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Updated Damping Guidelines for Railway Bridges: Insights From the InBridge4EU Project

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Abstract

Understanding and accurately estimating structural damping is essential for evaluating the dynamic performance of railway bridges, particularly under resonant conditions where responses can be amplified. Despite its importance, damping remains a complex and variable parameter, often influenced by factors such as soil–structure interaction, acceleration amplitude or damping estimation algorithms. Notably, recent measurement campaigns, including those carried out under the Shift2Rail In2Track2 and In2Track3 projects, have shown that damping levels in existing bridges frequently surpass the conservative values prescribed in current design codes, such as EN 1991-2. One of the most significant observations from these studies is the wide scatter of damping values, even among bridges with similar structural configurations. Recognising this, the European Union Agency for Railways (ERA), together with Europe's Rail Joint Undertaking (EU-Rail), launched the InBridge4EU project with the goal of improving guidance on dynamic behaviour, and particularly on damping, in the Eurocode framework. This paper presents major outcomes of the InBridge4EU project, which analysed over 1,000 traffic-induced acceleration records from 90 bridges across five European countries. Based on this extensive dataset, the project proposes updated damping curves and introduces a new classification of bridge typologies, aimed at better capturing the real-world variability of damping. These developments provide a more accurate and evidence-based foundation for the future revision of EN 1991-2.

Keywords: bridge damping, Eurocodes, railway bridges, damping estimation algorithms, InBridge4EU project, normative damping.

1 Introduction

Quantifying structural damping in railway bridges remains one of the most complex and uncertain tasks in dynamic analysis. This uncertainty stems from the wide range of energy dissipation mechanisms activated during train passages, involving not just the bridge structure but also the interaction between multiple subsystems, track, train, foundations, and soil [1]. Each of these interfaces, including track–bridge, soil–structure, and train–bridge interactions, can contribute differently to damping, complicating its estimation.

At the bridge level, damping arises primarily from internal material behaviour, but non-structural components such as bearings, expansion joints, and even handrails can have an effect [2]. Track–bridge interaction, for instance, involves longitudinal slip between rail and deck within the ballast, a process that can include nonlinearities and seasonal variations [3, 4]. Additionally, energy dissipation through the foundation into the surrounding soil, radiation damping, adds another layer of complexity [5].

Given the multidimensional nature of these sources, experimental characterisation of damping is particularly challenging. Several testing approaches have been adopted: ambient vibration tests [6], which use low-amplitude environmental excitation; operational monitoring under train traffic [7]; and forced vibration tests using artificial excitation [8]. While ambient tests are easier to perform, their low excitation levels may not reflect the real dynamic conditions caused by trains. Hence, tests based on actual railway traffic or external forcing are generally preferred for more accurate damping estimation.

Among damping identification methods, the Logarithmic Decrement approach is widely used [2], especially when the vibration response is dominated by a single mode. However, its application becomes problematic in cases involving close-mode frequencies, such as coupled bending and torsional vibrations [9]. For such situations, methods like Prony-Pisarenko or autoregressive modelling, adopted in the ERRI D 214/RP3 [10], offer better reliability.

The importance of damping becomes particularly evident under resonant conditions, where the bridge's dynamic response to train loads can be significantly amplified, especially at speeds above 200 km/h [10, 11]. The magnitude of these effects is highly sensitive to damping, in addition to factors like bridge stiffness and mass or train configuration and axle spacing [12]. Yet, due to the difficulty in determining accurate damping values, early regulatory efforts, such as those by the ERRI D214 committee, proposed conservative damping levels based on lower-bound estimates for various bridge types [10]. These recommendations were later embedded in EN 1991-2 [11], shaping the current Eurocode framework. While safe, these standardised damping values often overestimate the dynamic response, leading to unnecessarily conservative, and sometimes cost-inefficient, bridge designs.

To address these limitations, the European Union Agency for Railways (ERA), in collaboration with Europe’s Rail Joint Undertaking (EU-Rail), issued a call to improve the bridge dynamics provisions in existing standards. This led to the launch of the InBridge4EU project [13], which aims to update and refine several aspects of EN 1991-2, with a strong focus on damping criteria [11, 14].

This study reports key findings from the InBridge4EU initiative, which involved the analysis of more than 1,000 acceleration measurements recorded under train traffic on 90 bridges in five European countries. Drawing on this comprehensive dataset, this work offers revised damping curves and introduces a novel categorisation of bridge types to reflect the observed variability in damping behaviour. These results aim to support a more robust, data-driven revision of the damping provisions in EN 1991-2.

2 Studied bridges

In Part 2 of the ERRI D214/RP3 [15] report, the authors note that the available data for each bridge type was limited. In contrast, the present study, conducted within InBridge4EU, includes a significantly larger number of bridges, as shown in Table 1. Furthermore, ERRI D214/RP3 [15] does not specify the exact number of measurements per bridge. However, in this study, the total number of valid measurements, and consequently, the estimated damping ratios, reaches approximately 1150 from 89 different bridges, whose description can be found in [16], representing a substantial volume of processed data.

Bridge type \ Database	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 %

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

3 Damping estimation methods

3.1 Initial considerations

Damping has been estimated based on the bridge's free vibration response (acceleration) after the train passage. Naturally, the free response exhibits lower amplitudes compared to the forced regime, which can introduce bias if damping is amplitude-dependent, as is often observed. However, estimating damping while the train is still on the bridge presents a highly complex challenge and would not be feasible for the entire database. Therefore, all the damping estimations obtained from the tests under railway traffic were carried out based only on the free decay period of the bridge response.

For systems with linear damping, one may immediately see that the response is given by a periodic function modulated by a negative exponential, implying that the damping ratio can be directly evaluated from the free decay response through the classic Logarithmic Decrement (LD) method. These bridge's free decays measured after the train passage should only contain the contribution of a single mode, so the exponential functions can be directly fitted to the recorded time series. This classical approach, however, faces difficulties to isolate the contribution of modes with close natural frequencies. Therefore, more accurate methods should be adopted for better damping estimations in more complex systems, such as bridges. The following sections present the two methods used in this work to estimate damping through the free decay period, namely the Multi-Criteria Optimisation (MCO) and the Covariance Driven Stochastic Subspace Identification (SSI-COV) methods. The adoption of these two distinct methods has been employed to introduce redundancy and enhance the reliability of the results. A series of benchmarks were conducted to validate this approach, as detailed in [16].

3.2 Multi-Criteria Optimisation (MCO) method

The MCO method is based on the reconstruction of an analytic multi-degrees of freedom function matching the measured free response signal in both time and frequency domains. It is based on the MATLAB® multi-objective optimisation toolbox *GODLIKE* developed by [17], which implements the combination of 4 metaheuristics (solving procedures) to find an optimum of a problem involving several input variables and several objective functions.

In the present case, the damping estimation methods assumes that the measured vibration signal (acceleration) during free-responses of the bridge can be decomposed into a sum of exponentially decaying sines according to the Equation (1). Is it then assumed that the damping model is linear viscous and the amplitude and frequency parameters are constant over the response.

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

where N_{dof} is the number of considered modes, t is time, and A_i, ω_i, ξ_i and ϕ_i are the signal amplitude, the angular frequency, the damping ratio and the phase of mode i, respectively. This model allows to evaluate the superposition of several modes at once and does not require to heavily filter signals to process modes separately. Indeed, close modes can be difficult to isolate with filters and increasing the filter order can deform signals significantly. Additionally, the ability of the procedure to provide boundaries for variables helps eliminating spurious values and computing the cost function on time and frequency domains criteria also improves the ability of the method to deal with close modes which would be more difficult to separate in only one domain.

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

The SSI-COV method [18] has also been adopted to estimate damping based on the available measurements. This methodology is based on the identification of a state-space model of the recorded response ($\boldsymbol{y}_k$) as [19]

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

where $\boldsymbol{x}_k$ is the state vector, and $\boldsymbol{w}_k$ and $\boldsymbol{v}_k$ the process and measurement noise, respectively, and where the state matrix $\mathbf{A}$ contains all the relevant dynamic information of the system. Although originally designed for stochastic identification, this method can also extract modal parameters from free decays, such as the bridge response after train passage. Once the modal properties are identified, the measured decays can be decomposed into modal components using the output correlation matrix.

$$\mathbf{R}_y(j) = \mathbf{C} \cdot \mathbf{A}^{j-1} \cdot \mathbf{G} \tag{3}$$

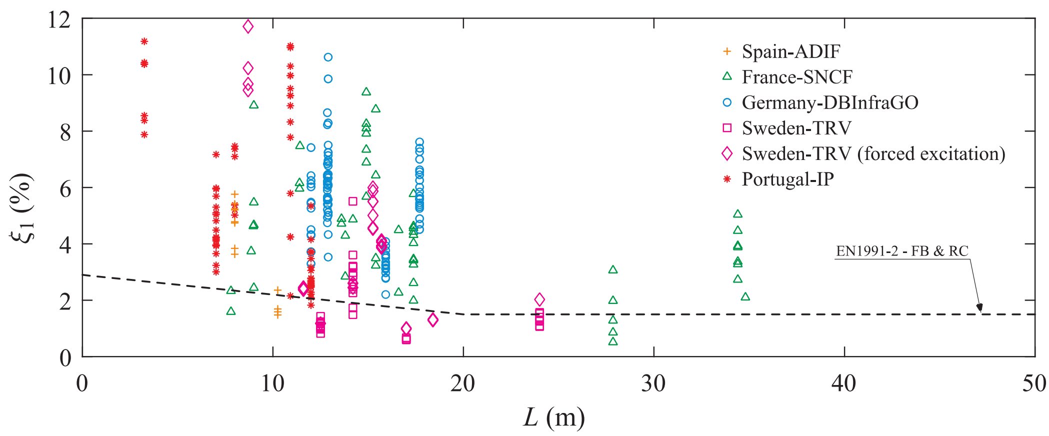
When the correlation matrix $\mathbf{R}_y$ is replaced by the measured free decays $\boldsymbol{y}_k$ and $\mathbf{A}$ substituted by its modal decomposition, the following expression is obtained:

$$\boldsymbol{y}_k = \mathbf{C} \cdot \boldsymbol{\Psi} \cdot \boldsymbol{\Lambda}^{k-1} \cdot \boldsymbol{\Psi}^{-1} \cdot \mathbf{G} \tag{4}$$

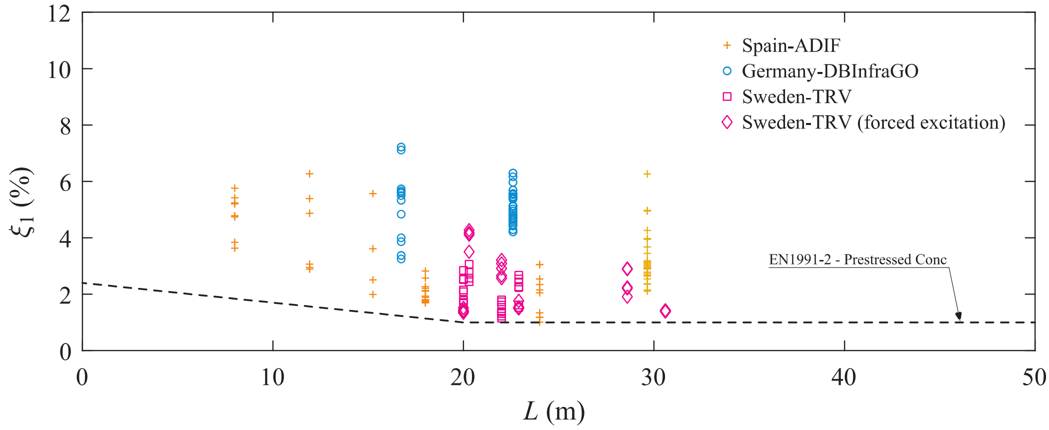
where $\boldsymbol{\Psi}$ contains in its columns the mode shapes, $\boldsymbol{\Lambda}$ is a diagonal matrix, whose elements are equal to $e^{\lambda_i \cdot \Delta t}$, Δt is the time interval between each sample and λ_i are the eigenvalues of the state-space model that are related with the natural frequencies and modal damping ratios of the tested structure. The contribution of a specific mode to the measured decay can be isolated using Eq. (4), retaining only the two complex conjugate eigenvalues corresponding to that mode in the diagonal matrix. Damping estimates for less excited modes are generally less reliable

4 Damping data processing and analysis

A total of around 1,150 train passages across approximately 90 railway bridges in Portugal, Sweden, Spain, Germany, and France were analysed. Damping estimates for most French and Spanish bridges were obtained using the MCO method, while the SSI-COV method was applied to the others. Figure 1 shows damping values as a function of span L for the three bridge types defined in [11], grouped by country. Some Swedish tests involved forced excitation with an external actuator; however, these are not detailed here due to space constraints (see [16] for more information). The damping coefficients ξ_1 presented correspond solely to the first fundamental vertical bending mode, which is most susceptible to resonance because of its low frequency. As expected, considerable scatter is observed, though most values lie well above the normative limits specified in [11]. Nonetheless, values below the normative curves also occur and will be addressed in Section 5.



a)



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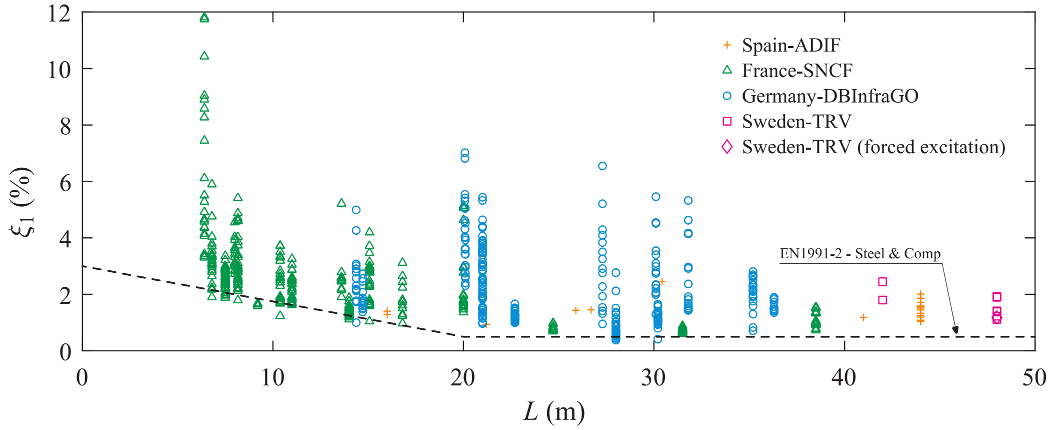


c)

Figure 1: Damping related to the fundamental bending mode as function of L for each bridge type and country: a) Filler beam and reinforced concrete (including portal frames), b) Prestressed concrete and c) Steel and composite.

5 Normative proposals for damping on railway bridges

5.1 Contribution of the fundamental bending mode for the bridge response

Damping is especially critical near resonance zones, yet not all measured scenarios correspond to these conditions. Estimating damping from non-resonant situations can produce misleading values that do not accurately reflect the structural behaviour relevant for bridge design. Therefore, it is essential to establish a clear and consistent method to identify near-resonant scenarios.

The approach to evaluate the damping in scenarios that most closely resemble those used in the design of bridges, i.e., scenarios within the resonance area, may be carried out through the following procedure:

1) Estimate the frequency of the bridge's fundamental vertical bending mode f_1 using dynamic reports from the Infrastructure Managers or through ambient vibration tests conducted during measurements. 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 cutoff frequency f_{cut} given by the following equation proposed by DB InfraGO in its dynamic reports:

$$f_{cut} = \max\{30 \text{ Hz}, 9f_1\} \tag{5}$$

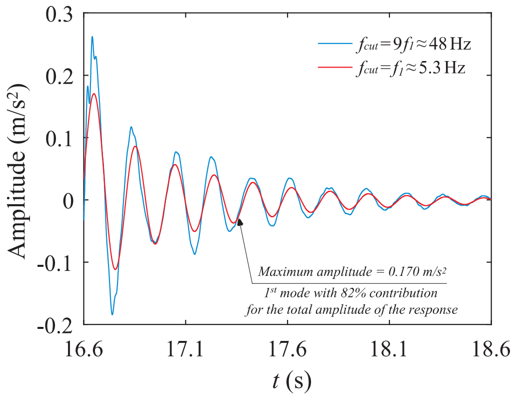
where the 30 Hz threshold is based on the procedure outlined in EN 1990-Annex A2 [20] for evaluating deck acceleration, while the $9f_1$ value corresponds to an internal procedure from DB InfraGO.

3) Isolate the free decay segment of the time series.

4) Estimate the damping of the fundamental vertical bending mode using one of the available methods (MCO or SSI-COV) based on the free decay segment identified in the previous step. Both methods provide not only the damping ratio ξ_1, but also the mode's frequency f_1. Additionally, it is also possible to extract the vibration (acceleration) amplitude A corresponding to the mode, along with its percentage contribution to the total acceleration response

5) Consider damping estimations only from measurements where the contribution of the fundamental bending mode of vibration is dominant, meaning its contribution to the overall response is the highest compared to other modes captured in the analysis.

As an example, Figure 2a illustrates the free decay response of one of the measurements carried out in one German bridge, where the fundamental first vertical bending mode is dominant. In this case, the percentage contribution to the total acceleration amplitude, after filtering the time-series with a cutoff frequency f_{cut}, i.e., considering a vast range of frequencies and modes, is 82 %. In contrast, Figure 2b depicts a scenario from a French bridge that is not clearly dominated by the fundamental mode, as its contribution to the global response is only 17 %. In the present work, only the damping ratios derived from scenarios equivalent to those shown in Figure 2a were considered for normative recommendations.



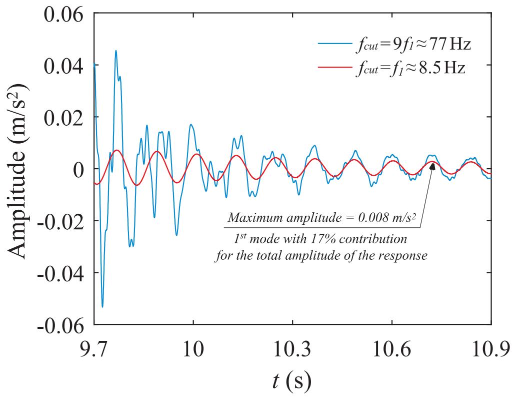


Figure 2: Example of a free decay response where: a) the fundamental first vertical bending mode is dominant, i.e., it has the highest contribution to the global response (bridge Ebr ü. Wenderterstraße in Germany); and b) the fundamental mode is not dominant, as higher modes, less prone to resonance, contribute more significantly to the overall response (bridge 001000_459+633 in France).

5.2 Normative recommendations for the "filler beam and reinforced concrete" bridge type

Following the exclusion of cases that did not correspond to near-resonant conditions, based on the procedure outlined in the previous section, the remaining damping estimates remain consistent with the existing normative curve for *"filler beam and reinforced concrete"* bridges. In several instances, particularly for bridges in Sweden and also some in Portugal and France, the estimated values are even lower than the current benchmark. Given this trend, and considering that the lower bound observed in the current dataset generally falls below that proposed by the ERRI D214 committee [15], the authors find no technical justification for increasing the normative damping curve for this specific bridge category.

However, Figure 1a reveals a noticeable trend: damping values derived from forced vibration tests tend to be marginally higher than those obtained from in-service railway traffic measurements, a pattern also reported by Andersson, Allahvirdizadeh [21]. This observation leads the authors to suggest that further forced excitation tests should be carried out on more common bridge forms, such as simply supported spans, to evaluate whether such configurations are conducive to achieving higher damping ratios.

5.3 Normative recommendations for the "portal frame" bridge type

The current version of EN 1991-2 [11] does not explicitly address damping values for portal frame bridges. However, findings from this study reveal that for spans shorter than 20 metres, these structures consistently exhibit noticeably higher damping levels than those associated with the most structurally comparable category, the *"reinforced concrete"* bridges. To reflect this behaviour, the revised damping curve plotted in Figure 3 and tailored specifically for portal frame bridges is proposed. This curve, developed from the project's dataset, envelopes the lower bound measurements, while also aligning with the values observed in the two long-span portal frames included in this study, Gesällgatan North and South in Sweden, which are constructed from prestressed concrete. In addition, despite methodological differences, the proposed curve encompasses the damping estimates previously reported by ÖBB-Infra in their study on the dynamic interface between bridges and rolling stock [22]. These estimates, which remain confidential in exact figures, suggest damping ratios ranging between 8.8% and 5.5% for spans of 4 to 16 metres and are visually represented as a cloud in Figure 3. This figure juxtaposes the original EN 1991-2 [11] damping curve for *"filler beam and reinforced concrete"* bridges with both the newly proposed curve and the collected lower-bound estimates. It clearly demonstrates that the new curve provides a better representation of portal frame behaviour, offering a significant increase over the standard while maintaining conservatism where appropriate. For spans exceeding 20 metres, available data remains limited; however, measurements from the two Swedish bridges mentioned earlier suggest that damping in these larger spans aligns with the current normative expectations. To ensure continuity, an intermediate segment is introduced between 15 and 20 metres, smoothing the transition and avoiding a sharp discontinuity at $L = 20$ m. The resulting piecewise damping function, plotted in Figure 3, defines damping as a function of span length L for portal frame bridges.

$$
\begin{aligned}
\xi &= 3.00 + 0.15 \cdot (20 - L) \quad ; \quad L < 15\ \text{m} \\
\xi &= 1.50 + 0.45 \cdot (20 - L) \quad ; \quad 15\ \text{m} \leq L < 20\ \text{m} \\
\xi &= 1.50 \quad ; \quad L \geq 20\ \text{m}
\end{aligned} \tag{6}
$$

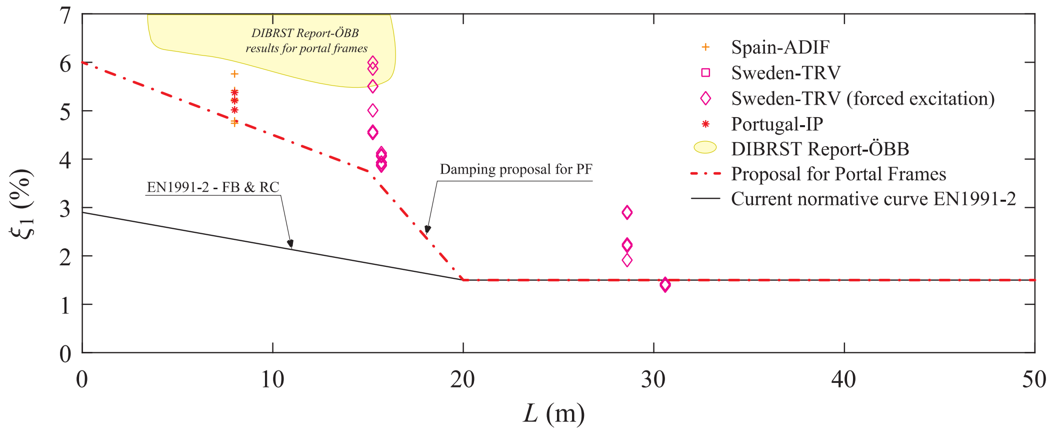


Figure 3: Comparison between the current stipulated curve in EN 1991-2 [11] for the "filler beam and reinforced concrete" bridge type and the newly proposed curve for the "portal frame" bridge category.

5.4 Normative recommendations for the "prestressed concrete" bridge type

By excluding cases that do not reflect near-resonant conditions, the observed lower bound of damping ratios for prestressed concrete bridges generally exceeds the values prescribed by the current normative curve. Furthermore, the ERRI D214/RP3 [15] report does not make a clear distinction between prestressed and reinforced concrete bridge types, as there are no data points uniquely attributed to prestressed concrete bridges situated between their respective normative damping curves. In fact, neither the main body nor the annexes of ERRI D214/RP3 [15] explicitly separate these two structural categories. In light of this, there is a strong rationale for revising the existing damping classification by consolidating the *"prestressed concrete"* category into the broader *"filler beam and reinforced concrete"* bridge family. This adjustment would permit a 0.5% increase in the prescribed damping values for prestressed concrete bridges, while still remaining below the empirically observed lower bounds.

Two notable exceptions to this trend are the Swedish Enköpingsvägen bridge (L = 20.0 m) and Gesällgatan North (L = 30.6 m). The former clearly stands out as an outlier, while the latter exhibits a minimum estimated damping ratio of ξ_1 = 1.38%, which lies just below the 1.5% threshold. It's worth noting that Swedish bridges across both reinforced and prestressed concrete categories generally show lower damping values. This can be attributed to their integral design: continuous decks with integrated wingwalls and backwalls that interact structurally with surrounding embankments, an arrangement known to suppress energy dissipation. For these reasons, the proposed updated normative curve intentionally excludes such atypical configurations (see Figure 4). The revised curve is defined by the same piecewise function currently applied to reinforced concrete bridges and is now extended to include prestressed concrete structures as well.

$$\begin{cases} \xi = 1.50 + 0.07 \cdot (20 - L) & ; \quad L < 20 \text{ m} \\ \xi = 1.50 & ; \quad L \geq 20 \text{ m} \end{cases} \tag{7}$$

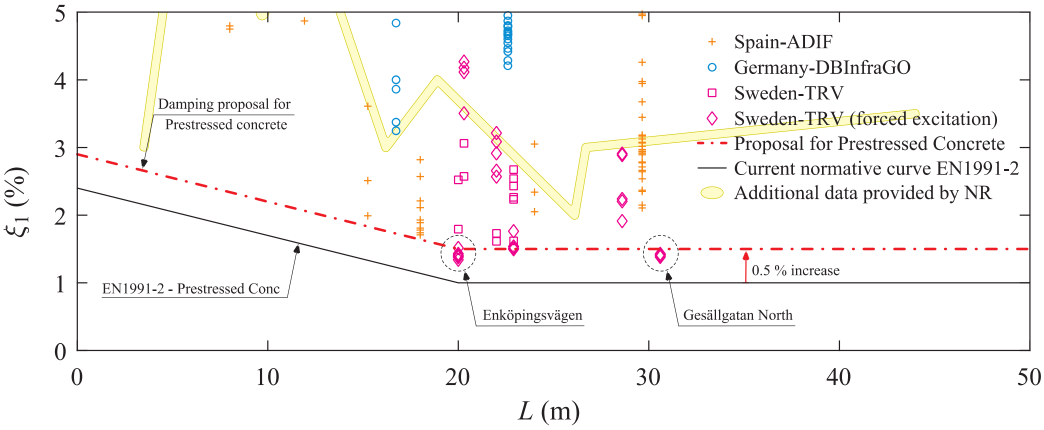


Figure 4: Comparison between the current stipulated curve in EN 1991-2 [11] for the "prestressed concrete" bridge type and the newly proposed curve for this category.

The data available to support this modification, however, pertains to bridges with spans ranging from approximately 8 m to 31 m. Nevertheless, data provided by Network Rail from 10 bridges with spans 3.5 m < L < 15.0 m, 3 bridges with spans 15.0 m < L < 20.0 m and 3 bridges with spans 25.0 m < L < 45.0 m indicate estimated critical damping values related to the first bending mode of 3% < ξ_1 < 10%, 3% < ξ_1 < 4% and 2% < ξ_1 < 3%, respectively. Since these values were not estimated using the same procedures as in InBridge4EU, they are presented here and plotted in Figure 4 for informational purposes only.

5.5 Normative recommendations for the "steel-concrete composite" bridge type

Steel-concrete composite bridges are currently grouped with steel bridges under the same normative damping classification, which sets a conservative lower bound of 0.5% for spans exceeding 20 meters. However, due to the inclusion of concrete elements, typically a top or bottom slab, composite bridges are generally heavier than pure steel bridges, which often leads to higher intrinsic damping levels. For instance, the composite bridges BadOldesloe (L = 30.10 m) and Banafjällsån (L = 42.00 m) exhibit structural weights of 13.14 t/m and 16.93 t/m, respectively. In contrast, steel bridges such as Braunschweig (L = 35.20 m) and Duisburg (L = 30.20 m) are significantly lighter, weighing 4.09 t/m and 5.90 t/m, respectively. In light of this substantial difference in mass, and the correlated impact on damping characteristics, this study advocates for a revised classification of damping behaviour for composite bridges. Specifically, it recommends decoupling them from the steel bridge category in the current normative framework.

The revised proposal retains the existing EN 1991-2 [11] damping curve for spans under 20 meters. However, for longer spans, a higher damping value of 1.0% is suggested, reflecting the intermediate dynamic behaviour of composite bridges relative to both steel and reinforced/prestressed concrete types. As was done in the case of portal frame bridges, a transitional segment has also been introduced between

15 m and 20 m to ensure continuity in the damping values and to prevent abrupt changes at L = 20 m. Accordingly, the updated damping characterization for steel-concrete composite bridges is defined by the piecewise function presented in *Figure 5*, expressed as follows:

$$\begin{cases} \xi = 0.50 + 0.125 \cdot (20 - L) & ; \quad L < 15 \text{ m} \\ \xi = 1.00 + 0.025 \cdot (20 - L) & ; \quad 15 \text{ m} \leq L < 20 \text{ m} \\ \xi = 1.00 & ; \quad L \geq 20 \text{ m} \end{cases} \tag{8}$$

Figure 5 displays the newly proposed damping curve for *"steel-concrete composite"* bridges, alongside the current normative curve and the lowest damping values from InBridge4EU. For spans under 20 m, a few French bridges fall below the current curve, but lowering it is not advised given successful design experience. For longer spans, two French bridges, 810000_097+770 (24.7 m) and 242000_138+166 (31.5 m), fall below the proposed 1.00% damping. This likely reflects their lighter structural type, featuring upper lateral inclined girders connected by spaced steel beams, which reduces damping compared to heavier composite bridges with concrete slabs. Therefore, unless new data emerges, these lighter composite bridges over 20 m should be excluded from this normative curve and instead classified with *"steel"* bridges. Developing a broader criterion incorporating concrete-to-steel ratios and their influence on global bending modes would be valuable but requires more data, representing an open topic for future code revisions.

Figure 5 includes damping values from composite bridges in ERRI D214/RP3 [15] for comparison with InBridge4EU results. Due to graph limits (L = 50 m), bridges longer than this are plotted at 50 m for consistency. While three bridges fall below the proposed recommendations, only Vieux Briollay PK 293.020 (L = 38 m) is a clear outlier. The Bip bridge's lowest damping (0.87%) is close to 1% and aligns with most ERRI data. Maison Lafitte's low value (0.70%) is considered unreliable due to questionable frequency assessment reported in [15] and excessive damping cycles. These damping estimates used the simpler Logarithmic Decrement method, which may lack accuracy in some cases. Given this, the Vieux Briollay outlier should not affect the proposed higher damping lower bound for longer-span composite bridges.

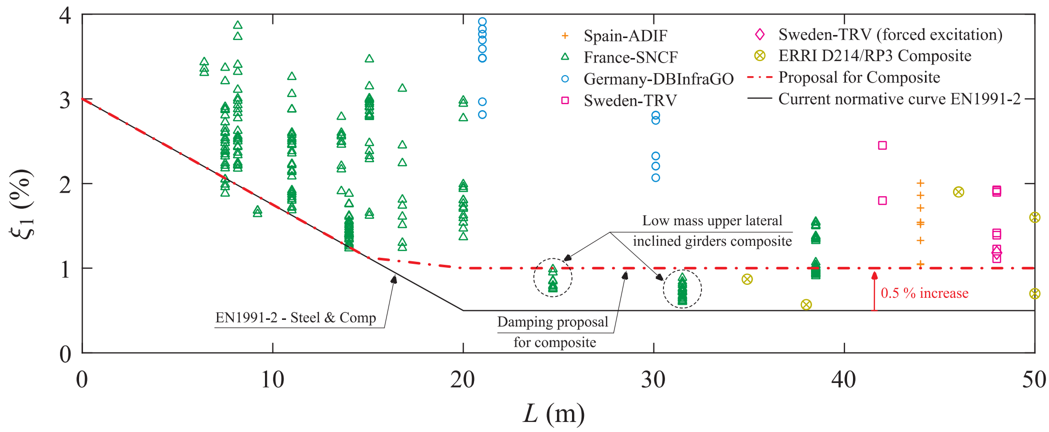


Figure 5: Comparison between the current stipulated curve in EN 1991-2 [11] for the "steel and composite" bridge type and the newly proposed curve for the "steel-concrete composite" bridge category.

5.6 *Normative recommendations for the "steel" bridge type*

For steel bridges, the estimated lower bound of damping, after filtering out non-resonant cases, remains consistent with the existing normative curve defined in EN 1991-2 [11]. The data collected in this study did not reveal any significant deviations or outliers that would warrant concern regarding the integrity of this lower bound. Moreover, the majority of the damping values obtained align well with those reported by the ERRI D214 committee, reinforcing the validity of the current standard. Although a few of the lowest values in both datasets fall slightly beneath the normative curve, these instances do not correlate with any identifiable structural feature or configuration. Consequently, the findings from this research offer no engineering basis to exclude specific results or to advocate for an increase in the prescribed damping values for steel bridges. Therefore, it is recommended that the current damping curve for this bridge category be retained without modification in future updates of the code.

6 Conclusions

Based on this work, the following can be concluded:

- More than 1,000 damping estimates were obtained from close to 90 railway bridges located across five European countries, France, Germany, Portugal, Spain, and Sweden, using two distinct identification algorithms: MCO and SSI-COV. This dataset significantly expands upon the scope of previous work by the ERRI D214 committee.
- To improve the reliability of the damping estimations, a dedicated methodology was introduced to isolate measurement scenarios most representative of near-resonance conditions. Since damping has the most pronounced effect on bridge dynamics during resonance, estimations derived from non-resonant cases may distort the interpretation. The proposed approach evaluates the influence of the first vertical bending mode, typically the most resonance-prone due to its low frequency, on the global response. By quantifying this contribution, the methodology allows for a more selective and accurate interpretation of the data. In contrast to the ERRI D214 study, which lacked such a diagnostic criterion, this refinement enables the exclusion of damping values arising from poorly correlated scenarios, many of which previously biased the lower bound in normative curves. In the authors' view, this represents a significant methodological advancement for informing future updates to standards.
- The work performed gave origin to the following recommendations (this proposals do not address any additional damping related to vehicle-bridge interaction, which has already been removed from the current version of EN 1991-2 [11]):
  i) The *"filler beam and reinforced concrete"* curve should remain unchanged.
  ii) The *"prestressed concrete"* bridges should be merged with the *"filler beam and reinforced concrete"*, forming a single bridge family.
  iii) Portal frames should be classified under a newly proposed *"portal frame"* bridge family.

iv) The current steel and composite bridge family should be split into two: a newly defined *“steel-concrete composite”* bridge type with higher damping for longer span bridges ($L > 20$ m) and a *“steel”* bridge type, which retains the existing curve.

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