# New Normative Recommendations for Damping in Railway Bridges within the InBridge4EU Project

Pedro Aires Montenegro[1*], Artur Silva[1], Francisco Pimenta[2], Etienne Laligant[3], Christophe Laurent[3]

[1] *CONSTRUCT-iRAIL, Faculty of Engineering, University of Porto, Portugal*
*paires@fe.up.pt, ajpsilva@fe.up.pt
[2] *CONSTRUCT-VIBEST, Faculty of Engineering, University of Porto, Portugal*
fnpimenta@fe.up.pt
[3] *AVLS, Bureau d'Études en Acoustique et Vibrations*
etienne.laligant@avls-fr.com, christophe.laurent@avls-fr.com

**Abstract.** The estimated damping value significantly affects the dynamic response of railway bridges, particularly at resonance, which can influence both the design of new structures and the assessment of existing ones. Experimental data from projects such as Shift2Rail In2Track2 and In2Track3 indicate that damping in existing bridges often exceeds the conservative values prescribed in EN 1991-2, originally based on ERRI D214/RP3 studies. This discrepancy is especially notable in structures with prominent soil-structure interaction, like portal frame bridges. Furthermore, a wide range of damping values has been observed even within similar structures, highlighting the complexity of this parameter. To address this challenge, the European Union Agency for Railways (ERA) and Europe's Rail Joint Undertaking (EU-Rail) launched the InBridge4EU project to propose new recommendations for improving the current standards that deal with bridge dynamics, in particular the damping criteria specified in EN1991-2. This paper presents key findings from the project, which analysed over 1000 acceleration measurements under railway traffic from 90 bridges across five countries. The outcomes include new damping curves and a classification of bridge families, offering a more accurate and representative basis for future updates to EN 1991-2.

**Keywords:** Damping, Railway bridges, Normative recommendations, Eurocodes, InBridge4EU project

## 1 Introduction

Quantifying damping in railway bridges remains uncertain within dynamic analysis. Energy dissipation during train passages involves not only the bridge structure, but also track, vehicle, foundation, and soil interactions. Each interface, track–bridge, soil–structure, train–bridge, contributes differently, complicating reliable estimation.

At the structural level, damping is linked to intrinsic material behaviour, but non-structural components such as bearings, expansion joints, or even handrails can also influence it. Track–bridge interaction adds further complexity through longitudinal slip within ballast, often nonlinear and seasonally variable [1]. Additionally, energy dissipation through the surrounding soil, such as the radiation damping in portal frame bridges, may add even more complexity to the phenomenon [2].

For identification, the Logarithmic Decrement method is common when a single mode dominates, but less effective with closely spaced modes such as coupled bending–torsion [3]. In such cases, techniques like Prony-Pisarenko or autoregressive modelling, as applied in ERRI D 214/RP3 [4], are more robust.

The influence of damping is most evident under resonance, when dynamic responses to train loads are amplified, particularly above 200 km/h [4]. Response magnitudes depend strongly on damping, but also on stiffness, mass, train configuration, and axle spacing. Because of the difficulties in obtaining precise values, early regulatory efforts by ERRI D214 proposed conservative lower-bound levels for different bridge types [4]. These were later integrated into EN 1991-2 [5]. Although safe, such provisions often exaggerate dynamic effects, leading to conservative and cost-inefficient bridge designs.

To address this issue, the European Union Agency for Railways (ERA) and Europe’s Rail Joint Undertaking (EU-Rail) issued a technical note [6] to resolve existing gaps in bridge dynamics codes, which ultimately led to the launch of the InBridge4EU project [7], focused on updating and refining the damping criteria in EN 1991-2 [5].

This paper presents key findings from InBridge4EU, based on more than 1,000 acceleration records from 90 bridges in five European countries. The analysis leads to revised damping curves and a new bridge categorisation that reflects observed variability. These results provide an evidence-based contribution to updating Eurocode provisions on railway bridge dynamics.

## 2 Studied bridges

Part 2 of the ERRI D214/RP3 [8] report highlights the limited data available for each bridge type. By contrast, the present InBridge4EU study covers a far larger sample, as shown in Table 1. Moreover, ERRI D214/RP3 [8] does not indicate how many measurements were taken per bridge, whereas here around 1,150 valid records were obtained from 89 bridges, providing a substantial dataset for estimating damping ratios.

**Table 1.** Comparison between the bridge data used in D214 and in InBridge4EU.

| Database / Bridge type | ERRI D214/RP3 | Inbridge4EU | Difference (%) |
| --- | --- | --- | --- |
| Steel | 24 | 20 | -17 % |
| Composite | 6 | 18 | +200 % |
| Prestressed concrete | 9 | 13 | +122 % |
| Reinforced concrete | | 7 | |
| Filler beam | 14 | 24 | +71 % |
| Portal Frame | 0 | 7 | - |
| Total | 53 | 89 | +68 % |

# 3 Damping estimation methods

### *3.1 Initial considerations*

Damping was estimated from the bridge's free vibration response following train passages. These decays occur at lower amplitudes than forced regimes, which can bias results if damping depends on amplitude. However, estimating damping while the train is still on the bridge is complex and impractical for large datasets, so only the decay phase was used.

For linear damping, the response is a periodic function modulated by a negative exponential, enabling damping ratios to be derived through the classical Logarithmic Decrement (LD) method. Since post-passage decays are usually dominated by a single mode, exponential functions can be fitted directly to the time series. However, the LD method becomes less reliable when closely spaced modes overlap.

To address this, two advanced approaches, Multi-Criteria Optimisation (MCO) and Covariance-Driven Stochastic Subspace Identification (SSI-COV), were applied. Their combined use introduces redundancy, improves accuracy, and enhances confidence in the estimates.

### *3.2 Multi-Criteria Optimisation (MCO) method*

The MCO method reconstructs a multi-degree-of-freedom function matching measured free vibrations in time and frequency domains. The method combines four metaheuristic solvers and estimates damping by expressing the bridge's free-response acceleration as a sum of exponentially decaying sines, as shown in Eq. (1), assuming a linear viscous model with constant amplitude and frequency parameters.

$$s(t) = \sum_{i=1}^{Ndof} A_i \cdot \exp(-\omega_i \cdot \xi_i \cdot t) \cdot \sin\left(\omega_i \cdot \sqrt{1-\xi_i^2} \cdot t + \phi_i\right) \tag{1}$$

where $N_{dof}$ is the number of modes, $t$ is time, and $A_i$, $\omega_i$, $\xi_i$ and $\phi_i$ represent amplitude, angular frequency, damping ratio, and phase of mode $i$, respectively. The model evaluates multiple modes simultaneously, reducing the need for heavy filtering that can distort signals. Parameter boundaries and combined time- and frequency-domain criteria further improve identification of overlapping modes.

### *3.3 Covariance Driven Stochastic Subspace Identification (SSI-COV) method*

The SSI-COV method [9] was also applied to estimate damping from the available measurements. This approach relies on identifying a state-space model of the recorded response ($\boldsymbol{y}_k$) as

$$\begin{aligned} \boldsymbol{x}_{k+1} &= \mathbf{A} \cdot \boldsymbol{x}_k + \boldsymbol{w}_k \\ \boldsymbol{y}_k &= \mathbf{C} \cdot \boldsymbol{x}_k + \boldsymbol{v}_k \end{aligned} \tag{2}$$

where $\boldsymbol{x}_k$ is the state vector, while $\boldsymbol{w}_k$ and $\boldsymbol{v}_k$ represent process and measurement noise. The state matrix **A** holds the system's dynamic properties. Though developed for stochastic identification, the method can also extract modal parameters from free decays, such as bridge responses after train passages.

# 4 Normative proposals for damping on railway bridges

### *4.1 Initial considerations*

About 1150 train passages on 90 bridges in Portugal, Sweden, Spain, Germany, and France were analysed. Damping in France and Spain was mostly estimated with MCO, while SSI-COV was applied elsewhere. Some Swedish forced-excitation tests are omitted. The reported $\xi_1$ values, forming the basis of the normative proposals, refer to the first vertical bending mode, which is most susceptible to resonance because of its low frequency. For the *"filler beam and reinforced concrete" and "steel" bridge types, the* estimated damping lower bounds remain consistent with EN 1991-2 [5], therefore, only normative recommendation for *"portal frame"*, *"prestressed concrete"* and *"steel-concrete composite"* are proposed in this work.

### *4.2 Contribution of the fundamental bending mode for the bridge response*

Damping plays a decisive role near resonance, yet many measured scenarios are non-resonant. Estimates derived from such cases may be misleading and fail to reflect the structural behaviour relevant for bridge design. It is therefore important to adopt a consistent procedure to identify and assess near-resonant conditions. The evaluation of damping in scenarios close to resonance can follow these steps:

1) Estimate the frequency of the bridge's fundamental vertical bending mode $f_1$ using Infrastructure Managers' dynamic reports or ambient vibration tests. This step will help determine the fundamental mode frequency in advance, making it easier to identify it in the subsequent analysis.
2) Apply a low-pass filter to the time series with a cut-off frequency $f_{cut} = \max\{30 \text{ Hz}, 9f_1\}$, combining EN 1990 [10] with DB InfraGO procedures.
3) Extract the free decay segment.
4) Use MCO or SSI-COV on this segment to estimate the damping ratio $\xi_1$, fundamental frequency $f_1$, and mode amplitude $A$ with its percentage contribution.
5) Retain only cases where the fundamental mode dominates the overall response.

For instance, the free decay response of the Ebr ü. Wenderterstraße bridge in Germany showed an 82% contribution from the fundamental first vertical bending mode, whereas in one of the responses from bridge 001000_459+633 in France, this contribution was only 17%. Hence, for this example, only the damping ratio from the German bridge was retained for normative proposals.

### *4.4 Normative recommendations for the "portal frame" bridge type*

EN 1991-2 [5] does not currently define damping values for portal frame bridges. Results from this study show that spans below 20 m consistently exhibit higher damping than the most comparable category, "reinforced concrete" bridges. To capture this behaviour, a revised curve is proposed in Fig. 1, derived from the project dataset. It envelopes the lower-bound values for bridges with spans bellow 20 m, but also matches observations from two long-span Swedish examples, Gesällgatan North and South, built in prestressed concrete.

The curve also encompasses estimates previously reported by ÖBB-Infra on dynamic interface between bridges and rolling stock, which indicated damping ratios between 8.8% and 5.5% for 4–16 m spans. In Fig. 1, these results are shown alongside the EN 1991-2 [5] *"filler beam and reinforced concrete"* curve, the new normative proposal for portal frames, and the measured lower bounds. The comparison demonstrates that the revised curve accurately represents portal frame damping behaviour, raising damping values where justified while remaining conservative.

For spans above 20 m, evidence is limited, though the two Swedish cases suggest consistency with current standards. To avoid a sharp change at 20 m, an intermediate segment between 15–20 m is introduced, producing a continuous piecewise function of damping versus span length defined by

$$\begin{aligned} \xi &= 3.00 + 0.15\cdot(20-L) \quad ; \quad L < 15\,\mathrm{m} \\ \xi &= 1.50 + 0.45\cdot(20-L) \quad ; \quad 15\,\mathrm{m} \le L < 20\,\mathrm{m} \\ \xi &= 1.50 \quad ; \quad L \ge 20\,\mathrm{m} \end{aligned} \tag{3}$$

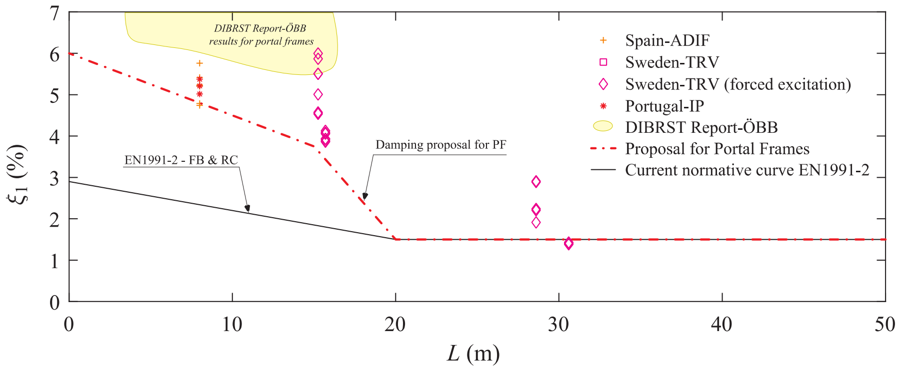


**Fig. 1.** Comparison between the EN 1991-2 [5] damping curve for the "filler beam and reinforced concrete" bridge type and the newly proposed curve for the "portal frame" category.

### *4.5 Normative recommendations for the "prestressed concrete" bridge type*

After excluding non-resonant cases, the lowest damping ratios for prestressed concrete bridges generally remain above the current normative curve. The ERRI D214/RP3 [8] report also makes no distinction between prestressed and reinforced concrete bridges, with no data uniquely assigned to prestressed types between the two curves. Neither the main report nor annexes separate these categories. This supports merging *"prestressed concrete"* into the broader *"filler beam and reinforced concrete"* family, with a proposed 0.5% increase in damping, still below measured lower bounds.

Two Swedish cases diverge: Enköpingsvägen ($L$=20.0 m), identified as an outlier, and Gesällgatan North ($L$=30.6 m), with a minimum damping of $\xi_1$=1.38%, just under the 1.5% threshold. Swedish bridges in both categories generally show lower damping likely due to their atypical specific structural configuration with over-sails and pot-bearing supports. For this reason, such atypical forms are excluded from the revised curve (see Fig. 2). The updated function applies the same piecewise definition as reinforced concrete bridges, now extended to prestressed structures.

$$\begin{cases} \xi = 1.50 + 0.07 \cdot (20 - L) & ; \quad L < 20 \text{ m} \\ \xi = 1.50 & ; \quad L \geq 20 \text{ m} \end{cases} \tag{4}$$

The evidence supporting this modification mainly covers bridges with spans of 8–31 m. Additional data from Network Railm covering 10 bridges with 3.5 m<$L$<15 m, 3 bridges with 15–20 m, and 3 bridges with 25–45 m, suggest critical damping values for the first bending mode of 3–10%, 3–4%, and 2–3%, respectively. As these estimates were not obtained using InBridge4EU procedures, they are included here in Fig. **2** for reference only.

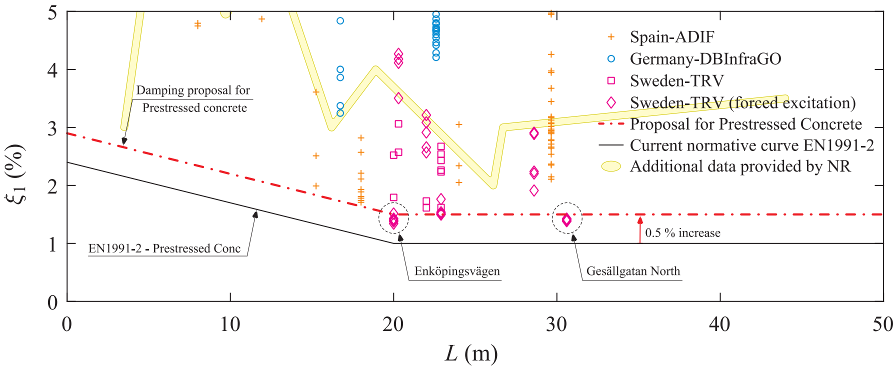


**Fig. 2.** Comparison between the EN 1991-2 [5] damping curve for the "prestressed concrete" bridge type and the newly proposed curve for this category.

### *4.6 Normative recommendations for the "steel-concrete composite" bridge type*

Steel–concrete composite bridges are currently classified with steel bridges, which EN 1991-2 [5] assigns a conservative damping lower bound of 0.5% for spans above 20 m. Yet, the presence of concrete slabs makes composite bridges significantly heavier, typically yielding higher damping. For example, BadOldesloe ($L$=30.10 m) and Banafjällsån ($L$=42.00 m) weigh 13.14 t/m and 16.93 t/m, while steel bridges such as Braunschweig ($L$=35.20 m) and Duisburg ($L$=30.20 m) weigh only 4.09 t/m and 5.90 t/m. This clear contrast supports treating composite bridges separately.

The proposed revision keeps the current EN 1991-2 [5] curve for spans under 20 m, but raises the damping value for longer spans to 1.0%, reflecting behaviour between steel and concrete types. As with portal frame bridges, a transitional segment between 15 m and 20 m ensures continuity, avoiding an abrupt step at $L$=20 m. The updated classification is defined by the following piecewise function (see Fig. 3):

$$\begin{cases} \xi = 0.50 + 0.125 \cdot (20 - L) & ; \quad L < 15 \text{ m} \\ \xi = 1.00 + 0.025 \cdot (20 - L) & ; \quad 15 \text{ m} \leq L < 20 \text{ m} \\ \xi = 1.00 & ; \quad L \geq 20 \text{ m} \end{cases} \tag{5}$$

Fig. 3 presents the proposed damping curve *for "steel–concrete composite"* bridges, alongside the current normative line and InBridge4EU lower bounds. For spans under

20 m, a few French bridges fall below the curve, but no reduction is advised given proven design performance. Above 20 m, two French bridges, 810000_097+770 (24.7 m) and 242000_138+166 (31.5 m), also fall below 1.0%. Their lighter design, with inclined girders and steel beams, likely explains the reduced damping compared with heavier slab composites. Such bridges should remain in the *"steel"* category until more evidence is available. A refined criterion linking damping to concrete–steel ratios could be developed but requires further data.
For comparison, Fig. 3 also shows ERRI D214/RP3 [8] results. To respect axis limits, bridges over 50 m are placed at that value. Three fall below the proposal, though only Vieux Briollay ($L$ = 38 m) is a clear outlier. The Bip bridge's 0.87% is close to 1.0%, and Maison Lafitte's 0.70% is unreliable due to frequency issues and excessive damping cycles, as explicitly referred in [8]. As these ERRI estimates used the simpler LD method, the Vieux Briollay outlier should not affect adoption of the higher lower bound for long-span composites.

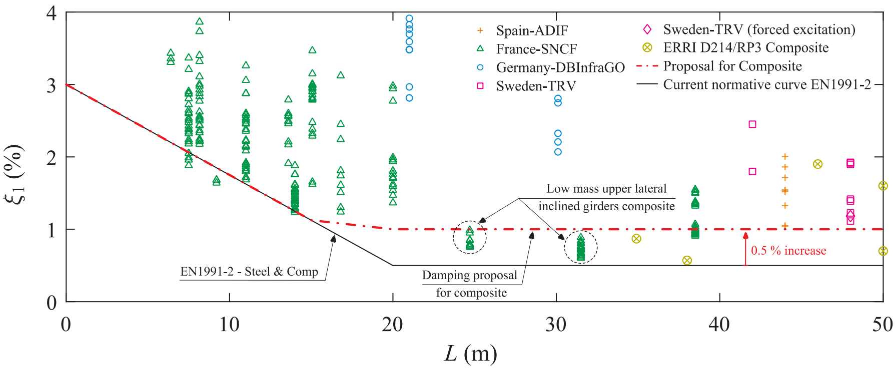


**Fig. 3.** Comparison between the EN 1991-2 [5] damping curve for the "steel and composite" bridge type and the newly proposed curve for the "steel-concrete composite" category.

## 5 Conclusions

Based on this work, the following can be concluded:

- Over 11500 damping estimates were derived from nearly 90 railway bridges in five European countries, using the MCO and SSI-COV algorithms. This represents a much broader dataset than that used in the ERRI D214 study.
- A methodology was developed to identify scenarios close to resonance, where damping has the greatest influence. By assessing the contribution of the first vertical bending mode, the approach allows the exclusion of non-representative cases that previously biased lower bounds. Unlike ERRI D214, this refinement ensures more selective and reliable interpretation and constitutes an important step forward for future standards.
- Based on the analysis, the following recommendations are made:
  i) Retain the *"filler beam and reinforced concrete"* curve.
  ii) Merge *"prestressed concrete"* with the above category.

iii) Define a new family for *"portal frame"* bridges.

iv) Separate steel and composite bridges, introducing a *"steel–concrete composite"* type with higher damping for spans $L$>20 m, while maintaining the existing curve for the *"steel"* bridge family.


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