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Theory and Application of a Method for Prioritizing Bridge Interventions

Alberto Contardi*

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Abstract: This article presents a calculation method developed by the company SINA with the aim of establishing a priority ranking for interventions involving the improvement, upgrading, or demolition and reconstruction of a set of bridges. As a measure of the urgency of an intervention, the "cost of postponement" for the community is proposed, defined as the increased risk associated with maintaining the current state of the structure for 1 year. Following the standard logic of risk analysis, this cost is estimated by multiplying the probability of bridge collapse, calculated over a 1-year period, by all the potential damages that such a collapse would cause.

Although the calculation is affected by several sources of uncertainty, the proposed procedure, based on the collection of a few essential data points and a limited number of simple steps, allows for sufficiently reliable comparisons between bridges. It highlights which characteristics most significantly influence risk and which can reasonably be neglected.

Author keywords: Risk analysis; failure costs; maintenance strategies; damages; aged bridges; priority ranking

Introduction

All calculation tools that enable a managing company to develop, in the most rational way possible, a program of maintenance or improvement interventions for the bridges within its road infrastructure, based on available knowledge of the structures, while minimizing economic resource consumption and maximizing user safety, are generally referred to as BMS (Bridge Management Systems).[1,2,3] The most recent examples of research in the field of BMS focus on the use of artificial intelligence for the preliminary classification of structures.[4–7]

When one of these evaluation methods is applied to the entire set of managed bridges, the result is always a more or less extensive list of bridges requiring intervention, along with an equally extensive list of various types of actions to be carried out on them. For example, in the case where the Italian Bridge Guidelines [CSLLPP[8]] are followed, at the end of the process of progressively increasing knowledge about the structures, a list of bridges classified as "Transitabili" (passable) is obtained. This refers to structures that require upgrading works so that they can be verified according to the currently applicable Technical Standards for Construction.

It should be noted that, in any case, regardless of the decision-making method adopted, the theoretical risk is minimized only when *all* the planned interventions are carried out *immediately*. However, when the workload is such that it makes the immediate execution of all interventions impossible, not only for economic reasons but also due to organizational and management constraints, it becomes necessary to determine which interventions can be postponed so that the associated increase in risk can be considered acceptable. A system is therefore needed to select the intervention program that represents the best compromise between risk reduction and operational management requirements.

This article presents the calculation method developed by the company SINA,[9] published in various papers and currently being tested,[10] with the aim of establishing a priority ranking for structures already classified as "Transitabili" according to the Italian Bridge Guidelines, that is, structures that must soon undergo repair, improvement, or demolition and reconstruction interventions. The method therefore serves as a necessary complement to the regulatory framework and is useful for estimating which structures (already identified for intervention) require more urgent action and which can reasonably tolerate a delay with an acceptable increase in risk.

General Considerations

The basic idea is that it is possible to estimate, for each bridge, the "cost of postponing" an intervention. This cost is represented by the increased risk associated with maintaining the current condition of the structure for another year instead of immediately carrying out the planned works. The higher the estimated cost, the greater the urgency of performing the improvement intervention without delay.

*Corresponding Author: Alberto Contardi.
Email: alberto.contardi@sina.it
SINA S.p.A., Milano, Italy

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The estimate is developed by considering a finite number of risk scenarios against which protection is sought. For each scenario, it is necessary to identify a limit state that must not be exceeded, the probability that it may be exceeded within the considered time period, and the material damage to society that would result from exceeding that limit state. The cost of postponement for a single risk scenario is given by the product of the probability that the aforementioned limit state will be exceeded within 1 year and the corresponding theoretical economic damage, as expressed in Eq. (1):

$$C_d = \sum D_j \left(p_j - p_{0j}\right) \tag{1}$$

where C_d is the total cost of postponement, D_j represents the economic damage associated with exceeding the jth limit state, p_j is the probability that the structure in its current condition will exceed that limit state within 1 year, and p_{0j} is the probability calculated for the structure after the planned restoration intervention has been carried out immediately.

Risk Scenarios

The first step is therefore to appropriately select the risk scenarios to be considered. In order for the costs associated with individual scenarios to be simply summed, as in Eq. (1), it is necessary that the scenarios be statistically independent. For bridges with typical characteristics, a reasonable choice could be the following:

- Collapse of part of the deck due to traffic overload.
- Failure of a slab panel due to traffic overload.
- Collapse of one or more piers caused by an earthquake (or by wind, where more significant).
- Collapse of the structure caused by a hydrogeological event.

Further addressing the issue of the statistical independence of the events, it should be noted that, for each scenario, the structural components considered (girders, deck slab, piers, and foundations) are different; they were constructed at different times, using different materials, and, in some cases, by different personnel. Moreover, the external loads are also entirely independent. The only exception concerns the first two events, both of which are related to traffic loads. However, under ordinary conditions, the most critical actions for the verification of the entire deck are generally associated with global load models, such as Load Model 1 of the Eurocode. For deck slab verification, on the other hand, provided that the spacing between girders is not excessively large, the most critical results are more frequently associated with Load Model 2 or other concentrated loads. In other words, the traffic and loading configurations leading to collapse in the two scenarios are generally substantially different and, for the purposes of the proposed method, may reasonably be considered statistically independent. For bridges with very short design spans or with widely spaced girders, a case-specific assessment may be appropriate, and a single traffic-load scenario could be considered instead.

As is well known, the events that cause the greatest number of bridge collapses worldwide are hydrogeological in nature. Unfortunately, however, the literature does not currently provide sufficiently reliable formulations to estimate the probability of bridge collapse due to landslides or floods. Therefore, in this publication, hydrogeological events have not been considered. However, if a planned intervention does not affect the structural resistance to this type of event, it can be observed that the difference $p_j - p_{0j}$ in Eq. (1) for the hydrogeological scenario is equal to zero. Therefore, the cost component associated with hydrogeological events may reasonably be neglected. Conversely, if the set of interventions under comparison includes measures aimed at improving bridge performance with respect to hydrogeological hazards, the collapse probabilities associated with such scenarios should also be evaluated. In these cases, the relevant information available in the specialized literature should be consulted.

Probability of Collapse

Since this calculation is performed at a stage when the decision about which bridges require intervention has already been made, it is assumed that the results of structural assessments are available for traffic loads, as well as for seismic and wind actions. It should be noted that these assessments do not necessarily need to have been carried out using highly sophisticated calculation methods (e.g., finite element analysis). All that is required for the proposed method is, for each of the three considered risk scenarios, the value of the structural resistance R at the most critical section, to be compared with the maximum effect of external loads E at the same section.

It is therefore possible to estimate the probability of exceeding the limit state based on the aforementioned resistance and load effect data, as illustrated in a recent publication by SINA,[11] to which the reader is referred for further details and for the derivation of the formulas. The calculation of the reliability index β, which expresses the probability of exceedance, can be represented by the following relationship:[12]

$$\beta = \beta_C + ln\,(R/E)\,/\sigma \tag{2}$$

where the additional symbols have the following meaning:

- β_C is a constant representing the reliability index when resistance and load effect are exactly equal;
- σ is a parameter expressing the statistical variability of $ln\,(R/E)$.

The practical application of Eq. (2) varies depending on the specific risk scenario considered, as described in the following sections.

Since the time period considered is only 1 year and, under ordinary conditions, material deterioration does not progress significantly over such a short interval, the probability of collapse is assumed to remain constant over time and to depend solely on the structural assessments performed with respect to the current condition of the bridge. Conversely, if the deterioration process is progressing very rapidly, the bridge should be excluded from the present prioritization procedure and immediate intervention should be undertaken without performing further analyses.

It should be noted that the choice of the time interval length—taken here, for convenience, as 1 year but potentially adjustable, provided that it remains within the range of short intervals and thus preserves the assumption that the evolution of the materials' condition state can be neglected—affects the absolute values of the collapse probabilities and, consequently, the final costs, but it does not affect the ranking of the bridges and is therefore not important for the purposes of the present method. It may be appropriate to set it equal to the frequency at which the calculations are periodically updated.

Failure due to traffic loads

In the case of global deck failure caused by traffic loads, the resistance R may be expressed as the bending moment capacity or shear capacity, calculated using the safety factors at the ultimate limit state required by current standards. Following the same logic, the maximum effect of external loads E may be represented by the corresponding bending moment or shear resulting from the most unfavorable traffic load configuration, including the relevant safety factors at the ultimate limit state, again according to current regulations. The section to be considered should clearly be the one yielding the lowest ratio R/E, selected among those of the main load-bearing structures, typically the beams or the entire deck in the case of box girder decks. In the case of local slab failure, the considered actions change, but the reasoning remains entirely analogous.

In calculating the resistance, material degradation must necessarily be taken into account. Since the time period considered for probability evaluation is very short, only 1 year, the deterioration of the bridge over time does not need to be modeled; it is sufficient to consider the current condition.

As for the constant β_C, since in this case the ratio R/E is equal to 1 when the structural resistance exactly meets the requirements of current standards, reference can be made to Eurocode 0,[13] Section B3.2, thus adopting $\beta_C = 3.8$ for a reference period of 50 years.

For the calculation of the statistical variability σ, reference is generally made to the previously mentioned SINA publication.[11] In brief, the parameter σ was calibrated to comply with the requirements of the Italian Bridge Guidelines, which prescribe reduced safety factors γ'_M, γ'_Q, and γ'_G when assessing a structure against a target reliability index of $\beta = 2.8$. Approximating R_d as $R_d \cong R_k/\gamma_M$, it can be demonstrated that the safety factor $(R/E)_{2.8}$ corresponding to a structure with $\beta = 2.8$ is such that $(R/E)_{2.8} \cong \gamma'_M(\gamma'_Q + \rho\gamma'_G)/(\gamma_M(\gamma_Q + \rho\gamma_G))$, where ρ denotes the ratio between permanent and variable loads. Since $\sigma = ln(R/E)/(\beta - \beta_C)$, by setting $\beta = 2.8$, it follows that $\sigma = -ln((R/E)_{2.8})$.

Formula (3) is therefore proposed, as it yields relatively low values and is thus highly conservative:

$$\sigma = -ln\,((1.1\,(1.2 + \rho \cdot 1.16))/(1.15\,(1.35 + \rho \cdot 1.25))) \tag{3}$$

Considering a variability of the ratio ρ between 0.5 and 2, the parameter σ ranges between 0.13 and 0.15. In the case of slab verification, ρ can be considered approximately equal to 0, resulting in $\sigma = 0.16$. Fig. 1 shows how the reliability index of the structure varies as a function of the ratios R/E and ρ.

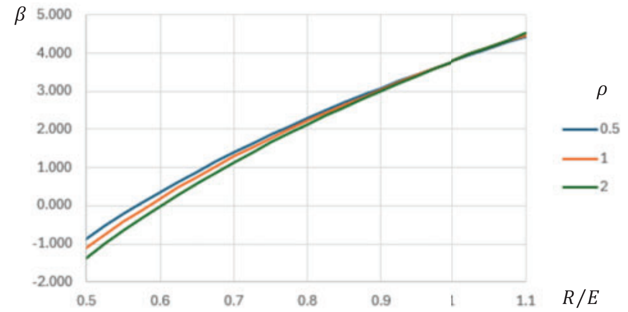

Figure 1. Reliability index as a function of the ratio between resistance and load effect

Recalling that $P = \Phi\,(-\beta)$, the variation in the probability of collapse, considering a reference period of 50 years, as a function of the ratio R/E is shown in Fig. 2.

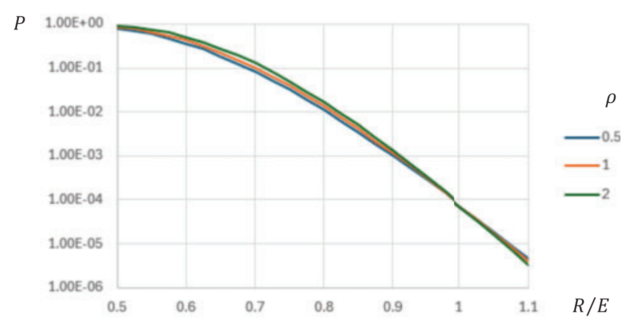

Figure 2. Probability of collapse (50 years) as a function of the ratio between resistance and load effect

The probability over 50 years P is then converted into the probability referring to a single year p using Eq. (4), based on the assumption that all years are equivalent (since only the current state of material degradation has been considered):

$$p = 1 - (1 - P)^{1/50} \tag{4}$$

As can be seen from the graphs and formulas, even a slight reduction in deck resistance leads to a significant increase in the probability of structural collapse, also due to the conservative value assigned to the standard deviation σ. For example, when the ratio R/E decreases from 0.9 to 0.85 (assuming $\rho = 1$), the annual probability of collapse increases from $2.3 \cdot 10^{-5}$ to $8.2 \cdot 10^{-5}$, effectively quadrupling. These values highlight how strongly the assessments presented in this article depend on the accuracy of the structural analyses: even minor approximations or simplifications made "on the safe side" can lead to a significant overestimation of the associated risk.

As an alternative to calculations performed using finite element models, where such models are not available and a rapid assessment is required, the external load effects may also be estimated using simplified methods, for example, the Courbon method. Similarly, without precise knowledge of reinforcement details or actual thicknesses, the structural resistance may be assumed equal to the load effect corresponding to the requirements of the original design standards. However, when proceeding in this way, the standard deviation σ should be adjusted accordingly to account for the increased uncertainty in the calculation. Nevertheless, for the reasons discussed above and since this method is applied only after gaining sufficient knowledge of the bridges and defining the maintenance strategy, it is recommended not to rely on results that have not been obtained through accurate analyses when determining the ratio R/E.

Failure due to seismic action

In the case of failure of one or more piers caused by horizontal external actions, such as earthquakes or wind, many of the considerations already discussed remain valid. However, seismic actions require specific considerations. In particular, the calculation of the standard deviation σ differs because the uncertainty associated with the computational model is much more significant. Moreover, the standard deviation of ground acceleration is not a fixed value but varies depending on the location.

The total standard deviation can be expressed as the contribution of three main factors, as shown in Eq. (5):

$$\sigma = \sqrt{\sigma_R^2 + \sigma_M^2 + \sigma_a^2} \tag{5}$$

where

- σ_R represents the variability of the structural resistance,
- σ_M represents uncertainties related to mathematical modeling, and
- σ_a is the variability of peak ground acceleration.

Compared to the static case, the second and third contributions take on much higher values, making the first contribution almost negligible, which can be generally assumed as $\sigma_R = 0.1$. Italian standards provide ground acceleration values for their territory with $\sigma_a = 0.4 \div 0.6$. If the reliability of structures with respect to seismic loads must be on the order of $\beta = 3.8$, it can consequently be shown[11] that the modeling uncertainty should be $\sigma_M = 0.5 \div 1.0$, a variability that in design practice is represented by the behavior factor q. Using intermediate values $\sigma_a = 0.5$ and $\sigma_M = 0.75$, we obtain $\sigma = 0.9$, which is significantly higher than the value used for traffic loads. Assuming R/E as the result obtained from a seismic vulnerability analysis, that is, the ratio between the collapse acceleration of the structure and the peak ground acceleration, the variation in the probability of collapse, considering a reference period of 50 years, is as shown in Fig. 3.

It is nevertheless clear, similarly to the static case, how significant the role of uncertainties is.

It should be noted that in the static case the load effect E includes not only traffic loads but also permanent loads,

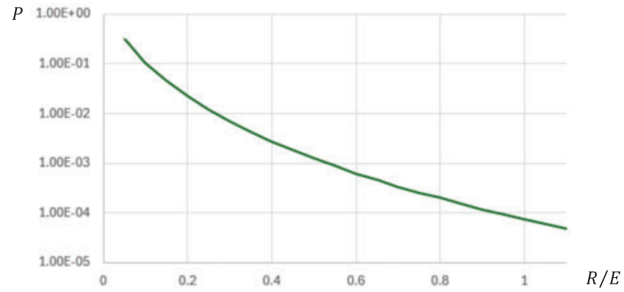

Figure 3. Probability of collapse (50 years) as a function of the ratio between accelerations

whereas in the seismic case the external action is given solely by forces induced by the earthquake, so the trends shown in Figs. 2 and 3 can be considered similar.

Damages

The analysis of damages suffered by society due to possible structural failures of a bridge has also been the subject of a dedicated publication,[14] to which the reader is referred for further details. In this article, only the main concepts are recalled, and above all the practical mathematical steps are presented that allow a credible estimate to be obtained starting from essential data about the structures. It should be noted that, in estimating damages, the aim is not to calculate their exact amount to the nearest unit; given the uncertainties involved, it is sufficient to capture their order of magnitude.

The figures presented in this chapter were calculated for Italy and refer to the year 2016. In particular, the values reported in Tables 1 and 2 may be affected by inflation and, of course, vary from country to country (and potentially also between regions with different economic conditions). For applications in countries other than Italy and in different time periods, it is recommended that the proposed values be adjusted using the international publications from which these data were derived.[15,16] However, for the sole purpose of bridge prioritization, these figures may be used without modification, since the effects of inflation and differing economic conditions are essentially the same for all bridges within a given infrastructure network and therefore do not affect the resulting priority ranking. Moreover, the uncertainties involved are such that corrections of only a few percentage points are unlikely to have a significant impact.

Equation (8), on the other hand, was derived empirically from the analysis of past bridge failure events in Italy. In different social contexts, particularly outside the European Union, where public perception and behavior may differ substantially, the psychological impact and reputational damage may be significantly different from those estimated herein. In such cases, a careful assessment is recommended before applying Eq. (8).

Only the most common cost items are reported here, as some types of damage are so specific that they would require case-by-case evaluation. Typically, when the interruption of the road crossing the collapsed bridge would isolate a town

Table 1. Hourly costs

	Light vehicles	Heavy vehicles
Short distances (<32 km)	13.7 €	56.1 €
Long distances (>32 km)	17.5 €	

Table 2. Congestion costs

	Flow-to-capacity ratio	Motorway	Other roads
Light vehicles	$r > 1.2$	0.223 €	0.452 €
	$1 < r < 1.2$	0.107 €	0.235 €
	$0.8 < r < 1$	0.044 €	0.105 €
Heavy vehicles	$r > 1.2$	0.936 €	1.572 €
	$1 < r < 1.2$	0.452 €	0.815 €
	$0.8 < r < 1$	0.185 €	0.365 €

Table 3. Probability of fatality as a function of height and velocity

v_m [km/h]	0	50	90	110
h [m]	Probability [%]			
2	0	5	63	93
5	1	8	72	95
10	3	18	83	98
15	9	33	90	99
20	18	50	95	100
25	33	66	98	100
30	51	78	99	100
35	66	87	100	100
40	79	94	100	100
45	88	97	100	100
50	94	99	100	100

or even an entire valley (a fortunately rare case), and considering that such an interruption may last several months, a specific study is required since the direct and indirect consequences can be extremely complex to estimate.

It should be noted that it is not necessary to include among the potential consequences those associated with the roadworks required to implement the proposed improvement measure. Since the comparison is made between the scenario in which the intervention is carried out immediately and the scenario in which the same intervention is postponed by 1 year, the construction site is assumed to be essentially identical in both cases. Consequently, any impacts associated with the works, whatever their nature, are the same in both scenarios and therefore do not affect the final result.

Finally, experience gained from analyzing numerous case studies shows that some types of damage, such as reconstruction costs and purely environmental damages, except in very particular cases, are numerically much less significant than the main ones; therefore, they will not be discussed in this article and can generally be neglected.

Estimation of the number of people involved

For obvious reasons, the first and most important evaluation when examining the consequences of a structural failure is the probability that loss of human life may occur in the event of a collapse.

According to the cited publications,[14] two different scenarios must be considered: the first is free-flowing traffic conditions, and the second is a situation in which the entire deck is occupied by stationary vehicles blocked in a queue or traffic jam. The free-flow scenario should be considered when the structural failure event is statistically independent of traffic conditions on the bridge: earthquakes, landslides, or floods do not depend on vehicular traffic and may occur at any time. Therefore, it is assumed that, at the moment the event occurs, traffic is in its most ordinary and typical condition.

Conversely, in the case of failure due to traffic overload, the event occurs (at least theoretically) when the vertical load on the structure is at its maximum. A queue of stationary vehicles, which may occur even on bridges with relatively low traffic volumes, represents the condition with the highest vehicle density and therefore the peak load. Unfortunately, it is also the condition with the greatest number of people present on the bridge.

In the free-flow traffic scenario, it is assumed that a number of vehicles equal to the average daily traffic travels over the bridge at an average speed (a good reference for speed is provided by travel times indicated by navigation systems for the specific road segment). The number of people involved in the collapse is given by Eq. (6):

$$n = ((L + d)/v_m)\,(1.2V_L + V_H)/(24 \cdot 60 \cdot 60) \qquad (6)$$

where L is the length of the portion of the structure involved in the failure, d is the stopping distance, v_m is the travel speed in m/s, V_L is the average daily number of light vehicles crossing the bridge, and V_H is the average daily number of heavy vehicles. Coefficient 1.2 represents the occupancy factor for light vehicles, while for heavy vehicles it is taken as 1. The stopping distance can be calculated as a function of the average speed by assuming a typical deceleration value of -6.5 m/s^2:[17] $d = v_m^2/13$.

To give a concrete example, the average daily traffic on Italian highways typically ranges between 10,000 and 30,000 vehicles per day. If $V_L = 12000$, $V_H = 3000$ and $v_m = 100\,km/h$, the number of people involved would be $n = 0.0073 \cdot L + 0.43$. For a collapsed length of 100 m, this corresponds to approximately 1.2 people, a relatively low number, despite the significant traffic volume and considerable length considered.

The same formula can be used to estimate the number of people involved when traffic passes beneath the structure rather than above it, as in the typical case of a highway overpass. In this case, L would represent the width of the overpass, generally a small value (usually $10 \div 15$ m).

The scenario involving a traffic queue on the bridge is entirely different. In this case, the empirical Eq. (7) is proposed, developed assuming one lane fully occupied by heavy vehicles (responsible for the overload) and the remaining lanes congested with passenger cars:

$$n = L/25 + 1.2\,(n_c - 1)\,L/10 \tag{7}$$

where, as before, L is the length of the portion of the structure involved in the failure and n_c is the number of lanes indicated by road markings. For roads with 1, 2, or 3 lanes, n is, respectively, $0.04\,L$, $0.16\,L$, and $0.28\,L$. It is clear that for large lengths, the number of people involved can become very significant.

Probability of fatalities

Once the most likely number of people involved has been estimated, it is necessary to determine what would happen to them, depending on the type of limit state considered. First, it should be clarified that in this publication the number of injured people is not calculated, since the associated damages can be considered negligible compared to those related to fatalities. Furthermore, in the case of local failures, such as slab collapse with a relatively small spacing between beams, given the limited extent of the damage, the possibility that such events could cause fatalities is excluded.

In general, it is assumed that, in the event of bridge collapse, the vehicles fall together with the structure. Consequently, the vertical impact velocity of the vehicles against the ground beneath is assumed to be equal to that of the bridge deck, neglecting the deceleration effect resulting from the interaction between the vehicles and the collapsing structure. To this vertical component, a horizontal component v_m is added, corresponding to the speed at which the vehicles were traveling at the instant the collapse began, assuming that the drivers had no time to brake.

For simplicity, by applying the free-fall equation in a vacuum, the impact velocity at ground level is given by $v = \sqrt{v_m^2 + 2gh}$, where h is the height of the bridge deck above ground level. For the scenario in which vehicles are stationary in a traffic queue, the same equation is used, with the assumption that $v_m = 0$.

It is then assumed that the consequences of impact are those indicated by a study published by the National Highway Traffic Safety Administration (NHTSA), part of the U.S. Department of Transportation for frontal vehicle collisions.[18] This is also an approximation, since during the fall each vehicle could overturn in various ways, but these results are considered reasonably applicable. See also[19] for other details. Table 3 and Fig. 4 show the probability that the fall is fatal as a function of velocity and height. The probability for the bridge under consideration can be obtained by interpolating the values in the table.

Let us now consider an example of application of the calculation just described. Consider a three-lane motorway viaduct, 20 m above ground level, consisting of three 30-m spans with a grillage of simply supported beams. Assume that the average daily traffic consists of 10,000 light vehicles

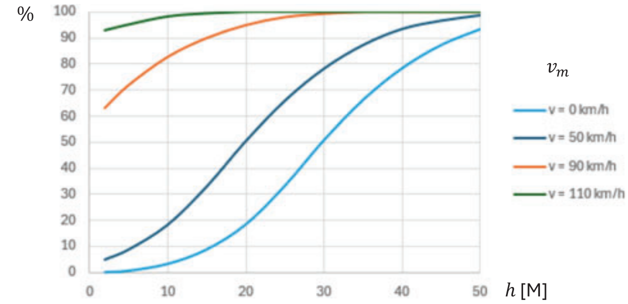

Figure 4. Probability of fatality as a function of height and velocity

and 2,000 heavy vehicles, and that the average speed is 110 km/h.

For the limit state involving failure of a beam, the collapsed bridge length would be 30 m and the number of people involved would be: $n = 30/25 + 1.2\,(3 - 1)\,30/10 = 8.4$. Since stationary queued traffic is considered, there would be an 18% probability of fatality, resulting in a total of: $8.4 \cdot 0.18 = 1.5$ potential fatalities. If the bridge were higher, the probability would increase significantly, leading to a much higher expected number of victims. Conversely, traffic volume has no effect.

For the limit state involving collapse caused by an earthquake, it is assumed that the effect of horizontal actions is critical for both piers and therefore that the collapsed bridge length is the entire structure. Thus, $n = \left((90 + 30.5^2/13)/30.5\right)(1.2 \cdot 10000 + 2000)/(24 \cdot 60 \cdot 60) = 0.86$. Although the probability of fatality is 100%, the expected number of victims remains approximately half that of the static case.

Finally, for the limit state involving slab failure, $n = 0$ is assumed.

Damages due to the presence of people on the bridge

In this section, an economic value is assigned to the loss of human lives. It is important to note that the aim is not to determine the value of human life, but rather to estimate *how much society is willing to pay* to avoid the risk of fatalities.

The total damage is given by the sum of a fixed compensation per victim and a contribution related to the psychological effect. This effect remains relatively limited when the number of victims is small, but increases exponentially when public perception of the bridge collapse shifts from a *tragic accident* to a *disaster*. The proposed empirical formula is given in Eq. (8), also illustrated in Fig. 5, and is based on experience from events that have occurred in Italy in the recent past:

$$\begin{cases} D = 3250000 \cdot n + 1000000 \cdot (3.523^n - 1) & \text{if } n < 5 \\ D = 3250000 \cdot n + 1000000 \cdot 1800 \log(n - 3) & \text{if } n \geq 5 \end{cases} \tag{8}$$

The first part of the equation ($3250000 \cdot n$) was derived from the data published in the "European DG MOVE Handbook on External Costs of Transport".[15] The second part was determined by assigning a value of approximately €2.9

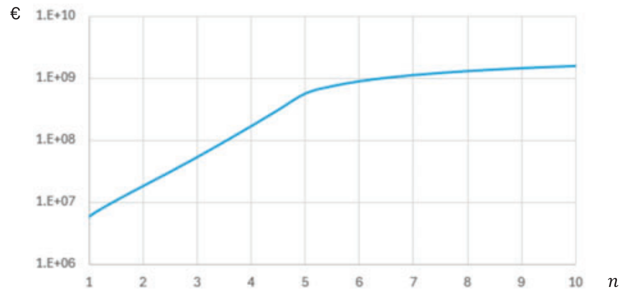

Figure 5. Damages as a function of the number of fatalities

billion to reputational damage—an estimate of the losses incurred by the concessionaire following the 2018 collapse of the Polcevera Viaduct;[20,21]—for $n = 43$, corresponding to the number of fatalities in that accident. Conversely, low values were assigned for $n < 3$, reflecting the observation that in all other bridge failure incidents involving at most two fatalities, no significant reputational damage was observed.

Returning to the previous example, $n = 1.5$ results in slightly more than 10 million euros in damages. From Fig. 5, it is clear that as the number increases, the calculated amount rapidly rises from millions to *billions* of euros, effectively making the postponement of improvement interventions inadvisable.

This effect makes the present method quite similar to what is proposed in Bulletin No. 80 of the FIB,[22] which suggests adopting a minimum reliability index of 2.8 for structures with short spans (less than about 30 meters) based solely on economic criteria, while the index increases rapidly for bridges with longer spans due to considerations related to the protection of human life.

Damages due to service interruption

As highlighted in the previous section, when a potential bridge collapse could result in numerous fatalities, the criterion of safeguarding human life becomes so important that all other considerations become secondary. When the size of the structure is smaller, however, damages caused by the interruption of the road service crossing the bridge also become significant.

Damages due to infrastructure interruption are typically calculated by estimating a daily cost and multiplying it by the duration of the service disruption. With regard to daily costs, the two main components are related to user delays (c_{del}) and traffic congestion (c_{con}), as expressed in Eq. (9):

$$D_{id} = T\,(c_{del} + c_{con}) \tag{9}$$

The time T during which the bridge is expected to remain closed was estimated on the basis of experience gained from past bridge failure events. In particular, it was observed that the total duration can be reasonably approximated as the sum of three distinct components:

- The first part is due to closure imposed by judicial authorities to allow investigations, only in cases where there are doubts about the causes of the collapse. It is proposed to consider 6 months for collapses caused by traffic overload and 0 months for seismic events (as well as hydrogeological events).
- The second part is required for the design of restoration or reconstruction works and for prefabrication of some structural elements. Assuming activities are organized to be completed as quickly as possible, a duration of 1 month is proposed.
- The third part concerns the execution of the works. Assuming maximum speed, the empirical Eq. (10) is proposed, where L is the length of the structure in meters:

$$t_w = 50\,log_{10}L \tag{10}$$

In the previously mentioned example of a motorway bridge 90 m long, the duration (assuming no additional bureaucratic delays) would be $T = 180 + 30 + 98 = 308$ days for traffic-related collapses and $T = 30 + 98 = 128$ days for seismic events.

Turning to the evaluation of daily costs, it is assumed that from the time of collapse until the bridge reopens, users will no longer be able to use the collapsed bridge and will have to take an alternative route, resulting in increased travel times and higher traffic on the alternative road. The proposed calculation approach considers these two effects separately; therefore, the increase in travel time is calculated independently of traffic congestion, to avoid double counting.

Drawing on the experience gained in the management of road infrastructure, we hypothesize that, when a road is unexpectedly closed for a short period, most drivers who would normally use it will follow the alternative route suggested by their navigation systems and will choose the same secondary road. For simplicity, it is also assumed that users already on the secondary road do not change their behavior and continue using the same route. This is particularly likely when the road crosses hilly or mountainous terrain, where alternative routes are very limited. However, even in flat areas, the road network is rarely dense enough to provide multiple substantially equivalent routes from which users can choose. As a result, traffic on the selected secondary road increases (being the sum of normal traffic and diverted traffic), while the impact on all other roads is negligible.

The damage due to increased travel time is calculated by summing the contributions of light and heavy vehicles, as shown in Eq. (11):

$$c_{del} = (D_L V_L + D_H V_H)\,(T - T_0) \tag{11}$$

where

- D_L is the hourly cost for a light vehicle (in euros),
- D_H is the hourly cost for a heavy vehicle (in euros),
- V_L is the average daily number of light vehicles crossing the bridge,
- V_H is the average daily number of heavy vehicles crossing the bridge,
- T is the travel time on the original route (before the collapse) (in hours), and
- T_0 is the travel time on the alternative route (before the collapse) (in hours).

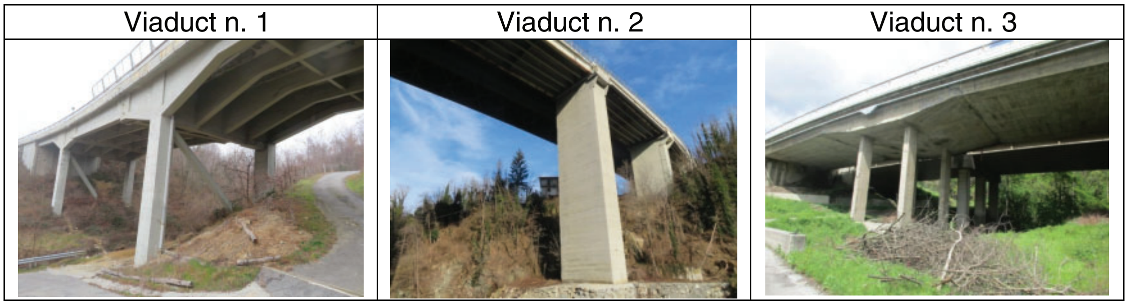

Figure 6. Case studies

Table 4. Congestion costs

				Bridge No. 1	Bridge No. 2	Bridge No. 3
Bridge characteristics						
	Deck length	L_D	[m]	56	508	53
	Maximum height above ground	H	[m]	16	45	16
Road section characteristics						
	Average daily traffic—light vehicles	V_L	[vehicles/day]	6340	8209	8028
	Average daily traffic—heavy vehicles	V_H	[vehicles/day]	1557	1848	1959
	Travel time (without interruption)	T_0	[min]	10	7	16
	Section length		[km]	13	11	17
	Average travel speed	v_m	[km/h]	78	94	64
Alternative route characteristics						
	Average daily traffic—light vehicles	V_{LS}	[vehicles/day]	4000	4000	6500
	Average daily traffic—heavy vehicles	V_{HS}	[vehicles/day]	500	500	550
	Travel time (without congestion)	T	[min]	18	14	24
	Section length	L_{AR}	[km]	13	12	16
	Traffic conditions during bridge closure	r		$> 1.0, < 1.2$	$> 1.0, < 1.2$	$r > 1.2$
	$c_{del} = (17.5 V_L + 56.1 V_H)(T - T_0)/(60 \cdot 1000)$					
	Delay cost	c_{del}	[k€/day]	26	29	33
	$\begin{cases} c_{con} = L_{AR}(0.235(V_L + V_{LS}) + 0.815(V_H + V_{HS}))/1000 & \text{if } 1 < r < 1.2 \\ c_{con} = L_{AR}(0.452(V_L + V_{LS}) + 1.572(V_H + V_{HS}))/1000 & \text{if } r > 1.2 \end{cases}$					
	Congestion cost	c_{con}	[k€/day]	53	57	168
Scenario No. 1: Deck collapse						
	Safety factor	R/E		0.51	0.83	0.69
	Ratio between loads	ρ		1.0	1.7	0.9
	$\beta = 3.8 - \ln(R/E)/\ln((1.1(1.2 + \rho \cdot 1.16))/(1.15(1.35 + \rho \cdot 1.25)))$					
	Reliability index	β		-1.00	2.41	1.19
	$p = 1 - (1 - \Phi(-\beta))^{1/50}$					

Table 4. (Continued)

				Bridge No. 1	Bridge No. 2	Bridge No. 3
	Annual probability of collapse	p		$3.6 \cdot 10^{-2}$	$1.6 \cdot 10^{-4}$	$2.5 \cdot 10^{-3}$
	Length of collapsed deck section	L	[m]	35	35	18
	Total number of lanes (excluding emergency lanes)	n_c		2	2	2
		$n = L/25 + 1.2\,(n_c - 1)\,L/10$				
	Number of people involved	n		5.60	5.54	2.94
	Estimated most probable number of fatalities (probability taken from Table 3)	n_f		0.58	4.85	0.31
		$\begin{cases} D = 3250 \cdot n_f + 1000 \cdot \left(3.523^{n_f} - 1\right) & if\ n < 5 \\ D = 3250 \cdot n_f + 1000 \cdot 1800 \log\left(n_f - 3\right) & if\ n \geq 5 \end{cases}$				
	Damages due to fatalities (including reputational damages)	D_f	[k€]	2900	463760	1010
		$T_{SI} = 180 + 30 + 50\, log_{10} L_D$				
	Duration of service interruption	T_{SI}	[days]	297	345	296
		$D_{id} = T_{SI}\,(c_{del} + c_{con})$				
	Service interruption cost	D_{id}	[k€]	23710	29757	59665
		$C_1 = \left(D_f + D_{id}\right) p$				
	Partial cost of postponement	C_1	[k€]	958	79	152
Scenario No. 2: Deck slab collapse						
	Safety factor	R/E		1.10	1.01	0.96
	Ratio between loads	ρ		0.10	0.10	0.12
		$\beta = 3.8 - \ln(R/E)/\ln((1.1\,(1.2 + \rho \cdot 1.16))/(1.15\,(1.35 + \rho \cdot 1.25)))$				
	Reliability index	β		4.40	3.87	3.55
		$p = 1 - (1 - \Phi(-\beta))^{1/50}$				
	Annual probability of collapse	p		1.1E-07	1.1E-06	3.9E-06
		$T_{SI} = 180 + 30 + 60$				
	Duration of service interruption	T_{SI}	[days]	270	270	270
		$D_{id} = T_{SI}\,(c_{del} + c_{con})$				
	Service interruption cost	D_{id}	[k€]	21554	23288	54424
		$C_2 = D_{id} p$				
	Partial cost of postponement	C_2	[k€]	0	0	0
Scenario No. 3: Piers collapse due to earthquake						
	PGA acceleration ratio	IR		0.53	1.32	1.14
		σ_a		0.44	0.44	0.44

Table 4. (Continued)

				Bridge No. 1	Bridge No. 2	Bridge No. 3
		$\beta = 3.8 - \ln(IR)/\sqrt{0.1^2 + \sigma_a^2 + (1.5\sigma_a)^2}$				
	Reliability index	β		3.00	4.15	3.96
		$p = 1 - (1 - \Phi(-\beta))^{1/50}$				
	Annual probability of collapse	p		2.7E-05	3.5E-07	7.7E-07
		$n = \left(\left(L_D + \left(v_m^2/13\right)\right)/v_m\right)(1.2V_L + V_H)/(24 \cdot 60 \cdot 60)$				
	Number of people involved	n		0.45	2.90	0.58
	Estimated most probable number of fatalities (probability taken from Table 3)	n_f		0.36	2.90	0.34
		$D = 3250 \cdot n_f$				
	Damages due to fatalities (including reputational damages)	D_f	[k€]	1160	9410	1100
		$T_{SI} = 30 + 50\,log_{10}L_D$				
	Duration of service interruption	T_{SI}	[days]	117	165	116
		$D_{id} = T_{SI}(c_{del} + c_{con})$				
	Service interruption cost	D_{id}	[k€]	9340	14231	23382
		$C_3 = \left(D_f + D_{id}\right)p$				
	Partial cost of postponement	C_3	[k€]	0	0	0
Total cost of postponement			**[k€]**	**958**	**79**	**152**

Using data available in the literature,[15,16] and assuming a traffic composition of 80% commuting/business and 20% personal travel, the proposed hourly costs are those shown in Table 1. Long-distance values should be used for motorway bridges, while short-distance values are more appropriate for local road bridges.

Travel times can be easily assessed using any navigation system by measuring the shortest travel time on the infrastructure under consideration and then forcing the route through the alternative secondary road.

On the selected secondary road, traffic volume increases, and therefore the speed of all vehicles decreases. In addition to the costs due to the longer route, the damage caused by the reduction in speed for all users must also be considered. The proposed formula, similar to the previous one, is Eq. (12):

$$c_{con} = L\left(C_L\left(V_L + V_{LS}\right) + C_H\left(V_H + V_{HS}\right)\right) \qquad (12)$$

where

- L is the length of the congested road section (in km),
- C_L is the cost for a light vehicle (in euros),
- C_H is the cost for a heavy vehicle (in euros),
- V_{LS} is the average daily number of light vehicles normally using the alternative road, and
- V_{HS} is the average daily number of heavy vehicles normally using the alternative road.

For the calculation, the appropriate values should be selected from Table 2.

As an example, consider again the 90-m bridge. Assume that the alternative road onto which motorway traffic would be diverted is a 10-km secondary rural road, $V_{LS} = 3000$ and $V_{HS} = 300$, so that the resulting traffic would be more than congested. Also assume that travel times increase from 6 minutes to 15 minutes (the travel time on the uncongested secondary road under ordinary conditions). The resulting values would be: $c_{del} = (17.5 \cdot 10000 + 56.1 \cdot 2000)(15 - 6)/60 = 43080€$ and $c_{con} = 10(0.452(10000 + 3000) + 1.572(2000 + 300)) = 94916€$. Summing these costs, which are obtained under entirely ordinary conditions, and multiplying them by the reconstruction duration of 308 days, the total damage exceeds 42 million euros. It is therefore clear that the reconstruction cost is, overall, negligible compared with the cost of service interruption.

Case Studies

The practical application of the method was tested through the classification of interventions on a significant number of real bridges. For the purposes of this paper, only the calculations performed for three bridges are presented for brevity shown in Fig. 6, as they are considered sufficiently representative.

All structures carry the same motorway and were built in the 1960s. Following the indications of the Italian guidelines, they were selected for rehabilitation interventions.

The first viaduct is a three-span reinforced concrete beam grillage, with the central deck section supported by Gerber hinges resting on cantilever brackets. The second is a larger bridge, both in terms of length and height, consisting of a grillage of post-tensioned prestressed concrete beams simply supported on the piers. The third bridge has only two spans and consists of a reinforced concrete box girder supported through Gerber hinges on the cap beam of the central pier.

Table 4 reports all calculation steps. The cells highlighted in gray contain the input data, whereas the remaining cells correspond to the calculations described in the previous sections. Given the context in which the bridges are located, environmental damages and damages to any underlying infrastructure were neglected.

For these three example bridges, it is evident that the parameter that most influenced the final ranking was not the exposure to risk, but rather the outcome of the structural assessment. In fact, the intervention on Bridge No. 2 is ranked as the least urgent, even though the consequences of a structural failure would be significantly greater than for the other two bridges. It can be observed that, if for this bridge the R/E ratio, equal to 0.83, were hypothetically 0.72 or lower, priority should instead be assigned to this structure.

It can also be seen that, in this case, the ranking was determined exclusively by the deck performance under traffic loads, whereas resistance to concentrated loads and seismic actions had no influence on the final classification.

Remarks

The estimation of the cost of postponement for each structure, obtained as described in the previous sections, makes it possible to establish a priority ranking of already planned improvement interventions, from the most urgent to those that can reasonably be deferred because the associated risk is sufficiently low.

It should always be remembered that the resulting monetary value has a purely conventional meaning. In other words, the estimated cost can be used in a relative sense to compare bridges with one another, but it cannot be used in an absolute sense to determine the actual cost to society of postponing an intervention. This issue arises not only from the uncertainties involved in estimating the consequences of bridge failure—particularly those related to psychological and social impacts—but, above all, from the choice of the design standards used for the structural assessment. Indeed, the primary requirement of a structural assessment report is that it be defensible, that is, capable of withstanding scrutiny by judicial or administrative authorities should the correctness of the engineer's work be challenged. For this reason, no engineer can disregard compliance with the applicable national regulations, even when less conservative assessment methods may be available in the scientific literature. The Eurocodes, as well as the national technical standards derived from them, were originally developed for the design of new structures rather than for the assessment of existing ones. Their application therefore tends to systematically underestimate the actual performance of existing bridges, as these standards incorporate numerous simplifications and assumptions made "on the safe side." As a consequence, the calculated probabilities of limit-state exceedance are generally much higher than the actual probabilities, and the associated estimated costs are correspondingly inflated.

As with any prioritization process, there is no absolute safety threshold that certifies risk acceptability; therefore, this method cannot be used to justify the decision not to carry out an intervention.

However, the proposed method has the advantage of not requiring projection of structural resistance into the future and therefore does not depend on knowledge of the evolution of material degradation over time. In fact, most bridges older than 50 years are periodically subject to maintenance interventions (such as concrete cover repairs, application of carbon fiber reinforcements, external prestressing, etc.), the long-term durability of which has not been adequately studied. For this reason, degradation models available in the literature, which typically consider only the original materials, are rarely applicable. By focusing solely on the probability of collapse over a 1-year period, the present method avoids these uncertainties.

The proposed procedure requires the collection of only a few essential data and a limited number of simple calculation steps. It enables sufficiently reliable comparisons between bridges, highlighting the characteristics that most strongly influence risk and those that can reasonably be neglected.

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