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Research Article

Life Cycle Analysis of a Steel Railway Bridge over the Operational Period considering Different Maintenance Scenarios: Application to a Case Study

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In the context of bridge management, three main types of maintenance actions can be considered. Maintenance actions can be taken preventively before the predefined limit condition is reached, or as a corrective measure in case those limits have been reached. The third possibility corresponds to the so-called "doing nothing" scenario, in which no action is taken on the bridge. To be able to implement preventive maintenance, it is necessary to know the current condition of the bridge, as well as to be able to predict its performance. On the other hand, it is also important to be able to identify potentially threatening events that might occur in the analysis life period. This paper describes an integrated methodology to help bridge managers in defining an efficient maintenance program, considering the specific case of a railway bridge. The novelty of the methodology is focused on updating an existing methodology proposed by COST TU1406, by extending it to railway bridges and also by including the resilience analysis in case of a sudden event occurrence. The analysis considers a multi-hazard future scenario, in which a flood event occurs while corrosion phenomena were already in place. The results show the feasibility of the proposed methodology as a support for the establishment of an efficient maintenance schedule to prevent bridge severe degradation, as well as to establish recovery plans in case of a sudden event.

1. Introduction

Asset Management (AM) is a multidisciplinary task that involves an extensive series of processes, such as those related to life cycle analysis, maintenance, risk analysis, and optimization [1, 2]. As a formal approach to proposing guidelines on the field of AM, the International Standard Organization (ISO) released in 2014 the ISO 55000 series, which are composed of three documents: (i) ISO 55000–Asset Management: Overview, Principles and Terminology; (ii) ISO 55001–Asset Management Systems: Requirements; (iii) ISO 55002–Asset Management Systems: Guidelines for the Application of ISO 55001. There is a wide range of definitions of AM depending on the field of evaluation. According to ISO 55000 [3], AM can be defined as a *"coordinated activity of an organization to realize value from assets"*.

In the context of bridge management, the processes mentioned before have been included in the bridge management systems utilizing three main modules [4]: (i) inventory database module that contains all the information to identify the bridge and its condition state; (ii) prediction module, encompassing degradation and cost models, in which all the predicted scenarios for the bridge are stored concerning its time-dependent performance, as well as all the costs involved in the maintenance; (iii) optimization module that includes a set of algorithms to support pointing out the best maintenance strategies to be applied on the bridge to get cost-effective maintenance during the analyzed period.

Nowadays, different bridge management systems are implemented worldwide. A nonexhaustive list includes Pontis, now denominated AASHTOWare, from the USA

[5]; KUBA from Switzerland [6]; DANBRO from Denmark [7]; and J-BMS from Japan [8]; the report of International Association for Bridge Maintenance and Safety (IABMAS) [9] can be consulted for other systems. Furthermore, several examples of research and development projects can be identified in the last years, namely (project name, duration, and reference), Sustainable Bridges, 2003–2007 [10]; NCHRP (National Cooperative Highway Research Program) [11–13]; SustIMS (Sustainable Infrastructure Management System), 2012–2015 [14]; and COST TU1406 - quality specifications for roadway bridges, standardization at a European level, 2015–2019 [15]—among others that have contributed to fostering the bridge management topic.

This work consists of the application of the abovementioned concepts to a steel railway bridge. Steel has been adopted as an alternative material in the construction of bridges since the second half of the nineteenth century. Thus, many of these old structures are still in service. The study of the life cycle of steel bridges is a very promising topic with a lot of challenges yet to be overcome. These days, a trend proving that a high number of bridges are starting to show large signs of degradation, thus being in need of intervention, has been registered. These issues bring inspection and maintenance to the spotlight in which readjustments on budgets for inspection and conservation should be optimized by the administrations.

In this context, the first step is the evaluation of steel bridge condition. Several research works have been developed in the past, addressing steel bridges assessment as shown in Table 1.

Moreover, additional works in the broader field of bridge management can be referred to, wherein predictive models, as well as proposals of life cycle management strategies and sustainability analysis, have been addressed [27–32].

Considering the aforementioned works (Table 1), some challenges remain open in the field of bridge management. Most of the works still rely mainly on the bridge structural analysis, while other important aspects such as safety of users, availability assessment, and response to unexpected sudden events remain scarce in the literature.

As an attempt to overcome some of these shortcomings, the present work proposes a bridge assessment methodology that combines four different key performance indicators (KPI): (i) reliability, (ii) safety of users, (iii) availability of the bridge, and (iv) costs associated with lifetime maintenance. Moreover, three different maintenance actions that can be taken during the bridge management are thoroughly discussed: (i) no maintenance, (ii) preventive maintenance, and (iii) corrective maintenance. The latter situation is also the subject of an additional study. This consists of simulating the influence of a multi-hazard context on the bridge in terms of its structural performance. This analysis includes the establishment of a recovery plan by estimating the bridge resilience parameter. It is noteworthy that this integrated methodology is proposed as an extension to the existing methodology proposed by COST TU1406 [15] by extending it to take account of railway bridges and also by considering the inclusion of sudden events as well as the establishment of recovery plans. Accordingly, the paper is divided into 4 sections. Following the Introduction, Section 2 is dedicated to the proposed methodology. Section 3 describes the application of the methodology to a case study of a steel railway bridge. Finally, in Section 4, the most important conclusions of the work are provided.

2. Methodology

This section describes a methodology for the assessment of existing bridges over their life cycle by combining the following different KPI: (i) reliability, (ii) safety, (iii) availability, and (iv) cost. The methodology is divided into two main stages: (i) current performance assessment and (ii) future performance prediction. Figure 1 depicts the flowchart of the proposed methodology.

2.1. Bridge Current Performance Assessment. The first steps of the assessment process refer to the acquisition and compilation of bridge characteristics and details. Whenever available, the information from previous inspection reports should also be compiled together with the remaining inventory information. Only with this data in hand, it is adequate to start the in situ bridge assessment. The information regarding the previous inspection reports should be thorough enough, including data about the condition state of both the bridge and its several components, as well as the cost information regarding any previous important interventions made.

Depending on the bridge structural type and loading conditions, among others, it should be possible to identify the potentially vulnerable zones. Those should be associated with the most relevant failure modes for each specific bridge.

2.1.1. Condition Assessment. After gathering all the details concerning the structural behavior of the bridge, as well as the previous reports, the next important step on the bridge assessment refers to the selection of the performance indicators (PI) more suitable to define bridge performance. Those PI can be then grouped into key performance indicators (KPI), which are classified on a 1 to 5 scale to ease their combination. Four different KPI are suggested: (i) reliability, (ii) safety, (iii) availability, and (iv) cost. Reliability KPI is estimated based on the homonymous PI reliability index, widely studied in the field of structural engineering. This PI is used to measure the structural performance given the existing uncertainties. It traduces the bridge failure probability, which is given by the violation of a given limit state. Nowadays, there are several codes wherein the assessment of existing bridges reliability is being addressed. Reliability KPI directly refers to the structural performance of the bridge, so it is useful for assessing the impact of the degradation mechanisms on bridges. Since the reliability index is computed using a continuous scale, Table 2 presents the corresponding reliability KPI scale using reliability index intervals.

As for safety, this KPI measures the ability of a bridge to minimize damage to its users. Damage herein means the possible injuries that might occur when using the bridge,

Table 1: Research works in the field of assessment of steel bridges.

Reference	Main contributions
Kim et al. [16]	Reliability index of the overall steel railroad bridge by evaluating fatigue over its lifetime adopting simplified, probabilistic, and deterministic procedures.
Lee et al. [17]	Life cycle cost approach and procedure for effective life cycle cost optimum design of steel bridges.
Akgül and Frangopol [18]	Lifetime analysis of superstructure components of a steel bridge, with initial reliability and lifetime reliability profiles being addressed.
Czarnecki and Nowak [19]	Time-variant reliability analysis of steel girder bridges.
Lee et al. [20]	Life cycle cost-effective optimum design of steel bridges considering the effects of corrosion and traffic.
Gervasio and Silva [21]	Complete life cycle analysis of a steel-concrete composite bridge.
Pipinato and Modena [22]	Time-dependent fatigue reliability assessment of a steel bridge.
Kwon and Frangopol [23]	Evaluation of the fatigue reliability at a given period considering crack growth and the probability of detection models.
Peng et al. [24]	Life cycle analysis of steel railway bridges based on the growth of cracks.
Kere and Huang [25]	Time-dependent reliability analysis considering four different maintenance strategies related to the corrosion of steel.
Lee et al. [26]	Improving the system reliability to handle the varying-amplitude load; proposing an analysis that enables updating the system-level risk of fatigue failure for railway bridges after inspection and repair.

with this being the associated PI. This KPI can be related to reliability KPI, for example, in the event of someone getting hit by a chunk of concrete spalling from underdeck cover. That, in turn, should be a sign of reliability loss. Moreover, safety is also related to the nonstructural element condition state (e.g., pavement, guards, and barriers). Table 3 summarizes how safety KPI can be computed from the corresponding safety PI.

The availability KPI quantifies the period in which the bridge is functioning adequately. Therefore, maintenance activities that restrict accessibility and disrupt traffic flows influence availability. Moreover, major disruptive events, such as sudden events, should be accounted for in the measurement of the availability.

While the previous two KPI were defined according to the Quality Control Plan of [33], availability KPI was defined differently. The availability KPI scale proposed by [33] was developed for roadway bridges. To make it general and applicable to other types of bridges, e.g., railway or railroad bridges, in the present methodology, it is suggested that speed restriction coefficient (SRC) is used as PI to quantify availability KPI. This PI represents the speed reduction, compared to normal speed, in case of an intervention. The more critical the intervention is, the highest the SRC is. Table 4 presents the proposed quantification scale for the availability KPI.

The cost KPI is also computed based on a homonymous cost PI, which addresses the long-term management cost. It can aid bridge managers in establishing proper budget strategies to minimize costs while maintaining an adequate performance level. Within a life cycle analysis concept, those costs are divided into the following: (i) direct costs, i.e., costs of construction, maintenance, and eventual demolition at the end of the bridge lifetime; (ii) indirect costs, i.e., costs related to inadequate performance of the bridge, such as extra time spent by users to use detour routes, due to maintenance actions. The maintenance component of cost (C_{maint}) is composed of different parts, namely, inspection costs (C_{insp}), maintenance actions costs (C_{action}), and rebuilding actions costs (C_{reb}). Hence, the maintenance cost can be computed by (1). To allow grouping cost PI into a set of five cost KPI values, the cost PI quantitative scale is normalized using (2). Note that these five groups were defined based on expert judgement.

$$C_{\mathrm{maint}} = C_{\mathrm{insp}} + C_{\mathrm{action}} + C_{\mathrm{reb}}. \quad (1)$$

$$\mathrm{COST}_{\mathrm{normalized}} = \frac{\mathrm{COST}_i}{\mathrm{COST}_r} \times 100\%, \quad (2)$$

where COST_i refers to the total cost in year i and COST_r corresponds to the cost of rebuilding the bridge. Table 5 depicts the adopted cost KPI scale.

The bridge's final condition is obtained by analyzing the four KPI estimated before. To this purpose, the value of each KPI can be plotted in a spider diagram as further discussed.

2.2. Bridge Future Performance Prediction. The lifetime analysis of a bridge is the subsequent step after its analysis in the present year. Such a task is of paramount importance since it provides valuable information about its behavior in medium-long term for the process of decision making.

Bridges are exposed to several aggressive environments and threats during their life cycle. Understanding how these aspects influence the bridge performance and establishing suitable degradation models constitute the first step. The literature offers several degradation models that explain the time-dependent bridge performance. Generally, they can be divided into deterministic, wherein the uncertainty effects are disregarded, and probabilistic models, in which uncertainties are considered. The most common bridge management systems rely on the latter, which in turn are usually supported by Markov-based stochastic deterioration models.

Besides the environmental conditions that cause progressive degradation over the bridge lifetime, sudden events

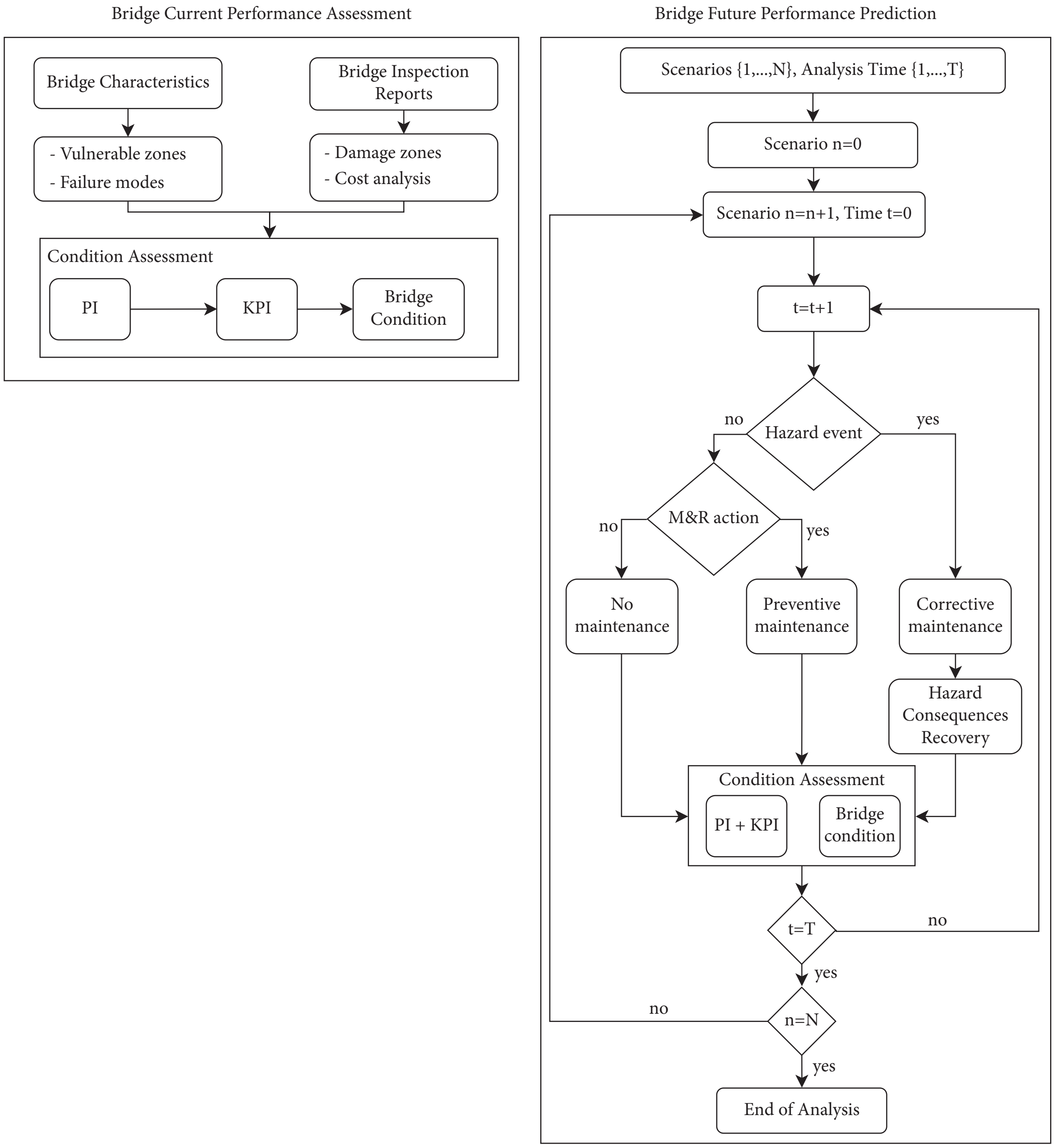

Figure 1: Flowchart of the presented methodology for the assessment of existing bridges.

represent another type of threats to the bridge that can result in bridge unavailability for major intervention, or even bridge failure. A sudden event is viewed as an event that drastically reduces the performance of the infrastructure in a short amount of time. According to [35], the most recurrent sudden events in bridges are related to hydraulic (e.g., floods, scour) and collision (e.g., vessel shocks) events.

Considering that the bridge can be subjected to those different processes of degradation, some scenarios of bridge performance should be defined to start the performance prediction analysis. Those scenarios can comprise situations where, in each specific year of analysis, there will be no maintenance applied on the bridge or some maintenance activities are carried out. In either of these situations, if a sudden event occurs, then corrective maintenance actions should be implemented. Regardless of the type of maintenance occurring in each scenario/year, the analysis is conducted for the entire years of each of the scenarios predefined. In the end, by employing a comparison between scenarios, boundaries for the bridge performance evolution can be drawn, and decisions regarding the best maintenance schedule can be taken. The following sections describe the above-mentioned three maintenance types in detail.

Table 2: Correlation between reliability KPI qualitative scale and reliability index (β) PI quantitative scale, adapted from [33].

Reliability KPI	Reliability PI (β)
1	$\beta \geq 4$
2	$3,25 \leq \beta < 4$
3	$2,50 \leq \beta < 3,25$
4	$2 \leq \beta < 2,50$
5	$\beta < 2$

Table 3: Correlation between safety KPI qualitative scale and safety PI quantitative scale, adapted from [33].

Safety KPI	Safety PI
1	No danger. It is very unlikely that a person could get injured because of the current bridge performance.
2	It is unlikely that a person could get injured because of the current bridge performance.
3	It is unlikely that a person could get injured because of the current bridge performance. Intervention shall be performed before the next inspection.
4	It is likely that a person could get injured because of the current bridge performance. Intervention shall be performed shortly after inspection.
5	Immediate danger. It is very likely that a person could get injured because of the current bridge performance. Immediate action is required.

Table 4: Correlation between availability KPI qualitative scale and speed restriction coefficient (SRC) PI quantitative scale, adapted from [34].

Availability KPI	Speed restriction coefficient PI
1	<10%
2	10%-40%-
3	40%–70%
4	70%–90%
5	>90%

Table 5: Correlation between cost KPI qualitative scale and cost PI quantitative scale.

Cost KPI	Cost PI
1	$\mathrm{COST}_{\mathrm{normalized}} < 20\%$
2	$20\% \leq \mathrm{COST}_{\mathrm{normalized}} < 40\%$
3	$40\% \leq \mathrm{COST}_{\mathrm{normalized}} < 60\%$
4	$60\% \leq \mathrm{COST}_{\mathrm{normalized}} < 80\%$
5	$80\% \leq \mathrm{COST}_{\mathrm{normalized}}$

2.2.1. No Maintenance. In this situation, it is assumed that there are no maintenance activities in the current year. This means that the bridge is going to degrade continuously. This is expected to be the most common option during the first years of the bridge. Sometimes, this is also the option even when the bridge is older. While that may seem inadequate, in many situations, the existence of large stocks of bridges and very limited budget leave no other option. In these situations, less important bridges can be successively left behind and have their maintenance postponed in favor of other more relevant bridges. However, even in these extreme situations, minor inspection actions are assumed to occur, at least to update the evolution of bridge's performance and ensure it is safe. Those inspection actions depend on several factors like the condition of the bridge, the type of inspection, the skills of the inspector, and the type of material.

In the current work, the proposal of [36] is adopted for the estimation of the inspection costs, given by the following equation:

$$C_{\mathrm{insp}} = \left[\frac{2\ d}{80} + \left(\frac{(20 + 0.5L)H \times S \times I \times M}{60}\right)\right] \times (C_l + C_v), \tag{3}$$

where d is the distance from the depot in km, L the length of the bridge, H the condition of the bridge, S the skill of the inspector, I the inspection type, M the bridge material, C_l the labor costs (€/h), C_v the vehicle costs (€/h), and r the discount rate.

2.2.2. Preventive Maintenance. Apart from the inspection actions, which are quantified using (3), in any specific year, there can be considered some preventive maintenance actions to reduce the degradation rate. The literature offers several models to compute costs of intervention on bridges. Nevertheless, the general approach of the cost calculation is computed by the following equation:

$$C_{\mathrm{action}} = \sum_{i=1}^{m} \mathrm{UC}_i \times \mathrm{Aq}_i \times \psi, \tag{4}$$

where C_{action} is the direct maintenance action cost per year (€), i is one of the m activities composing the action, UC_i is the unit cost of each activity (€/unit), Aq_i is the number of units of activity i (unit), and ψ is a reduction factor of costs according to the condition state of the bridge.

As the maintenance is carried out, there are indirect costs related to the delay imposed by the work ongoing on the bridge. This work might reduce the availability of the bridge, or even close it, thus forcing drivers to use alternative detour routes. On the other hand, this maintenance work can affect the availability of the bridge at different levels.

2.2.3. Corrective Maintenance. This situation is related to the occurrence of unexpected events that might lead to reducing, or even closing, the bridge in a specific year. Those unexpected events can cause disruptions on the network and thus significant impacts on traffic management. In the context of resilient management, this behavior is conceptually defined in Figure 2 and analytically expressed by the following equation:

$$\text{Resilience} = \int_{t_0}^{t_0+t_R} Q(t)\mathrm{d}t, \quad (5)$$

where $Q(t)$ is the time-dependent functionality, t_0 is the event occurrence time, and t_R is the time to complete recovery of the bridge under analysis.

It can be seen that a bridge with a certain functionality level is affected by some disruptive event at year t_0. Then, after a first moment in which the impact of such an event is being accommodated, a restoration process needs to be started, with the bridge being unavailable (partially or totally) during that process. The amount of time the recovery lasts, i.e., the bridge resilience, is a function of the observed damage. On the other hand, the response to the hazard event is highly dependent on a previous estimation of the potential consequences, as well as the definition of an adequate recovery plan.

A recovery curve of a bridge can be defined as a function that describes the process for restoring a bridge to its initial performance after a disruptive event. However, bridge recovery is a complex process as it is affected by several parameters, many of which have a high level of uncertainty. Therefore, the recovery models must have a simple structure such that they can be easily adapted to fit real or numerical observations. Several models have been proposed to describe recovery functions, which can be either empirical or analytical, depending on the source of data and the type of analysis [37–39].

Hazard estimation. The process of natural degradation that a bridge undergoes throughout its lifetime is usually designated an interceptable event. If a noninterceptable event occurs in a specific year, those two events must be combined to obtain a fair estimation of their joint effect on the bridge as given by the following equation:

$$P(H_A H_B) = P(H_A) + P(H_B) - P(H_A) \times P(H_B), \quad (6)$$

where $P(H_A)$ and $P(H_B)$ are the probability of bridge collapse occurrence after A and B events, respectively. It is worth mentioning that this formulation assumes that their occurrence is statistically independent and collectively exhaustive.

Consequences. Consequence estimation has been proposed by several authors in the literature for the field of the infrastructures at the network level, as it can be seen in [40]. Generally, these consequences are related to rebuilding the system given by the following equation:

$$C_{\text{reb}} = c \times W \times L, \quad (7)$$

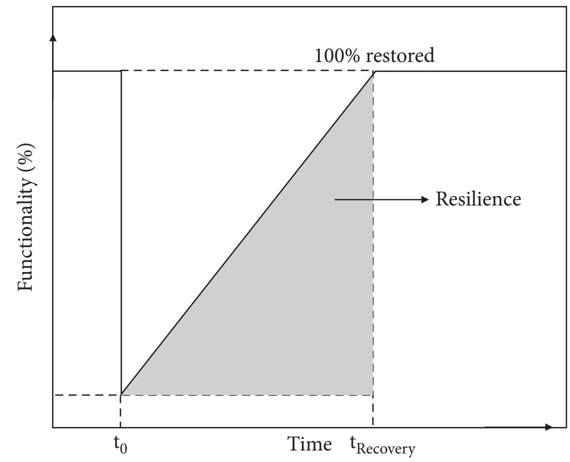

Figure 2: Resilience illustration.

where c is the cost per square meter (€/m^2), W the bridge width (m), and L the bridge length (m). Like other cost components, consequences also have an indirect part, which considers all the impacts that, despite being not directly related, followed the hazard occurrence.

3. Demonstration of the Methodology: Application to a Railway Steel Bridge

The methodology discussed in Section 2 was applied to a steel railway bridge located in Óbidos region, Portugal. Note that the developed methodology is consistent for any other type of bridge within reasonable assumptions. The bridge was originally built in 1886. However, due to the need for modernizing the rail line, the bridge was renewed in 1990. In this work, considering that a major intervention was made in 1990, the lifetime analysis of this bridge was considered ever since. The studied bridge is made of steel with the reticular structure shown in Figure 3, with a total length of 27.25 m and a width of 5.3 m. The average daily traffic is 30 trains.

3.1. Bridge Current Performance Assessment. The structural scheme adopted for this application was based on a truss bridge; see Figure 4. Note that the truss bridge is symmetric wherein the distance between adjacent points is 4.30 m with a corresponding height of 6.2 m. While there are different failure modes to be analyzed in a truss bridge, for the sake of brevity, in this case study, only the axial buckling failure mode is considered.

This bridge was subjected to two visual inspections recently. In the first inspection (2011), evident signs of decay and ageing were found, essentially related to corrosion and oxidation of the elements. In the second inspection (2015), corrosion was again the main problem of the bridge, with the deck being the most affected component as depicted in Figure 5. As for the cost analysis, there were no reports about its quantification of interventions or inspections on this bridge.

Figure 3: Óbidos bridge view.

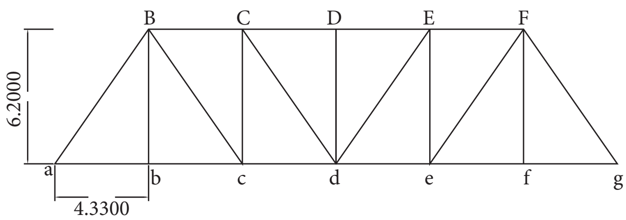

Figure 4: Structural scheme adopted [dimensions in meters].

Figure 5: Corrosion of the deck's steel members (bottom view).

3.1.1. Condition Assessment. For the condition assessment, the KPI previously presented in Section 2 were estimated. For reliability, only the superstructure, i.e., the deck, was considered. For this truss bridge, the limit state function can be calculated by considering the difference between the resistance axial strength and the axial action as expressed in the following equation:

$$g = f_y \times A - N_S\,(\mathrm{PL}, \mathrm{LM}\,71), \tag{8}$$

where f_y is the yield strength of steel in MPa and A is the cross section in mm^2. The axial load N_S is given as a function of the permanent loads (PL) and the live loads. The later was based on the LM71 load model provided by Eurocode [41]. Considering that uncertainty quantification is needed to have a proper definition of the reliability index, the involved variables were defined probabilistically by considering probabilistic normal distributions with mean and coefficient of variation parameters as provided in Table 6.

After having defined all the resistance and demand variables, a structural analysis was made, and the limit state equations were defined. A first-order reliability method (FORM) analysis was used to calculate the reliability index. Since the structure is isostatic, the obtained global reliability index was given by the minimum value obtained for each bar element. Thus, the obtained reliability index was $\beta = 4.87$, corresponding to the central vertical bar (bar Dd in Figure 4). This reliability index corresponds to the initial reliability. To consider the structural performance for the remaining years of the bridge life cycle, an estimation of the time-dependent reliability index was assumed based on an analytical model proposed by [42] and given by the following equation:

$$C = \mathrm{A}t^{B}, \tag{9}$$

in which C is the average corrosion penetration rate in micrometre, t is the time in years, A and B are regression parameters determined from analysis of experimental data under different environmental conditions. For this situation, the regression parameters A and B were assumed to be 34.0 and 0.65, respectively, for a rural environment and unprotected carbon steel.

Table 7 shows the obtained reliability index, for a situation of corrosion, until the time of the last inspection (2015). Note that the calculations were assumed for the year 1990. Because the first available reports of inspection were from 2011, a detailed calculation of the true initial time of corrosion was impracticable. Therefore, an alternative was found in the study of [43], which developed reliability-based degradation models for steel bridges, that is, a rate of corrosion being practically zero between 10 and 15 years. Likewise, for this study, no degradation in the first 10 years was assumed.

In this case study, safety of users was associated with the nonstructural element condition rather than the superstructure itself. Inspection reports state, back in 2011, that the pedestrian crossing was very much damaged, constituting a form of dangerous hazard for the operators of the line as observed in Figure 6.

The condition state on the sidewalks and parapets was classified by the inspector in 2011 and 2015. According to the scale proposed in this paper, in 2011, the safety was classified as 4. The inspection of the year 2015 showed that some sidewalks were replaced, thus denoting an improvement on the safety indicator. Considering this intervention, a classification of 3 was assigned to the safety.

The inspection of 2011 was merely visual with no signs of activities of maintenance on the bridge. Thus, the availability in that year was classified as 1, according to Table 4. In 2015, due to some repair activities on the sidewalks of the bridge, a value of 3 was assigned to the availability since trains were expected to pass slower during the period in which maintenance teams were working.

Regarding the KPI cost, the report of the inspections of 2011 and 2015 did not reveal any kind of expenses. Although the improvements on the sidewalks in 2015 were made, no costs were reported. Nevertheless, it is estimated that the intervention costs fall within a condition state level of 1, according to Table 5.

Table 6: Random variables quantification.

	Variable	Mean	CoV*	Reference
Resistance	Cross section, A (mm^2)	Nominal value	4%	JCSS 2001
	Yielding strength, f_{ym} (MPa)	202.16 MPa	7%	JCSS 2001
Actions	Permanent Loads (PL)	23 kN/m	10%	Assumed
	Live loads (LM71)	207.4 kN 63.4 kN/m	10%	CEN 2004

*Coefficient of variation

Table 7: Reliability value of the critical bar for each inspection year.

Year	Reliability index
1990	$\beta = 4.87$
2000	$\beta = 4.87$
2011	$\beta = 4.50$
2015	$\beta = 4.40$

(a) (b)

Figure 6: User safety condition assessment: (a) sidewalks; (b) parapets.

Considering the scales proposed in Section 2, Table 8 summarizes the evolution of all the KPI until the last year of inspection. Note that the bridge was in a good structural condition; hence, the reliability KPI is graded 1 in the entire column. Similarly, the cost KPI is also graded 1 since there were no indications from the reports of major interventions until 2015. Contrarily, safety of users was identified as a serious threat given the condition state of the parapets and the sidewalks. Because there was an improvement of those elements from 2011 to 2015, the availability KPI was graded 3 given the interventions. Figure 7 depicts these results, using a spider diagram, to ease comparison between the different years considered in the analysis.

3.2. Bridge Future Performance Condition. This section addresses the lifetime analysis of the bridge. For a demonstration of the methodology, the period of analysis was assumed to be 20 years starting in the last inspection year (2015). Moreover, such period was considered based on the periodicity of the inspections since there is a high probability of their occurrence in this time horizon. Some possible future scenarios were considered given different levels of loss of performance. Following the methodology discussed in Section 2, in this study, three types of scenarios were discussed: (i) natural scenario, (ii) preventive scenario, and (iii) corrective scenario. Each scenario was considered isolated to prove the calculation details associated with each one. However, scenarios combining years without maintenance actions, years with maintenance actions, and years in which some sudden event might occur can and should be considered.

3.3. Natural Scenario. In this scenario, only routine inspections were assumed, so natural evolution of bridge condition is considered. The time-dependent KPI are illustrated in Figure 8. The nonnormalized costs are also presented. It is observed that, in terms of reliability, the bridge presents a good structural performance. This was expected since the bridge was designed for a period higher than 20 years. Nevertheless, since reliability PI is progressively reducing, from year seven onwards, the corresponding KPI changed from 1 to 2 (Figure 8(a)).

The safety KPI, which in the beginning had a value of 3, decreased to a value of 5 around year 16. This reveals that

Table 8: Assessment of the bridge at the year of the last inspection.

Year	Reliability	Safety	Availability	Cost
1990	1	1	1	1
2000	1	1	1	1
2011	1	4	1	1
2015	1	3	3	1

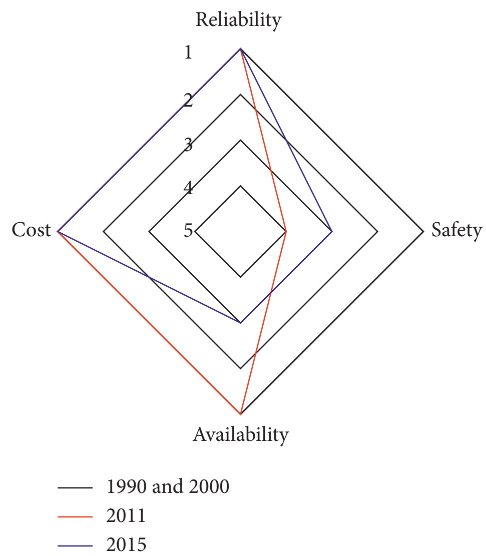

Figure 7: Bridge condition evolution until the last year of inspection.

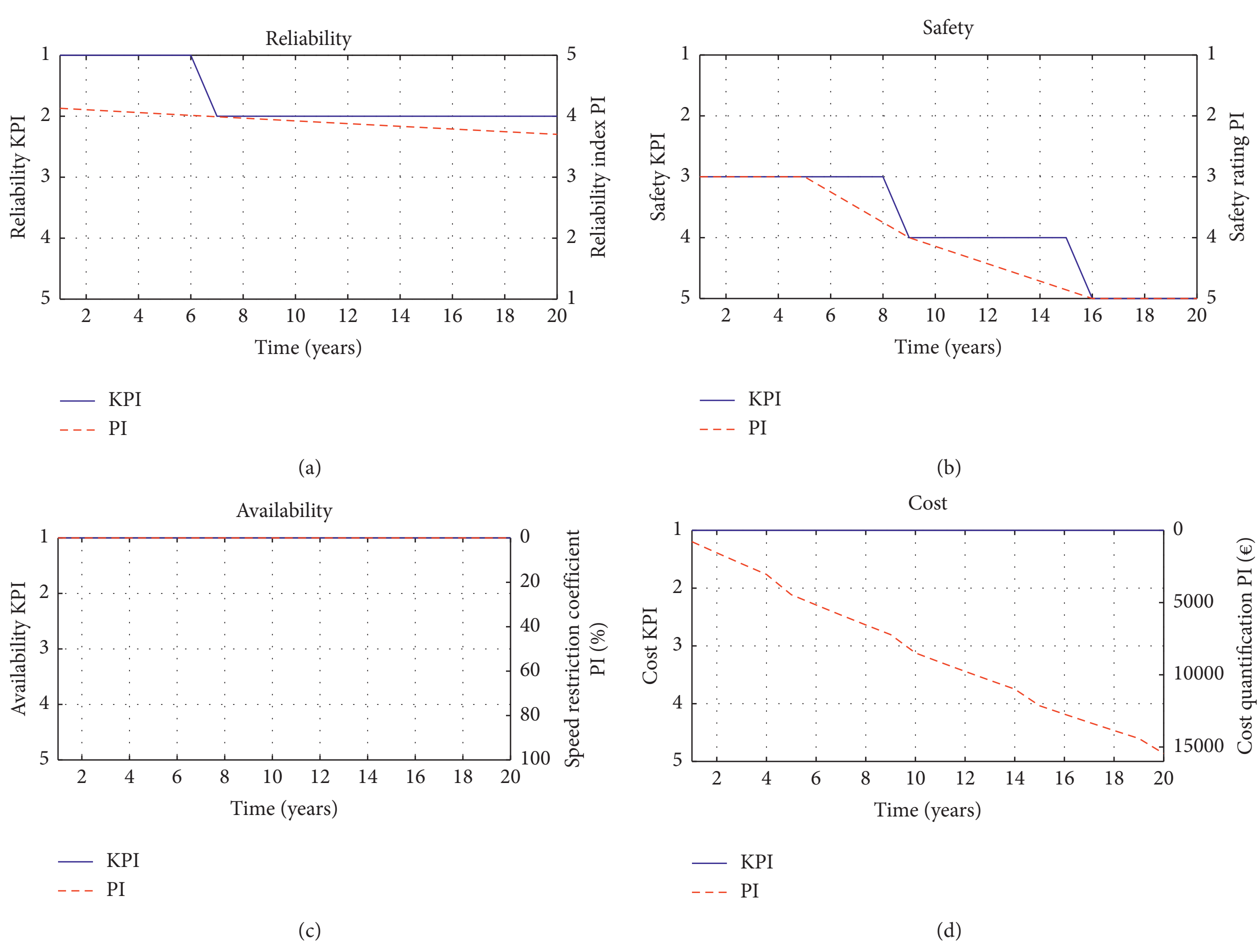

Figure 8: Bridge performance condition considering natural scenario: (a) reliability; (b) safety; (c) availability; (d) cost.

some actions should be taken to avoid major consequences for the users (Figure 8(b)).

Availability maintains a value of 1, for both PI and KPI, throughout all the lifetime analysis. This occurs since the system is considered fully available since there is no intervention; thus, no speed restrictions and no extreme disruptive events were considered (Figure 8(c)).

The cost KPI also presents a value of 1 during the entire lifetime of the bridge (Figure 8(d)). Nevertheless, in this case, the PI value is constantly changing since inspection costs were considered each year. The calculation of the costs for this scenario was based on (3). Their calculation was based on the parameters quantified in Table 9.

3.4. Preventive Scenario. Apart from the inspection actions, the preventive scenario includes preventive maintenance actions that are carried out to reduce the deterioration rate. Such maintenance actions include associated costs. The quantification of both direct and indirect costs, associated, respectively, with bridge managers and user costs, was taken into consideration in this work. For the sake of brevity, in the present work, only two maintenance actions were considered. Table 10 shows the effects of applying the maintenance actions on the bridge, as well as the unit costs and the frequency of application, based on expert opinion [44]. The direct maintenance cost calculation is given by (4).

The indirect costs can be computed by the following equation:

$$C_{\text{action,ind}} = \left[\text{DC} \times \text{DUR} \times \text{TMD} \times \gamma_{\text{prev}} \times (\text{SR} + \text{BRK})\right], \tag{10}$$

where DC are the delay costs (€/min) that the infrastructure company must pay to the train operator in case of maintenance activities, obtained according to an asset owner; DUR is the duration of the maintenance activity (days); TMD is the average daily traffic of trains; γ_{prev} is the speed restriction related to the preventive action; SR and BRK are delays related to the speed reductions and braking, respectively. Those parameters are herein estimated according to [45] and given by the following equations:

$$\text{SR} = 60 \times (\text{BL} + 0.15) \times \left(\frac{1}{S_r} - \frac{1}{S_n}\right). \tag{11}$$

$$\text{BRK} = \frac{1000}{60 \times 60 \times 60} \times (S_r - S_n) \times (2.2 - 0.0105 \times S_r), \tag{12}$$

where S_r and S_n are the reduced and normal speed in km/h, respectively, and BL is the bridge length, in km, with 150 meters added when there is reduced speed. Both these equations include the conversion factors to convert from *km* to *m* and hours to minutes. Table 11 shows the variable quantification adopted in this study.

The corresponding time-dependent KPI are illustrated in Figure 9. As the first tentative for a preventive scenario, actions were considered in the years in which performance changes were found in the natural scenario (see Figure 8). Hence, in year 6 an action was taken to delay the corrosion process, thus maintaining reliability in the best value for two more years (Figure 9(a)). Likewise, in years 8, sidewalks were replaced to improve the safety level; when considering the natural scenario, it was expected to decrease (Figure 9(b)). Moreover, with the preventive actions applied on the bridge, the availability KPI slightly decreased in the periods when they were being applied, due to some speed restrictions (Figure 9(c)). The cost KPI remained at a maximum value of 1. However, it can be observed that the cost PI experienced a sudden increase in the years when the preventive actions were applied (Figure 9(d)).

3.5. Corrective Scenario. As stated in Section 2, the corrective scenario aims at accounting for situations wherein an unexpected event occurs forcing the closure of the bridge for its rehabilitation. This scenario is yet somehow different from the previous one in terms of assessment. Thus, this section is divided into three steps: (i) hazard analysis, (ii) consequence estimation, and (iii) recovery plan.

3.5.1. Hazard Analysis. For the hazard analysis, a multihazard event is herein applied following (6). Here, the events A and B were defined according to the case study implemented. Hence, the event A stands for the corrosion process while event B stands for the flood.

Floods were reported to be a common event in the bridge's location wherein the level of the flood results in water reaching the deck of the bridge in some of the worst past floods. In this way, an estimation of the bridge damage due to the flood is addressed, as well as consequence estimation to highlight the potential threats of such event and possible consequences for the bridge and thus for the network to which it belongs.

The flood event estimation follows the formulations of [46, 47]. When the deck is partially or completely submerged, the main forces involved are the dragging, F_D, and the lifting, F_L, forces given by the following equations, respectively:

$$\frac{F_D}{L} = \frac{1}{2} \times C_D \times \rho \times v^2 \times s\,(\text{kN}/m). \tag{13}$$

$$\frac{F_L}{L} = \frac{1}{2} \times C_L \times \rho \times v^2 \times W\,(\text{kN}/m), \tag{14}$$

where C_D is the drag coefficient, C_L the lift coefficient, ρ the density of water, v the flow velocity, s the deck thickness, L the bridge length, and W the width of the bridge deck.

Deck failure occurs when there is transverse or uplift failure. Transverse failure is defined as the event where the drag force exceeds the transverse resistance between the deck and the piers and the uplift force does not exceed the uplift capacity of the bridge. Here, transversal capacity was considered to be the friction force as μPL, with μ being the friction coefficient and *PL* the permanent loads [48]. Thus, the limit state function for transverse failure is given by (15). Uplift failure is defined as the event where the uplift forces exceed the uplift resistance of the bridge and the drag force is

Table 9: Quantification of the variables for inspection costs.

Parameters	Notation	Quantification
Distance from the depot (km)	D	Approximately 344 km
Length (m)	L	27.25
Condition of the bridge (H)*	H	0.9
Skills of the inspector (S)*	S	1
Inspection type (I)*	I	1
Bridge material (M)*	M	1.2
Labor costs (€/day)*	C_l	Technician: 207.66 Supervision: 119.66 Operator: 109.07
Vehicle costs (€/km)	C_v	0.40

*Values provided by bridge owner. The inspection team is composed of 1 technician, 1 supervisor, and 3 operators. The inspection takes one day and is made on an annual basis.

Table 10: Effects for the maintenance actions.

Preventive maintenance action	Effect of the maintenance	Frequency of application	Cost
Anticorrosive painting	Delay of corrosion process for 2 years	10 years	1400 €/m
Sidewalk replacement	Restoring safety level	15 years	100 €/un

Table 11: Variable quantification for indirect costs.

Parameters	Quantification	
DUR	Expert opinion	
l_r	27.25 m	
l_t	200 m	
Type of train	Regional trains	Medium-long trip trains
DC	4€/min	2.5€/min
S_r	30 km/h	30 km/h
S_n	90 km/h	90 km/h
TMD	30	5
γ_{prev}	40%	

higher than zero. The uplift capacity of the bridge is permanent loads. The limit state function associated with this event is given by (16). Then, the final probability failure of the deck is given as the combination of these two events according to (17).

$$g_{\text{transverse}} = p\left[F_D > \mu\left(\text{PL} - F_L\right)\right] \cap p\left(F_L \leq \text{PL}\right). \quad (15)$$

$$g_{\text{uplift}} = p\left(F_L > \text{PL}\right) \cap p\left(F_D > 0\right). \quad (16)$$

$$p_{f,\text{final}} = g_{\text{transverse}} \cup g_{\text{uplift}}. \quad (17)$$

For this case study, the stream is assumed to have a trapezoidal cross section with a 45° wall inclination, a bottom width of 17 m, a top width of 27 m, and a height of water of 3.525 m. Thus, assuming Manning's equation, the discharge can be obtained by the following equation:

$$Q = \frac{A}{n} \times \left(\frac{A}{P}\right)^{2/3} \times i^{1/2}\left(m^3/s\right), \quad (18)$$

where A is the cross section of the flow, P is the wetted perimeter, i is the slope of the channel, and n is the Manning roughness coefficient. Therefore, the velocity is given by the following equation:

$$v = \frac{Q}{A}\,(m/s). \quad (19)$$

Uncertainty of the input variables was estimated by their mean value and coefficient of variation (CoV) as shown in Table 12.

By applying the FORM analysis and considering the limit state function given by (15) and (16) and the parameters in Table 13, the obtained reliability index considering the effect of the flood event was $\beta = 2.00$. Note that, for computing the reliability index, it was assumed that the wetted perimeter reached the height of the deck.

Moreover, considering the combination of the hazards, the joint failure probability, defined in (6), is given as follows:

$$P\left(H_C H_F\right) = P\left(H_C\right) + P\left(H_F\right) - P\left(H_C\right) \times P\left(H_F\right), \quad (20)$$

where $P(H_C)$ and $P(H_F)$ are the failure probabilities given the hazards of corrosion and floods, respectively. The obtained probabilities given the hazard of corrosion and flood were $4.81e-5$ and 0.030, respectively. The resulting joint failure probability was around 0.030. It should be highlighted that this formulation was adopted for the calculation of the reliability index of the deck. The obtained value can thus be considered conservative since the whole deck-pier-foundation system was not considered due to the lack of information regarding the pier and the foundations.

3.5.2. Consequence Estimation. Direct consequences on the system are here estimated based on (7). As for the effects, the bridge is assumed to return to as-built conditions, with a total rebuilding cost of 8000€/m^2. This value was based on expert opinion. The estimation of the evolution of the performance indicators over time is illustrated in Figure 10. Due to the high uncertainty of a sudden event, the time of its occurrence was assumed to happen at year 12 just to

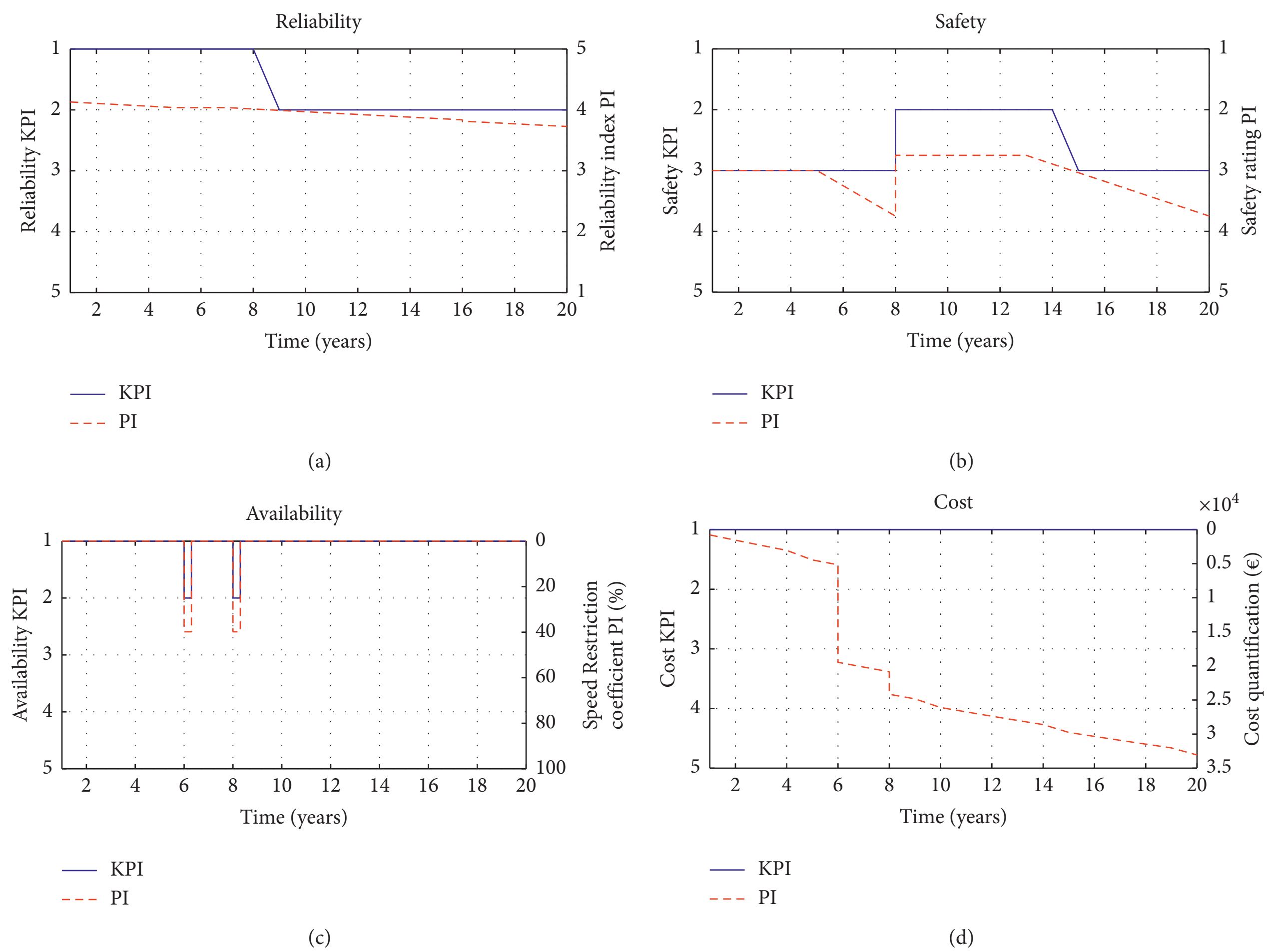

Figure 9: Bridge performance considering preventive scenario: (a) reliability; (b) safety; (c) availability; (d) cost.

Table 12: Variable quantification for flow quantification.

Variable	Mean	CoV	Distribution	Observation	Reference
Channel's slope, i (m/m)	0.005	0.053	Normal	Measured from topographic data	[49]
Manning's roughness, n	0.060	0.068	Normal	Assuming natural channel	[50]
Model uncertainty factor, k_v	1	0.15	Lognormal	Factor related to flow velocity	[46]
Drag coefficient	1.10	—	Deterministic	—	[47]
Lift coefficient	−1.60	—	Deterministic	—	
Thickness of the deck, s (m)	1.525	—	Deterministic	From drawings' information	—

Table 13: Variable quantification for indirect consequences.

Description	Notation	Value		
Traffic conditioned percentage	γ_{corr}		100%	
Average daily traffic	TMD	Cars 950		Trucks 50
Cost per kilometre (€/km)	C_K	0.18		0.68
Cost per hour (€/h)	C_H	8.4		10.1
Normal speed (km/h)*	S_n		120	
Restricted speed (km/h)	S_r	70		50
Detour route (km)	L_D		8.700	
Normal route (km)	L_P		5.000	

*Normal speed of the train for that zone of the line.

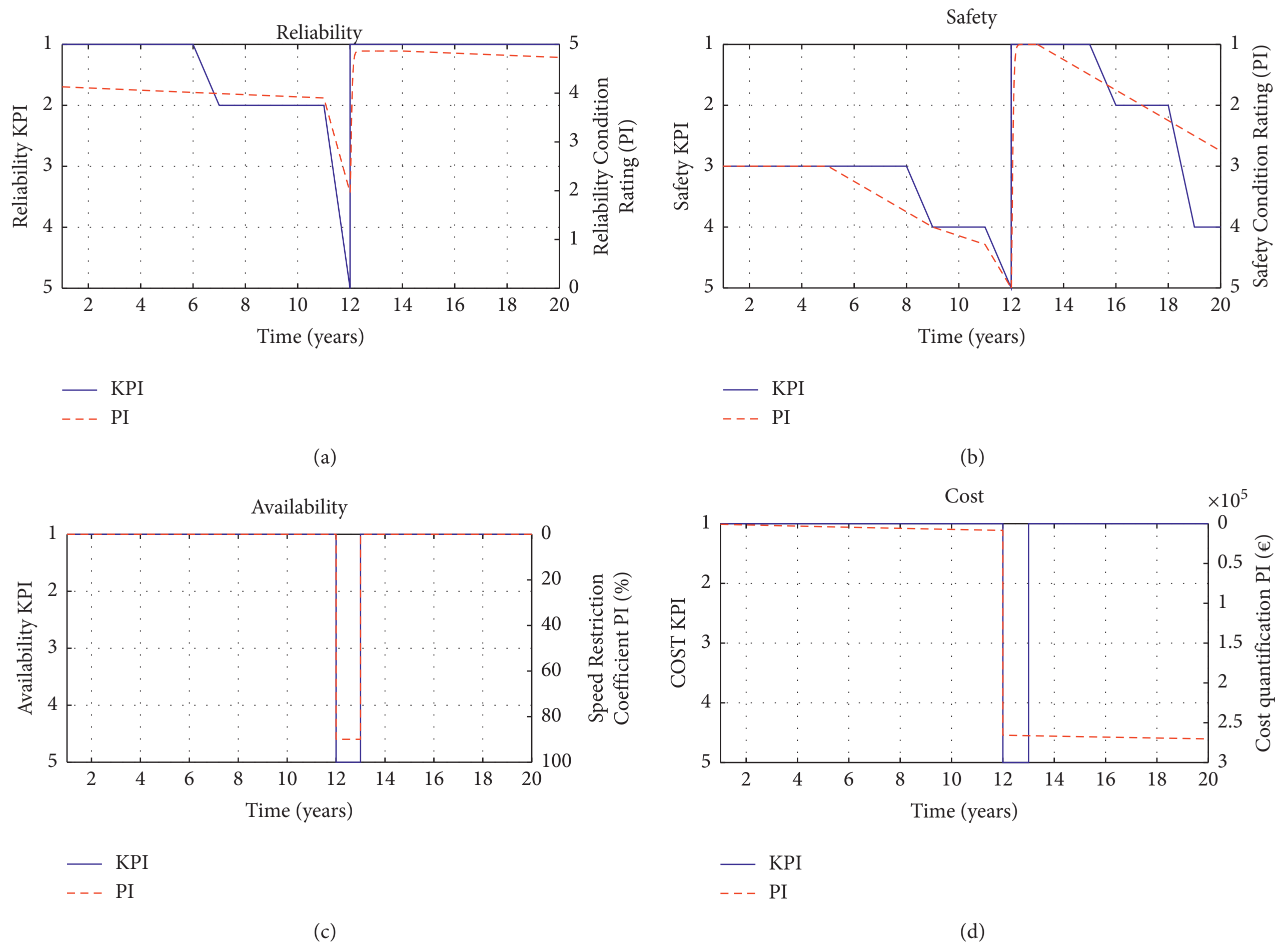

Figure 10: Bridge performance condition considering corrective scenario: (a) reliability; (b) safety; (c) availability; (d) cost.

exemplify the application of corrective scenario. Furthermore, in this figure, only the direct consequences, i.e., rebuilding of the system, are presented. All the indirect consequences are covered in the next section regarding the recovery plan of the system.

3.5.3. Recovery Plan. In this section, the year of the occurrence of the event is thoroughly discussed. For this study, it is assumed that the bridge is meant to be fully recovered, i.e., return to as-built condition. Concerning the indirect consequences, their calculation was based on (21), with the variables being quantified according to Table 13. Note that, in this work, the considered indirect consequences were related to the detour of the vehicles, i.e., when finding an alternative route. With most of the railway tracks being not redundant as the roadway roads, most of the time, an alternative route is defined through roadways. Bearing this in mind, the calculation of the indirect consequences is based on (21) provided by the study of [34].

$$C_{\text{detour}} = \text{DUR} \times \gamma_{corr} \times \sum_{v=1}^{2} \text{TMD} \times \left[C_K \times (L_D - L_P) + C_H \times \left(\frac{L_D}{S_r} - \frac{L_P}{S_n} \right) \right], \tag{21}$$

where DUR is the duration of the activity (days), γ_{corr} is the speed restriction for the corrective intervention, v is a variable that considers the vehicle type (for cars $v = 1$ and trucks $v = 2$), TMD is the average daily traffic, L_D is the detour route length (km), L_P is the normal route length (km), S_n is the normal speed (km/h), S_r is the restricted speed (km/h), C_K is the unit cost per kilometre (€/km), and C_H is the unit cost per hour (€/h).

The DUR variable is an unknown parameter as there is no real information about the recovery time of the bridge. Thus, recovery time values were assumed based on the literature review on bridge resilience topic. The study of [51]

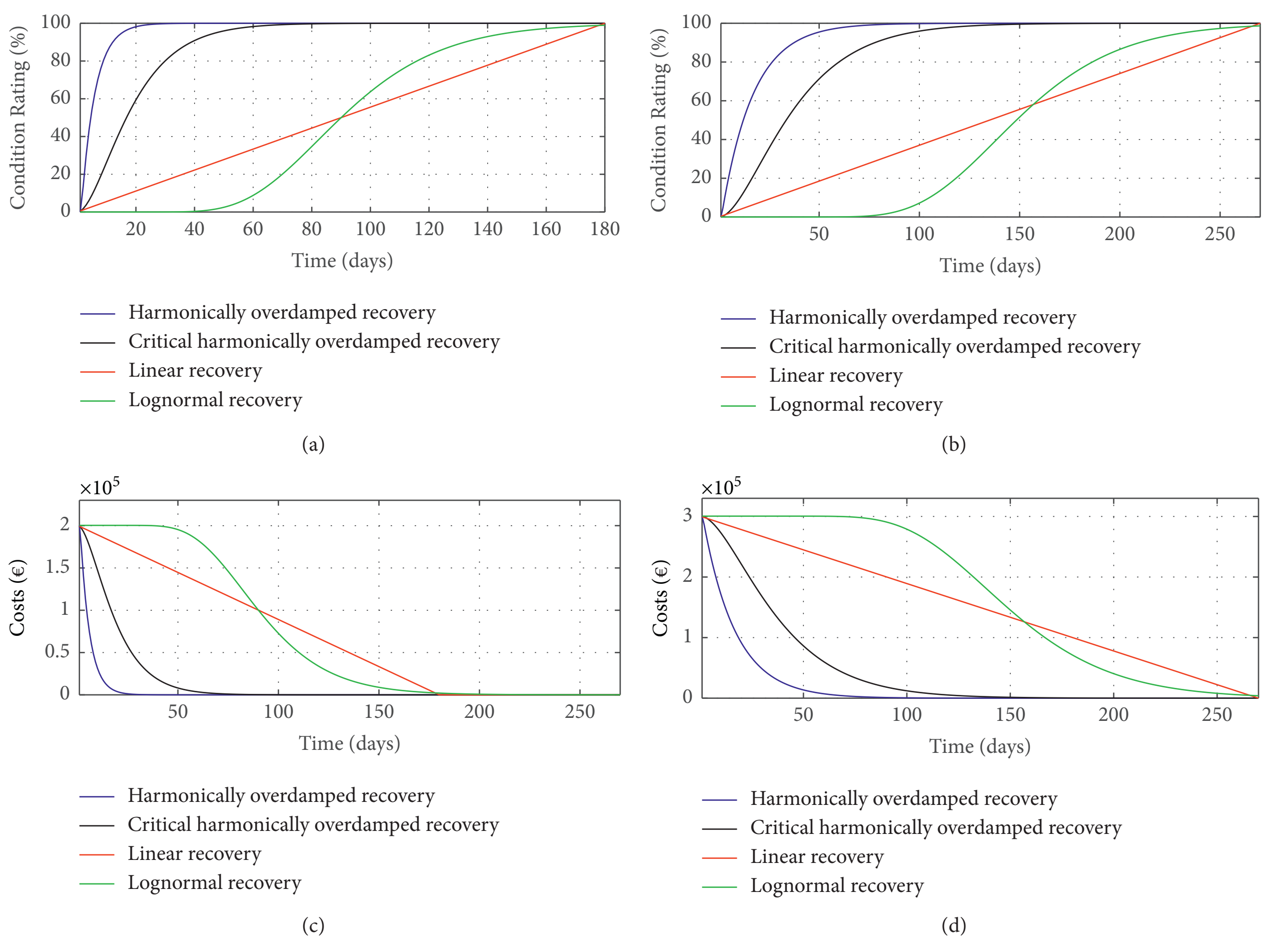

Figure 11: Recovery scenario and consequence estimation for different recovery functions: (a) recovery for 180 days; (b) recovery for 270 days; (c) consequences for 180 days; (d) consequences for 270 days; (e) legend of the recovery functions.

Table 14: Resilience estimation using different recovery functions.

Recovery function	Resilience (%)	
	Recov. time of 180 days (%)	Recov. time of 270 days (%)
Harmonically overdamped	96.7	93.7
Critical harmonically overdamped	88.9	85.1
Liner	50.0	50.0
Lognormal	47.8	43.1

proposes the recovery time for bridges according to different levels of severity. In this case study, a moderate and high severity were considered resulting in a recovery of 180 and 270 days, respectively.

The recovery functions were obtained following the methodology in Section 2. The selection of the best parameters is a difficult task since society preparedness and response are quite variable. However, some functions fit better for a fast recovery while others fit better for a slow recovery. Since the present case study has no available information regarding those parameters, a parametric study is proposed based on the recovery functions proposed by [37–39]: (i) harmonically overdamped recovery, (ii) critical harmonically overdamped recovery, (iii) linear recovery, and (iv) lognormal recovery. Thus, for each recovery time, the corresponding indirect consequences for the closed-system, recovery functions as well as the resilience for each recovery function were estimated based on (21). Figure 11 depicts the recovery functions as well as the estimation of the consequences for 180 days and 270 days. Table 14 resumes the resilience estimation for different recovery functions. To ease comparison, the recovery functions were normalized and then converted into percentages.

Observing the obtained results, we find that the harmonically as well as critical overdamped recovering functions present the highest resilience, being thereby the functions that correspond to a well-prepared recovery. Contrarily, the linear and the lognormal functions present

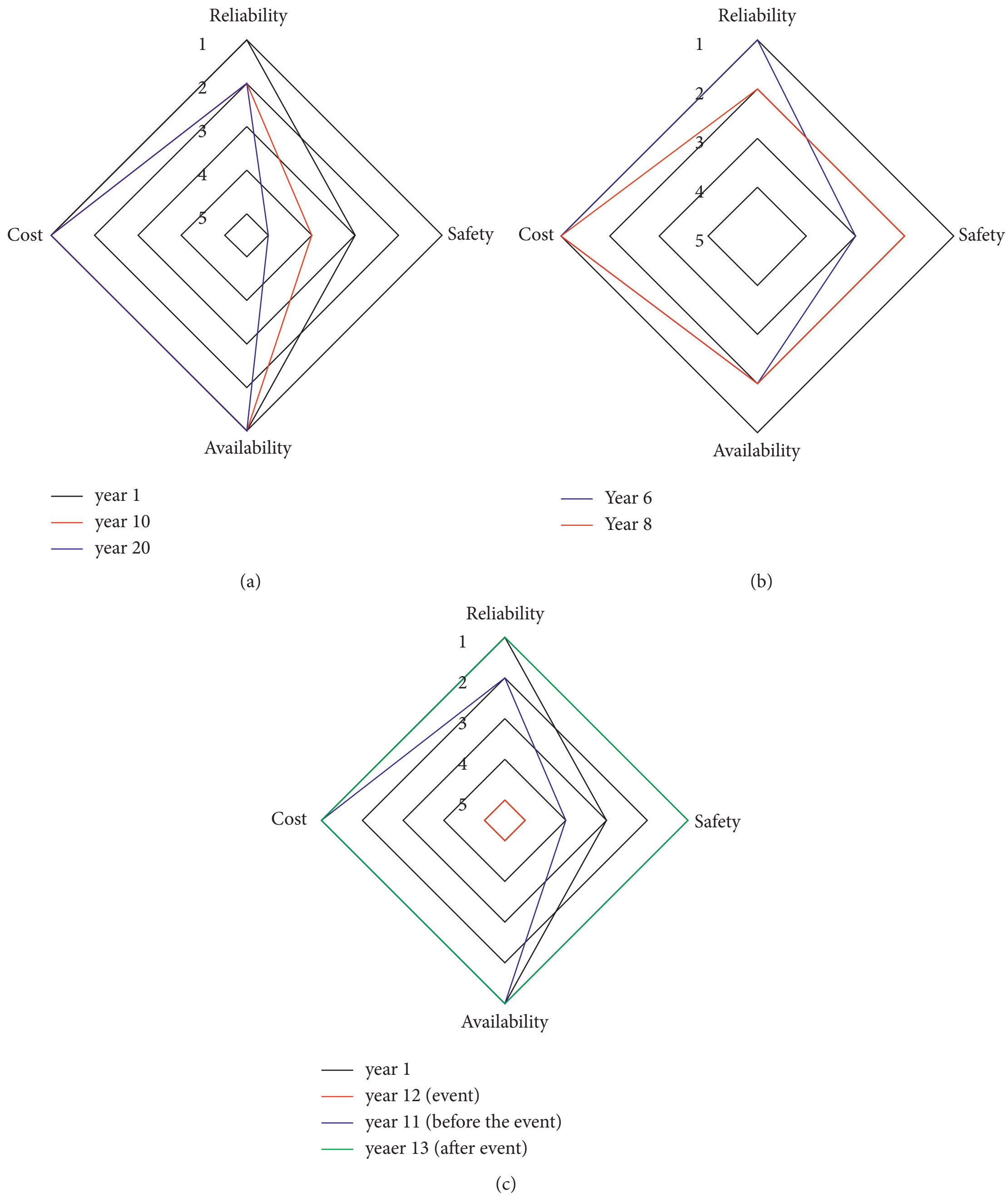

Figure 12: Bridge performance condition in different scenarios: (a) natural; (b) preventive; (c) corrective.

the lowest recovery capacity. Regarding the estimation of the indirect consequences, it is observed in Figure 11 that, as expected, their values decrease while the recovery is occurring. The high influence of the recovery time on the indirect consequences is also observed; i.e., lower recovery times lead to considerably lower indirect consequences.

3.6. Comparison of the Key Performance Indicators. The last step of the proposed framework summarizes all the results obtained in the previous sections. Spider diagrams were used for this purpose as they are very useful for showing results combining different indicators simultaneously. The results for the three different scenarios analyzed can be seen in Figure 12. The years selected for result visualization were based on the years before and after the most relevant events, i.e., maintenance for the preventive scenario and sudden event for the corrective scenario. As for the no maintenance scenario, three results, for 10-year-spaced periods, were considered.

Considering all the analyses presented in the previous sections, it can be concluded that, in the analyzed period of

20 years, the safety KPI is the most relevant for the performance condition of the bridge. Therefore, maintenance actions should be carefully considered to maintain proper safety to the users. On the other hand, the natural scenario, at year 8, already presented minor signs of degradation. These findings were the premises for the preventive scenario. With the introduced preventive actions, the safety KPI presented an improvement, while reliability KPI was kept better for a longer period. These introduced some losses in terms of cost and availability KPI.

This kind of conflict between bridge performance increase and budget/time decrease shall be considered very carefully by the manager of the bridge. As for the corrective scenario, a sudden event of the flood was simulated. In year 12, corresponding to the event occurrence, all KPI were graded 5, thus assuming the worst scenario possible. After the recovery period, the bridge was rebuilt, and all KPI were updated accordingly.

4. Conclusions

Different management scenarios considering the methodology of this work were presented. The methodology combines two different assessment moments: (i) assessment of the bridge at the current year, considering the inspection reports of the bridge; (ii) lifetime analysis in which different scenarios were proposed and discussed. Besides, this work introduced a proposal of a recovery plan in the corrective scenario with an estimation of the resilience for different recovery functions and periods. Thus, the main contributions of this work focused on the following:

(i) Updating the existing Quality Control Plan, proposed by COST TU1406 (TU1406 2018), by introducing resilience concepts in case of an extreme event occurrence, in addition to proposing a recovery plan after its occurrence.

(ii) Extending the methodology to other types of bridges, e.g., railway bridges, by proposing specific scales for computing KPI in their context.

The approach was validated in a truss railway bridge located in Portugal. It must be noted that, although the methodology may be considered for similar assets, the conclusions of the case study are dependent on the characteristics and conditions of the case study itself and must not be extrapolated to other cases without carrying out the full framework analysis. The assessment of the bridge in terms of reliability revealed that the bridge presents a good condition in terms of structural analysis. On the other hand, the safety of users was somehow compromised since the parapets and sidewalks showed poor condition.

Regarding the lifetime assessment, three different scenarios were considered in an analysis period of 20 years. The no maintenance scenario has shown that the reliability presents a good performance. On the other hand, safety was compromised, reaching the worst possible grade at year 16. Even in this unsafe scenario, since there were no interventions on the bridge, the availability KPI was classified as 1. Likewise, the cost only included visual inspections and thus was graded 1.

For the preventive scenario, two interventions were considered revealing a good improvement on the bridge performance, mainly on the safety KPI. Accordingly, in the interventions' years, the availability decreased since speed restrictions needed to be considered. The corresponding costs of maintenance were also calculated. Despite an additional cost from the preventive maintenance actions, a grade of 1 was achieved.

The corrective scenario covered the possibility of sudden event occurrence. In the present work, the impact of a flood was simulated to be estimated. A conservative approach was adopted by considering only the bridge's deck. Moreover, a recovery plan was proposed to estimate the consequences of the flood occurrence, as well as the bridge's resilience in the postevent period. Several recovery functions were applied considering a well-prepared and a not-prepared system. The results have shown considerable differences in the obtained resilience for each recovery function, with the critical and harmonically overdamped recovery functions being the best, and the lognormal and the linear recovery functions being the worst. It was then emphasized that defining proper recovery plans is of utmost importance.

Future developments in this field must deal with some of the limitations identified in this work, namely, the following:

(i) Quality and quantity of information to quantify the performance indicators. The lack of information about inspection reports, as well as damage quantification, forced the authors to solve this issue using models adopted in the literature. On the other hand, the quantification of the condition state is known to be subjective since it normally includes parameters defined based on expert judgement. Strategies to overcome this aspect should also be sought.

(ii) Quantification of a sudden event. Due to the fact of not having in-site information about the flood event, the authors proposed quantification based on some studies about hydrological events based on the literature. The methodology presented should be tested in new case studies in which more complete information is available; thus, fewer assumptions need to be made.

(iii) Quantification of direct and indirect consequences. In this regard, it was necessary again to take advantage of reasonable formulations and values adopted in similar case studies presented in the literature.

(iv) Resilience quantification. Given the lack of information about recovery time for the present study, the authors proposed different recovery times based on the literature. The same happened for the recovery function given the lack of historical information on recovery systems. In the end, the sensitivity analysis conducted could be revisited in case new information becomes available to pick the most suitable recovery times/functions.

Data Availability

Some or all data, models, or code used during the study were provided by a third party (inspection records). Direct request for these materials may be made to the provider as indicated in the Acknowledgments.

Disclosure

The sole responsibility for the content of this publication lies with the authors. It does not necessarily reflect the opinion of the European Union. Neither the Innovation and Networks Executive Agency (INEA) nor the European Commission is responsible for any use that may be made of the information contained therein.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

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