

Life-Cycle Performance of Deteriorating Structural Systems under Uncertainty: Review

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Abstract: In recent years, significant advances have been accomplished in the fields of modeling, analysis, and design of deteriorating civil engineering systems, and novel approaches to life-cycle assessment, maintenance planning, and optimal design of structural systems have been proposed. Despite these advances, life-cycle concepts are far from being explicitly addressed in design and assessment codes and effectively implemented into practice. There is, therefore, a need to promote further research in the field of life-cycle performance of structural systems under uncertainty and to fill the gap between theory and practice by incorporating life-cycle concepts in structural design and assessment codes. An effort is currently ongoing within the Structural Engineering Institute (SEI)/ASCE to meet this need. This paper is part of this effort and is aimed at presenting a review of the main principles, concepts, methods, and strategies for life-cycle assessment and design of deteriorating structural systems under uncertainty. General criteria for deterioration modeling are first presented, with emphasis on the effects of corrosion and fatigue in steel structures and chloride-induced corrosion in concrete structures. The time-variant structural performance is then investigated with reference to a set of probabilistic performance indicators, and the structural lifetime associated with a reliability target is formulated. The role of inspection and monitoring, the effects of maintenance and repair interventions, and the definition of cost-effective maintenance strategies are discussed. The concepts of life-cycle performance assessment and maintenance planning are used to formulate the life-cycle reliability-based design problem in an optimization context. DOI: [10.1061/\(ASCE\)ST.1943-541X.0001544](https://doi.org/10.1061/(ASCE)ST.1943-541X.0001544). © 2016 American Society of Civil Engineers.

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Introduction

The economic growth and sustainable development of modern society need to rely on reliable and durable civil engineering structures and infrastructure facilities. However, structure and infrastructure systems, owing to their inherent vulnerability, are at risk from aging, fatigue, and deterioration processes resulting from aggressive chemical attacks and other physical damage mechanisms (Ellingwood 2005). The detrimental effects of these phenomena can lead over time to unsatisfactory structural performance under service loadings or accidental actions and extreme events, such as natural hazards, e.g., earthquakes, hurricanes, and floods, and human-made disasters, e.g., vehicular collisions, fires, and explosion blasts due to terrorists' attacks.

Over the past decade, significant attention has been devoted in several countries to the condition rating of huge stocks of existing structures and infrastructures, including buildings, bridges, roads, railways, dams, ports, and other construction facilities (NCHRP 2006; ASCE 2013). The economic impact of aging and deterioration processes on all such systems is exceptionally high, particularly

for bridges and infrastructure networks. According to ASCE (2013), "In total, one in nine of the nation's bridges are rated as structurally deficient, while the average age of the nation's 607,380 bridges is currently 42 years. The Federal Highway Administration (FHWA) estimates that to eliminate the nation's bridge backlog by 2028, we would need to invest \$20.5 billion annually, while only \$12.8 billion is being spent currently. The challenge for federal, state, and local governments is to increase bridge investments by \$8 billion annually to address the identified \$76 billion in needs for deficient bridges across the United States."

These problems pose a major challenge to the field of structural engineering because the classical time-invariant structural design criteria and methodologies need to be revised to account for a proper modeling of the structural system over its entire life-cycle by taking into account the effects of deterioration processes, time-variant loadings, and maintenance and repair interventions, among others. In addition, because of the uncertainty in material and geometrical properties, in the physical models of the deterioration process, and in the mechanical and environmental stressors, a probabilistic measure of the time-variant structural performance is necessary for realistic results (Ang and Tang 2007). The evolution over time of the aleatory and epistemic uncertainty effects needs also to be properly considered. In fact, whereas aleatory uncertainty owing to randomness cannot be reduced, epistemic uncertainty associated with incomplete information and knowledge could be effectively reduced by improving knowledge and accuracy of predictive models by means of inspection and monitoring (Frangopol et al. 2008).

In recent years, significant advances have been accomplished in the fields of modeling, analysis, maintenance, repair, and design of deteriorating civil engineering systems, and novel approaches to life-cycle reliability assessment, maintenance planning, and optimal design of structural systems, have been proposed

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(Ellingwood and Mori 1993; Mori and Ellingwood 1994a, b; Frangopol et al. 1997a, b, 2001, 2002, 2004, 2008, 2012; Ellingwood 1998; Enright and Frangopol 1998b, c; Frangopol 1999; Stewart and Rosowsky 1998; Val et al. 1998; Enright and Frangopol 1999a, b; Estes and Frangopol 1999, 2001a, b, 2005; Vu and Stewart 2000; Ciampoli and Ellingwood 2002; Kong and Frangopol 2003b, 2004; Biondini et al. 2004, 2006a, b, 2008, 2011, 2014; Ellingwood 2005; Frangopol and Liu 2007; Biondini and Frangopol 2008, 2009, 2014; Ang 2011; Biondini 2011; Esteve et al. 2011; Moan 2011; Padgett et al. 2010; Sánchez-Silva et al. 2011; Alipour et al. 2013). These advances are of crucial importance to establish guiding policies and support decision-making processes for reliable design of durable structures and rational planning of maintenance, repair, or replacement of deteriorated existing structures. Furthermore, the availability of quantitative life-cycle performance metrics allows to effectively incorporate emerging sustainability and environmental issues in structural design, such as the effects of global warming and climate change (Bastidas-Arteaga et al. 2010, 2013; Stewart et al. 2011, 2012; Flint et al. 2014). Societal issues in adopting life-cycle concepts in the decision-making process may also play an important role within the political system to comply with the different methods, metrics, needs, and priorities addressed by public officials, infrastructure users, and owners (Corotis 2009). Further developments along these lines of research represent a priority and, nowadays, such developments are perceived to be at the heart of civil engineering, which is undergoing a transition toward a life-cycle-oriented design philosophy (Frangopol 2011).

Despite this research trend, life-cycle concepts are not yet explicitly addressed in structural design codes, and the checking of system performance requirements is referred to the initial time of construction when the system is intact (CEN 2002; CSA 2006; AASHTO 2010). In this approach, design for durability with respect to chemical-physical damage phenomena is based on simplified criteria associated with classes of environmental conditions. As an example, for concrete structures, such criteria introduce threshold values for concrete cover, water-cement ratio, amount and type of cement, among others, to limit the effects of local damage resulting from carbonation of concrete and corrosion of reinforcement. However, a durable design cannot be based only on such indirect evaluations of the effects of structural damage, but also needs to take into account the global effects of the local damage phenomena on the overall performance of the structure.

These considerations indicate that there is a strong need to promote further research in the field of life-cycle performance of structural systems under uncertainty and to fill the gap between theory and practice by incorporating life-cycle concepts in structural design codes and standards. To meet this need, an effort is currently ongoing within the SEI-ASCE Technical Council on Life-Cycle Performance, Safety, Reliability and Risk of Structural Systems, Task Group 1 on Life-Cycle Performance of Structural Systems under Uncertainty (Frangopol and Ellingwood 2010). This paper is part of this effort, and it is aimed at presenting a general review of the main principles, concepts, methods, and strategies for a life-cycle probability-based approach to assessment and design of deteriorating structural systems under uncertainty.

Deterioration Modeling

The life-cycle performance of structural systems is affected by time-variant deterioration effects of aging and damage processes of structural materials and components (Estes and Frangopol 2005). Aging and deterioration from environmental aggressiveness may also interact over time with damage induced by other natural

hazards. As an example, these effects can be particularly relevant for aging bridges exposed to excessive traffic loads or earthquakes and flood-induced scour (Stein et al. 1999; Guo and Chen 2015; Zhu and Frangopol 2016). This indicates that deterioration mechanisms are generally complex, and their effects and evolution over time depend on both the damage mechanisms and type of materials and structures. For steel structures, the main causes of lifetime deterioration are corrosion and fatigue. For concrete structures, there is a wider spectrum of aging and damage mechanisms that may seriously affect the life-cycle performance. These mechanisms include chemical processes associated with carbonation, leaching, sulfate and chloride attacks, reinforcing steel corrosion, and alkali-silica reactions; physical processes because of freeze/thaw cycles and thermal cycles; and mechanical processes such as cracking, abrasion, erosion, and fatigue (Ellingwood 2005). This paper is mainly concerned with the effects of structural deterioration at the component and system levels, and a comprehensive description of chemistry, physics, and mechanics of these damage processes and their effects at the material level is well beyond the scope of this paper. They are extensively described by, for instance, Fong (1979), Clifton and Knab (1989), CEB (1992), Basheer et al. (1996), Fisher et al. (1998), Bertolini et al. (2004), Delatte (2009), Melchers and Li (2009), and Collepardi (2010).

A life-cycle probabilistic-oriented approach to the assessment and design of structural systems must be based on a reliable and effective modeling of structural deterioration mechanisms. Deterioration processes may involve different types of damage mechanisms with different consequences on the structural performance. Moreover, several damage mechanisms might be simultaneously active, but progressing at different rates, including the sequential occurrence and interaction of continuous and sudden damage processes. A discussion of these aspects can be found in Sánchez-Silva et al. (2011, 2016). In the following, general criteria for deterioration modeling are presented, with emphasis on the effects of corrosion and fatigue in steel structures and chloride-induced corrosion in concrete structures, which are among the most common and detrimental deterioration processes that may affect structural systems, such as buildings and bridges, over their service life. Deterioration models could be developed on empirical bases, as it is generally necessary for rate-controlled damage processes, or founded on mathematical descriptions of the underlying physical mechanisms, as it is often feasible for diffusion-controlled damage processes (Ellingwood 2005). In any case, the parameters which define the deterioration processes are always affected by uncertainties. Consequently, the life-cycle prediction model has to be formulated in probabilistic terms, and all parameters of the model have to be considered as random variables or processes.

Deterioration Patterns

A mathematical description of deterioration processes may be complex and not always feasible because of incomplete information and knowledge of the damage mechanisms. Despite these complexities and drawbacks, effective models can often be established for practical applications by assuming the structural damage as a progressive deterioration of materials and components. According to this approach, the amount of deterioration is generally specified at the member level by means of time-variant damage indices $\delta = \delta(t) \in [0; 1]$ associated with prescribed patterns of deterioration, with $\delta = 0$ and $\delta = 1$ for the undamaged and fully damaged states, respectively (Frangopol and Curley 1987; Biondini et al. 2004; Biondini and Frangopol 2014).

Several damage mechanisms, including uniform corrosion in steel structures, as well as crushing, cracking, abrasion, and erosion

in concrete structures, can be effectively represented at the member level by a progressive reduction of the cross section. The damage index δ is related to a deterioration parameter that represents damage penetration, and proper correlation laws are introduced to define the geometrical properties of the damaged cross section such as area and moment of inertia, among others. A similar approach can be adopted to represent the damage effects of reinforcement corrosion in concrete structures. In this manner, the system performance associated with a specified level of damage δ can be evaluated based on the structural properties of the damaged members (Biondini and Frangopol 2014).

By denoting p the damage penetration depth, the damage index δ can be defined as follows:

$$\delta = \frac{p}{s} \quad (1)$$

where s = characteristic geometrical dimension of the structural component, such as the web thickness of a steel I-beam or the diameter of a reinforcing steel bar. Specific patterns of deterioration are needed when localized damage occurs. As an example, in concrete structures, localized corrosion (pitting corrosion) of reinforcing steel bars may frequently occur. The amount of pitting corrosion is often characterized by means of a pitting factor R_p defined as the ratio between the maximum depth $p_{\max}$ measured at pit and the average penetration $\bar{p}$ as calculated indirectly from the weight loss of the steel bar

$$R_p = \frac{p_{\max}}{\bar{p}} \quad (2)$$

Typical values of the pitting factor R_p vary between 4 and 8 for natural corrosion and between 5 and 13 for accelerated corrosion tests (Gonzalez et al. 1995).

Deterioration Rate

The evolution over time of the deterioration process needs to be described by suitable models of time-variant deterioration rate. However, a mathematical description of the physical mechanism underlying the deterioration process is often not available. In such case, empirical models can be successfully adopted, for example in the following form (Ellingwood 2005):

$$\delta(t) = \kappa(t - t_i)^\eta, \quad t \geq t_i \quad (3)$$

where t_i = initiation time; and κ and η = parameters determined from regression of available data, that are usually limited (Bartlett and Sexsmith 1991; Granata et al. 1996; Liu and Weyers 1998b; Naus et al. 1999). These parameters are generally considered constants. However, depending on availability of data over time, they could be also formulated as time-dependent and estimated by time-variant regression procedures.

For empirical deterioration models, a probabilistic formulation is necessary to account for the relevant uncertainty associated with natural randomness, as well as with imperfections in modeling and prediction of reality (Ang and De Leon 2005). More specifically, in time-dependent reliability analysis, a significant part of epistemic uncertainty derives from the selection of the deterioration model owing to the complexity of the damage processes and lack of experimental data (Zhang and Mahadevan 2000). Model uncertainty has been widely discussed in literature (Ditlevsen 1982; Der Kiureghian 1989, 2008; Guedes Soares 1997; Devooght 1998; Oberkampf et al. 2002; Nilsen and Aven 2003; Reinert and Apostolakis 2006; Der Kiureghian and Ditlevsen 2009; Mahsuli and Haukaas 2013). However, no consensus seems to exist on its definition and measure (Nilsen and Aven 2003; Bjerga et al. 2014).

Uncertainty associated with model errors is often quantified by using Bayesian approaches (Beck and Yuen 2004; Park et al. 2010; Haukaas and Gardoni 2011; Zhu and Frangopol 2013a) and is incorporated in design codes and standards on a simplified basis (Ellingwood et al. 1980; Bulleit 2008). In empirical deterioration models, model uncertainty is generally covered by means of error terms included in the model formulation, for example

$$\delta(t) = k(t - t_i)^\eta \varepsilon_1(t) + \varepsilon_2(t), \quad t \geq t_i \quad (4)$$

where the initiation time t_i is a random variable; and the error terms $\varepsilon_1 = \varepsilon_1(t)$ and $\varepsilon_2 = \varepsilon_2(t)$ are modeled as random variables or processes (Melchers 2003; Ellingwood 2005).

Empirical deterioration models are amenable to an efficient implementation in life-cycle prediction probabilistic frameworks. Moreover, they often represent the only feasible approach to model rate-controlled damage processes. On the other hand, the parameters of these empirical models are sensitive to several factors that characterize the problem and, in most cases, such sensitivity does not allow for a generalization to situations that are not covered by the available database. Generalization is particularly important when the parameters of the model may significantly change over time, for example because of the sequential occurrence and interaction of continuous and sudden damage processes, or as a result of maintenance interventions and repair actions (Sánchez-Silva et al. 2016). For this reason, when possible, more complex and comprehensive mathematical models have to be developed to represent the actual deterioration mechanisms and their effects on the life-cycle structural performance.

A mathematical description of time-variant deterioration may be feasible for diffusion-controlled damage processes, in which the deterioration rate generally depends on the concentration of the diffusive agents. This is the typical case of concrete structures, where damage induced by the diffusive attack of aggressive agents, such as sulfates and chlorides, may involve deterioration of concrete and corrosion of reinforcement (CEB 1992; Bertolini et al. 2004; Bertolini 2008). In such processes, damage induced by mechanical loading interacts with the environmental factors and accelerates both diffusion and deterioration. Therefore, the dynamics of the process is generally complex, and the available information about environmental factors and material characteristics is usually not sufficient for a detailed modeling. However, despite such complexities and drawbacks, simple degradation models may be often successfully adopted for an overall evaluation of the life-cycle structural performance by relating the rate of damage to the concentration $C = C(t)$ of the diffusive agent

$$\frac{\partial \delta(t)}{\partial t} = r(C, t), \quad t \geq t_i \quad (5)$$

where $r = r(C, t)$ is a rate function; $t_i = \min\{t | C(t) \geq C_{cr}\}$ is the corrosion initiation time; and C_{cr} = critical threshold of concentration (Biondini et al. 2004, 2006a; Biondini and Frangopol 2008, 2009).

Based on available data for chloride attacks (Pastore and Pedferri 1994) and correlations between chloride content and corrosion current density in concrete (Liu and Weyers 1998a; Thoft-Christensen 1998; Bertolini et al. 2004), a linear relationship between rate of corrosion in the range 0–200 mm/year and chloride content in the range 0–3% could be reasonable for structures exposed to severe environmental conditions. However, the parameters related to corrosion propagation are affected by relevant uncertainties (Andrade et al. 1990; Andrade and Alonso 1994). In particular, the corrosion rate depends on several parameters related to characteristics of concrete and climatic conditions (El Hassan et al. 2010). The available experimental data on this dependency

is limited, and further research is needed for an accurate calibration of the corrosion rate model. A probabilistic approach is clearly necessary to cover the relevant aleatory and epistemic uncertainties involved in this process, including randomness and modeling associated with material strengths, geometrical parameters of the concrete members, location and diameters of the steel bars, concrete diffusivity, steel corrosion rate, environmental aggressiveness, and exposure scenarios.

Fatigue and Corrosion in Steel Structures

Steel structures are subjected to time-dependent deterioration and aging effects resulting from multiple factors such as corrosion induced by harsh environmental conditions and fatigue damage. The economic impact of these effects are particularly relevant for steel bridges owing to their widespread use in many countries worldwide. For example, as of 2012, steel bridges represented approximately 30% of the bridge inventory in the United States (FHWA 2015). Moreover, they represent approximately 20% of the newly built bridges (FHWA 2011).

Corrosion deterioration of steel girders occurs from salt water exposure and atmospheric corrosion of the metal. Corrosion reduces the original thickness of the web and flanges of steel I-girders as indicated in Estes and Frangopol (1999), among others. Because of heavier exposure to leaking salt water, corrosion can be assumed to occur throughout the web height at the supports, but only at the bottom quarter of the web height along the rest of the girder length including the midspan location (Akgül and Frangopol 2004). Because of reductions in web and flange thicknesses, the values of time-variant geometrical properties of a steel girder must be computed based on a corrosion penetration predicting model.

To this purpose, proper correlation laws may be introduced to define the variation of the cross-sectional properties, such as area A and moment of inertia I , as a function of the damage index δ

$$A(\delta) = [1 - \delta_A(\delta)]A_0 \quad (6)$$

$$I(\delta) = [1 - \delta_I(\delta)]I_0 \quad (7)$$

where A_0 and I_0 = area and moment of inertia of the undamaged cross section; and $\delta_A = \delta_A(\delta)$ and $\delta_I = \delta_I(\delta)$ are dimensionless damage functions which provide a measure of cross-sectional damage in the range $[0; 1]$. The damage functions depend on the type of cross section and deterioration mechanism (Biondini and Frangopol 2014).

Severity of steel corrosion, in general, depends on the material properties (composition of alloys in metal), local atmosphere (important environmental conditions affecting steel corrosion include temperature and relative humidity), and exposure conditions such as initial climate, sheltering, orientation, angle of exposure, time of wetness, atmospheric pollutants, deicing salt, and debris (NCHRP 1984). Models developed to predict time-variant corrosion penetration in steel are usually empirical formulas intending to capture the actual corrosion process. In most studies, the power function given by Eq. (3) is used for the corrosion model. The values of the parameters in this prediction model are discussed in details in McCuen and Albrecht (1995).

Fatigue, on the other hand, may reduce the structure integrity owing to the initiation and propagation of cracks, which may cause failure of the damaged member at stress levels well below those associated with static loading conditions (Fisher et al. 1998). Fatigue effects can be reduced by adopting better details, avoiding stress concentrations, and decreasing the number of welded attachments, among others. Currently, two approaches are widely used

for fatigue analysis and design, namely, the S-N (i.e., stress-life) approach and the crack growth approach. The former is adopted by most of the design guidelines, such as the AASHTO LRFD design specifications (AASHTO 2010). However, it cannot be used for studying the crack conditions at a cracked detail. In contrast, the crack growth approach, based on linear elastic fracture mechanics, can be used to investigate the crack propagation at a damaged detail. Both approaches can be used for the life-cycle assessment and management of steel bridges (Kwon and Frangopol 2011; Soliman et al. 2013).

Corrosion and fatigue are synergistic phenomena. Corrosion pits may grow into cracks under fatigue loading, and corrosion-fatigue damage may develop more rapidly than would be expected from the separate damage mechanisms. Corrosion-fatigue is a complex phenomenon that is highly dependent on the material characteristics and surrounding environment (Crooker and Leis 1983; Out et al. 1984; Zuraski and Johnson 1990; Hahin 1994; Albrecht and Lenwari 2009; Kwon and Frangopol 2012; El Aghoury and Galal 2014; Dong and Frangopol 2015). The corrosion-fatigue problem is generally solved using fracture mechanics (Crooker and Leis 1983; Bolotin and Shipkov 2001). In design practice, there is a lack of simple approaches to evaluate the fatigue life of corroded steel members. This primarily results from the complexity of the problem and the scarcity of experimental data on structural elements cyclically loaded under simultaneous environmental effects of corrosion (El Aghoury and Galal 2014). Weathering steel can be used to avoid repainting over time and reduce the life-cycle cost of the structure (Melchers 2008). However, protective coating should be adopted because weathering steel corrodes at the same rate as ordinary steel if the material remains wet for long time or is salt-contaminated (Albrecht and Lenwari 2009). Finally, it is recalled that simultaneous fatigue and creep damage may also occur for steel structures subjected to long-term cyclic loading at high temperature. This issue can be of particular relevance for special structures used in nuclear power generation, chemical and aerospace industries (Nikbin 2013).

Corrosion of Steel in Reinforced Concrete Structures

The life-cycle performance assessment of concrete structures in aggressive environment should be able to account for both the diffusion process of aggressive agents, such as chlorides, and the corresponding mechanical damage induced by diffusion, which usually involves deterioration of the concrete matrix and corrosion of the steel reinforcement (Kilaeski 1980; CEB 1992; Bertolini 2008).

Diffusion Processes

The corrosion rate in reinforcing steel can be related to the concentration of diffusive aggressive agents as indicated by Eq. (5). The diffusion process can be described by the Fick's laws which, in the case of a single component diffusion in isotropic, homogeneous and time-invariant media, can be reduced to the following second-order linear partial differential equation (Glicksman 2000):

$$D\nabla^2 C = \frac{\partial C}{\partial t} \quad (8)$$

where D = diffusivity coefficient of the medium; $C = C(\mathbf{z}, t)$ is the concentration of the chemical component at point $\mathbf{z} = (x, y, z)$ and time t ; $\nabla C = \mathbf{grad} C(\mathbf{z}, t)$; and $\nabla^2 = \nabla \cdot \nabla$. In general, these equations need to be solved by means of numerical methods. However,

for one-dimensional diffusion (1D) the Fick's laws can be solved analytically to obtain:

$$C(x, t) = C_0 \left[1 - \operatorname{erf} \left(\frac{x}{2\sqrt{Dt}} \right) \right] \quad (9)$$

where C_0 = surface concentration of the diffusive chemical component; and $\operatorname{erf}(\bullet)$ = Gauss error function.

The Fick's 1D model is generally used to investigate the chloride diffusion process in concrete structures. Improved formulations can also be developed to account for different factors that may be involved in the diffusion process, including the apparent value of chloride diffusion coefficient over the investigated time interval, the initial chloride content in the cement paste, and the depth of the convection zone, i.e., the concrete layer up to which the process of chloride penetration differs from Fick's model (fib 2006). The 1D model also allows to directly evaluate the corrosion initiation time t_i based on the limit state condition $C(x = c, t = t_i) = C_{cr}$ at the concrete cover dept $x = c$

$$t_i = \frac{c^2}{4D} \left[\operatorname{erf}^{-1} \left(\frac{C_0 - C_{cr}}{C_0} \right) \right]^2 \quad (10)$$

Fick's 1D model is therefore a convenient mathematical tool for practical applications. However, the actual diffusion process is generally characterized by 2D or 3D patterns of concentration gradients. Recent studies demonstrated that simplified 1D diffusion models can lead to a significant loss of accuracy depending on the exposure conditions, geometrical shape ratio of the cross section, and location of points where the concentration is evaluated (Titi and Biondini 2015). As an example, 1D models can lead to inaccurate results in case of rectangular cross sections exposed to chloride attacks on the shorter sides. For this reason, a numerical solution of the general Fick's diffusion laws may be necessary for life-cycle assessment of concrete structures exposed to corrosion (Biondini et al. 2004, 2006a).

Finally, the key parameters D , C_0 , and C_{cr} that determine the diffusion process and corrosion initiation depend on several uncertain factors, including concrete mix, concrete cure, temperature, and humidity, and are characterized by high coefficients of variations (Ellingwood 2005). The dependency of the corrosion initiation time on different parameters, such as concrete cover thickness or quality and type of cement, has been widely researched (Browne et al. 1983). However, further research is needed to improve available data and models for chloride diffusion and reduce in this way the level of uncertainty in life-cycle prediction of corroded concrete structures.

Corrosion Damage

For concrete structures, damage starts to develop locally in the reinforcing steel bars and propagates affecting both the corroded steel bars and the surrounding volume of concrete.

The most relevant effect of corrosion is the reduction of the cross section of the reinforcing steel bars. The area A_s of a corroded steel bar can be represented as a function of the corrosion penetration index δ as follows:

$$A_s(\delta) = [1 - \delta_s(\delta)] A_{s0} \quad (11)$$

where $A_{s0} = \pi D_0^2/4$ is the area of the undamaged steel bar; and $\delta_s = \delta_s(\delta)$ is a dimensionless damage function that provides a measure of cross section reduction in the range [0; 1]. The damage function depends on the type of corrosion mechanism (Biondini and Vergani 2015). In carbonated concrete without relevant

chloride content, corrosion tends to develop uniformly on the steel bars. In the presence of chlorides, corrosion tends instead to localize (pitting corrosion). Pits have irregular shape, and the cross section reduction attributable to pitting corrosion is described through simplified models, for example, by assuming at pit a circular corrosion front (Val and Melchers 1997; Stewart 2009). A model of pitting corrosion associated with mixed corrosion mechanisms, with a component of uniform corrosion, can also be found in accelerated corrosion tests (Zhang et al. 2010).

Depending on the amount of steel mass loss, nonuniform corrosion may also involve a remarkable reduction of steel ductility. Tensile tests on corroded bars show that for a quite limited mass loss (approximately 13%), steel behavior may become brittle (Almusallam 2001). The results of experimental tests reported in Apostolopoulos and Papadakis (2008) indicate that ductility reduction is a function of the cross section loss. Based on these results, the steel ultimate strain ε_{su} can be related to the damage function $\delta_s = \delta_s(\delta)$ (Biondini and Vergani 2015). A limited reduction of steel strength is also observed for corroded bars (Du et al. 2005).

The effects of corrosion are not limited to damage of reinforcing steel bars. In fact, particularly in the case of uniform corrosion with low penetration rate, the formation of oxidation products may lead to propagation of longitudinal cracks and concrete cover spalling (Cabrera 1996; Vidal et al. 2004; Al-Harthy et al. 2011; Guzmán et al. 2011). This local deterioration of concrete can effectively be modeled by means of a degradation law of the resistant area of concrete matrix A_c (Biondini et al. 2004)

$$A_c = [1 - \delta_c(\delta)] A_{c0} \quad (12)$$

where A_{c0} = area of undamaged concrete; and $\delta_c = \delta_c(\delta)$ = dimensionless damage function which provides a measure of concrete damage in the range [0; 1]. However, in this form, it may be not straightforward to establish a relationship between the damage function δ_c and the corrosion penetration index δ .

Alternatively, concrete degradation can be taken into account by modeling the reduction of concrete compression strength f_c attributable to cover cracking

$$f_c = [1 - \delta_c(\delta)] f_{c0} \quad (13)$$

where f_{c0} = strength of undamaged concrete. The damage function $\delta_c = \delta_c(\delta)$ can be defined by relating the mean crack opening to the steel mass loss (Vidal et al. 2004).

The crack opening increases with the expansion of corrosion products up to a critical width. The spalling of concrete cover is assumed to occur when this threshold is reached. The reduction of concrete strength is generally applied to the entire concrete cover (Coronelli and Gambarova 2004). However, the longitudinal crack pattern depends on the arrangement of reinforcing bars, and cracking propagation induced by corrosion should be limited to the zones adjacent to the reinforcing bars to effectively reproduce the mechanism of cover spalling. This mechanism is characterized by inclined fracture planes for wide bar spacing, and parallel fracture planes (delamination) for closely spaced bars, as shown in Biondini and Vergani (2015).

Life-Cycle Structural Performance under Uncertainty

In structural design codes, the level of structural performance is generally specified with reference to structural safety. However, when aging and deterioration are considered, the evaluation of the system performance should account for additional performance indicators aimed to provide a comprehensive description of the life-cycle structural resources (Saydam and Frangopol 2011;

Zhu and Frangopol 2012, 2013b; Biondini and Frangopol 2014; Frangopol and Saydam 2014). In addition, the progressive and/or sudden deterioration of structural performance develops under significant uncertainties related to material and geometrical properties, damage mechanisms, mechanical and environmental stressors, inspection and monitoring outcomes, maintenance and repair interventions, life-cycle cost models, and decision-making processes. A proper modeling of these uncertainties and of their propagation over time is therefore essential to quantify the life-cycle structural performance (Ang 2011; Frangopol 2011).

Structural Performance Indicators

A failure of a system is generally associated with the violation of limit states. At the system level, limit states of interest are the occurrence of the first local failure, which represents a warning for initiation of damage propagation, and the global collapse. For structural systems, the identification of the local failure modes and of their occurrence in time can represent crucial information to maintain a suitable level of performance and to avoid collapse over the structural lifetime. In fact, repairable local failures can be considered as a warning of possible occurrence of more severe and nonrepairable failures (Frangopol and Nakib 1991).

There are several time-variant performance indicators that can be related to the possible occurrence of local and global failures, including system ductility, redundancy, failure times, robustness, and resilience.

System ductility is the ratio of ultimate displacement at collapse to the corresponding displacement at first failure. This performance indicator is related to the energy dissipation capacity of the structural system and is generally a key factor in seismic design procedures based on capacity design criteria. These criteria are assumed to be time-invariant in seismic design codes. However, the system ductility and the hierarchy of member strengths, and hence the energy-dissipating failure mode claimed for a capacity design of the structure, may change over time depending on the environmental exposure of the structure (Biondini and Frangopol 2008). This confirms the importance of a proper combination of seismic and environmental hazards in the evaluation of the life-cycle seismic performance of structural systems (Akiyama et al. 2011, 2012; Biondini et al. 2011, 2014; Celarec et al. 2011; Yalciner et al. 2012; Titi and Biondini 2014).

Structural redundancy denotes the ability of the system to redistribute among its members the loading that can no longer be sustained by damaged members after the occurrence of a local failure (Frangopol and Curley 1987; Frangopol et al. 1992; Decò et al. 2011). The load redistribution capacity is a desirable structural feature to ensure suitable system performance under accidental actions and extreme events, such as earthquakes (Bertero and Bertero 1999). The concept of structural redundancy can be extended over time to account for the time-evolution of the redistribution mechanisms owing to deterioration processes (Biondini and Frangopol 2008, 2014; Okasha and Frangopol 2010b).

Structural redundancy refers to a prescribed point in time and does not provide a measure of the failure rate, which depends on the damage scenario and damage propagation mechanism. Failure times should be computed to this purpose, and the time interval between subsequent failures, such as the first local failure and structural collapse, could represent an effective indicator of the damage tolerance of the system and its ability to be repaired after local failures (Biondini 2012; Biondini and Frangopol 2014).

Structural robustness can be viewed as the ability of the system to suffer an amount of damage not disproportionate with respect to

the causes of the damage itself. According to this definition, a measure of robustness should arise by comparing the system performance in the original state, in which the structure is fully intact, and in a perturbed state, in which a prescribed damage scenario is applied (Frangopol and Curley 1987). Robustness is generally evaluated with respect to damage suddenly provoked by accidental actions and abnormal loads (Ellingwood 2006; Ghosn et al. 2010; Saydam and Frangopol 2011). However, depending on the damage propagation mechanism, aging and progressive deterioration may also involve disproportionate effects (Biondini and Frangopol 2014).

Finally, resilience can be related to the capability of structures, infrastructure systems, and entire communities, to withstand the effects of extreme events and to recover efficiently the original performance and functionality (Bruneau et al. 2003; Lounis and McAllister 2016). Resilience is often investigated with reference to damage and disruption caused by seismic events (Bruneau et al. 2003; Chang and Shinozuka 2004; Bruneau and Reinhorn 2007; Padgett and DesRoches 2007; Cimellaro et al. 2010; Bocchini and Frangopol 2012; Frangopol and Bocchini 2012; Decò et al. 2013). In this context, a life-cycle approach is necessary because the effects of aging and environmental aggressiveness can modify the seismic performance and functionality and, consequently, make the system resilience depending on the time of occurrence of the seismic event (Biondini et al. 2015).

This brief overview of structural performance indicators highlights the role of damage on the time evolution of the system resources and emphasizes the importance of considering a suitable set of indicators for a comprehensive evaluation of the life-cycle system performance. The formulation and quantitative definition of these indicators are out of the scope of this paper. A review of performance indicators and metrics is available in Ghosn et al. (2016a).

Reliability-Based Structural Performance Criteria

Structural models and their idealizations, deterioration mechanisms, material resistances, geometries, and loads are uncertain. Therefore, a probabilistic approach is necessary to quantify the reliability of structural systems (Ang and Tang 2007). A review of uncertainty modeling and reliability-based performance criteria used to calibrate design and evaluation codes and standards for assessing the strength, serviceability, and fatigue resistance of structural components is provided in Ghosn et al. (2016b). The review shows that the target reliability levels adopted for evaluating the strength of various types of structural members and materials depend on many factors, including intended structure design/service life, expected member modes of failure (e.g., ductile or brittle), importance of the individual member to overall system integrity, experiences with previous designs, material and construction costs, structure type, and occupancy risk tolerance of the engineering community and the public within a code's jurisdiction.

Current specifications remain primarily focused on the design and evaluation of individual structural members and components, and member-level performance and reliability assessment procedures are currently well-established. However, it is widely recognized that a member-oriented approach does not necessarily lead to an efficient utilization of limited resources when making decisions related to the management of existing deteriorating structures or lifeline systems, especially those that may be exposed to extreme events. A review of proposals for the development and implementation of system-level assessment methods and performance-based criteria for structural systems and infrastructure networks is presented in Ghosn et al. (2016a). The review addresses system

reliability methods and probabilistic system-level performance metrics and characteristics, such as reliability, redundancy, robustness, resilience, and risk of structural systems under uncertainty. A framework for risk-informed decision making for the life-cycle performance of infrastructure facilities that includes consideration of sustainability and resilience is also presented in Lounis and McAllister (2016).

The review papers by Ghosn et al. (2016a, b) and Lounis and McAllister (2016) clearly indicate that the definition and calibration of reliability-based durability criteria in design standards are still open issues. This is primarily because of the difficulties encountered in modeling material degradation mechanisms and their interactions. In addition, there is a need to incorporate durability evaluations at the system level and to pursue practical and calibrated system-level performance indicators that support life-cycle performance, safety, reliability and risk of structures, and infrastructure systems as integral parts of resilient communities.

Definition and quantification of uncertainty for the prediction of the system performance over time is discussed by Sanchez-Silva et al. (2016) in the context of maintenance and operation of structures and civil infrastructure. It is shown that a key factor associated with maintenance programs is the reduction of uncertainty by finding cost-effective reliable monitoring systems for data acquisition, as well as better decision-making strategies that can capture the dynamics of the system performance and the role and interaction of all individuals involved in the process (owner, users, operators, among others). However, a proper modeling of the remaining uncertainties associated with the performance of structures and their propagation in time is of essence to reliably predict the life-cycle performance of structural systems.

Uncertainty over the Life-Cycle

The life-cycle performance profile of a structural system can be considered as shown in Fig. 1, in which uncertainties are associated with initial performance indicator, damage initiation, deterioration rate, performance improvement after maintenance/repair interventions, and service life without or with maintenance/repair (Frangopol et al. 2001; Frangopol 2011). The uncertainty associated with the prediction of service life increases with time.

Uncertainty in the modeling of structures and randomness in loading phenomena require the use of probabilistic methods (Benjamin and Cornell 1970; Ang and Cornell 1974). In the context of life-cycle assessment, uncertainty analysis is used to better explain and support decision-making processes (Ditlevsen 1982, 2003; ISO 1998a; Faber 2005; Lloyd and Ries 2007). Efforts are

made to distinguish between variability owing to inherent differences within a population and uncertainty resulting from lack of knowledge (Morgan and Henrion 1990; Hoffman and Hammonds 1994). Explicitly distinguishing the two types of uncertainty, namely, the aleatory and epistemic, is crucial for the proper handling of a probabilistic analysis approach (Oberkampf et al. 2004; Ang and Tang 2007; Der Kiureghian and Ditlevsen 2009). In fact, as already mentioned, whereas aleatory uncertainty cannot be reduced, improvement in knowledge or in the accuracy of predictive models will reduce the epistemic uncertainty (Ang and De Leon 2005; Goulet et al. 2015). The nature of uncertainty may change over time, for example, during the design phase, the uncertainties in structural properties are inherently random and, therefore, aleatory in nature. However, once the structure is constructed, such uncertainties become epistemic in nature (Faber 2000, 2005; Goulet et al. 2015). Moreover, in uncertainty modeling and propagation, it may be difficult to categorize a particular uncertainty as aleatory or epistemic, and this distinction is frequently determined by modeling choices (Der Kiureghian and Ditlevsen 2009).

In the past decades, design methodologies have shifted from deterministic-based approaches, such as allowable stress design, to the semiprobabilistic approaches found in current codes such as the LRFD design specifications (AASHTO 2010), the Canadian Highway Bridge Design Code (CSA 2006), and the Eurocodes (CEN 2002). However, life-cycle concepts are not yet explicitly incorporated in structural design codes and standards. Several approaches to quantifying uncertainty and its propagation over time have been proposed in the broad area of life-cycle assessment, including interval analysis, scenario modeling, fuzzy data sets, analytical uncertainty propagation, probabilistic simulation, and Bayesian statistics, among others (Björklund 2002). However, probability-based concepts and methods are generally recognized to provide a rational and more scientific basis for treating and combine inherent variability associated with natural randomness and uncertainty arising from imperfections in modeling and prediction of reality (Ang and Tang 1984, 2007; Ang 2011). Along these lines, classical time-dependent reliability methods can be used to perform life-cycle performance prediction under uncertainty (Mori and Ellingwood 1993, 1994a; Enright and Frangopol 1998a, c). More efficient alternatives are also available, including the use of lifetime functions (Hoyland and Rausand 1994; Leemis 1995; Yang et al. 2004; Okasha and Frangopol 2009; Barone and Frangopol 2014).

Despite differences in their treatment of uncertainty, each method (deterministic, semiprobabilistic, and probabilistic) seeks an optimal balance between economical design and safe performance from the standpoint of life-cycle cost considerations (Ang and Lee 2001; Esteve et al. 2002). With respect to cost, it is important to estimate the whole life cost of the system, which must necessarily include the costs of construction, operation, maintenance, inspection, monitoring, repair, and demolition, as well as the indirect costs of nonperformance or failure (Frangopol et al. 1997a; Ang 2011). An important uncertainty that arises in this context is related to the discount rate, particularly when conventional life-cycle engineering decision methods are applied to long-term predictions, such as 75 or 100 years, involving intergenerational event horizons (Newell and Pizer 2003; Lee and Ellingwood 2015).

When possible, to reduce the uncertainty, it is extremely advisable to integrate life-cycle predictions with the results of inspection and monitoring activities (Mori and Ellingwood 1994a, b; Glaser et al. 2007; Catbas et al. 2008; Frangopol et al. 2008; Hosser et al. 2008; Kim and Frangopol 2012; Soliman et al. 2013; Budelmann et al. 2014; Malerba 2014; Watanabe et al. 2014; Brownjohn et al. 2015; Godart 2015). In fact, life-cycle models can be very sensitive to change of the parameters of the input random variables, and

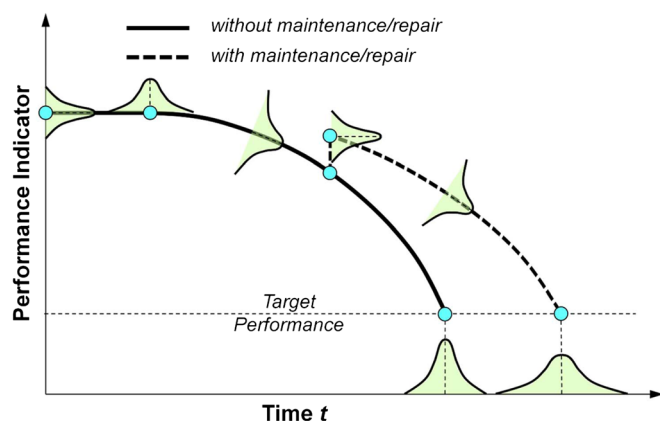


Fig. 1. Life-cycle performance profile under uncertainty

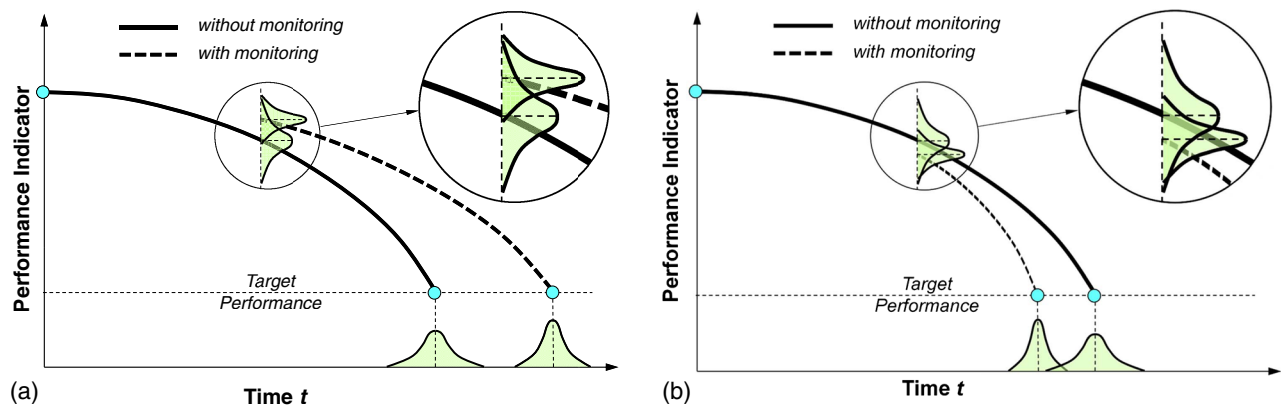


Fig. 2. Updating of the performance profile based on the results of monitoring: without monitoring, service life is (a) underestimated or (b) overestimated

monitoring may provide a powerful aid to reduce the level of epistemic uncertainty and to improve in this way the accuracy of predictive probabilistic models (Frangopol 2011). The effect of monitoring on the life-cycle performance prediction is qualitatively shown in Fig. 2 for both underestimation [Fig. 2(a)] and overestimation [Fig. 2(b)] of service life.

Notably, the selection of optimal inspection/repair strategies requires consideration not only of uncertainties related to degradation phenomena, but also quality of inspections, attitudes toward repair, and costs of both inspections and repairs, among others (Frangopol 2011). Inspection results are characterized by uncertainty related to damage detection. Probabilistic approaches have been proposed to model such uncertainty as a function of the damage level, taking into account probabilities of damage detection, false alarms (Frangopol et al. 1997b; Faber and Sorensen 2002), and correct or incorrect assessments after inspection (Orcesi and Frangopol 2011; Sheils et al. 2012).

Life-Cycle Reliability and Structural Lifetime

For structures exposed to damaging environments, the life-cycle performance must be considered as time-dependent. As mentioned previously, because of the uncertainty in material and geometrical properties, in the physical models of the damage process, and in the mechanical and environmental stressors, a measure of the time-variant structural performance is realistically possible only in probabilistic terms (Ang and Tang 2007).

Probability of Failure and Reliability Index

Let $R = R(t)$ and $S = S(t)$ be time-variant measures of the structural resistance and demand, respectively. Because of the uncertainty, both functions $R = R(t)$ and $S = S(t)$ have to be considered as random variables or processes. By denoting r_k and s_k the outcomes of the random variables $R_k = R(t_k)$ and $S_k = S(t_k)$, respectively, the probability of failure at given time instants $t = t_k$ can be evaluated by the integration of the joint density function $f_{R,S}^k(r, s)$ within the time-variant failure domain $\mathfrak{F}_k = \mathfrak{F}(t_k) = \{r, s | r_k < s_k\}$

$$P_F(t_k) = P[R_k < S_k] = \int_{\mathfrak{F}_k} f_{R,S}^k(r, s) dr ds \quad (14)$$

The analytical solution of this integral may be not feasible. Alternatively, the structural safety can also be measured by means of the time-variant reliability index $\beta_k = \beta(t_k)$

$$\beta(t_k) = \frac{\mu_{R,k} - \mu_{S,k}}{\sqrt{\sigma_{R,k}^2 + \sigma_{S,k}^2 - 2\rho_k \sigma_{R,k} \sigma_{S,k}}} \quad (15)$$

where μ and σ = mean and standard deviation, respectively; and ρ = correlation coefficient between R and S . If resistance and demand are normal variates, the following relationship holds:

$$P_F(t_k) = \Phi(-\beta_k) \quad (16)$$

where $\Phi(\bullet)$ = standard normal cumulative probability function. This relationship is often generalized and applied to estimate the reliability index for non-normal variates.

In practice, the statistical parameters and joint probability density distribution of R and S are generally not known, and at most, some information is available about a set of basic random variables $\mathbf{x} = [x_1 x_2 \dots]^T$, which defines the structural problem at the initial time $t = t_0$. In this case, by denoting $\mathbf{g}(\mathbf{x}, t) = \mathbf{0}$ a set of time-variant limit state functions $g_j(\mathbf{x}, t) = 0, j = 1, 2, \dots$, the probability of failure at time $t = t_k$ can be alternatively evaluated by the integration of the joint density function $f_{\mathbf{x}}^k(\mathbf{x})$ within the failure domain $\mathfrak{F}_k = \mathfrak{F}(t_k) = \{\mathbf{x} | g_{k,j}(\mathbf{x}) = g_j(\mathbf{x}, t_k) < 0, j = 1, 2, \dots\}$

$$P_F(t_k) = P[g_j(\mathbf{x}, t_k) < 0, j = 1, 2, \dots] = \int_{\mathfrak{F}_k} f_{\mathbf{x}}^k(\mathbf{x}) d\mathbf{x} = \Phi(-\beta_k) \quad (17)$$

Finally, in structural design, the levels of verification are usually formulated in terms of functions of random variables $\mathbf{y} = \mathbf{y}(\mathbf{x})$ which describe the structural response at each time instant, and such derivation is generally only available in an implicit form. For this reason, in most cases, a numerical approach, for example based on Monte Carlo simulation, is required to perform a lifetime reliability analysis for nonlinear analysis problems and realistic deterioration mechanisms (Ciampoli and Ellingwood 2002; Biondini et al. 2004). Suitable importance sampling techniques can be used to reduce the computational cost, and Latin hypercube sampling is suggested to improve their efficiency (Bucher 2009; Dolsek 2009; Mitropoulou et al. 2011).

Additional information and references on criteria and methods for uncertainty quantification, component and system reliability analysis, and probabilistic simulation in structural reliability, with emphasis on the assessment of reliability-based performance indicators for structural members, structural systems, and infrastructure networks, can be found in Ghosn et al. (2016a, b).

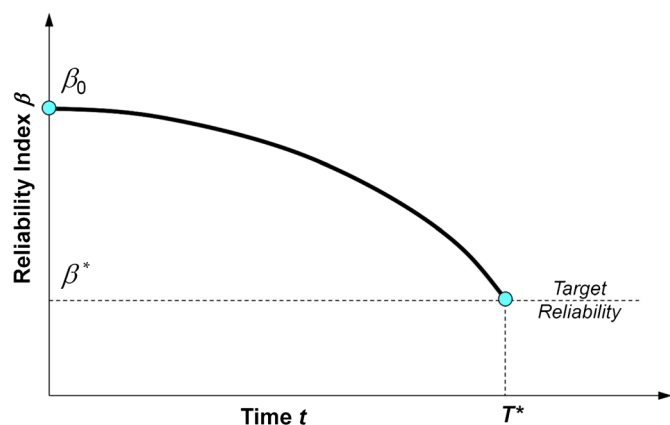


Fig. 3. Structural lifetime T^* associated to a target reliability index β^*

Structural Lifetime

The probabilistic assessment of the time-variant structural reliability allows evaluating the lifetime T of the structure (Biondini et al. 2006a)

$$T = \min\{(t - t_0) | R(t) < S(t)\} = \min_{j=1,2,\dots} \{(t - t_0) | g_j(\mathbf{x}, t) < 0\} \quad (18)$$

where t_0 = time instant at the end of the construction phase. In particular, the threshold T^* of the random variable T associated with a given target reliability level, for example expressed in terms of acceptable values of probability of failure $P_F^* = P_F^*(t)$ or reliability index $\beta^* = \beta^*(t)$, can be directly evaluated as follows (Fig. 3):

$$T^* = \min\{(t - t_0) | P_F(t) > P_F^*(t)\} = \min\{(t - t_0) | \beta(t) < \beta^*(t)\} \quad (19)$$

Based on this formulation, a proper design strategy can also be identified to achieve a structural lifetime $T^* \geq T_d$, where T_d is design requirement for the structure service life. Fig. 4 shows the effects of two design strategies aimed at improving a noncompliant design by increasing the initial reliability level and/or reducing the deterioration rate to achieve a structural lifetime $T^* \geq T_d$.

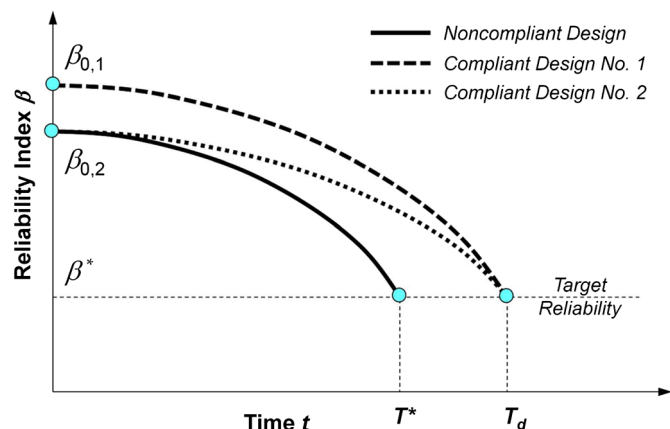


Fig. 4. Design strategies aimed at improving a noncompliant design by increasing the initial reliability level and/or reducing the deterioration rate to achieve a structural lifetime $T^* \geq T_d$

Lifetime Assessment and Maintenance Planning

The proper design and management of structures under time-dependent deterioration requires performing frequent inspections and maintenance actions. Thus, the application of life-cycle design and management concepts in selecting the materials and structural attributes can play a significant role in minimizing the total life-cycle cost associated with the initial design, construction, and lifetime inspection and maintenance costs.

Role of Maintenance and Repair

Repair or maintenance interventions, based on preventive and/or essential strategies, as shown in Fig. 5, can also be planned to achieve a structural lifetime $T^* \geq T_d$ (Frangopol et al. 2004).

The reliability index $\beta(t)$ of the repaired structure can be obtained by superposing the initial reliability index $\beta_0(t)$ and its modifications $\Delta\beta_k(t)$ associated with the subsequent interventions $k = 1, \dots, n$ applied at time instants $t = t_k$ (Kong and Frangopol 2003b)

$$\beta(t) = \beta_0(t) + \sum_{k=1}^n \Delta\beta_k(t) \quad (20)$$

Different maintenance scenarios with essential and/or preventive interventions with different reliability increments $\Delta\beta_k(t)$ can be considered to extend the structural lifetime T^* to achieve the desired value of service life T_d .

Minimum Expected Life-Cycle Cost

The maintenance program should be selected to minimize the total expected life-cycle cost (Chang and Shinozuka 1996; Ang and De Leon 1997; Frangopol et al. 1997a; Frangopol 1999; Estes and Frangopol 2001b; Wen and Kang 2001a, b; Kong and Frangopol 2003a; Furuta et al. 2011; Barone et al. 2014). To this purpose, the cost of maintenance C_M can be evaluated by summing the costs C_k of the individual interventions

$$C_M = \sum_{k=1}^n \frac{C_k}{(1+v)^{(t_k-t_0)}} = \sum_{k=1}^n C_{0k} \quad (21)$$

where the cost C_k of the k -th repair intervention has been referred to the initial time t_0 by taking a proper discount rate of money v into account (Kong and Frangopol 2003a).

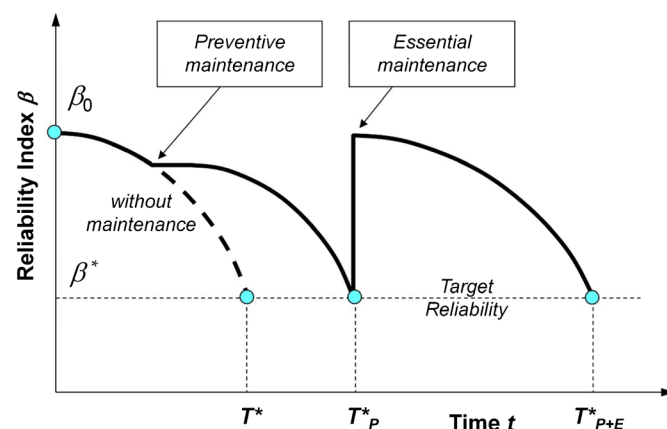


Fig. 5. Maintenance planning based on preventive and/or essential interventions aimed at extending the structural lifetime T^* without maintenance to achieve a lifetime $T^*_{P+E} \geq T_d$

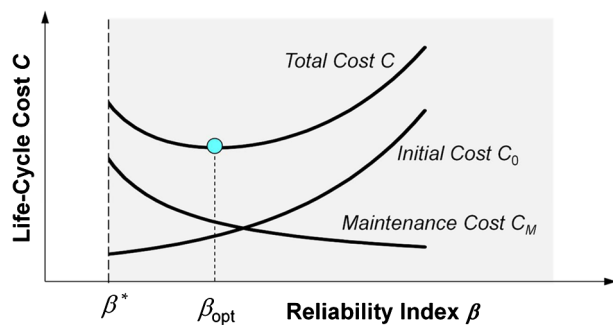


Fig. 6. Optimal design solution associated with minimum life-cycle cost under maintenance

In this way, based on a comparison among the costs of maintenance associated with different scenarios, the proper repair and maintenance strategies could finally be selected. However, as shown in Fig. 6 for a structural system under maintenance, a life-cycle approach to structural design and maintenance planning need to consider trade-off optimal solutions able to optimize the effects of several cost components, including the initial construction cost and the costs of operation, maintenance, inspection, monitoring, repair, and demolition, as well as the indirect costs of nonperformance or failure (Frangopol et al. 2002; Kong and Frangopol 2003b; Liu and Frangopol 2004, 2005, 2006; Ang 2011).

Optimal Maintenance Strategies

Optimal life-cycle maintenance strategies have been extensively and successfully applied to concrete bridges (Enright and Frangopol 1999a, b; Biondini et al. 2004; Frangopol and Liu 2007; Frangopol et al. 2012). However, life-cycle cost considerations can be especially beneficial for the case of steel bridges, in which different materials and structural configurations are available for the designer to choose from. Each material has its own strength and resistance to environmental conditions, whereas the structural configuration affects the fatigue resistance. Thus, each design alternative will yield a different total cost that can be calculated by means of the life-cycle analysis. An outcome of such analysis is shown qualitatively in Fig. 7, in which the life-cycle costs of two design alternatives are compared. As shown, although some cases may have a lower initial design and construction cost, on the long

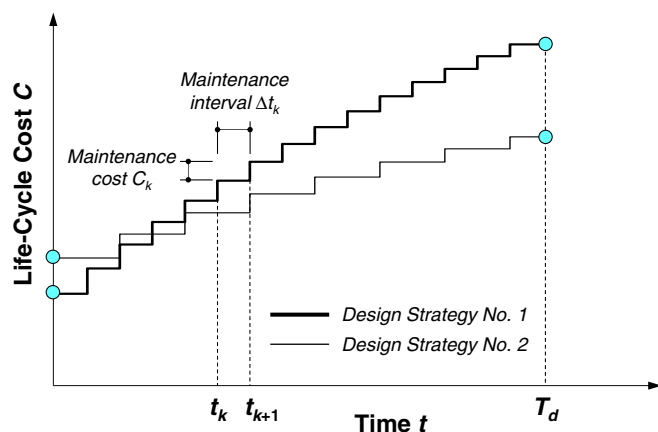


Fig. 7. Life-cycle cost evaluation of two design and maintenance strategies

term, they may require a significantly higher cost to maintain their functionality and reliability. Recent studies by Okasha et al. (2012) and Soliman and Frangopol (2015) clearly showed the benefits of the life-cycle analysis by comparing the life-cycle cost of using maintenance-free steel versus regular carbon steel for bridge construction. The study considered practical costs of repainting and construction, as well as repainting intervals, and concluded that the use of the maintenance-free steel, even for the lowest considered repainting cost and the longest interval between repainting, yields a lower life-cycle cost when compared to the regular construction steel after its first repainting action. Similar concepts can be applied to bridges under rehabilitation, in which different retrofit options can be evaluated on a life-cycle basis, and an optimal solution which fits the lifetime reliability and monetary constraints can be achieved (Liu et al. 2010a, b).

Fatigue and corrosion have been also successfully integrated into the life-cycle management of steel bridges under uncertainty. In these management schemes, the optimal intervention times and types, which fulfill the management goals, are established using optimization techniques. Up to date, these goals included maximizing the service life, minimizing the life-cycle cost, maximizing the lifetime performance, minimizing the lifetime expected failure rate, and minimizing the damage detection delay, among others (Okasha and Frangopol 2010a; Kim and Frangopol 2012; Kim et al. 2013; Barone et al. 2014).

Life-Cycle Reliability-Based Design

The concepts of life-cycle performance assessment and maintenance planning can be used to formulate the life-cycle reliability-based design problem in the context of structural optimization (Frangopol et al. 1997b; Frangopol and Maute 2003; Frangopol 2011; Frangopol and Soliman 2016). In the following, a general formulation of the optimization problem is presented to provide an overview of the existing research on this subject.

The purpose of a lifetime design optimization process is to find a vector of design variables $\mathbf{x}$ which optimizes the value of a set of objective functions $\mathbf{f}(\mathbf{x})$, according to both side constraints with bounds $\mathbf{x}^-$ and $\mathbf{x}^+$, and inequality time-variant behavioral constraints $\mathbf{g}(\mathbf{x}, t) \geq \mathbf{0}$ over a design lifetime T_d . The deterministic life-cycle multiobjective design optimization problem can be formulated as follows (Biondini and Frangopol 2009):

$$\min_{\mathbf{x} \in D} \mathbf{f}(\mathbf{x}) \quad (22a)$$

$$D = \{\mathbf{x} | \mathbf{x}^- \leq \mathbf{x} \leq \mathbf{x}^+, \mathbf{g}(\mathbf{x}, t) \geq \mathbf{0}, t_0 \leq t \leq T_d\} \quad (22b)$$

The objective functions $\mathbf{f}(\mathbf{x})$ representing the target requirements for the optimal design are generally related to the cost of the structure (Frangopol 1999), including the initial construction cost and the costs of maintenance, repair and demolition, as well as to additional structural performance indicators (Furuta et al. 2006, 2011; Frangopol and Liu 2007) such as safety, redundancy, and robustness, among others. The design variables $\mathbf{x}$ may include the geometrical and mechanical properties of the structural system (Frangopol et al. 1997a; Estes and Frangopol 2001b; Biondini and Marchiondelli 2008; Biondini and Frangopol 2009; Kwon and Frangopol 2010), as well as the time and reliability increments of maintenance and repair interventions (Frangopol et al. 2002; Kong and Frangopol 2003b; Okasha and Frangopol 2009; Neves and Frangopol 2006a, b).

Reliability-based design associated to a set of design variables $\mathbf{x}$ is concerned with the evaluation of the probability of failure

$$P_F(\mathbf{x}, t) = P[g_j(\mathbf{x}, t) < 0, j = 1, 2, \dots], \quad t_0 \leq t \leq T_d \quad (23)$$

or the corresponding reliability index

$$\beta(\mathbf{x}, t) = -\Phi^{-1}[P_F(\mathbf{x}, t)], \quad t_0 \leq t \leq T_d \quad (24)$$

Therefore, a general formulation of the life-cycle reliability-based design optimization problem can be cast in the following form (Biondini and Frangopol 2009):

$$\min_{\mathbf{x} \in D} f(\mathbf{x}) \quad (25a)$$

$$D = \{\mathbf{x} | \mathbf{x}^- \leq \mathbf{x} \leq \mathbf{x}^+, \beta(\mathbf{x}, t) \geq \beta^*(t), t_0 \leq t \leq T_d\} \quad (25b)$$

The target reliability $\beta^* = \beta^*(t)$ is in general time-variant because it reflects several factors which may change over time, including type and importance of the structure, possible failure consequences, warning of failure occurrence, and socioeconomic criteria (ISO 1998b, 2001). Further research on determination of optimal target reliabilities for design and upgrading of structures is however necessary (Ang and De Leon 1997).

The computational cost of optimization processes depends on the size of the problem and number of function evaluations requested by the solution method, including objective functions, design constraints, and for gradient-based methods, sensitivities with respect to the design variables. Life-cycle reliability-based optimization problems usually involve multiple deterministic function evaluations, and despite the availability of efficient analytical procedures for design sensitivity analysis of reliability criteria (Hohenbichler and Rackwitz 1986; Madsen and Tvedt 1990; Karamchandani and Cornell 1992), the computational cost of the solution process can be significantly high. For this reason, the implementation in design practice of life-cycle reliability-based optimization methods is still not advanced as would be desirable (Frangopol and Maute 2003). Sensitivity analysis procedures and preanalysis and postanalysis strategies are generally necessary to support the model development and make the size of the design problem practicable (Enevoldsen and Sørensen 1994). Uncertainty importance measures should be also used to quantify the relative importance of the involved variables (Haukaas and Der Kiureghian 2005) and to capture the time-variant role played by the uncertainty effects (Biondini et al. 2008). Nevertheless, these methods have been successfully used in reliability-based code calibration procedures for buildings and bridges (Ellingwood and Galambos 1982; Galambos et al. 1982; Nowak 1995; Ellingwood 1996) and, as reported in the quoted literature, applied to a wide range of structural systems.

Conclusions

This paper provided a review of advances in the fields of life-cycle reliability assessment, maintenance planning, and optimal design of structures and infrastructures. In this context, the paper summarized the principles, concepts, methods, and strategies for life-cycle assessment and design of deteriorating structural systems under uncertainty. It has been shown how the classical time-invariant structural design criteria and methodologies need to be revised to account for a proper modeling of the structural system over its entire life-cycle by taking the effects of deterioration processes, time-variant loadings, maintenance actions, and repair interventions into account. The discussed topics include the modeling of aging and structural deterioration, the simulation of diffusion processes, the modeling of corrosion effects in concrete structures, the formulation of time-variant structural performance indicators—such as system reliability, ductility, redundancy, elapsed time

between failures, robustness, and resilience—the evaluation of structural lifetime, the role of inspection and monitoring, the effects of maintenance and repair interventions, the definition of cost-effective maintenance strategies, and the formulation the life-cycle structural design as a multiobjective structural optimization problem. Several comprehensive realistic examples of probabilistic life-cycle methodologies applied to buildings, bridges, and ships are provided in Mori and Ellingwood (1994b), Thoft-Christensen (1998), Wen and Kang (2001b), Akgül and Frangopol (2004), Biondini et al. (2004, 2006a, b, 2008, 2011, 2014), Ang and De Leon (2005), Neves et al. (2006a, b), Biondini and Frangopol (2008, 2009), Catbas et al. (2008), Frangopol et al. (2008), Ghosn et al. (2010), Liu et al. (2010a, b), Akiyama et al. (2011), Decò et al. (2011, 2013), Moan (2005, 2011), Okasha et al. (2012), Zhu and Frangopol (2012, 2013b), Alipour et al. (2013), Soliman et al. (2013), Kim et al. (2013), Barone et al. (2014), Malerba (2014), and Biondini and Vergani (2015).

The main goal of this review paper was to provide a basis for further research advances in the field of life-cycle performance of structural systems under uncertainty and to promote the incorporation of life-cycle concepts in structural design codes and standards to support and advance the civil engineering profession. Clearly, a life-cycle-oriented design framework is intrinsically more demanding than the standard time-invariant design approach because it involves the modeling of complex deterioration processes and the evaluation of several performance indicators over the structural lifetime. In particular, a robust prediction of the time-variant structural performance must rely on a reliable and computationally efficient probabilistic deterioration modeling of materials and structural components.

Advanced models are well established for some of the most detrimental damage processes, such as corrosion and fatigue, and are rapidly becoming available for a wider range of deterioration mechanisms. However, deterioration models are generally very sensitive to change of the probabilistic parameters of the input random variables, and their robust validation and accurate calibration are difficult tasks to be performed because of the limited availability of data. Further efforts in this direction, aimed at gathering new data from both existing structures and experimental tests, are crucial for a successful implementation in practice of life-cycle methods. In this context, inspection and monitoring activities could provide a powerful aid to reduce the level of epistemic uncertainty and to improve the accuracy of predictive probabilistic models.

Significant efforts are also needed to advance the implementation in design practice of life-cycle reliability-based multiobjective optimization methods. These methods can be extremely useful to support the decision-making process involved in the design of new structures and maintenance and rehabilitation of existing structures, particularly under conflicting objectives. However, their computational cost is generally high and can rapidly become prohibitive when the number of design variables, objective functions, and design constraints, becomes excessive.

Finally, further developments to address societal issues in adopting life-cycle concepts in the decision-making process are necessary to raise awareness within the political system to allocate funds according to methods, metrics, needs, and priorities addressed by public officials, infrastructure users, and owners. Civil infrastructure systems are the backbone of modern society and among the major drivers of the economic growth and sustainable development of countries. It is hence a strategic priority to consolidate and enhance criteria, methods, and procedures to protect, maintain, and improve the safety, durability, efficiency, and resilience of critical structure and infrastructure systems under uncertainty.

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