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Mechanics-based Fragility Assessment of Existing Prestressed Concrete Bridges Under Traffic Loads

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ABSTRACT: After catastrophic collapse of several major bridges in Italy, the safety evaluation and retrofit interventions on existing bridges became a top priority for the engineering community, road management companies and government. This promoted the publication of Italian guidelines (GL) for risk classification, structural assessment, and structural health monitoring of existing roadway bridges. In this regard, existing bridges that do not meet the performance objectives of the Italian building code (NTC) can be checked under GL-conforming traffic load models (TLMs). This study deals with a class of simply supported, beam-type, prestressed concrete bridge decks built between 1970 and 1980 in Italy. The traffic-load fragility was evaluated by implementing a Monte Carlo sampling technique to generate simplified deck models, which can be used to assess large bridge portfolios. Based on actual data, geometric and material properties, loads, and capacity model error were modelled as random variables. Fragility analysis was carried out under different TLMs provided by GL and NTC for comparative purposes. Those TLMs were combined with bridge usage limitations, such as reduced distance of external load lane from the kerb or reduced number of lanes, to support decision-making by road management companies. In the end, a European weigh-in-motion database was used to evaluate the annual failure probability of selected bridges. Analysis results show that the load pattern plays a key role in fragility estimation, underling that realistic vehicles models should be developed to achieve a target safety level conforming to current structural codes.

1. INTRODUCTION

Statistics on the construction age of existing bridges show that a huge number of such facilities are approaching their design service life, needing specific maintenance. First-order reliability methods were recently used for safety evaluation of existing bridges (Allaix et al., 2016). Additionally, full-scale experimental tests have been recently carried out on prestressed concrete (PC) beams, which are known to be among the most vulnerable elements in existing bridges (Huber et al., 2018). In 2020, the Italian High Council of Public Works (CSLLPP) issued novel technical Guidelines (CSLLPP, 2020), referred to as GL hereafter, for classification, safety checks, and structural health monitoring of existing bridges in response to recent cases of bridge collapse due to vehicle overloads, collisions, floods, landslides, and lack of maintenance (di Prisco, 2019; Nuti et al., 2020).

GL proposed new traffic load models (GL-TLMs) according to vehicle classes defined by the Italian road code (MIT, 1992), differently from those commonly adopted for design of new bridges (CEN, 2003; MIT, 2018). Depending on the traffic control system, GL defined different values of partial safety factors for traffic loads. Cosenza and Losanno (2021) presented the multi-level approach, GL-TLMs and safety checks of existing bridges according to GL.

Miluccio et al. (2021) found that simply supported, beam-type, PC bridges constructed before 1980 are more prone to bending rather than shear failure mechanism. This finding can be considered for prioritization of retrofit interventions. In this study, fragility analysis for the same class of bridges under various TLMs is presented, showing the effects of GL provisions and usage limitations. Under various load patterns, edge girders' structural capacity and strength demand for the ultimate limit state (ULS) were assessed. Using Monte Carlo simulation (MCS) in MATLAB (The MathWorks Inc., 2022), fragility was assessed as the conditional probability of exceeding the ULS given the load pattern. Last section presents a preliminary assessment of the unconditional collapse failure probability.

2. ITALIAN CODE PROVISIONS FOR EXISTING BRIDGES

In case an existing bridge does not meet safety requirements for new structures (MIT, 2018) limitations in usage (e.g., a reduced number of lanes or a different lane position) and/or gross vehicle weight (*GVW*) can be implemented in safety checks. Since GL-TLMs were not derived via stochastic analysis of road traffic, each TLM was deterministically set based on the Italian road code classification (MIT, 1992).

Four different TLMs are provided by GL to be representative of different vehicle classes: (i) heavy, (ii) medium, (iii) light and (iv) ultralight. In this study, the heaviest vehicle class (*GVW* = 440 kN) is considered and the corresponding model is defined as heavy GL-TLM.

In addition to different tandem loads, distributed loads in GL-TLMs may only be applied outside the silhouette of concentrated loads in contrast to NTC-conforming TLMs (NTC-TLMs hereafter). Further details on GL- and NTC-TLM load pattern definitions can be found in Cosenza and Losanno (2021).

3. METHODOLOGY OF THE STUDY

According to a single-component reliability-based safety assessment, fragility analysis was performed based on the modelling of uncertainties about loads and the structural capacity of the deck to determine the conditional probability of exceeding ULS in the edge girder. The methodology for fragility analysis was based on the following steps (Figure 1):

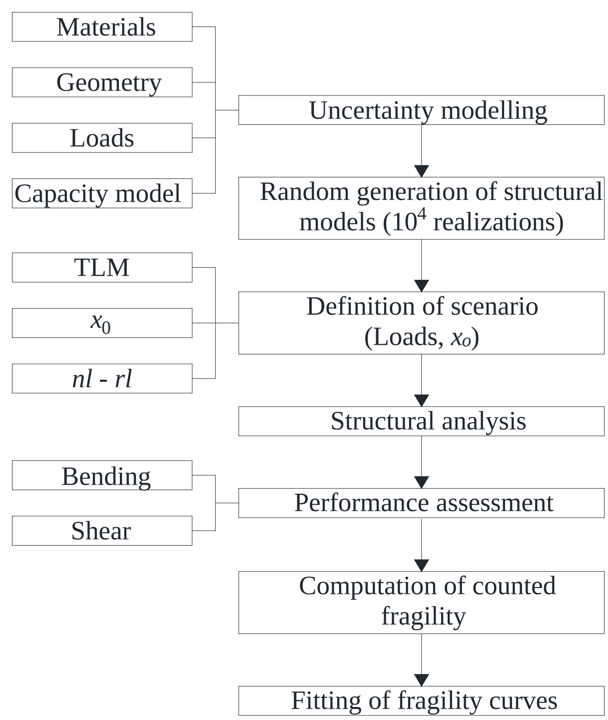

Figure 1: Flowchart of traffic-load fragility analysis of PC girder decks.

- Uncertainty modelling related to geometric and material properties, permanent loads, and capacity model, which was based on both discrete and continuous probability distributions for random variables (RVs) as well as regression models.

- The longitudinal PC bridge deck girder flexural nd shear strength determined using code-conforming capacity models.
- Random generation of structural deck models via MCS.
- Selection of TLM and usage limitation.
- Structural analysis of each deck realization under increasing traffic load intensity, in accordance with either NTC or GL provisions (CSLLPP, 2020), in the latter case with or without usage limitations.

3.1. Uncertainty modelling

This study adopted probability distributions and regression models developed in Miluccio et al. (2021) to represent geometric and material properties as RVs. These latter variables were defined through either probability density functions (PDFs) or probability mass functions (PMFs). Regression models were also developed and implemented to account for statistical dependency of some properties upon RVs. For the sake of brevity, Table 1 shows the probability distributions only.

Table 1. Distributions and statistics of random variables

Category	RV	μ	CoV [%]	Distribution
Materials	f_c [MPa]	38.50	11.40	Lognormal
	f_y [MPa]	451.00	7.20	Lognormal
	$f_{p,01}$ [MPa]	1665.00	2.50	Lognormal
Geometry	L [m]	33.20	13.60	Lognormal
	W [m]	8.50, 12.25, 16.00	–	PMF
	s [m]	0.25	12.00	Uniform
	ρ_{sw} [mm^2/m]	715.00	34.00	Uniform
Loads	$\sigma_{sp}/f_{p,01}$ [%]	50.00	12.00	Uniform
	γ_c [kN/m^3]	25.00	5.00	Normal
	g_{2k} [kN/m^2]	2.00	10.00	Normal

Interested readers can refer to the paper by Miluccio et al. (2021) for further information on regression models. It should be noted that such models were effectively used to generate deck models. Figure 2a and Figure 2b show the cross sections of the deck and longitudinal girders, respectively. Model uncertainty related to capacity models was duly considered (Section 4.2).

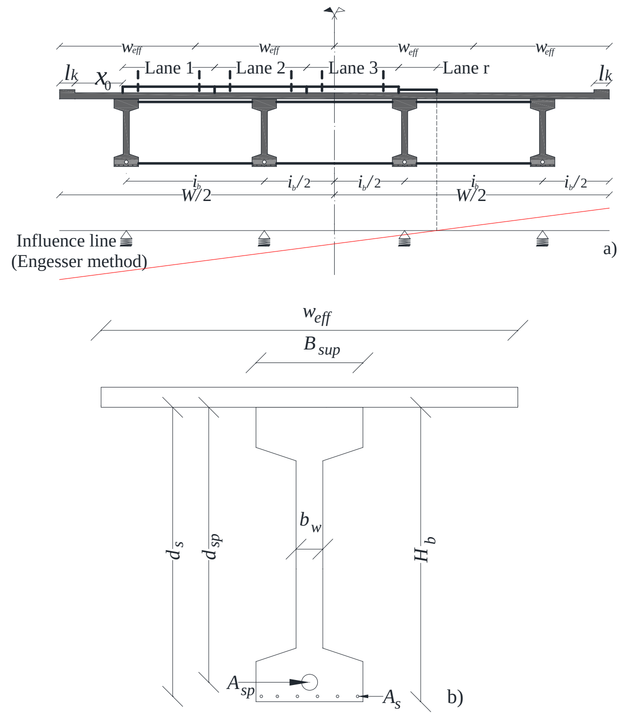

Figure 2: (a) Cross section of typical PC bridge deck with traffic load lanes (first lane at x_0 from kerb) and (b) longitudinal girder cross section.

3.2. Capacity modelling

The flexural and shear capacity models suggested by current codes for design of reinforced concrete (RC) structures (CEN, 2004; MIT, 2018) were adopted to assess the ULS capacity of the edge girder. The flexural capacity model M_r is defined in Eq. (1) derived by taking into account both the prestressing steel area A_{sp} and the reinforcing steel area A_s to be lumped in their centroid with internal lever arm d^*:

$$M_r = \left(A_{sp} f_{p,01} + A_s f_y\right) d^* \qquad (1)$$

Shear capacity for an existing PC bridge girder (i.e., V_r) can be calculated in its uncracked configuration due to limited transverse reinforcement and allowable shear stress design criteria at the time of construction, provided that pre-existing shear cracks due to, for example, construction defects and/or damage, are lacking (Eq. (2)):

$$V_r = 0.7 b_w d \sqrt{f_{ct}^2 + \sigma_{cp} f_{ct}} \qquad (2)$$

where: b_w is the girder web width; d is the distance of reinforcing steel rebar from the top compression side; f_{ct} is the concrete tensile strength; and σ_{cp} is the average compressive concrete stress due to the residual prestressing action σ_{sp}. Previous sensitivity analysis showed that the shear capacity truss model would underestimate the actual capacity due to limited amount of transverse reinforcement (Miluccio et al., 2021).

3.3. Structural analysis procedure

To automatically run a large number of analysis cases through a computationally efficient method while taking into account a common design practice at the time of construction, a rigid deck cross-section was assumed, and the strength demand on the edge girder under traffic loads was assessed using the Courbon-Engesser formulation (Raithel, 1977). Following a random sampling of a set of bridge deck models, either NTC-TLM or GL-TLM traffic loads were applied to determine the strength demand on the edge girder. A clear distance from the kerb (denoted as x_0 below) was used to accommodate for a geometric limitation on the position of the first notional lane (Figure 2a).

Based on geometric statistics of the modelled bridges, the maximum value of n_l would be equal to 3. In order to assess its impact on structural fragility, the residual load area was always considered by a factor r_l to take into account its usage (r_l = 1). After the TLM, n_l, x_0, and r_l were selected, the bridge model was automatically created and analysed under the traffic load pattern associated with the greatest demand on the edge girder.

4. FRAGILITY ANALYSIS OF SELECTED BRIDGE DECKS

4.1. Fragility analysis procedure

MCS was used to obtain 10^4 random realisations of a real vector $\boldsymbol{\theta}$ that includes the selected RVs. The ratio α between the incremental and design values of first-lane tandem load was used as intensity measure (IM), which ranged from 0 to 5 with a step of 0.01. With reference to the edge girder, the demand-to-capacity ratio (*DCR*) was used to assess the performance of each sample deck.

According to earlier studies (Castaldo et al., 2018b), the fragility assessment in a nonlinear structural analysis may be impacted by convergence problems and instability events. However, the methodology adopted in this study is based on a linear deck analysis, enabling fragility to be calculated as follows (Eq. 3):

$$P\left[DCR \geq 1 \mid IM = im\right] = \frac{\sum_{i=1}^{N_{sim}} I_{DCR|IM}\left(\boldsymbol{\theta}\right)}{N_{sim}} \qquad (3)$$

where *im* is a given IM value and *DCR* = 1 is defined as limit state function (Miluccio et al., 2021). Fragility is defined as the conditional probability of exceeding ULS given IM. The right-hand side of Eq. (3) is equal to the expected value of $I_{DCR|IM}\left(\boldsymbol{\theta}\right)$, namely the ratio between the summation of all values of $I_{DCR|IM}\left(\boldsymbol{\theta}\right)$ over the number of simulations $N_{sim} = 10^4$. $I_{DCR|IM}\left(\boldsymbol{\theta}\right)$ is a Bernoulli-type variable that is used to count the attainment of failure and is defined as follows (Eq. (4)):

$$\begin{aligned} I_{DCR|IM}\left(\boldsymbol{\theta}\right) &= 0 \qquad \text{if } DCR < 1 \\ I_{DCR|IM}\left(\boldsymbol{\theta}\right) &= 1 \qquad \text{if } DCR \geq 1 \end{aligned} \qquad (4)$$

Then, the so-called counted (or empirical) fragility values corresponding to each given IM level were fitted through a lognormal distribution.

4.2. Discussion of results

This section presents the ULS fragility curves. The IM (i.e., α) is represented on the bottom *x*-axis of each fragility plot, while the top *x*-axis shows the equivalent first-lane tandem load $Q_1 = \alpha\, Q_{1d}$. If α is set to 1, the design traffic load of the chosen TLM is Q_{1d} = 440 kN for heavy GL-TLM and Q_{1d} = 600 kN for NTC-TLM. This study aims at assessing how the following parameters affect traffic-load fragility: (i) TLM, (ii) limitation of transverse eccentricity, i.e., $0 \le x_0 \le 2.0$ m, (iii) number of lanes, i.e., $1 \le n_l \le 3$. Because shear failure mechanism results in negligible impact on deck fragility under traffic loads (Miluccio et al., 2021), flexural capacity model uncertainties have been taken into consideration while ignoring shear capacity uncertainties. The model error was defined as the difference between the experimental and theoretical strength values for each specimen (Castaldo et al., 2018a), namely $\theta = M_{r,exp}/M_{r,th}$, in accordance with experimental data from the literature (Elsharkawy et al., 2013; Harajli, 1990; Hussien et al., 2012; Vu et al., 2010; Zhang et al., 2017). As suggested in a previous study (Castaldo et al., 2018a), a lognormal probability distribution of θ was derived and characterised by a mean value μ_θ= 0.99 and a logarithmic standard deviation σ_θ = 0.15, or alternatively, a coefficient of variation CoV = 15%. The logarithmic standard deviation (or dispersion) of each fragility curve was thus estimated as follows:

$$\beta = \sqrt{\beta_{al}^2 + \beta_{ep}^2} \tag{5}$$

where β_{al} is the dispersion associated with aleatory uncertainties (representative of the inherent variability of geometric and material properties, as well as loads) and β_{ep} is the dispersion associated with epistemic uncertainties that turns out to be the standard deviation of the model error θ, hence resulting in $\beta_{ep} = \sigma_\theta$.

Assuming the maximum number of load lanes (i.e., the value of n_l that maximises strength demand), Figure 3 shows the fragility curves under varying x_0. If x_0 = 0, a NTC-conforming load distribution is obtained in the case of NTC-TLM. When the maximum eccentricity of the first load lane is restricted, a considerable decrease in traffic-load fragility may be observed. Due to the assumption of equal traffic loads on separate lanes when GL-TLM is adopted (Figure 3b), the rate of fragility reduction under increasing value of x_0 is larger than obtained under NTC-TLM. Fragility curves for heavy GL-TLM result in a lower conditional probability of failure than those for NTC-TLM due to (i) a lower tandem load (440 kN in contrast to 600 kN) and (ii) a different distribution of tandems (i.e., 5 axles over 11 m instead of 2 axles with 1.2 m spacing).

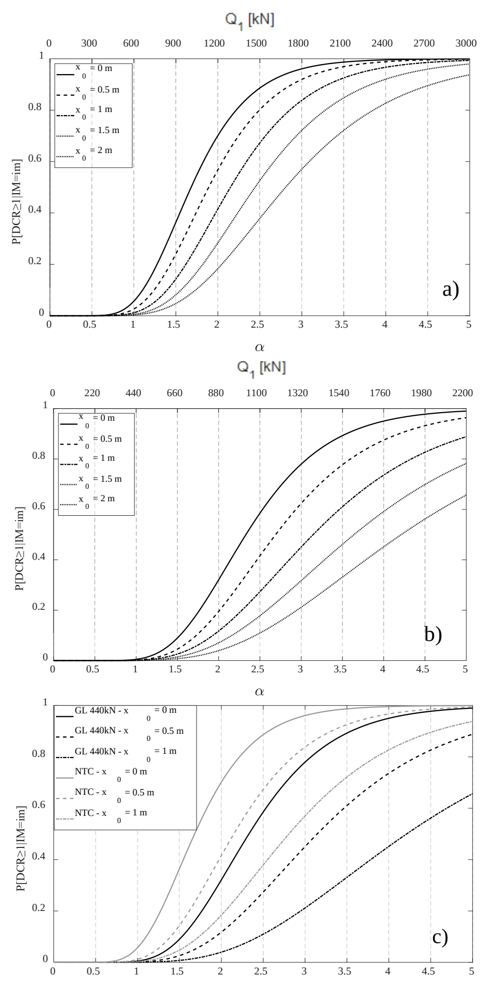

Figure 3. Fragility curves corresponding to the maximum number of load lanes: (a) NTC-TLM; (b) GL-TLM; (c) comparison between fragility curves under $0 \le x_0 \le 1$ m.

Table 2 outlines the fragility parameters (i.e., median value μ and dispersion σ) and fragility levels corresponding to the design traffic loads (therein abbreviated as $P[C|\alpha = 1]$), given the TLM under varying x_0. $P[C|\alpha = 1] = 5.7 \cdot 10^{-2}$ was calculated for NTC-compliant load pattern (i.e., NTC-TLM and $x_0 = 0$), whereas heavy GL-TLM reduces fragility to $4.9 \cdot 10^{-3}$ (i.e., approximately 10 times lower). If x_0 is increased from 0 to 1.0 m, $P[C|\alpha = 1]$ decreases to $1.2 \cdot 10^{-2}$ and $1.3 \cdot 10^{-3}$ under NTC-TLM and GL-TLM, respectively. This demonstrates how decisions regarding carriageway limitations can immediately result in major advantages in terms of traffic-load fragility.

Table 2. Fragility parameters, coefficient of determination and code-related fragility corresponding to NTC-TLM and GL-TLM under varying x_0.

Provisions	*x_0 [m]*	*n_l*		*β*	*$P[C\|\alpha = 1]$*
NTC	*0*	*3*	*1.68*	*0.33*	*$5.7 \cdot 10^{-2}$*
	0.5	*3*	*1.89*	*0.33*	*$2.6 \cdot 10^{-2}$*
	1.0	*3*	*2.15*	*0.34*	*$1.2 \cdot 10^{-2}$*
	1.5	*3*	*2.44*	*0.35*	*$5.3 \cdot 10^{-3}$*
	2.0	*3*	*2.81*	*0.38*	*$3.0 \cdot 10^{-3}$*
GL	*0*	*3*	*2.33*	*0.33*	*$4.9 \cdot 10^{-3}$*
	0.5	*3*	*2.69*	*0.35*	*$2.1 \cdot 10^{-3}$*
	1.0	*3*	*3.15*	*0.38*	*$1.3 \cdot 10^{-3}$*
	1.5	*3*	*3.64*	*0.41*	*$8.0 \cdot 10^{-4}$*
	2.0	*3*	*4.21*	*0.42*	*$3.0 \cdot 10^{-4}$*

The ratio between values of both $P[C|\alpha = 1]$ and η related to NTC- and GL-TLMs decreases as x_0 increases. The coefficient of determination for each fragility curve in this study is extremely close to unity ($R^2 = 0.99$), indicating that the lognormal distribution function fits very well each set of fragility points.

5. REMARKS ON FAILURE PROBABILITY UNDER TRAFFIC LOADS

The fragility (i.e., $P[C|H]$) and traffic-related hazard (i.e., $P[H]$) can be multiplied to determine the failure probability conditioned on code-based TLM (i.e., $P[C]_{TLM}$) as follows:

$$P[C]_{TLM} = P[C|H]_{TLM} \cdot P[H] \qquad (6)$$

Although the proposed work focuses on $P[C|H]$, $P[H]$ would need a probabilistic model of traffic loads on existing bridges. NTC-TLM is defined as a characteristic value with 5% chance of being exceeded in 50 years and hence a return period of 1000 years (CEN, 2003). Therefore, $P[H]$ may be calculated as the ratio between 0.05 and 50 years in order to determine the mean annual rate of recurrence of the NTC-TLM.

A value of $P[C]_{NTC} = 5.7 \cdot 10^{-5}$ was obtained considering fragility analysis under NTC-TLM, which would meet the upper bound to the failure probability range expected for design of new bridges, i.e., between 10^{-7} and 10^{-5} depending on the consequence class (CEN 2006). Since no special derailment devices were used for illegal vehicles (i.e., GVW > 440 kN), the cumulative frequencies obtained by a previous weigh-in-motion (WIM) system and reported in Maljaars (2020) are taken into consideration as representative of unconditional traffic load composition of a European motorway. The information was processed from a one-month observation in 2018. The monthly frequency may be used as the yearly frequency assuming that a stationary stochastic model accurately describes traffic loads. Maljaars found out a cumulative frequency of GVW exceeding 440 kN, i.e., approximately one vehicle out of ten, under a conservative assumption of equal distribution of traffic vehicles on fast and slow lanes. With $n_l = 3$ and $x_0 = 0$, the heavy GL-TLM conditional failure probability $P[C|H]$ was calculated in Section 4.2 to be equal to $4.9 \cdot 10^{-3}$, yielding a collapse probability $P[C]_{GL,440kN} = 4.4 \cdot 10^{-4}$. The outcome $P[C]_{GL,440kN} > P[C]_{NTC}$ indicates that the assumption of equal traffic load distribution across multiple lanes may be overly conservative. An additional term for traffic hazard conditioned on congestion state can be introduced as GL-TLMs are meant to be representative of congested traffic with no space between vehicles (i.e., a very low distance between the rear and front axles of two consecutive vehicles). $P[H]$ is the probability that the gross vehicle weight (GVW) is greater than the threshold value of 440 kN (e.g.,

$P[GVW > \overline{GVW}]$), given that the distance d between two heavy vehicles is less than $\overline{d}$ = 16 m (Eq. 7).

$$P\left[H\right]=P\left[GVW>\overline{GVW}|d<\overline{d}\right]P\left[d<\overline{d}\right] \quad (7)$$

If $P\left[d < \overline{d}\right]$ = $1 \cdot 10^{-2}$ is assumed according to Maljaars (2020), $P[C]_{\mathrm{TLM}}$ turns out to be $4.4 \cdot 10^{-6}$.

The convolution of fragility (i.e., $P[C|H]$) and traffic-related hazard (i.e., $P[H]$) may be used to determine the unconditional collapse failure probability $P[C]$ as follows:

$$P\left[C\right]=\sum_{i} P[C|\alpha=\alpha_i,TLM]\cdot P[\alpha=\alpha_i|TLM] \quad (8)$$

where α is the IM described above and TLM is the conventional load model assigned to traffic.

The collapse probability is determined to be $P[C] = 5.8 \cdot 10^{-4}$ based on the fragility curves generated under heavy GL-TLM with n_l = 3 and x_0 = 0 (i.e., in accordance with the assumption of uniform load pattern on bridge deck). Additionally, the above-described implementation of the crowded traffic hypothesis results in $P[C] = 5.8 \cdot 10^{-6}$. This shows that, under actual traffic loads, the annual rate of collapse associated with GVW = 440 kN ranges between 10^{-6} and 10^{-4}, including the value $P[C]_{NTC}$ = $5.7 \cdot 10^{-5}$. This section shows after selecting the allowable vehicle classes, provided fragility curves can help road management companies appropriately establish limitations on the maximum number of lanes and distance from the kerb in order to meet a target safety level.

6. CONCLUSIONS

This paper has presented the fragility analysis of a class of existing PC bridge decks that were built in Italy between 1970 and 1980. The analysis procedure was fully implemented and automatically run in MATLAB, using Monte Carlo simulation. The scale factor α of the first-lane tandem load of the TLM was used as intensity measure. The performance of the bridge deck was then assessed, considering the ultimate limit state of edge girders. Based on a huge number of realisations, a frequentist approach was used to calculate fragility points, which were fitted through lognormal probability distributions. The conditional failure probability associated with heavy traffic loads imposed by new Italian guidelines for existing bridges decreases to $4.9 \cdot 10^{-3}$ compared to the failure probability associated with the NTC-TLM (i.e., $5.7 \cdot 10^{-2}$), which is related to the nominal value of traffic loads.

According to the findings of a fragility analysis, a combination of traffic load restrictions and/or newly suggested TLMs by new Italian guidelines may result in lower fragility levels than those produced by NTC-TLMs. Indeed, fragility of existing PC bridge decks can significantly reduce to the range $[10^{-6}, 10^{-4}]$ due to limitations on the number and/or location of load lanes.

Based on WIM data available from the literature for actual traffic and code-based return period for NTC-TLM, the authors of this study have provided some considerations on the unconditional failure probability corresponding to various load patterns. This study will support the calibration of new TLMs in a forthcoming updated version of Italian guidelines for existing bridges as well as the development of risk-informed traffic limitations for high-risk bridges by road management companies.

7. ACKNOWLEDGEMENTS

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