Bridge Maintenance, Safety, Management, Digitalization and Sustainability – Jensen, Frangopol & Schmidt (eds)

Reliability of I-girder PC bridges through proof load testing: Preliminary results

Gianmarco Addonizio
Department of Structures for Engineering and Architecture, University of Naples Federico II, Naples, Italy

Rein de Vries
Faculty of Civil Engineering and Geosciences, Delft University of Technology, Delft, The Netherlands
Structural Reliability, Netherlands Organization for Applied Scientific Research, Delft, The Netherlands

Eva O.L. Lantsoght
Faculty of Civil Engineering and Geosciences, Delft University of Technology, Delft, The Netherlands
College of Sciences and Engineering, Universidad San Francisco de Quito, Quito, Ecuador

Daniele Losanno
Department of Structures for Engineering and Architecture, University of Naples Federico II, Naples, Italy

ABSTRACT: Structural assessment of existing bridges is one of the most challenging aspects for the correct and sustainable management of civil infrastructures that emerged in the last few years. Specifically, Italy has a substantial number of bridges over 50 years old, mainly represented by simply supported prestressed concrete I-girder-type with a cast-in-situ deck. Proof load testing represents an empirical alternative to the standard calculation methods commonly adopted for safety assessment of existing structures. This article investigates the reliability of a bridge in different configurations including damaged and undamaged scenario through site-specific traffic data collected by Weigh-In-Motion (WIM) systems in the Netherlands. Assuming a target load corresponding to characteristic load combination according to Eurocode provisions, the proof load test resulted into slightly reduced reliability indexes during the test while achieving higher values under successful completion. Preliminary results contribute to demonstrate proof load test can represent a valuable method for assessment of existing bridges.

1 INTRODUCTION

1.1 *Motivation for proof load testing*

Looking at the high number of ageing structures, management and safety assessment of existing bridges have emerged as increasingly crucial global concerns. Taking into account mainly the European and North American panorama (Calvi, et al., 2019), a pervasive imperative has arisen to understand the condition of our infrastructure to create prioritisation for good asset management and ensure ongoing safety and efficiency. Many bridges are reaching their nominal design life in the short term. For example, in Italy, the majority of the modern road network (total 840,000 km) was built during the two decades from 1955 to 1975 (Pinto & Franchin, 2010). Moreover, the intricate topography of Italy is a cause of a dense distribution of bridges (Calvi, et al., 2019), with an approximate frequency of one every two kilometres of highways (Miluccio, Losanno, Parisi, & Cosenza, 2021). Additionally, in (ANSFISA, 2022) it is possible to see that the Italian road network is mainly owned by Municipalities (~80%). Similar considerations for ageing bridges can be made in the Netherlands and the United States (Lantsoght, 2023).

DOI: 10.1201/9781003483755-32

In such a challenging framework, different safety assessment methods should be evaluated for the existing bridges (Lantsoght, 2023). This paper focuses on Proof Load Testing (PLT), i.e. a complementary method to the numerical desk-study assessment for evaluating bridges' structural safety against traffic loads. Basically, through a prescribed static loading protocol, the aim is to check actual bridge conditions by monitoring its structural response and at the same time obtaining a minor of its capacity. PLT assumes particular relevance when standard calculation methods encounter limitations, such as in case of lacking original plans (Shenton, J.Chajes, & J.Huang, 2007) (Anay, Cortez, Jáuregui, ElBatanouny, & Ziehl, 2015) or when numerical models are limited by a number of assumptions regarding structural analysis. Moreover, sometimes analytical safety checks are too demanding and time-consuming for several local authorities, when the level of knowledge is low and the number of bridges is very high with a limited budget for an in-depth safety assessment. In this case, exceptional vehicles that passed over the bridge during the past would represent a proof load and could be taken into account for reliability assessment in absence of deterioration phenomena.

1.2 *Codes and state-of-art overview*

As regards international code provisions, mainly two approaches can be acknowledged all over the world. The former approach is typical in the USA and is highlighted by the Manual for Bridge Evaluation (MBE) (AASHTO, 2011), which prescribes the so-called "Proof Load Test" to demonstrate the ability of the bridge to carry some "magnified" live load that is correlated to a specific load rating vehicle. The second approach is more typical in Europe, where an "acceptance" load test after bridge completion or extraordinary maintenance operations is conceived and it is based on an equivalent sectional effect between the load applied during the test and the design unfactored live load. Unfortunately, a fragmented outline persists in the European scenario regarding codes and guidelines for PLT (Lantsoght, 2023). For example, the Italian Building Code (Italian Ministry of Infrastructures and Transportation, 2018) only prescribes the so-called "acceptance" test. Recently, in 2020, the Italian High Council of Public Works (HCPW) issued new guidelines for classification and risk management, safety assessment, and structural health monitoring of existing bridges (Italian High Council of Public Works, 2020) introducing a "temporary operational" condition to be proved by load testing. Nevertheless, this PLT has to be followed by a conventional, i.e. analytical-based, safety assessment (to be completed within 60 days from the test). In (AASHTO, 2011) the calibration of proof load factors is based on literature review, (Lichtenstein, 1993) and (Transportation Research Board, 1998). In the European framework (Casas & Gómez, 2013), a methodology to calibrate proof loads to be applied to existing highway bridges was proposed, based on available WIM data for traffic modelling. Several modifications have been recently proposed by (de Vries, Lantsoght, & Steenbergen, 2023) to improve the PLT practice.

1.3 *Research objective*

After reviewing the current literature on PLT, this study aims to highlight the potential of an empirical alternative to the standard calculation methods commonly adopted for the safety assessment of existing I-girder PC bridges, linking proof load testing and structural reliability. Based on restricted but realistic hypotheses, theoretical benefits in terms of increasing reliability have been proved for different traffic load combinations and damage scenarios.

2 PROOF LOADING RELIABILITY

According to (Melchers & Beck, 2018), structural reliability is related to the probability of a limit state being exceeded for an engineered structural system at any stage during its life. The main concern of PLT is the calibration of the load to be applied in order to achieve target reliability, assuming that a trade-off between collapse risk during the test and posterior benefit regarding proved capacity has to be found. Indeed, the use of relatively high target loads entails an increased risk during the test itself, but once the test has been successfully completed the reliability of the structure would be higher. For this reason, a reliability-based approach is required for a research-oriented analysis and calibration of proof load targets, stop criteria and methodology (Spaethe, 1994). In a probabilistic framework, all types of uncertainties and randomness affecting initial assumptions can be accounted for. The nominal probability of failure P_f (Melchers & Beck, 2018) and the relative Reliability Index β at a structural element level can be considered exhaustive safety measures. Considering all the variables as mutually independent, an example of general limit state function equation Z before any proof load test, adopted in (de Vries, Lantsoght, & Steenbergen, 2023) and (de Vries R., Lantsoght, Steenbergen, & Fennis, 2023), is represented by Equation 1:

$$Z = [\theta_R R - \theta_E G] - \theta_E C_{0T} T = R' - \theta_E C_{0T} T \tag{1}$$

where R represents a capacity measure, G is the permanent load (i.e. the self-weight plus the load due to non-structural elements) and T is the traffic live load. Both for resistance and loads, model uncertainties (θ_R; θ_E) may be considered in the analysis to account for approximation when calculating resistance and load effects, respectively. Finally, the random variable C_{0T} may account for time-independent uncertainties of traffic trend effects. During the PLT, Z_{PL} of Equation (2) may be considered as a limit state function, assuming that in the meantime of the test, the bridge is closed to traffic:

$$Z_{PL} = [\theta_R R - \theta_E G] - \theta_{E,PL} Q_{PL} = R' - \theta_{E,PL} Q_{PL} \tag{2}$$

where R, G, T, θ_R and θ_E are the same as Equation (1) and Q_{PL} represents the effect due to the proof load PL and $\theta_{E,\ PL}$ is the model uncertainty for the controlled proof load effect. Both Q_{PL} and $\theta_{E,\ PL}$ can be modelled as random or deterministic variables. Looking at the posterior analysis, at the state-of-the-art, different ways exist to consider the actual benefit due to a PLT in a reliability framework. The most consolidated and spread approach is to consider the effect generated by a specific load as a lower bound of the actual capacity of the bridge through a truncation of its probability distribution (Christensen, et al., 2023), (Transportation Research Board, 2019), (Casas & Gómez, 2013) and (Faber, Val, & Stewart, 1999). For instance, Equation (1) may still be considered valid with a truncate distribution of R'. Other methods to consider the additional knowledge about the bridge behaviour, such as a bayesian updating, can be found in (K. Nishijima, 2006), (Schmidt, et al., 2020), (Kapoor, Christensen, Schmidt, Sørensen, & Thons, 2021) and (de Vries R., Lantsoght, Steenbergen, & Fennis, 2023).

3 CASE STUDY BRIDGE

3.1 *General description*

The I-girder prestressed concrete (PC) deck archetype represents a significant number of Italian roadway bridges built during past decades. Most of the bridge stock is represented by simply supported decks of 30 to 40 m span with PC girders designed according to an allowable stress method in force at the time (Pinto & Franchin, 2010; Borzi, et al., 2015). In (Miluccio, Losanno, Parisi, & Cosenza, 2021) and (Miluccio, Losanno, Parisi, & Cosenza, 2022), the authors investigated the traffic-load fragility of a portfolio of I-girder PC bridges assumed to be representative of many existing bridges. At the Ultimate Limit State (ULS), the case-study bridges showed higher vulnerability (i.e. higher values of conditional failure probability) to flexural mechanisms rather than shear under traffic load model 1 (TLM 1) (Comité Européen de Normalisation, 2003). Grounding on these preliminary results, the authors of this paper assumed a case study bridge built during the '60s along an important highway in the South of Italy. With a span length of 42m and a total length of around 1km, the bridge is simply supported and it is composed of 3 m high PC I-girders, a cast in situ concrete slab (variable thickness between 0.20 and 0.30m) and four diaphragms (Figure 1).

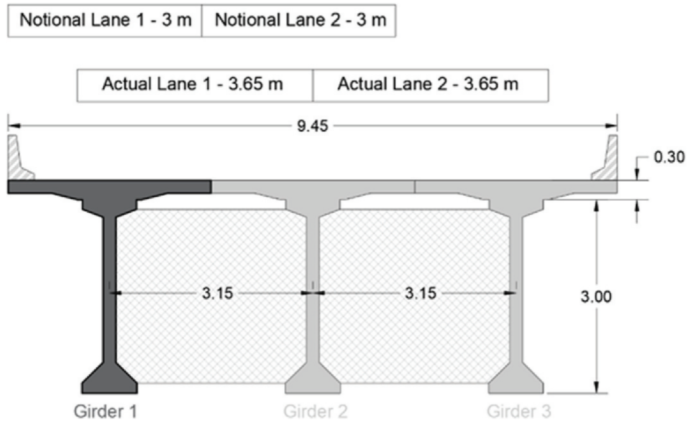

Figure 1. Deck cross-section including traffic load configurations.

Due to the deteriorating state of the viaduct, retrofit interventions were designed in the last few years in order to restore the full bearing capacity of the bridge. Original design and retrofit plans were made available. Additionally, an acceptance load test (i.e. imposed traffic load) after restoration was recently performed to reproduce the TLM1 bending moment. This effect was achieved with four steps of loading, each with two 42t heavy trucks. In the final loading configuration, the eight trucks were arranged according to the notional configuration of traffic (Figure 1). The structural response during the PLT was monitored through displacement transducers, demonstrating linear behaviour and no damage upon unloading.

3.2 *Capacity modelling*

At ULS, the flexural capacity model M_R (Equation 3) has been adopted by considering contributions from both prestressing steel area A_{sp} and reinforcing steel area A_s to be lumped in their centroid with internal lever arm $d \approx 0.9h_{section}$, being $f_{p,\,01}$ the conventional yield strength of prestressing steel and f_y the yield strength of mild steel:

$$M_R = d \cdot \left(A_{sp}f_{p,01} + A_s f_y\right) \tag{3}$$

During on-site visual inspections, different corrosion scenarios were detected in a number of girders prior to retrofitting. In the following analyses, both undamaged and damaged configurations have been assumed; in the latter case the most severe corrosion was considered in terms of 10% loss of total prestressing steel area (i.e. 8 out of 80 strands in total) and total loss of reinforcing steel area at the bottom of the beam (Table 1). Based on original plans, mechanical properties f_y and $f_{p,\,01}$ were assumed as lognormal random variables defined in the next section.

Table 1. Deterministic geometric variables.

Undamaged	Reinforcing steel area Prestressing steel area	$A_s A_{sp}$	6ϕ10 + 7ϕ20 80 strands*	2670 mm^2 7440 mm^2
Damaged	Reinforcing steel area Prestressing steel area	$A_{s,\,dam} A_{sp,dam}$	- 72 strands*	0 mm^2 6696 mm^2

*1 strand = 7 wires 1/2".

Safety checks in terms of internal bending moment accounting for partial safety factors at ULS yielded capacity-to-demand ratios (CDR) equal to 1.36 and 1.26 in undamaged and damaged configuration, respectively. The PLT was performed on the retrofitted bridge whose intervention aimed at restoring the original undamaged safety level (i.e., CDR =1.36).

3.3 *Load distribution factors*

Load distribution due to traffic load is paramount in assessing existing structures. In a few circumstances, simplification may be assumed. Because of the presence of a discrete number of diaphragms, the Engesser-Courbon method (Raithel, 1978) was adopted to calculate load distribution factors for the case study bridge. Basically, the deck deflection profile of the deck is assumed to be rigid in the transverse direction and defined in terms of downward displacement plus rotation only. The primary torsional stiffness of the single girder is neglected therefore deck torsional stiffness is associated with flexural stiffness of the girders. Additionally, the vertical displacements obtained during the load tests validated this assumption with a maximum error of less than 5%. Due to equal girders and traffic load eccentricity, the maximum bending moment M was expected at midspan in the edge right girder (i.e. girder 1) located beneath the slow lane where trucks primarily drive (Figure 1). Two different lane configurations were considered, one having notional lanes (3m width) and one with actual lanes (3.65m width) (Figure 1). In both cases, two load lanes are assumed to obtain maximum effects from the analysis. The effects of each lane in terms of distribution load factors of the edge girder are summarised in Table 2.

Table 2. Load distribution factors.

Lane configuration	Factor lane 1	Factor lane 2
Notional	0.83	0.35
Actual	0.64	0.03

3.4 *Traffic effect modelling*

Acknowledging the present absence of robust Italian Weigh-In-Motion (WIM) data and the methodological purpose of this article, for the present case study a statistical characterisation of traffic load is derived from WIM measurements, utilising an extensive dataset from the Netherlands. Preliminary WIM data from Italian highways is going to be made available in the near future also due to new guidelines promoting traffic monitoring of existing bridges, (Iervolino, et al., 2023) and (Cosenza & Losanno, 2021). WIM recording stations considered in this study are strategically positioned in the Netherlands along high-intensity highway locations and are assumed representative of the traffic load on the case study bridge. Considering that Netherlands traffic is some of the heaviest in Europe (O'Brien, O'Connor, & Arrigan, 2012), the former hypothesis could be considered conservative in terms of intensity and loads. Data from four specific locations (highways A16L, A27L, A50L, A67L) featuring 2-3 traffic lanes were selected due to their high quality. The weekly block maxima of the load effect were obtained to determine the distribution of the load effect in terms of maximum bending moment at midspan. Subsequently, the Gumbel extreme value distribution was fitted to the data using the maximum likelihood estimation (MLE) method. A threshold value (probability of exceedance S = 0.25) was selected to capture the linearly descending right tail of the distribution on the log scale. Lastly, the distribution of weekly maxima was converted to the distribution of annual maxima (de Vries, Lantsoght e Steenbergen 2023). Considering the load distribution factors of Table 2, the simulation was conducted for both notional and actual lane configurations, yielding two sets of parameters for the Gumbel distributions (Table 3), represented in Table 4 as T_{Not} and T_{Act}, respectively. The mean value of the traffic load in the

Table 3. Load effects due to Gumbel distributions for notional and actual traffic lane configurations.

Lane configuration	Annual mean [MNm]	COV [-]
Notional	9.13	0.039
Actual	6.63	0.019

notional lane configuration significantly exceeds that in the actual configuration due to higher eccentricity. Furthermore, contributions from the second lane amplify the coefficient of variation. The realism of the notional lane configuration is a subject of inquiry, especially when considering the layout of the bridge, including the presence of guardrails. Such a configuration would be code-compliant that rarely happens.

3.5 *Permanent load effect modelling*

Permanent load G (Table 4) is considered to model deck self-weight plus non-structural components. The mean value of the associated bending moment (11.70 MNm) is assumed to be equal to the value computed deterministically considering original plans, with a CoV supposed to be equal to 0.10.

3.6 *Proof load effect modelling*

The effect due to a proof load PL, α is modelled both deterministically and probabilistically. The Deterministic Proof Load (DPL) initial assumption considers $Q_{PL,\ \alpha}$ as a deterministic variable equal to α times the value M_{LM1}, which represents the unfactored TLM1 effect (Comité Européen de Normalisation, 2003). According to Italian code provisions for acceptance test (Italian Ministry of Infrastructures and Transportation, 2018), M_{LM1} was computed considering the notional configuration (Figure 1) with tandem (Q1K, Q2K) and distributed loads (q1k, q2k) for the first and the second lane, respectively. A remaining area uniformly distributed load qrk has been considered for the unfavourable parts of the influence lane. $\theta_{\mathrm{E,\ PL}}$ is modelled as a deterministic variable equal to 1. For the posterior analysis, DPL truncates the lower tail of the random variable $R' = \theta_R R + \theta_G G$ at the value $\alpha * M_{LM1}$ and uses this distribution when computing Z. Two different values of α (1 and 1.5) are taken into account to investigate the influence of a PLT with a target effect equal to 100% and 150% LM1, respectively. The Probabilistic Proof Load (PPL) initial assumption considers $Q_{PL,\alpha}$ as a normal random variable with a mean equal to α times the value M_{LM1} computed as in the case of DPL. A CoV equal to 0.05 is considered to account for geometric uncertainty when positioning the trucks. $\theta_{\mathrm{E,\ PL}}$ is modelled as a normal variable with mean and CoV respectively equal to 1 and 0.10. In the posterior evaluation of reliability, PPL considers only samples for which there is no failure during the PLT.

3.7 *Probabilistic models*

All the random variables used in the analysis are defined in Table 4. Bending moment probability distribution functions are displayed in Figure 2 without model uncertainties.

Table 4. Random variables.

Variable	Symbol	Unit	Mean	CoV	Distribution
Yield strength mild steel	f_y	MPa	440	0.075	Lognormal
Yield strength prestressing steel	$f_{p,\ 01}$	MPa	1600	0.050	Lognormal
Resistance Model Uncertainty	θ_R	-	1.00	0.050	Lognormal
Load Model Uncertainty	θ_E	-	1.00	0.100	Lognormal
Proof Load Model Uncertainty	$\theta_{E,\ PL}$	-	1.00	0.100	Lognormal
Time-independent Traffic Load Uncertainty	C_{0T}	-	1.10	0.100	Lognormal
Notional Maxima Traffic Load Effect	T_{Not}	MNm	9.13	0.039	Gumbel
Actual Maxima Traffic Load Effect	T_{Act}	MNm	6.63	0.019	Gumbel
Permanent Load Effect	G	MNm	11.70	0.100	Normal
Proof Load Effect (α=1)	QPL, 1	MNm	12.58	0.050	Normal
Proof Load Effect (α=1.5)	QPL, 1.5	MNm	18.87	0.050	Normal

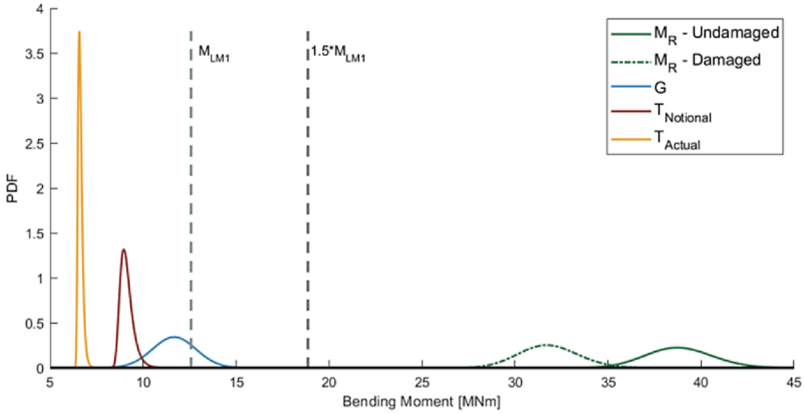

Figure 2. Distribution of bending moment demand due to different loads and capacity scenarios.

Considering the random variable $R' = \theta_R R - \theta_G G$ (undamaged) with mean value μ_R' and standard deviation $\sigma_{R'}$, Table 5 displays proof load magnitudes and mean values of traffic load distributions.

Table 5. Distances of load random variables from the mean value of R' undamaged.

$\mu_{Q_{PL,1}} = \mathrm{M_{LM1}}$	12.58 MNm	$\mu_{R'} - 4.7\ \sigma_{R'}$
$\mu_{Q_{PL,1.5}} = 1.5 \cdot \mathrm{M_{LM1}}$	18.87 MNm	$\mu_{R'} - 2.7\ \sigma_{R'}$
$\mu_{T_{Not}}$	9.13 MNm	$\mu_{R'} - 5.8\ \sigma_{R'}$
$\mu_{T_{Act}}$	6.63MNm	$\mu_{R'} - 6.0\ \sigma_{R'}$

3.8 *Methodology and results*

Before and during the PLT, P_f has been estimated as the probability of the limit state functions Z and Z_{PL} being not positive. After the PLT, the nominal probability of failure P_f (Melchers & Beck, 2018) has been estimated through the Monte Carlo Simulation (MCS) method for a time-invariant analysis, counting the number of successes over the total number of simulations. Considering the two Proof Load Effect Modelling (PPL and DPL), P_f at ULS has been estimated for each traffic load configuration, before, during and after the PLT, in both undamaged and damaged scenarios. Once P_f was estimated, reliability indexes β were calculated and are summarised in Tables 6 and 7. Percentage variations of β are computed according to Equation 4 and reported in Tables 6 and 7.

$$\Delta\beta_i\ [\%] = \left(\beta_i - \beta_{before}\right)/\beta_{before} \tag{4}$$

where β_{before} is the reliability indexes before the PLT and β_i is the reliability index during or after the PLT.

Table 6. Reliability indexes for undamaged scenario.

			During PLT				After PLT			
Traffic	Before PLT	α	PPL	[%]	DPL	[%]	PPL	[%]	DPL	[%]
Notional	4.25	1	3.67	-14	4.80	+13	4.58	+8	4.28	+1
Notional	4.25	1.5	1.95	-54	2.83	-33	5.88	+38	4.96	+17
Actual	5.26	1	3.67	-30	4.80	-9	> 6.0	> +14	5.61	+7
Actual	5.26	1.5	1.95	-63	2.83	-46	> 6.0	> +14	> 5.5*	> +5

*e10 trials without failure samples

Table 7. Reliability indexes for damaged scenario.

			During PLT				After PLT			
Traffic	Before PLT	α	PPL	[%]	DPL	[%]	PPL	[%]	DPL	[%]
Notional	2.79	1	2.13	-24	2.80	0	3.56	+28	2.99	+7
Notional	2.79	1.5	0.41	-85	0.55	-80	5.60	+101	4.93	+77
Actual	3.77	1	2.13	-44	2.80	-26	5.92	+57	4.67	+24
Actual	3.77	1.5	0.41	-89	0.55	-85	> 6.0	> +59	> 5.5*	> +46

*e10 trials without failure samples

It can be seen that the higher the value of the PLT, the higher the posterior value of β compared to its initial value. Assuming a PLT with $Q_{PL,\ 1}$ in the PPL hypothesis, in the case of undamaged configuration, the variation of β with the notional and actual lane is equal to +8% and more than +14%, respectively. Limited effectiveness in the case of a notional lane can be explained in terms of CDR larger than unit (1.36). In the case of lower CDRs as per damaged configuration, this benefit increased up to +28% and +57% for notional and actual lanes, respectively. As regards the differences between PPL and DPL, data indicates slight differences with higher values in case of PPL. Considering variability during the proof load test PPL results in lower reliability than neglecting proof load model uncertainty in DPL. As a matter of fact, for the notional configuration and according to DPL the reliability index doesn't reduce during the PLT. The lower risk during the PLT is reflected in a lower benefit after that, i.e. there is always a positive difference between β_{after} computed probabilistically and deterministically compared to β_{before}. This outcome suggests that the model uncertainty may have important effects on reliability and more investigation is needed.

4 CONCLUSIONS

Proof load testing is a powerful empirical method for the assessment of existing bridges, especially when the level of knowledge of the structure is limited. Specific considerations have to be drawn in terms of damage prevention during the test, target PLT and reliability levels during and after the test as well as in terms of feasibility. The paper presented a preliminary study on the positive impact of a PLT on an existing I-girder PC bridge deck, considering different traffic load combinations and both undamaged and damaged girder conditions. The higher the imposed proof load, the lower the reliability index during the test but the higher the benefit once it is passed successfully. In contrast to CDRs showing a relative difference of less than 10% between undamaged and damaged configurations, the impact of the PLT in terms of posterior reliability would be significantly higher. The methodology could be particularly effective for those bridges interested by exceptional traffic loads during the past which did not report any damage. Even if the preliminary results address the positive impact of PLT on existing bridges, a number of limitations have to be acknowledged from the present study. Further refinement of capacity modelling is recommended considering different failure mechanisms and limit states, such as shear mechanism and cracking. Experimental tests, including laboratory testing on prototype girders, could represent a benchmark for calibrating the PLT and reducing model uncertainties. Dynamic amplification factors should be taken into account when calculating the live load effects. WIM data represent a major breakthrough in traffic modelling to be developed in the near future for the reliability assessment of existing bridges.

ACKNOWLEDGMENT

This study was developed within the CSLLP-ReLUIS project, funded by the Italian High Council of Public Works.

REFERENCES

AASHTO. (2011). *Manual for Bridge Evaluation*. American Association of State Highway and Transportation Officials, Washington, D.C.

Anay, R., Cortez, T. M., Jáuregui, D. V., ElBatanouny, M. K., & Ziehl, P. (2015). On-Site Acoustic-Emission Monitoring for Assessment of a Prestressed Concrete Double-Tee-BeamBridge without Plans. *Journal of Performance of Constructed Facilities*.

ANSFISA. (2022). *Relazione annuale sulla sicurezza delle ferrovie e delle instrastrutture stradali e autostradali.* Agenzia Nazionale per la Sicurezza delle Ferrovie e delle Infrastrutture Stradali ed Autostradali, Roma.

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*.

Calvi, G. M., Moratti, M., O'Reilly, G. J., Scattarreggia, N., Monteiro, R., Malomo, D., Pinho, R. (2019). Once upon a Time in Italy: The Tale of the Morandi Bridge. *Structural Engineering International*.

Casas, J., & Gómez, J. (2013). Load Rating of Highway Bridges by Proof-loading. *KSCE Journal of Civil Engineering*.

Christensen, C. O., Damsgaard, K. D., Sørensen, J. D., Engelund, S., Goltermann, P., & Schmidt, J. W. (2023). Reliability-Based Proof Load Factors for Assessment of Bridges. *Buildings*.

Comité Européen de Normalisation. (2003). EN1991-2 - Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges. Brussels.

Cosenza, E., & Losanno, D. (2021). Assessment of existing reinforced-concrete bridges under road-traffic loads according to the new Italian guidelines. *Structural Concrete*.

de Vries, R., Lantsoght, E. O., Steenbergen, R. D., & Fennis, S. A. (2023). Time-dependent reliability assessment of existing concrete bridges with varying knowledge levels by proof load testing. *Structure and infrastructure engineering*.

de Vries, R., Lantsoght, E., & Steenbergen, R. N. (2023). Proof Load Testing Method by the American Association of State Highway and Transportation Officials and Suggestions for Improvement. *Journal of Transportation Research Record*.

Faber, M. H., Val, D. V., & Stewart, M. G. (1999). Proof load testing for bridge assessment and upgrading. *Engineering Structures*.

Iervolino, I., Baltzopoulos, G., Vitale, A., Grella, A., Bonini, G., & Iannaccone, A. (2023). Empirical distributions of traffic loads from one year of weigh-in-motion data. *Scientific Data*.

Italian High Council of Public Works. (2020). *Linee guida per la classificazione e gestione del rischio, la valutazione della sicurezza ed il monitoraggio dei ponti esistenti*. Rome, Italy.

Italian Ministry of Infrastructures and Transportation. (2018). DM 17/01/2018. Aggiornamento delle "Norme tecniche per le costruzioni'. Rome, Italy.

K. Nishijima, M.F. (2006). Bayesian approach to proof loading of quasi-identical multi-components structural systems. *Civil Engineering and Environmental Systems*.

Kapoor, M., Christensen, C. O., Schmidt, J. W., Sørensen, J. D., & Thons, S. (2021). Decision analytic approach for the reclassification of concrete bridges by using elastic limit information from proof loading. *Reliability Engineering & System Safety*.

Lantsoght, E. O. (2023). Assessment of existing concrete bridges by load testing: barriers to code implementation and proposed solutions. *Structure and Infrastructure Engineering*.

Lichtenstein, A. (1993). Bridge Rating Through Nondestructive Load Testing - NCHRP 12-28(13)A.

Melchers, R. E., & Beck, A. T. (2018). *Structural Reliability Analysis and Prediction*. Wiley.

Miluccio, G., Losanno, D., Parisi, F., & Cosenza, E. (2022). Fragility analysis of existing prestressed concrete bridges under traffic loads according to new Italian guidelines. *Structural Concrete*.

Miluccio, G., Losanno, D., Parisi, F., & Cosenza, E. (2021). Traffic-load fragility models for prestressed concrete girder decks of existing Italian highway bridges. *Structural Concrete*.

O'Brien, E., O'Connor, A., & Arrigan, J. (2012). Procedures for Calibrating Eurocode Traffic Load Model 1 for National Conditions. *Bridge Maintenance, Safety, Management, Resilience and Sustainability*.

Pinto, P., & Franchin, P. (2010). Issues in the Upgrade of Italian Highway Structures. *Journal of Earthquake Engineering*.

Raithel, A. (1978). *Ponti a travata*. Napoli: Liguori Editore.

Schmidt, J. W., Thöns, S., Kapoor, M., Christensen, C. O., Engelund, S., & Sørensen, J. D. (2020). Challenges Related to Probabilistic Decision Analysis for Bridge Testing and Reclassification. *Frontiers in Built Environment*.

Shenton, H. W., M. J. Chajes, & J. Huang. (2007). Load Rating of Bridges Without Plans. *Journal of Performance of Constructed Facilities*.

Spaethe, G. (1994). Die Beeinflussung del Sicherheit eines Tragwerkes durch Probebelastung. *Bauingenieur*.

Transportation Research Board. (1998). Manual for Bridge Rating Through Load Testing. (234).

Transportation Research Board. (2019). Primer on Bridge Load Testing (E-C257)