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Vulnerability to traffic loads of typical Italian bridges in relation to the evolution of the code framework

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Abstract

Over the last century, the definition of the design traffic loads for Italian road bridges has undergone a continuous evolution, starting from the first code in 1933 up to the current code. The changes in the arrangement and in the intensity of the design loads that have occurred over the years resulted in significant variations, both decreasing and increasing the overall static demand required to the bridges compared to previous codes.

As part of a broader study on the structural risk of existing bridges with respect to traffic loads, it is certainly of interest to investigate the influence of the adopted design code on the risk itself. The paper presents some of the results of a study conducted with this purpose, regarding Prestressed Concrete beam-and slab bridges with simply supported spans, one of the most common typology among bridges with medium-small span.

The comparison between historical codes and current one is carried out in terms of internal forces of the most stressed girder, taking into account the transverse distribution of loads. The study can provide useful elements for the definition of a fast procedure for the preliminary assessment of typical existing Italian bridges based on the geometric characteristics of the decks and their design age.

Keywords

Traffic loads code evolution, vulnerability, PC girder bridges, transverse load distribution

1 Introduction

Bridges are critical elements of roads networks, and they can be subjected to both natural hazards [1], [2] and man-made hazards (e.g. overloading [3]). Furthermore, structural performance of bridges decrease over time due to ageing effects, especially if there is lack of maintenance.

In Italy, a large part of the road bridges were built between the 1950s and the 1970s, and are mainly simply supported Reinforced Concrete (RC) or Prestressed Concrete (PC) bridges [4], [5]. Most of the bridges currently in service have been designed according to outdated codes and many of them could be inadequate to modern heavy traffic. In fact, over the last century, the definition of the design traffic loads for roads bridges has undergone a continuos evolution: starting from the first normative references [6]-[8], which provided loading schemes composed by some well-defined vehicles, passing through ministerial decrees [9]-[11], which defined conventional traffic loads, up to the latest technical codes [12], [13], which provide for the same traffic load models as the Eurocode. The changes in the arrangement and in the intensity of the design loads that have occourred resulted in significant variations, both decreasing and increasing the overall static demand required to the bridges [14]. Thus, the adopted design code affects the bridges vulnerability to traffic loads, and it is certainly of interest to investigate this aspect.

This paper present the results of a parametric study about the comparison of the effects induced by the loads provided by outdated codes and those induced by the loads provided by the current technical code; the study focuses on PC beam-and-slab bridges with simply supported spans, one of the most common typology among bridges with medium-small span. The comparison is performed in terms of maximum bending moment of the most stressed girder, accounting for the transverse load distribution via the methodology proposed by Guyon, Massonnet and Bareš [15].

2 Traffic loads code evolution

The main steps of the evolution of the Italian codes on traffic loads are [6]-[13]:

- Decree no. 8 of September 15, 1933 (D.1933);
- Circular no. 6018 of June 09, 1945 (C.1945);
- Circular no. 384 of February 14, 1962 (C.1962);
- Ministry Decree no. 308 of August 02, 1980 (M.D.1980);
- Ministry Decree of May 04, 1990 (M.D.1990);
- Ministry Decree no. 222 of September 14, 2005 (M.D.2005);
- Ministry Decree no. 29 of January 14, 2008 (M.D.2008);
- Ministry Decree no. 8 of January 17, 2018 (M.D.2018).

The current technical code (M.D.2018) provide for the same traffic load models as the Eurocode 1 part 2 [16].

For the sake of brevity, the traffic load models are only briefly described hereafter; more detailed descriptions are reported in [14], [17].

Each code defines a heavy lane and one or more light lanes, which have to be arranged on the carriageway in the most adverse configuration. Usually, the loads of first-class and second-class bridges differ only in the heavy lane, which is less loaded for second-class bridges. The current code provide for only one class of road bridges.

Before M.D.1980, bridges were designed for the heaviest vehicles that could cross them (heavy lorries or military vehicles); only from 1980, conventional traffic loads have been considered. The lane width changed over time: before 1962, all lanes were 3.00 m wide, while in 1962 they were set to 3.50 m for military loads and to 3.11 m for civil loads. Conventional lanes were set 3.50 m wide from 1980 to 2005 and 3.00 m from 2005 onward, with the exception of narrow bridges. In fact, it was necessary to consider at least two lanes unless the bridge was narrower than 5.00 m, 5.50 m or 5.40 m respectively for M.D.1980, M.D.1990 and subsequent codes. The third class bridges defined in D.1933 and the second class bridges defined in C.1945 are not considered in the present study. Traffic loads expressed in tons are converted into SI units by approximating one ton equal to ten kilo-Newton, as indicated by some Italian regulations.

All the codes before M.D.2008 consider a dynamic amplification factor (φ) of the traffic load as follow:

$$\varphi_{1933} = 1.25 \qquad (1)$$

$$\varphi_{1945}(L) = 1 + \frac{16}{L+40} \qquad (2)$$

$$\varphi_{1962}(L) = 1 + \frac{(100-L)^2}{100\,(250-L)} \qquad (3)$$

$$\varphi_{1980}\left(L, \frac{g}{q}\right) = 1.4 - 0.002\left(\frac{g}{q} + 1\right) \cdot L \geq 1.0 \qquad (4)$$

$$\varphi_{1990}(L) = \varphi_{2005}(L) = \begin{cases} 1.4, & L \leq 10m \\ 1.4 - \frac{L-10}{150}, & 10m < L < 70m \\ 1.0, & L \geq 70m \end{cases} \qquad (5)$$

Where L is the element length in meter, g is the permanent load, q is the variable load. Starting from M.D.2008, the dynamic effects are already taken into account in the load models.

3 Parametric study

3.1 Guyon – Massonnet – Bareš method

The comparison between the effects induced by the outdated traffic load models and those induced by the loads provided by the current technical code is performed in terms of maximum bending moment of the most stressed girder. Other studies suggest that similar results are obtained when the maximum shear forces are considered [14],[17]. The transverse load distribution is taken into account via the methodology proposed by Guyon, Massonnet and Bareš (GMB hereafter) [15]. This method has the advantage of not introducing any assumptions regarding the flexural or torsional stiffness of longitudinal girders and transverse diaphragms.

The GMB method is based on three hypotheses (Figure 1):

- The deck composed by longitudinal girders and transverse diaphragms can be modelled as an equivalent orthotropic plate, with flexural and torsional average stiffness in both longitudinal and transverse directions;
- The orthotropic plate is simply supported at the ends, while the lateral edges are free;
- The load is sinusoidal in the longitudinal direction.

The structural behaviour of the equivalent orthotropic plate is characterised by two parameters, the torsion parameter α and the transverse deformability parameter θ:

$$\alpha = \frac{\gamma_P + \gamma_E}{2\sqrt{\rho_P \cdot \rho_E}} \qquad (6)$$

$$\theta = \frac{b}{L} \cdot \sqrt[4]{\frac{\rho_P}{\rho_E}} \qquad (7)$$

where b is the half-width of the deck; γ_P, γ_E are the torsional stiffness per unit of width of longitudinal girders and transverse diaphragms; ρ_E, ρ_P are the bending stiffness per unit of width of longitudinal girders and transverse diaphragms.

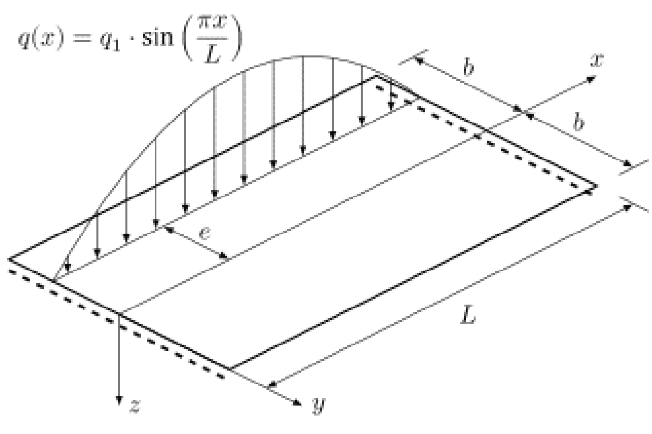

Figure 1 Equivalent orthotropic plate scheme

Guyon, Massonnet and Bareš derived coefficients that allow to calculate the stresses on the equivalent orthotropic plate for the cases $\alpha = 0$ and $\alpha = 1$, and proposed an interpolation for intermediate values of α. In particular, with reference to Figure 1, the bending moment per unit of width in the longitudinal direction m_x can be written as:

$$m_x(x,y) = K_\alpha\left(\theta, \frac{e}{b}, \frac{y}{b}\right) \cdot q_1 \cdot \frac{l^2}{2b\pi^2} \cdot \sin\left(\frac{\pi x}{L}\right) \qquad (8)$$

$$K_\alpha \approx K_0 + (K_1 - K_0)\sqrt{\alpha} \qquad (9)$$

where K_α, K_0 and K_1 are coefficients calculated respectively for α, $\alpha = 0$ and $\alpha = 1$.

Should be noted that (8) can be seen as K_α times the average bending moment of the orthotropic plate (the bending moment induced by the same load uniformly distributer in the transverse direction), so it is a transverse load distribution coefficient.

In order to apply the GMB method analytically, generic loads should be approximated by a proper superposition of sinusoidal loads. The θ_n value for each one must be calculated according to (7) but considering L as the length of the half sine wave. Then, to determine the bending moment induced on a girder it is necessary to integrate with respect to y over its associated tributary width.

The GMB method is usually used in an approximated way, calculating K_α of a sinusoidal load at the coordinate y of a girder and using this value as the transverse distribution coefficient of the generic load.

In this paper, the GMB method is used in the latter way, so the bending moment on the i-th girder induced by N_k loads is calculated as:

$$M_i(x) = \sum_{k=1}^{N_k} K_\alpha\left(\theta, \frac{e_k}{b}, \frac{y_i}{b}\right) \cdot \frac{M_k(x)}{N_i} \qquad (10)$$

where N_i is the number of the girders, y_i is the transverse coordinate of the i-th girder, e_k is the transverse coordinate of the k-th load, $M_k(x)$ is the total bending moment induced by the k-th load.

3.2 Equivalent uniformly distributed loads

The first traffic load models [6]-[8] consisted of sets of concentrated forces, arranged according to well-defined geometries, to represent the axles of civil or military heavy vehicles. To simplify the static analysis involving these sometimes quite complex loading schemes, D.1933 reported five tables with the maximum effects (bending moments and shear forces) induced by each lane loads on a simply supported beam for different span values. Similarly, C.1962 reported the equivalent uniformly distributed loads for both maximum bending moments and shear forces, defined as the uniformly distributed loads that induce the same maximum effects as lane loads placed in the most unfavourable position on a simply supported beam.

M.D.1980 defined lane loads as uniformly distributed loads, whose intensity was a function of the bridge span, while subsequent codes reported traffic load models composed of a few concentrated forces and distributed loads.

In this study, all the lane load models are replaced with equivalent uniformly distributed loads (EUDLs) for maximum bending moments, calculated as:

$$q_{k,eq} = \frac{8 \cdot \max_x(M_k(x))}{L^2} \qquad (11)$$

Under the assumption of transverse load distribution independent of the longitudinal position of the loads, this simplification does not introduce errors in the calculation of the maximum bending moments.

3.3 Description of the parametric study

To investigate the difference in structural performance required by historical codes compared with the current one, different deck geometries were considered. In particular, simply supported bridges with span ranging from 10 m to 40 m (discretized every 1 m) and width ranging from 8 m to 16 m (discretized every 2 m) were taken into account. The carriageway was considered both 1.0 m and 2.0 m narrower than the total width to take into account two lateral kerbs 0.5 m wide for safety barriers or two lateral sidewalks 1.0 m wide, respectively. For each bridge width, four numbers of PC beams were considered; the numbers of beams were selected to obtain spacing as close as possible to 1.0 m, 1.5 m, 2.0 m, 2.5 m and 3.0 m, discarding choices with spacing smaller than 1.0 m or greater than 3.0 m. The PC beams were assumed equally spaced along the width of the deck.

The values of the parameters assumed in the present study are summarized in Table 1. The parametric study involved 41 spans, 5 width values, 2 carriageway configurations and 4 number of beams each, for a total of 1640 geometries analysed for both outdated and current codes.

Table 1 Values of the variable parameters assumed in this study

Parameter	Min value	Max value	Step
L	10 m	40 m	1 m
W	8 m	16 m	2 m
$w_{k/s}$	0.5 m	1.0 m	0.5 m
i_g	1.0 m	3.0 m	Variable

The number of transverse diaphragms was chosen to obtain spacing as close as possible to 10 m. The height of the PC beams was chosen as close as possible to 1/18 of the bridge span and not higher than 1/20 of the span; the other cross section dimensions were defined based on a database of precast PC sections, graphically reported in Figure 2. The slab thickness was assumed equal to 0.20 m, the transverse diaphragm thickness was assumed equal to 0.30 m and the transverse diaphragm height was assumed 0.15 m lower than the girder height.

To evaluate α and θ, the beam sections were approximated as an I-shape, the elastic modulus of the precast concrete (beams) was assumed equal to 40249 MPa, the elastic modulus of the cast-in-place concrete (slab and transverse diaphragms) was assumed equal to 31137 MPa and the Poisson ration was assumed equal to 0.2.

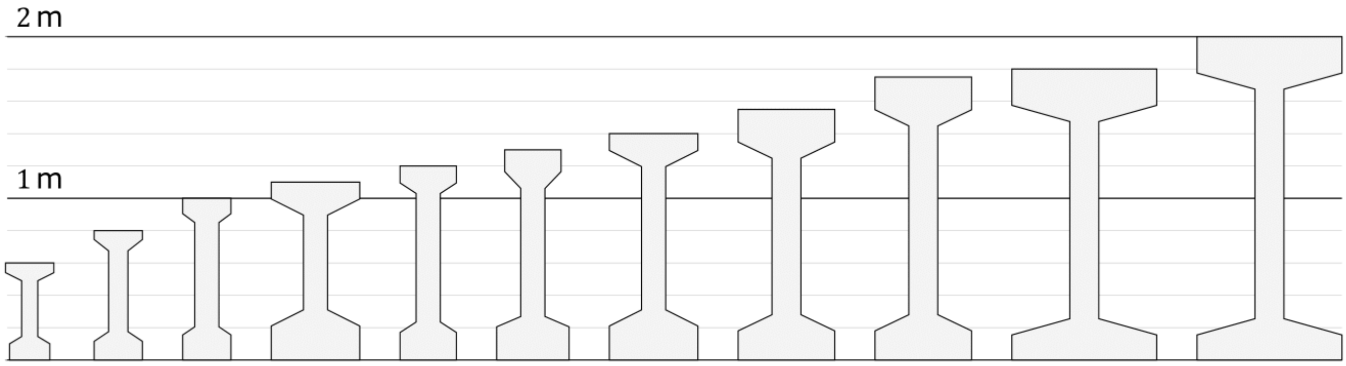

Figure 2 Precast prestressed concrete sections adopted in this study

Dead loads were calculated considering a specific weight of reinforced concrete equal to 25 kN/m^3, kerbs/sidewalk 0.25 m thick (25 kN/m^3) and road pavements 0.10 m thick (20 kN/m^3).

The exact traffic load models were replaced with EUDLs (as described in 3.2) and positioned on the carriageway to maximize the load eccentricity, which is usually the most adverse configuration for the considered bridge typology.

Bending moments were calculated for each girder of the deck in order to determine the maximum value on all girders, taking into account the transverse load distribution by the GMB method (as described in 3.1) , and the maximum bending moment was used to compare the performance required by different codes.

Figure 3 shows an outline of the methodology adopted.

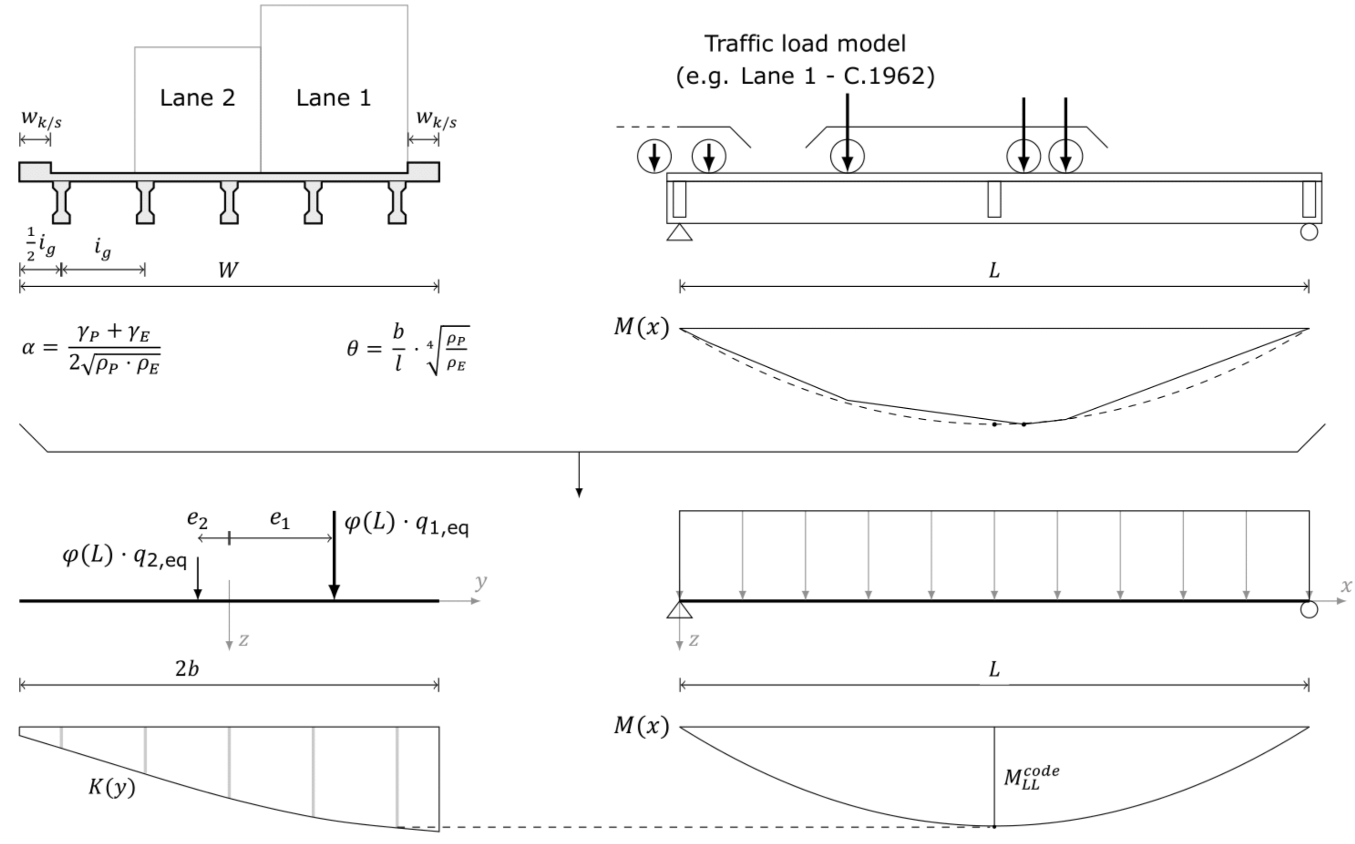

Figure 3 Outline of the methodology adopted: structural behaviour of the deck modelled according to the GMB method and use of equivalent uniformly distributed loads

4 Results

To compare the performance required by different codes, maximum bending moments were normalised with respect to those induced by the current traffic load model:

$$\rho_M = \frac{M_{LL}^{code}}{M_{LL}^{M.D.2018}} \qquad (12)$$

where M_{LL} is the live load maximum bending moment, i.e. that induced by a traffic load model. This ratio is also the performance indicator proposed by the new Italian guidelines [18] for the preliminary assessment of bridges (level 3 of the multi-level assessment method). Results in terms of ρ_M are reported in Figure 4.

Total bending moments were also normalised with respect to those calculated according with M.D.2018:

$$\rho_{M,tot} = \frac{M_{tot}^{code}}{M_{tot}^{M.D.2018}} = \frac{M_D + M_{LL}^{code}}{M_D + M_{LL}^{M.D.2018}} \qquad (13)$$

where M_D is the dead load maximum bending moment. Assuming that the safety levels obtained by complying with the technical requirements of historical codes are equal to that required by the current one, this ratio could be seen as the demand/capacity ratio of existing bridges: $\rho_{M,tot} \geq 1$ denotes adequate bridges and $\rho_{M,tot} < 1$ denotes inadequate bridges. This value is therefore an indicator related to the structural vulnerability of existing bridges, neglecting the effects of degradation over time. Results in terms of $\rho_{M,tot}$ are reported in Figure 5.

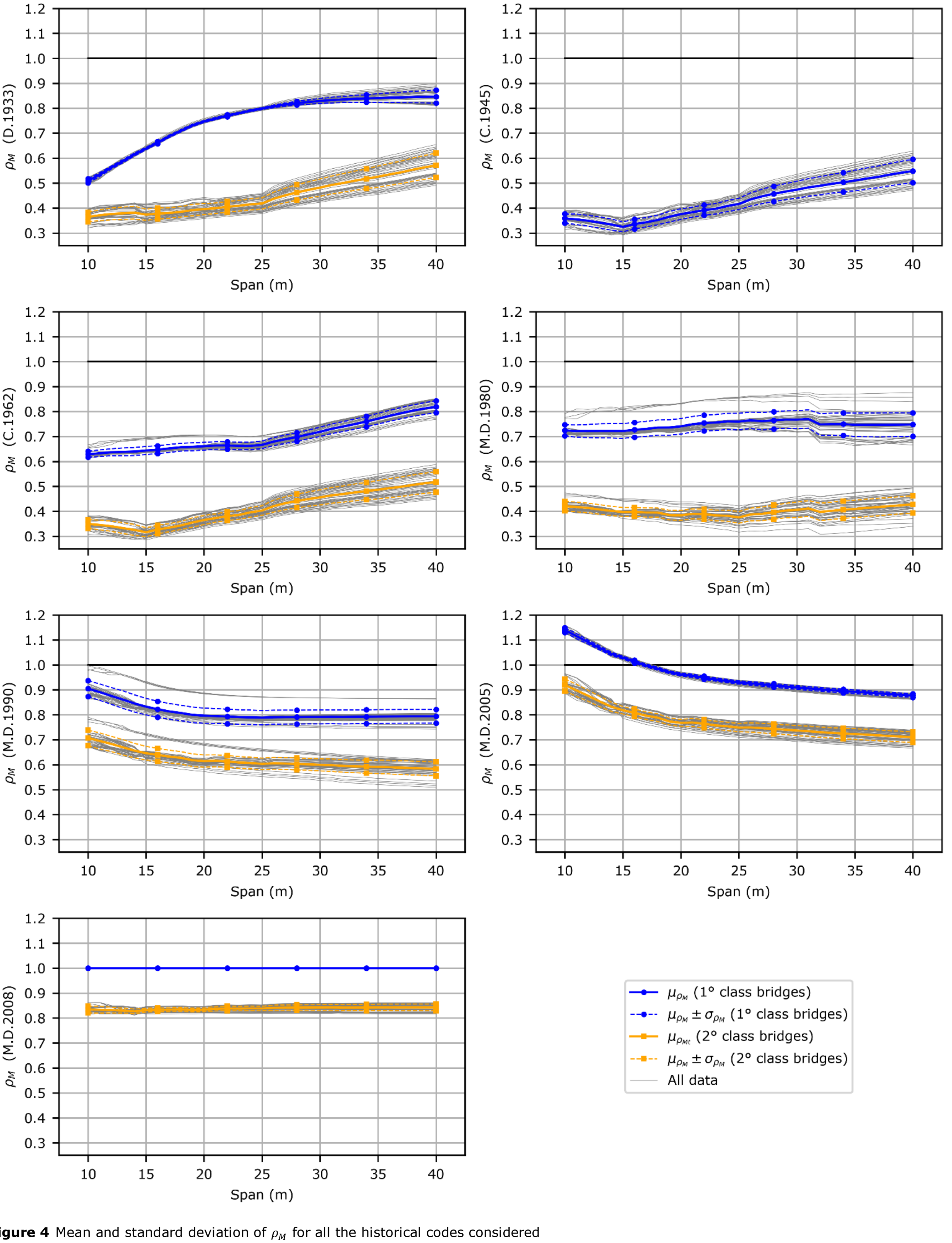

Figure 4 Mean and standard deviation of ρ_M for all the historical codes considered

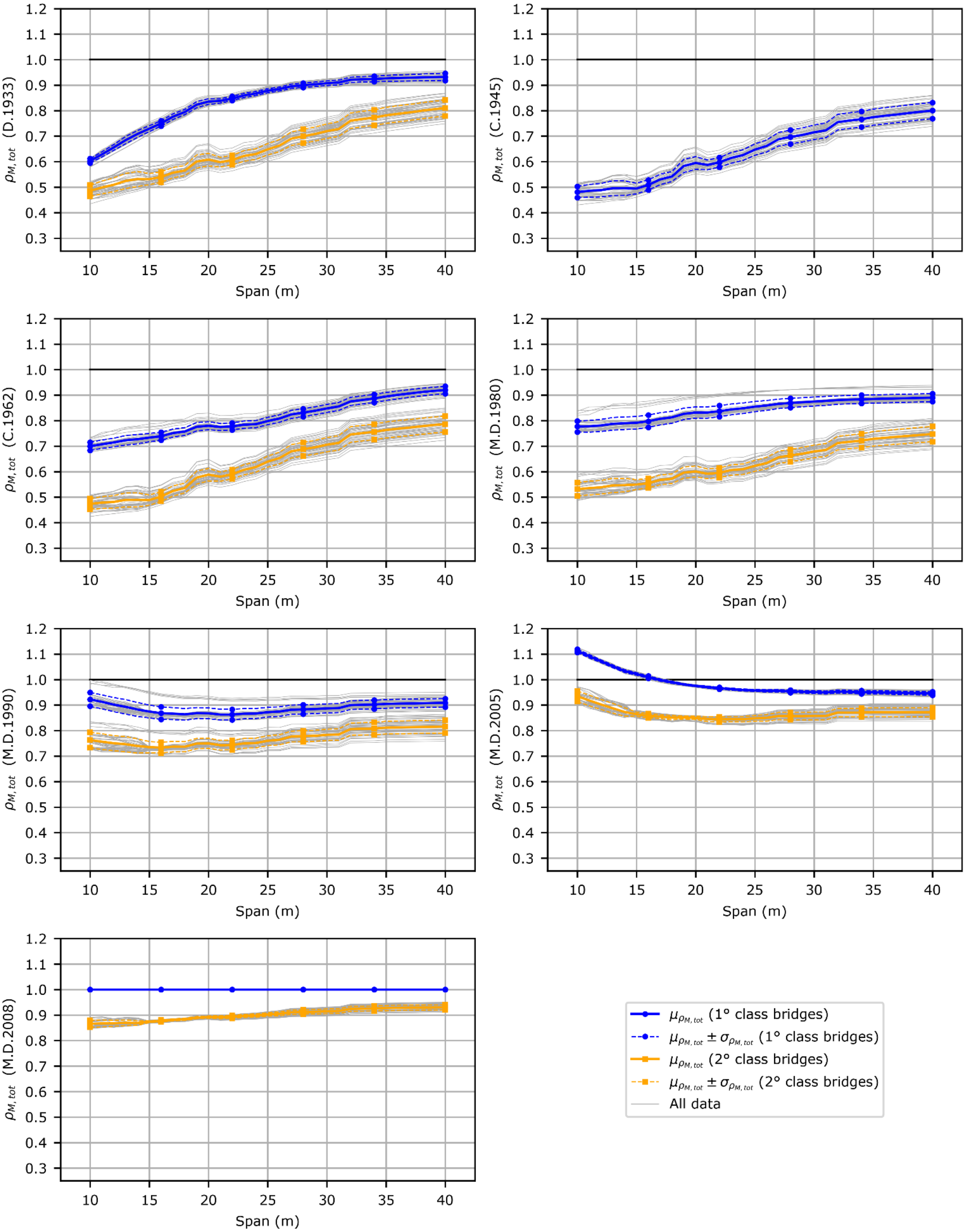

Figure 5 Mean and standard deviation of $\rho_{M,tot}$ for all the historical codes considered

First class bridges designed according to D.1933 show $\rho_{M,tot}$ increasing from about 0.6 to 0.9 for spans ranging from 10 m to 25 m, while for longer spans $\rho_{M,tot}$ values remain almost constant. Second class bridges designed according to D.1933 or C.1962, as well as those designed according to C.1945, show very similar trends, with $\rho_{M,tot}$ values increasing from about 0.5 to about 0.8 in the range of spans considered. These bridges were designed for civil loads only, with similar loading patterns and dynamic amplification factors. First class bridges designed according to C.1962 show also a similar trend but with higher values, varying from 0.7 for 10 m span to 0.9 for 40 m span.

Despite the introduction of conventional loads, bridges designed according to M.D.1980 show $\rho_{M,tot}$ trends similar to those of the previous code, with slightly higher values for short spans and very close values for spans of 35-40 m. Bridges designed according to M.D.1990 present different trends instead, with $\rho_{M,tot}$ initially decreasing and then increasing as the span increases. First class bridges have $\rho_{M,tot}$ values varying between about 0.85 and 1.0, while those referring to second class vary between 0.7 and about 0.85. First class bridges designed according to M.D.2005 show $\rho_{M,tot}$ decreasing with span, with values greater than one for short span bridges and values around 0.95 for bridges with span equal or greater than 25 m. Second class bridges have $\rho_{M,tot}$ values very close to those of first class ones designed according to the previous code.

For first class bridges, M.D.2008 provides for the same traffic load model as M.D.2018, therefore $\rho_{M,tot}$ is always equal to one. Second class bridges, on the other hand, have values around 0.9 (slightly lower for shorter bridges and slightly higher for longer ones).

It is worth noticing that the most structurally vulnerable bridges according to this study are second class bridges designed before 1990 together with bridges designed according to C.1945, followed by first class bridges up to 1945 with short spans. For bridges built up to 1990, shorter decks are more vulnerable, while those longer than 30-35 m show similar performances to more recent bridges. Since 1990, bridges have been designed for increasingly heavy loads, and therefore are less structurally vulnerable; the difference between first and second class bridges is less marked than in the older codes.

5 Conclusions

This paper reports a parametric study that investigates the structural vulnerability of existing Italian bridges in relation to the evolution of the code framework. The study focuses on simply supported Prestressed Concrete bridges, one of the most common typology in Italy. The study takes into account several geometries with different span, width and number of beams and all the traffic load models issued over the last century.

The maximum bending moments of the most stressed girders induced by traffic loads and those induced by dead loads and traffic loads were used as engineering demand parameters. The effects of the transverse load distribution between girders were taken into account by the GMB method. The results were normalized with respect to the bending moments induced by the current traffic load model. These ratios provide an estimate of the structural vulnerability of the considered existent bridges related to the design code as a function of the construction period. These results are also useful for the preliminary assessment of existing bridges according to the new Italian guidelines.

The influence of technical requirements at the time of construction and the effects of material degradation over time have been neglected. Further studies could be carried out to evaluate the influence of these aspects on the structural vulnerability of existent bridges.

References

[1] Ren, J.; Song, J.; Ellingwood, B. R. (2021) *Reliability assessment framework of deteriorating reinforced concrete bridges subjected to earthquake and pier scour*, Engineering Structures 239, 112363. https://doi.org/10.1016/j.engstruct.2021.112363

[2] Argyroudis, S. A.; Mitoulis, S. A. (2021). *Vulnerability of bridges to individual and multiple hazards - floods and earthquakes*, Reliability Engineering & System Safety 210, 107564. https://doi.org/10.1016/j.ress.2021.107564

[3] Yan, W.; Deng, L.; Zhang, F.; Li, T.; Li, S. (2019) *Probabilistic machine learning approach to bridge fatigue failure analysis due to vehicular overloading*, Engineering Structures 193, p. 91–99. https://doi.org/10.1016/j.engstruct.2019.05.028.

[4] Pinto, P. E.; Franchin, P. (2010). Issues in the Upgrade of Italian Highway Structures. *Journal of Earthquake Engineering,* 14, 1221-1252.

[5] Borzi, B.; Ceresa, P.; Franchin, P.; Noto, F.; Calvi, G. M.; Pinto, P. E. (2015) Seismic vulnerability of the italian roadway bridge stock, Earthquake Spectra 31, 4. *https://doi.org/10.1193/070413EQS190M*

[6] Ministero dei Lavori Pubblici (1933). *Normale n.8 – Carichi da considerare nel calcolo dei ponti per strade ordinarie*.

[7] Ministero dei Lavori Pubblici (1945). *Circolare n.6018 del 09/06/1945*.

[8] Ministero dei Lavori Pubblici (1962). *Circolare n.384 – Norme relative ai carichi per il calcolo dei ponti stradali*.

[9] Ministero dei Lavori Pubblici (1980). *Decreto ministeriale del 02/08/1980 – Criteri generali e prescrizioni tecniche per la progettazione, esecuzione e il collaudo dei ponti stradali*.

[10] Ministero dei Lavori Pubblici (1990). *Decreto ministeriale del 04/05/1990 – Aggiornamento alle norme tecniche per la progettazione, esecuzione e il collaudo di ponti stradali*.

[11] Ministero delle Infrastrutture e dei Trasporti (2005). *Decreto Ministeriale 14/09/2005 – Norme tecniche per le costruzioni*.

[12] Ministero delle Infrastrutture e dei Trasporti (2008). *Decreto Ministeriale 14/01/2008 – Norme tecniche per le costruzioni*.

[13] Ministero delle Infrastrutture e dei Trasporti

(2018). *Decreto Ministeriale 17/01/2008 – Aggiornamento delle «Norme tecniche per le costruzioni».*

[14] Bencivenga, P. et al. (2022). *Evolution of Design Traffic Loads for Italian Road Bridges*. In: Pellegrino, C., Faleschini, F., Zanini, M.A., Matos, J.C., Casas, J.R., Strauss, A. (eds) Proceedings of the 1st Conference of the European Association on Quality Control of Bridges and Structures. EUROSTRUCT 2021. Lecture Notes in Civil Engineering, vol 200. Springer, Cham. https://doi.org/10.1007/978-3-030-91877-4_154

[15] Bareš, R.; Massonnet, C. (1966). *Les calcul des grillages de poutres et dalles orthotropes*. Paris: Dunod.

[16] CEN (2005). *Eurocode 1: Actions on structures – Part 2: traffic loads on bridges.* EN 1991-2 :2005, European Committee for Standardization. Brussels.

[17] Buratti, G.; Cosentino, A.; Morelli, F.; Salvatore, W.; Bencivenga, P.; Zizi, M.; De Matteis, G. (2019) Alcune considerazioni sull'evoluzione *normativa dei carichi da traffico nella progettazione dei ponti stradali in Italia*. In: XVIII Convegno ANIDIS, Ascoli Piceno.

[18] Ministero delle Infrastrutture e dei Trasporti (2020). *Linee Guida per la classificazione e gestione del rischio, la valutazione della sicurezza ed il monitoraggio dei ponti esistenti.*