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Probabilistic Approach for Assessment of Track Stability Safety Factor on Ballasted Railway Bridges

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Abstract

Railways are increasingly an essential part of a greener future. In Europe, the Shift2Rail programme has promoted research projects related to rail infrastructure, including the enhancement of building codes. Concerning the EN 1990-Annex A2, this study discusses its limit for vertical deck acceleration on ballasted track railway bridges and the partial safety factor associated with it. A novel methodology to calculate such factors is presented, based on a probabilistic approach that accounts for the variability in geometrical and mechanical properties of the bridge. A case study bridge is used to calculate new partial safety factors, considering the High Speed Load Model. The results indicate that there is a margin for discussion of that partial safety factor in the Eurocode.
Keywords: High-speed railway bridges, Probabilistic Analysis, Eurocode

1 Introduction

In Europe, there is a raising awareness of the rail sector as an essential player for the guarantee of a greener future. Initiatives such as the European Year of Rail, aligned with the European Green Deal, highlight climate and environmental concerns, with the European Union Agency for Railways [1] underscoring that railways are the backbone of intermodal transportation, foreseeing development in high speed lines.

To connect regulators, corporations, public entities and academia, the Shift2Rail Joint Undertaking was formed, and it promotes research projects related to rail infrastructure (among other subjects), such as the In2Track2 project [2], which the work presented in this paper is a part of.

One of Shift2Rail's its objectives is to enhance building codes in order to reduce uncertainties and costs. Concerning railway bridges, the EN 1990-Annex A2 [3] limits vertical deck acceleration to 3.5 m/s2 on ballasted tracks. In experimental tests, it has been found that the ballast layer's loss of stability occurs at levels close to 7 m/s2 [4], confirming that a safety factor of 2 is associated with the normative limit [5]. This study proposes a discussion of this safety factor by presenting a methodology that accounts for uncertainties in the material and geometrical properties of the structural system. Using a probabilistic approach, a new formulation for this safety factor is introduced.

2 Methods

In this work, the aforementioned partial safety factor γ_{bt} is defined as the ratio between the limit acceleration a_{Rk} of 7 m/s^2 and the maximum acceleration calculated in a design scenario a_{Ed}, corresponding to a critical speed. The proposed methodology is divided into three steps, as follows.

Firstly, design scenarios are defined, considering the EN 1991-2 [6] dispositions concerning the estimations of stiffness and mass. Several random variables related to the bridge are defined and a sensitivity analysis is performed, to understand which of them are the most conditioning. This is achieved by comparing each variable's upper and lower bounds with a baseline scenario of all average values. Consequently, design scenarios can be determined and each of them is subjected to a dynamic analysis, registering the maximum vertical deck acceleration for each speed value.

Afterwards, the critical speed can be calculated, employing a probabilistic analysis. For this Monte Carlo trial, the random variables are sampled, generating a number of bridges, and dynamic analyses are ran to obtain the maximum acceleration for each speed. The critical speed corresponds to the lowest speed at which the probability of surpassing the limit acceleration p_f is greater or equal to 10^{-4} (target from [7]). To reduce computation costs, the initial assessment is made with a number of samples n of 1.000 on the entire speed range, and it is increased afterwards, for speeds close to the critical, until n=100.000 ([8]).

Lastly, the partial safety factor associated with the probabilistic analysis can be obtained. This factor can be interpreted as a value by which the design acceleration has to be multiplied to guarantee that the probability of exceeding the acceleration limit is less that p_f. Fig. 1. Represents a schematic overview of the methodology.

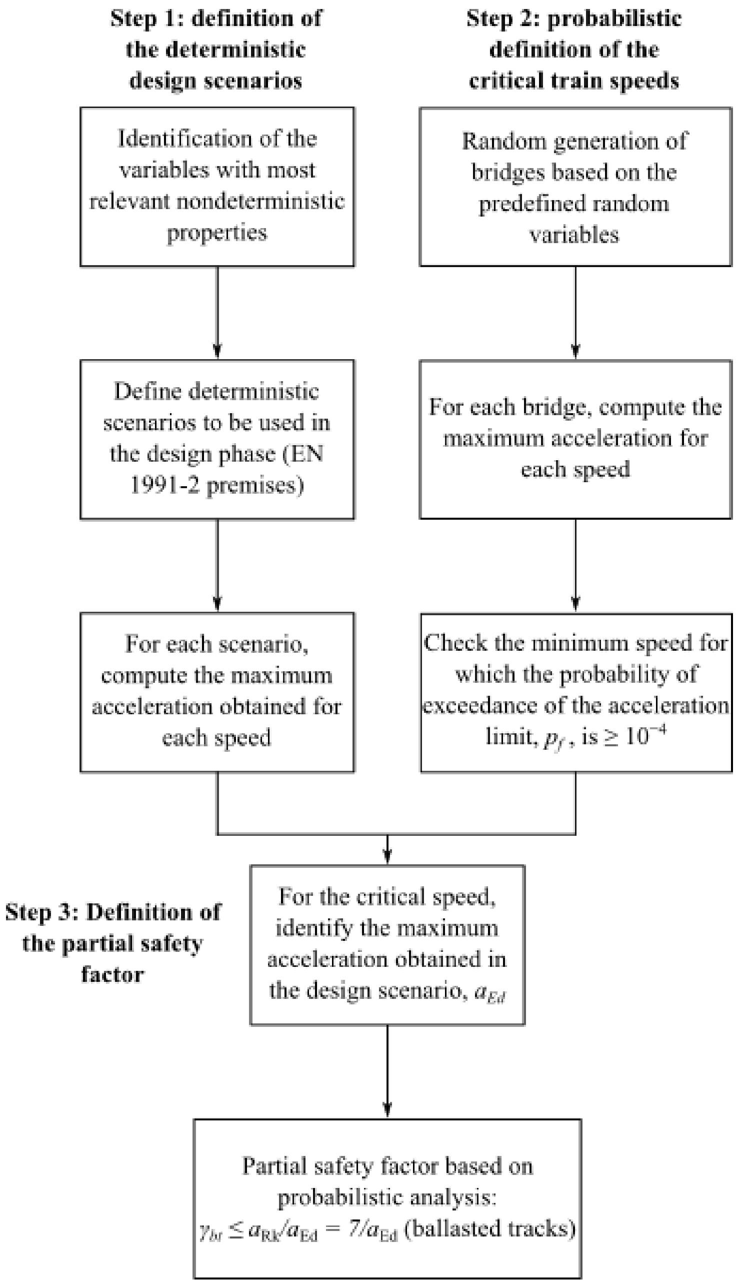

Figure 1: Methodology

3 Results

The Canelas bridge was selected as a case study, given the previously performed numerical studies ([9]). A single simply supported 12 m span of this filler-beam bridge was modelled. The deck measures 4.5 by 0.7 m, embedded with HEB 500 steel profiles, and it is supported by a set of bearings. For the assessment of vertical acceleration, a 2D FE mode suffices, and the dynamic analyses are achieved by the moving loads methodology. Table 1 lists the selected random variables (adapted from [10]) and Fig. 2 represents how the variables relate to the FE model. The results from the sensitivity analysis indicated the importance of the variables seen in Table 2, which lists the design scenarios.

Structure variables (Gaussian)	Mean	Standard deviation
Reinforced concrete density (γ_C)	2.5 t/m³	0.1 t/m³
Concrete elasticity modulus (E_C)	36.1 GPa	2.888 GPa
Slab thickness (t_{slab})	0.7 m	0.01 m
Slab width (b_{slab})	4.475 m	0.005 m
Area of the steel profiles (A_S)	0.01975 m²	0.00079 m²
Structural damping (ξ)	2 %	0.3 %
Track variables (Uniform)	**Minimum**	**Maximum**
Ballast density (γ_b)	1.5 t/m³	2.1 t/m³
Ballast elasticity modulus (E_b)	80 GPa	160 GPa
Ballast height (h_b)	300 mm	600 mm
Load distribution angle (α)	15°	35°
Sleeper mass (m_S)	220 kg	325 kg
Rail pad stiffness (k_p)	100 kN/mm	600 kN/mm
Track shear stiffness (k_t)	1×10^4 kN/m/m	3×10^4 kN/m/m
Other variables (Uniform)	**Minimum**	**Maximum**
Shear modulus of the neoprene from the bearing supports (G_n)	0.75 MPa	1.20 MPa

Table 1: Random variables

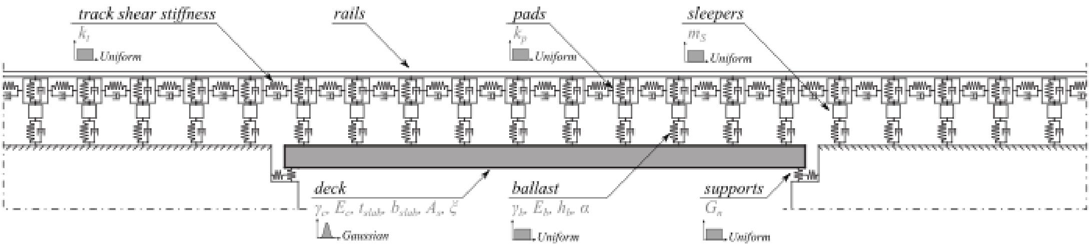

Figure 2: Schematic representation of the FE model and random variables

Scenario	E_C, ξ	γ_C, t_{slab}	γ_b, h_b	G_n
F1	$\mu - 1.64\sigma$	$\mu - 1.64\sigma$	min.	min.
F2	$\mu - 1.64\sigma$	$\mu + 1.64\sigma$	max.	min.
F3	$\mu - 1.64\sigma$	$\mu - 1.64\sigma$	min.	max.
F4	$\mu - 1.64\sigma$	$\mu + 1.64\sigma$	max.	max.

Table 2: Deterministic design scenarios

The probabilistic analyses were conducted for the 10 Universal Trains of the High Speed Load Model [6]. Figs. 3 and 4 show the simulation results for the HSLM-A1 and HSLM-A3 trains with the initial sample size of 1.000 and a speed range from 140 km/h to 500 km/h in 10 km/h intervals. The second iteration, with

n=5.000, is shown in Figs. 5 and 6, for the same load models. It can be seen that the range was reduced and the interval changed to 5 km/h. The simulation results for n=100.000 are represented in Figs. 7 and 8, in terms of the evolution of p_f, for a selection of speed values. A summary of the calculated critical speeds and corresponding p_f values can be consulted in Table 3.

Having found the critical speed values, the dynamic scenarios are consulted to determine the maximum design acceleration, and consequently, the ratio that defines γ_{bt} can be calculated. Table 4 contains these values, as well as the deterministic scenario that led to them.

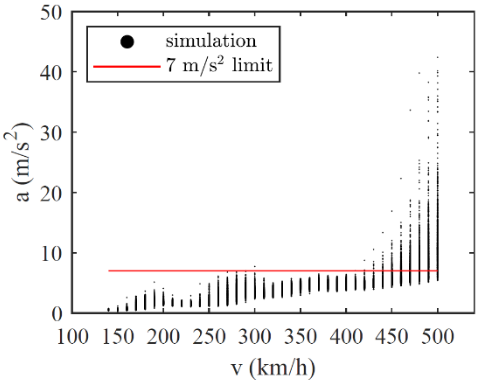

Figure 3: Initial assessment of critical speed (n=1.000) – HSLM-A1

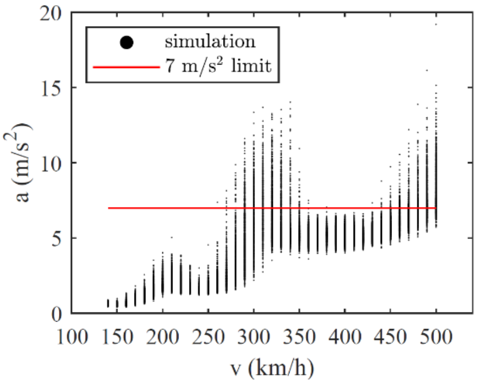

Figure 4: Initial assessment of critical speed (n=1.000) – HSLM-A3

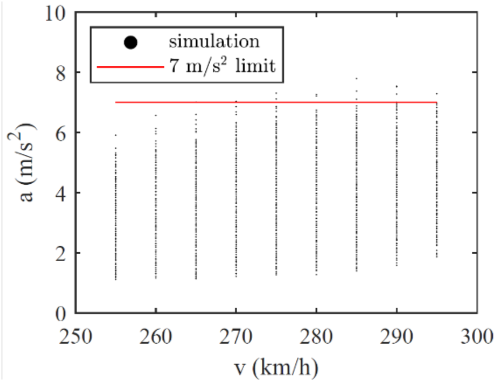

Figure 5: Refined assessment of critical speed (n=5.000) – HSLM-A1

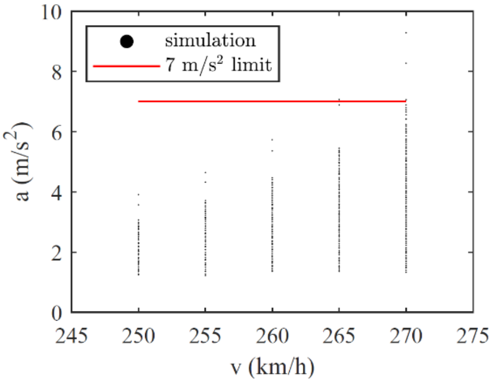

Figure 6: Refined assessment of critical speed (*n*=5.000) – HSLM-A3

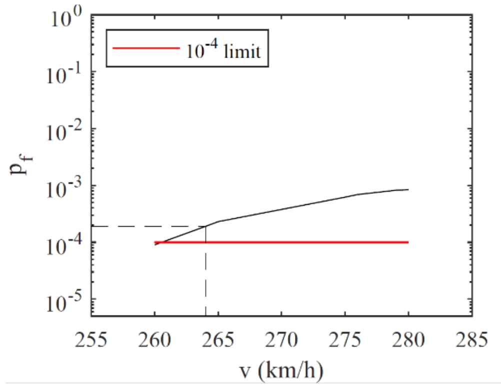

Figure 7: Refined assessment of critical speed (*n*=100.000) – HSLM-A1

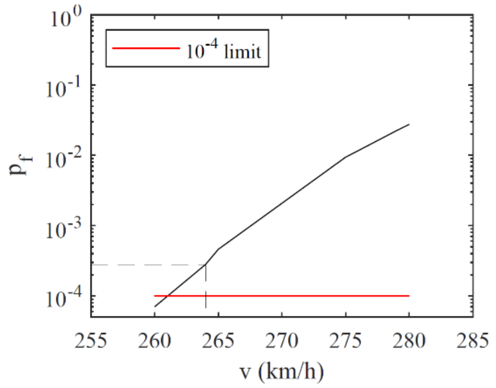

Figure 8: Refined assessment of critical speed (*n*=100.000) – HSLM-A3

	$n = 1000$		$n = 5000$		$n = 100000$	
Load model	Critical speed	p_f	Critical speed	p_f	Critical speed	p_f
HSLM-A1	290 km/h	10×10^{-4}	265 km/h	2×10^{-4}	264 km/h	1.9×10^{-4}
HSLM-A2	360 km/h	10×10^{-4}	280 km/h	2×10^{-4}	—	—
HSLM-A3	260 km/h	10×10^{-4}	265 km/h	2×10^{-4}	264 km/h	2.8×10^{-4}
HSLM-A4	280 km/h	20×10^{-4}	275 km/h	2×10^{-4}	276 km/h	1.4×10^{-4}
HSLM-A5	280 km/h	10×10^{-4}	285 km/h	2×10^{-4}	—	—
HSLM-A6	290 km/h	10×10^{-4}	290 km/h	2×10^{-4}	—	—
HSLM-A7	300 km/h	10×10^{-4}	300km/h	2×10^{-4}	—	—
HSLM-A8	320 km/h	10×10^{-4}	310 km/h	2×10^{-4}	—	—
HSLM-A9	320 km/h	30×10^{-4}	310 km/h	2×10^{-4}	—	—
HSLM-A10	330 km/h	10×10^{-4}	325 km/h	4×10^{-4}	—	—

Table 3: Critical speeds

Load model	Critical speed	a_{Ed}	Scenario	γ_{bt}
HSLM-A1	264 km/h	3.30 m/s^2	F1	2.12
HSLM-A2	280 km/h	3.25 m/s^2	F1	2.15
HSLM-A3	242 km/h	5.26 m/s^2	F2	1.33
HSLM-A4	276 km/h	4.92 m/s^2	F2	1.42
HSLM-A5	285 km/h	5.98 m/s^2	F2	1.17
HSLM-A6	290 km/h	5.77 m/s^2	F2	1.21
HSLM-A7	300 km/h	5.89 m/s^2	F2	1.19
HSLM-A8	310 km/h	5.44 m/s^2	F2	1.29
HSLM-A9	310 km/h	5.95 m/s^2	F2	1.18
HSLM-A10	264 km/h	6.35 m/s^2	F1	1.10

Table 4: Partial safety factors

4 Conclusions and Contributions

Following the presented methodology, the partial safety factors for the load models other than the HSLM-A1 and HSLM-A2 are lower than 1.5. That is, for those load models, the design acceleration only has to be majored by a factor less than 1.5 to ensure that the probability of exceeding the limit criterion is less than 10^{-4}. In other words, for those results, the design acceleration considering these load models does not need to be limited to 7/2=3.5 m/s^2. Contrariwise, for the first two Universal Trains, critical speeds correspond to design acceleration maxima below 3.5m/s2, and therefore the resulting partial safety factors are above 2.0. It is noted that there is margin for improvement in the EN 1990-Annex A2 regarding the clarification of the acceleration limit and of the implied partial safety factor.

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