Service-life extension of transport infrastructure through Structural Control

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ABSTRACT: Transportation Infrastructure Systems are recognized as essential for economic development, territorial cohesion, and social transformation. Due to the increasing age of bridges, and given that a large part of the existing stock was built several decades ago, some of their key structural components, such as bridges, are getting older while loading conditions are often exceeding those initially envisaged as they are subjected to harsher natural events and growing levels of traffic. The increasing age of bridges, the deterioration phenomena and the increase in service conditions, exceeding those used in the initial design, contribute to reduce their reliability level. This contribution firstly explores the role that structural control can play, then it proposes a suitable measure for the formalization of this role within the life-cycle assessment of bridges and overcrossing structures. The effects of structural control are evaluated for the case study of a cable-supported bridge subjected to fatigue deterioration due to wind actions.

1 INTRODUCTION

Management of the life-cycle of products and components has received a great deal of attention starting from the first works that appeared at the half of the last century (SAIC 2006), evolving into a probabilistic approach (see the recent review in Biondini & Frangopol 2016 for applications to structures and infrastructures). In the field of structural engineering, a proper life-cycle assessment of a structure involves taking into account several time-variant aspects, ranging from the aging of the structure itself, to the evolution of the loading and the effects of maintenance and repair interventions. From the economic point of view, besides the cost of construction and maintenance of the structure, other aspects, often of greater importance, are also of interest, such as the social costs associated with the presence of an infrastructure and those related to its loss of functionality.

Transportation infrastructure systems are recognized as essential for an area's economic development, territorial cohesion, and social transformation. This opens up the issue of their aging. In many cases, some of their key structural components, such as bridges, are getting closes to their intended life while climate changes and society evolution are driving the loading conditions to exceed those initially envisaged at the design stage. In such scenario they can be indeed subjected to increasing hazards, such as natural events or manmade phenomena, and larger levels of traffic loads. Due to the increasing age of bridges, and given that a large part of the existing stock was built several decades ago (ASCE 2013), the deterioration phenomena and the increase in service condition loading, larger than those used in the initial design, might contribute to reducing the reliability level if countermeasures are not promptly taken. Therefore, the assessment of the current state and the prediction of the future condition of bridges, as well as their protection against external hazards, has become an essential management challenge that needs to be addressed.

This work aims at contributing toward this practical goal by firstly exploring a suitable measure for the formalization of the role of structural control systems useful for the life-cycle assessment of bridges, and of overcrossing structures; subsequently, by evaluating the effects of structural control with reference to the case study of a cable-supported bridge subjected to fatigue deterioration due to wind action. The concepts explored are not restricted only to bridge structure, and can be applied, in principle, also to other structurally controlled structures.

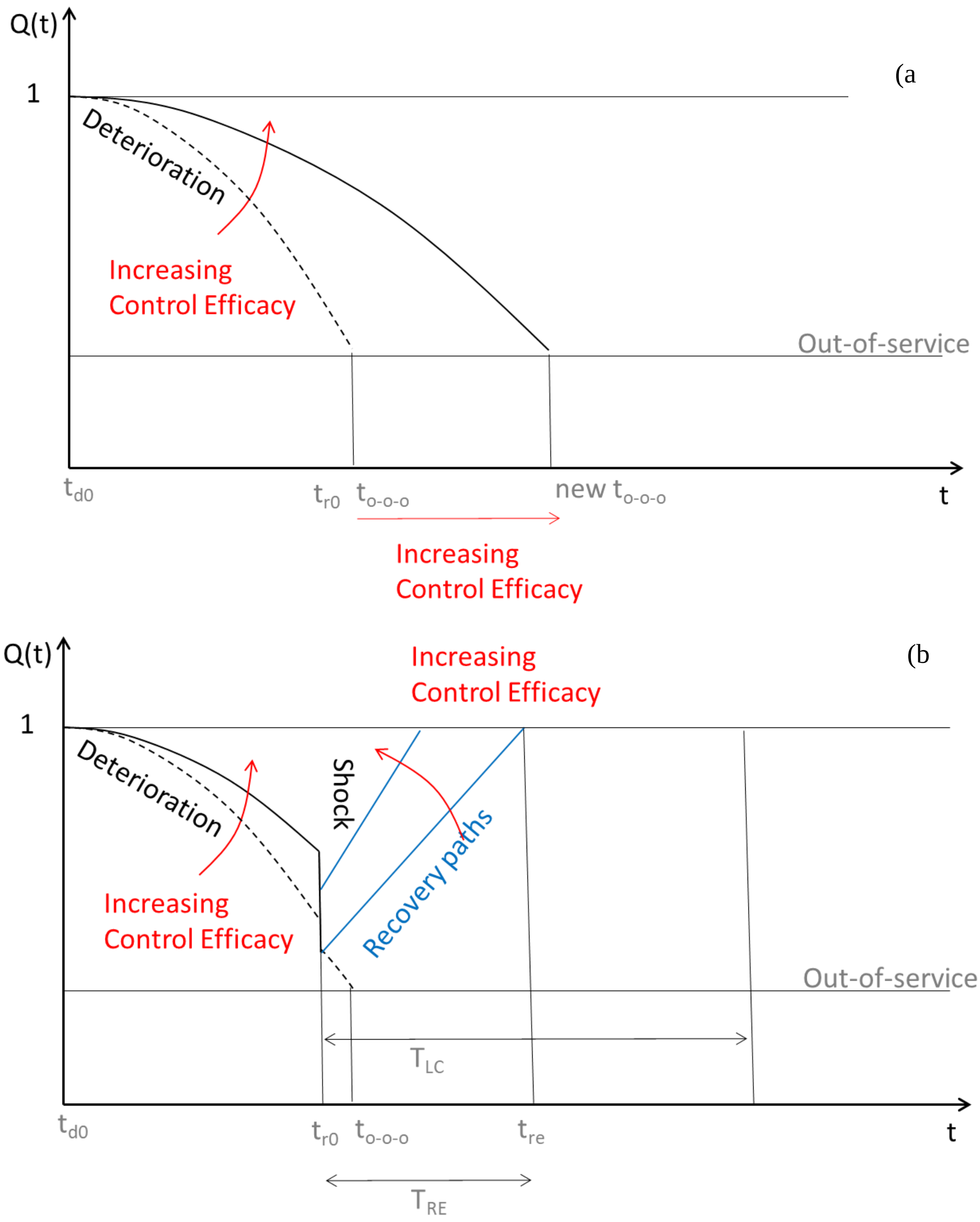

Figure 1. Graphical representation of the possible effects of Structural Control on the Functionality function: (a) in case no extreme events will happen; (b) in case of a single extreme event at time t_{r0}.

2 FOMALIZATION OF STRUCTURAL CONTROL EFFECTS ON THE LIFE CYCLE OF OF STRUCTURES

Structural Control refers to the application of techniques, such as active and passive control systems, to manage the structural behavior of a structure. The use of such techniques can have a significant impact on the life cycle analysis of a structure.

An important aspect of a Life-Cycle Assessment (LCA) is the ability to assess the system performance considering the aging and the deterioration of a system. This capability is a key constituent of LCA since the very purpose of an LCA exercise is usually to project into the future the life of a structure and assess the related costs.

Since Structural Control is able both to reduce the extreme values of the loading effects, as well as their variances (e.g. Casciati et al 2012), it will reduce damage, and hence it will extend the service life of the structure: time $t_{o\text{-}o\text{-}o}$ (out-of-service time) in Figure 1a. Time t_{d0} , which is the time at which damage will start, can be, for practical purposes, assumed as the time at which the structure has been completed. The other marked time instants in Figure 1 make reference at the case in which a damaging extreme event will happen. They are: t_{re} = the time at end of recovery phase, ($t_{o\text{-}o\text{-}o}$ - t_{r0}) = the residual service life after a shock at time t_{r0}, T_{RE} = (t_{re}- t_{r0}) = the time interval to recover full functionality, T_{LC} = a time interval limit set by the stakeholders to recover the functionality of the infrastructure.

In case of the occurrence of an extreme event at time t_{r0} Structural Control will have several positive effects since it will: a) reduce the structural deterioration up to time t_{r0}; b) reduce the final deteriorated state of the structure after the extreme event occurs, since the structure will be found in a better shape in the controlled configuration with respect to the uncontrolled one; c) speed up the recovery rate, and shorten the recovery time T_{RE} of the structure, since being the structure in a better shape before the damaging event it will be less damaged after the occurrence of the extreme event and easier to repair. All these aspects will impact the Life-Cycle Assessment of the structure.

An indicator aimed to provide a comprehensive description of the time evolution of available structural resources, which includes besides deterioration also the possible occurrence of local and global failures, is the Functionality function $Q(t)$, which is at the base of the measure R of Resilience (see, e.g. Cimellaro et al. 2010). According to well known the definition in Manyena (2006), Resilience is the ‘‘intrinsic capacity of a system, community or society predisposed to a shock or stress to adapt and survive by changing its non-essential attributes and rebuilding itself’’.

In essence, Resilience is related to the capability of a system to withstand the effects of extreme events and to recover efficiently the original performance and functionality (Bruneau et al. 2003). A well know measure R of Resilience is to defined R as the normalized area underneath the functionality function $Q(t)$ of a system (Fig. 1):

$$R = \left(\frac{1}{T_{LC}}\right) \cdot \int_{t_{r0}}^{t_{r0}+T_{LC}} Q(t)dt \tag{1}$$

In Equation 1 the functionality function $Q(t)$ is a dimensionless function of time t that describes the level of functionality possessed by the structure, to a value of $Q(t)$ = 1 it corresponds no loss-of-function for the system, while to a suitable small value of $Q(t)$, not necessarily zero if only a Serviceability Limit State has been exceeded, it corresponds the out-of-service of the system or of the structure; t_{r0} is the time at which recovery starts after a damaging event; T_{LC} is a time set by the stakeholders to recover the functionality of the infrastructure.

Aiming at applications within a Life Cycle Assessment procedure, in this work an extended version of the Resilience measure R is proposed which includes in Equation 1 the effects of Structural Control.

In the proposed definition the normalized area subtended by the functionality function $Q(t)$ is that from the time t_{d0} at which structural deterioration starts (see Figure 1b) to include all the life of the structure:

$$R = \left(\frac{1}{\left(t_{r0} - t_{d0} + T_{LC} \right)} \right) \cdot \int_{t_{d0}}^{t_{r0} + T_{LC}} Q(t) dt \tag{2}$$

Since it is very difficult to precisely define the part of curve $Q(t)$ when a damaging process is in action, the curves in Figure 1 are only indicative. Note, however, that previous experience (Martinelli & Domaneschi 2017) pointed out that the specific shape of deterioration part of the functionality function $Q(t)$ is not changed by Structural Control, but the curve is made less steep as structural control will reduce damages and structural degrade.

3 APPLICATION OF STRUCTURAL CONTROL TO WIND-INDUCED FATIGUE LIFE

Long-span suspension bridges must satisfy reliability and safety requirements. Amongst these, Eurocode 3 (C.E.N. 2004) lists fatigue during the intended life of the structure.

Indeed, wind-induced motion has been recognized (Chen & Cai 2007, Seo & Caracoglia 2013, Seo & Caracoglia 2015, Sacconi et al. 2021, Zheng & Li 2022) as an important problem for long-span bridges, and cyclic loading stemming from wind fluid-structure interaction is regarded as an important issue.

The effects of Structural Control on the fatigue life of a suspension bridge under wind loading have been studied in Martinelli & Domaneschi (2017) with reference to a numerical model of a suspension bridge inspired by the Shimotsui-Seto Bridge, in Japan. The bridge model (Fig. 2) has a total length of 1400 m, a span between the towers of 940 m, a towers height of 149 m, and a vertical distance of 31 m from the main girder to the foundations. The bridge has a steel truss main girder of rectangular transversal section of 30x13 m, width and thickness respectively. A complete description and validation of the model can be found in Domaneschi & Martinelli (2013a).

In the work by Martinelli & Domaneschi (2017), the standard deviation of the internal forces is assumed as a valuable parameter for evaluating fatigue effects in the main girder following general approaches of the existent literature, and typical structural details of the deck, having opposite characteristics in relation to fatigue life, are analyzed for fatigue failure using the stresses derived form the value of the wind induced bending moment in the main girder of the bridge. Prevention of fatigue damage is a complex problem that requires the identification and characterization of the structural element most prone to fatigue effects. Typical components suffering from fatigue are the connections between members of the structure, as the stresses are typically higher at these locations. The stress components in the connections depend, besides on the time history of the loading, also on the specific geometry, technology and typology of the connection itself.

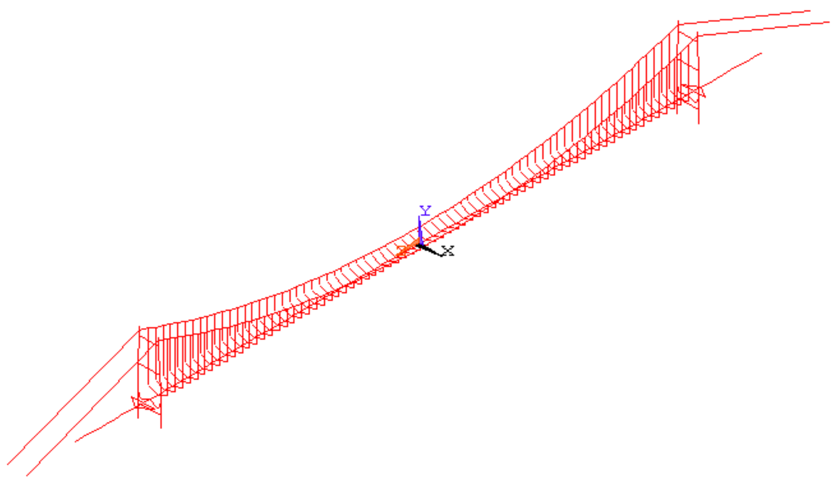

Figure 2. General view of the bridge model.

In the work by Martinelli & Domaneschi (2017) the representation of wind forces is based on the modeling of the drag force in time domain, as a completely non-linear force within the quasi-steady theory. For the lift force and the aerodynamic moment, instead, it is adopted a linearized form of the aerodynamic force and moment, with corrections for frequency-dependent loading using indicial functions. Details on the modelling of the wind loading, accounting for turbulence, can be found in Domaneschi & Martinelli (2013b).

All the wind interaction forces obviously depend on the wind velocity. The intensity of the wind loading is characterized through a representative statistical description of the average wind velocity as set forth in the IEC 61400-1 (2005) international standard, as this norm focuses specifically on applications that require a reliable statistics of the wind velocity.

The wind velocity mean value V over a time period of 10 min is assumed to follow the Rayleigh distribution, given as:

$$P_V\left(V_0\right) = 1 - \exp\left[-\pi\left(1 - \frac{V_0}{2V_a}\right)^2\right] \tag{3}$$

In Equation 3 $P_V(V_0)$ denotes the cumulative probability function for V (the probability that $V<V_0$) while V_a is the yearly average value of V.

This yearly average wind velocity value is derived from the value, V_{50}, relative to the extreme value, that has a recurrence period of 50 years, of the 10 min average wind speed. The relation between V_a and V_{50} adopted is the one proposed in IEC 61400-1: $V_a = 0.2\ V_{50}$. With V_{50}.= 39.09 m/s.

The value of V_{50} has been derived from the one, V_{150}, relative to the extreme 10 minutes mean velocity having a recurrence period of 150 years that was taken from the design specification for the bridge under study: V_{150} = 43 m/s.

The probability density function of the average 10 min wind velocity will be used in the following to describe the statistics of the wind loading for evaluating the occurrence of fatigue damage in selected structural details of the bridge deck.

In Martinelli & Domaneschi (2017) two hypothetical different details, having opposite characteristics in relation to fatigue life, were considered. A nominal stress component S was derived at a critical position in a deck connection from the lateral bending moment, M_y, in the girder on the base of the following linear law:

$$S = b_1 b_2 M_y \tag{4}$$

In Equation 4 the proportionality factor b_1 summarizes the effects of the deck connection details and of the girder section properties, while the factor b_2 scales the intensity of the input. Dependence on the other components of the main girder's internal forces is neglected.

A simple damage accumulation strategy, of the Miner's type (Committee on Fatigue and Fracture Reliability of the Committee on Structural Safety and Reliability of the Structural Division, American Society of Civil Engineers 1992) is then used to find the resulting effects of the constant amplitude cycles:

$$D = \sum_{i=1}^{d} D_{i,Si} = \sum_{i=1}^{d} \frac{n_{i,Si}}{N_{i,Si}} \tag{5}$$

In Equation 5 D is the cumulated damage over the life of the structure, $D_{i,Si}$ is the fraction of damage suffered by the material due to a number $n_{i,Si}$ of cycles at stress level S_i while $N_{i,Si}$ is the fatigue strength (i.e. the number of cycles to failure at stress level S_i given by a so-called S-N curves). The S-N curves herein adopted are those reported in Eurocode 3 (fatigue) (C.E.N. 2004), and failure is assumed to occur when $D = 1$.

To better frame the impact of fatigue effects, the structural details studied were at the extremes of the sensitivity to fatigue effects among the details for which Eurocode 3 (fatigue) (C.E.N. 2004) gives direct guidance. The more sensitive detail is characterized by a constant stress amplitude $\Delta\sigma_C$ = 36 MPa at N = 2 million cycles, with a yield strength of the steel f_y = 430 MPa. The less sensitive detail is characterized by $\Delta\sigma_C$ = 90 MPa at N = 2 million cycles, with a yield strength of the steel f_y = 430 MPa.

3.1 *Considered Structural Control settings*

Different control settings are considered to assess the effects of Structural Control. These correspond to the ones already considered also in Martinelli & Domaneschi (2010). The control devices considered belong to the quite general fluid dynamic dampers family, able to perform either in a passive configuration or in a semi-active one. These control devices are positioned so that they connect the deck to the towers (Fig. 3). Two orientations of the dampers are defined: transversal to the deck, denoted as TD, and longitudinal to the deck, indicated as LD. Transversal devices TD give a large contribution to the reduction of the deck transversal oscillations, while the LD devices help create a dissipative torque.

The three control settings considered are: passive-optimal (PO), passive-semi-active (SA), and uncontrolled (UNC). All controlled configurations comprise both TD and LD devices. The passive-semi-active configuration SA is obtained assuming that the transversal devices TD can act as semi-active ones. The longitudinal devices LD are always of the passive type.

Figure 4 and 5 depict the degrading part of the functionality function $Q(t)$ due to wind-induced fatigue for the most sensitive structural detail ($\Delta\sigma_C$ = 36 MPa at N = 2 million cycles) and for the least sensitive one ($\Delta\sigma_C$ = 90 MPa at N = 2 million cycles). The degrading part corresponds to the time interval $t_{d0}=0 < t < t_{r0}=150$ years, and no extreme events have been assumed to happen. The curves in Figures 4 and 5 have been obtained by recalculating the data presented in Martinelli & Domaneschi (2017).

The effects of structural control are massively beneficial (Fig. 4) for the most sensitive detail, showing a huge reduction in the predicted value of the damage. The curves in the figure are linear since the same distribution of average hourly wind velocity has been assumed for each year. Note that very little is gained in terms of fatigue life going from the passive optimal PO scheme to the semi-active SA one. With the hypotheses on the distribution of the wind velocity made, the uncontrolled structure life exceeds the 100 years mark.

The less sensitive detail, instead, has no problems (Fig. 5) in reaching the intended design life of the structure, even in the uncontrolled case.

Application of Equation 2 to the curves in Figure 4 leads, for $t_{r0}+T_{LC}$ = 100 years to a value of the extended Resilience R = 0.5, 0.9, 0.9 for the UNC, PO, SA cases, respectively, highlight as the proposed index can be useful in assessing the structure’s resources within a Life Cycle Analysis.

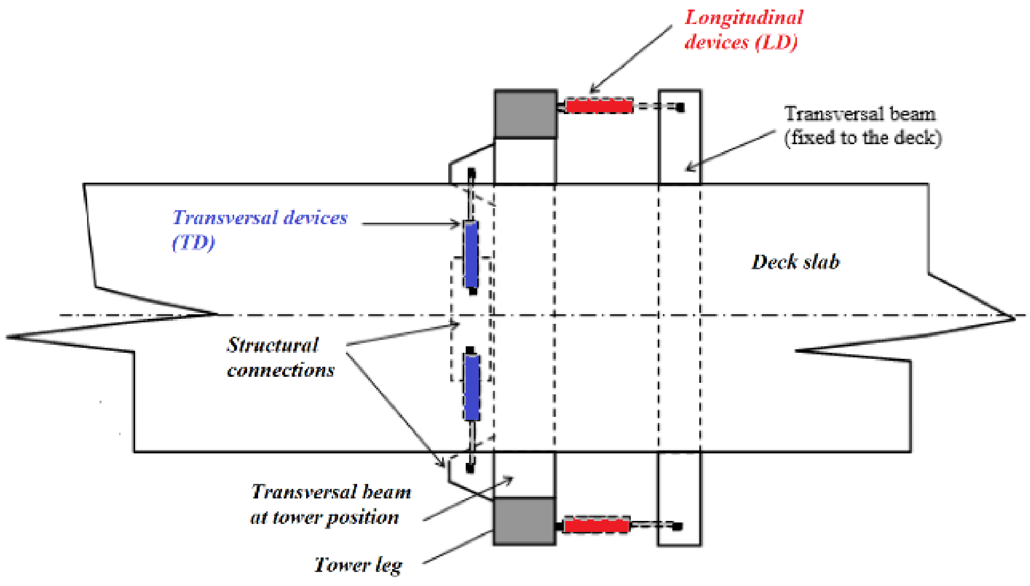

Figure 3. Top view of the deck at the towers, with the position of longitudinal LD and transversal TD devices.

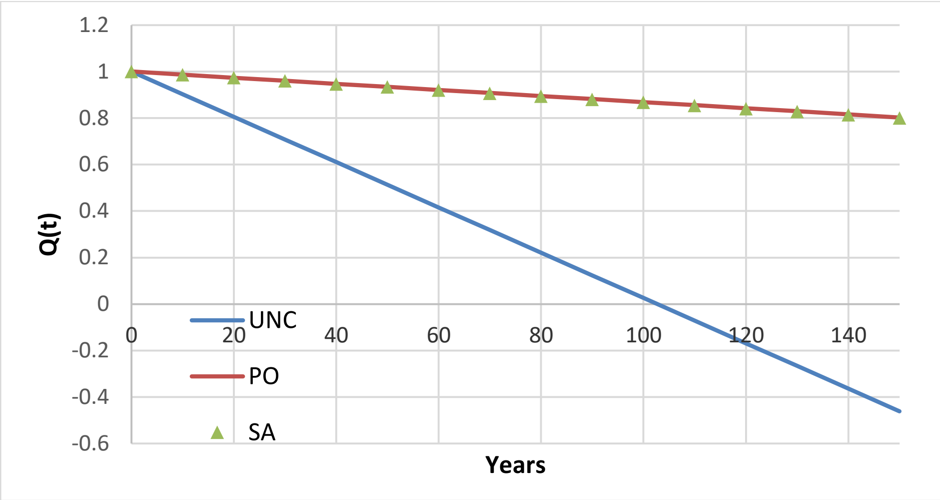

Figure 4. Deteriorating part of the functionality function $Q(t)$ due to wind-induced fatigue for the most sensitive structural detail.

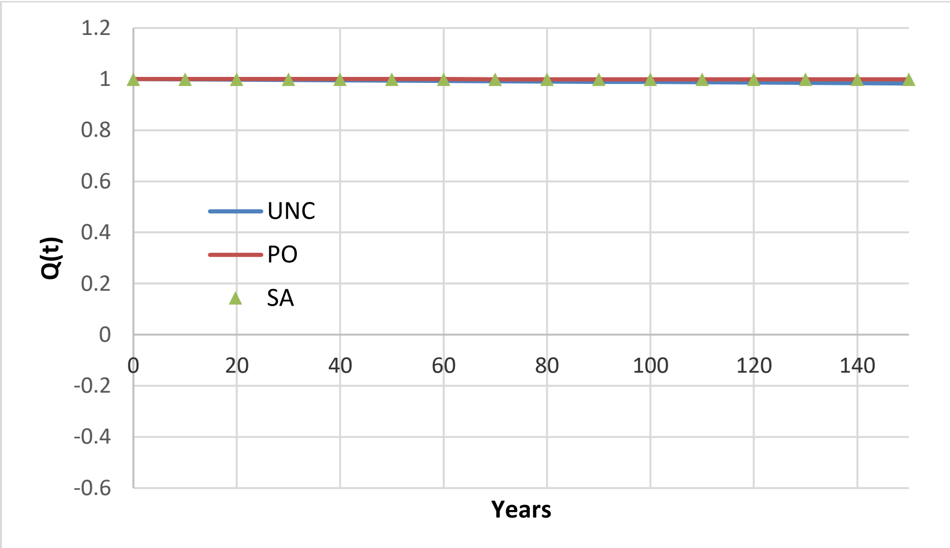

Figure 5. Deteriorating part of the functionality function $Q(t)$ due to wind-induced fatigue for the least sensitive structural detail.

4 CONCLUSIONS

Life cycle analysis involves assessing the environmental and economic impact of a structure over its entire life cycle, from construction to demolition. This analysis takes into account factors such as the use of materials, energy consumption, maintenance, and disposal. In this respect, structural control can be of help under several points of view.

Improved durability: Structural control techniques can help to minimize the impact of environmental factors such as wind, on the bridge structure. By reducing the damage caused by these factors, the bridge can last longer and require less frequent repairs and maintenance.

Increased safety: By monitoring and controlling the behavior of the bridge, structural control systems can help to detect and respond to any anomalies or potential failures before they become serious safety concerns. This can help to minimize the risk of accidents and ensure the safety of users.

Reduced environmental impact: Structural control systems can help to minimize the use of materials and energy during construction and operation of the bridge. By reducing the amount of resources required, the environmental impact of the bridge can be minimized.

Improved cost-effectiveness: By reducing the need for maintenance and repairs, structural control can help to reduce the lifetime cost of the bridge. This can make the bridge more cost-effective and help to ensure that it provides value for money over its entire life cycle.

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