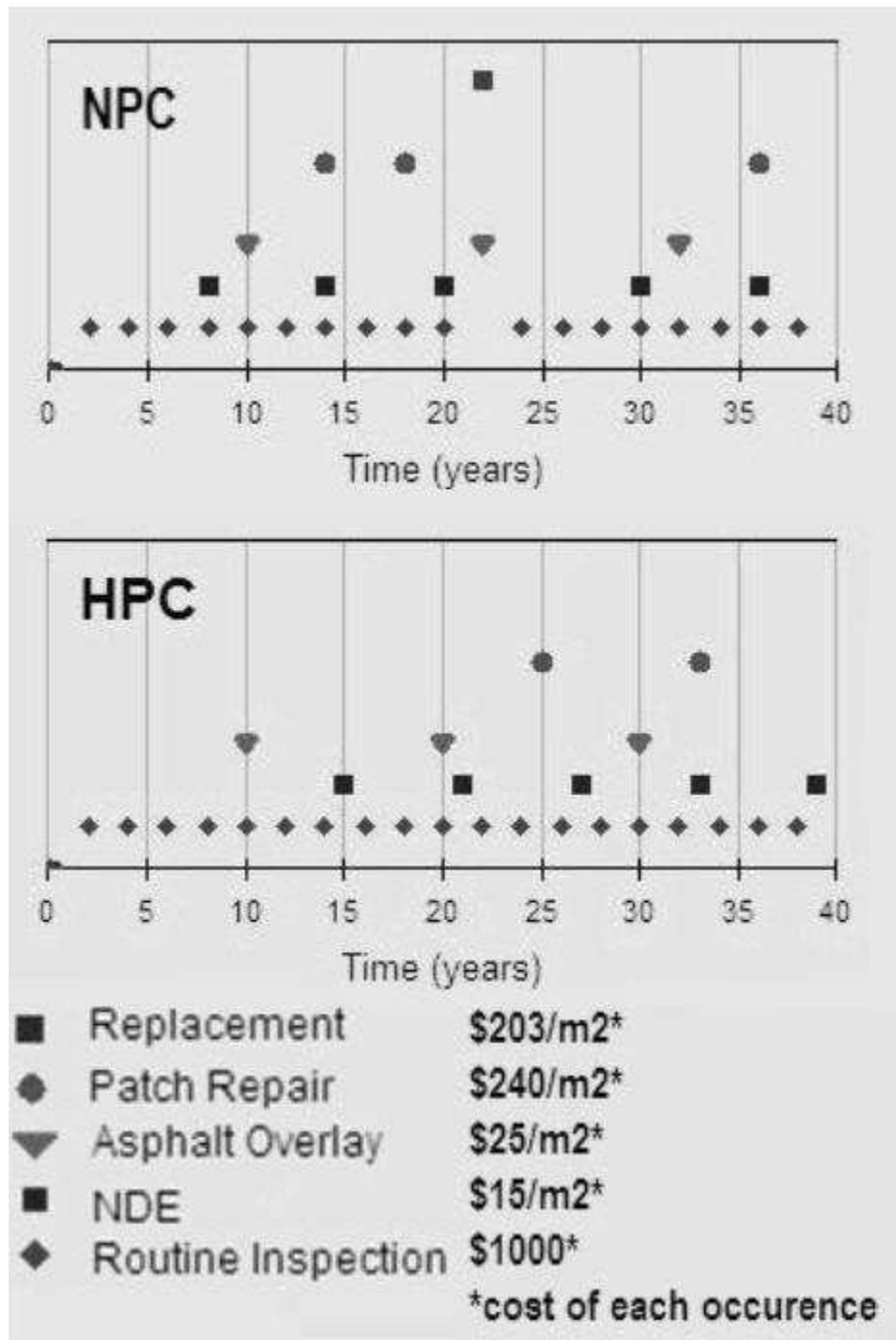


Figure 7. Initial unit construction costs and life cycle costs for NPC and HPC decks.

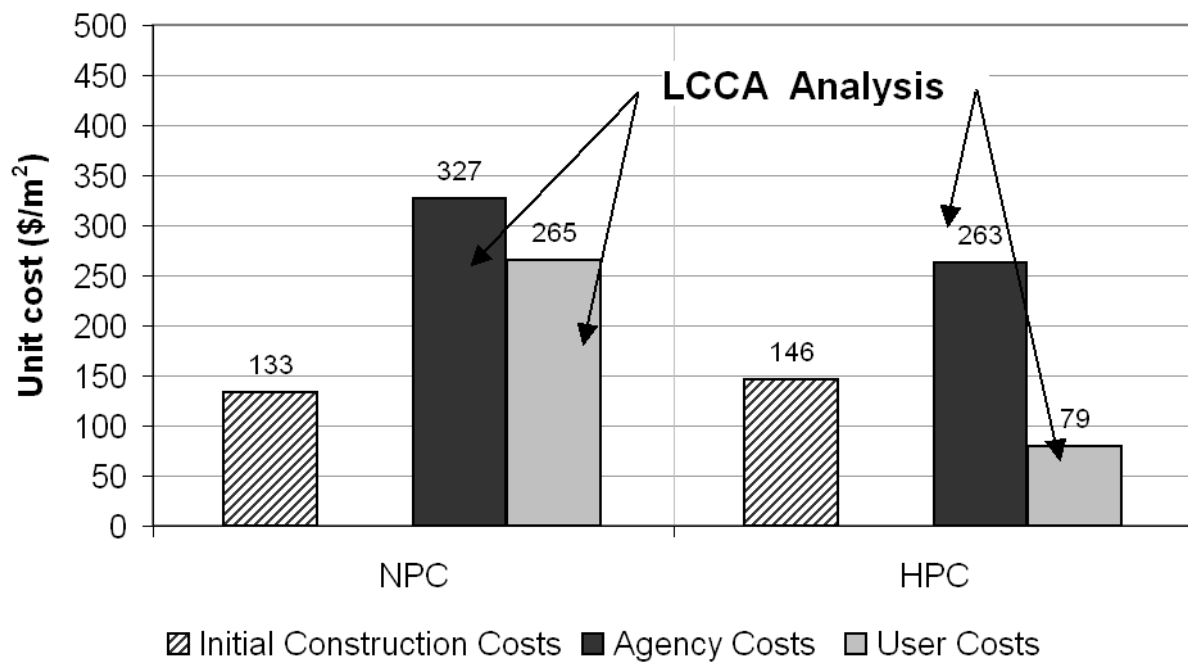


Figure 8. CO₂ emissions over life cycles of NPC and HPC bridge decks.

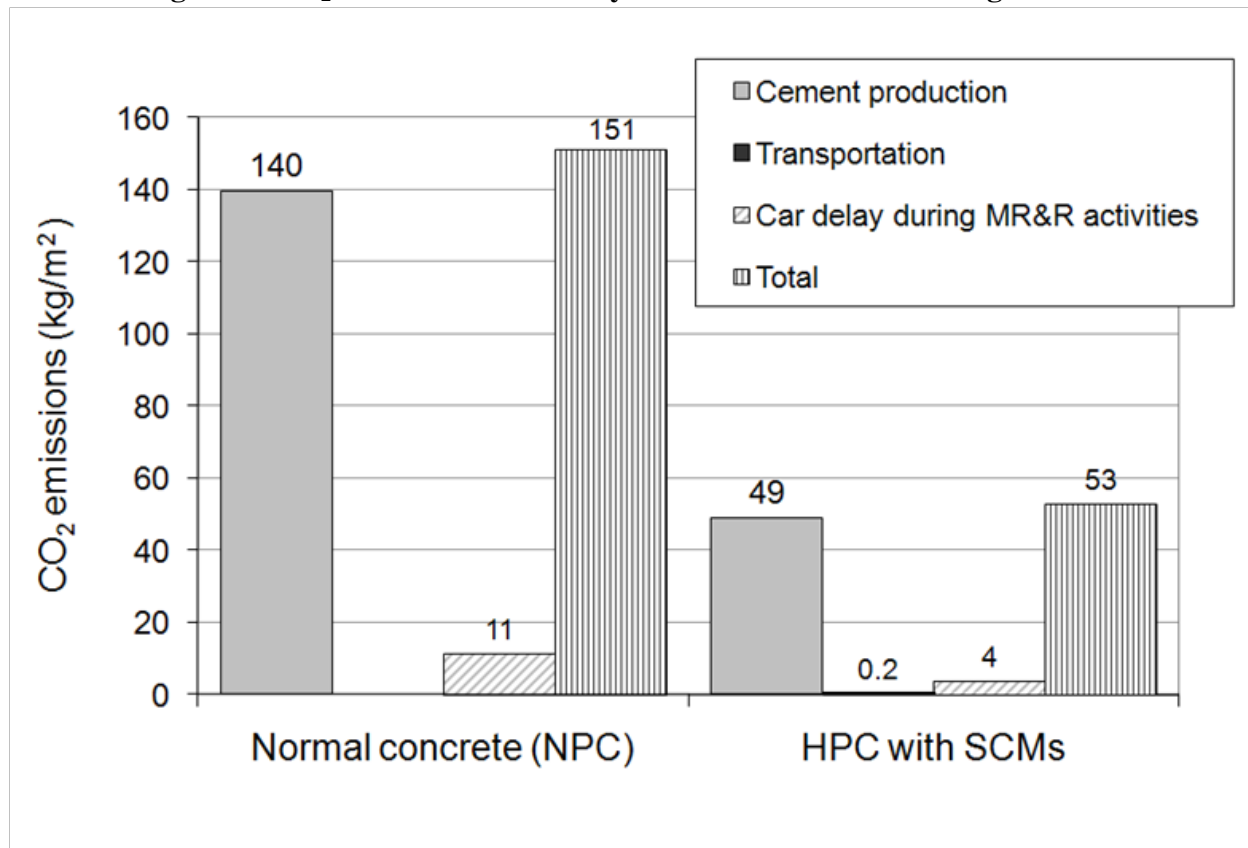


Figure 9. Volume of construction waste materials from NPC and HPC bridge decks.

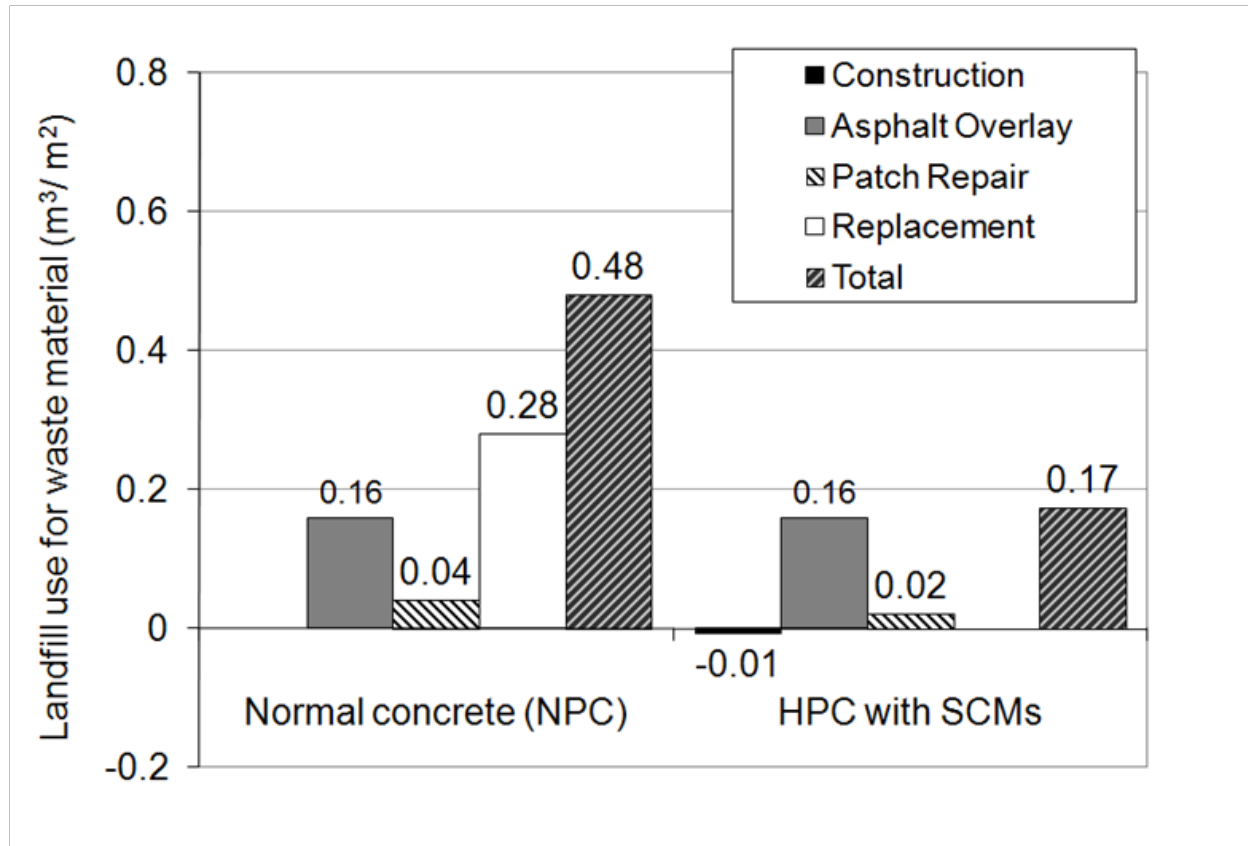


Figure 10. Unit accident unit costs and time lost for NPC and HPC deck alternatives.

