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Influence of track-bridge interaction in the comparison of load model HSLM-A vs. conventional and regular high-speed convoys derived from EN1991-2

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Abstract

This paper investigates the influence of track-bridge interaction (TBI) in the comparison of dynamic effects of actual (or envisaged) high-speed trains vs. load models prescribed by national or international standards. The focus is set on concentrated load models of high-speed vehicles travelling over simple bridges, following a general methodology previously presented by the authors (*Museros, P., A. Andersson, V. Martí, and R. Karoumi. Dynamic behaviour of bridges under critical articulated trains: Signature and bogie factor applied to the review of some regulations included in en 1991-2. J. Rail and Rapid Transit, 1–21, 2020*). For such comparison, two variables named *exceedance* (in amplitude) and *required speed increase* were used to analyse whether a real train is or is not duly covered by a model prescribed in a standard. Two particular cases are analysed here of one *regular* and one *conventional* train, both compliant with Annex E in EN1991-2, which are compared vs model HSLM-A in two slab-type bridges of spans 10.8 m and 25.2 m. Two different mathematical models are considered for each bridge, characterised by very different levels of refinement. The frequencies of the bridges are selected intentionally low, close to the limit of the frequency band defined in EN1991-2, section 6.4.4; therefore, it is of interest to note how the resonant speeds are also low, particularly for the second example. The main conclusions of this work are two. First, the difference between the response predicted at mid-span by a simple beam model (1-DOF) and a much more elaborated 2D FE model with TBI is very small (in the second example, totally negligible). The reason behind it is that resonances of the fundamental mode prevail in the envelope response at mid-span of these two examples, as in the majority of simply-supported bridges. The remarkable similitude is obtained particularly because damping is derived from free vibration in the FE model, and subsequently assigned to the 1-DOF model. Secondly, since the responses obtained with both models are so similar, their influence in the comparison of real trains vs. HSLM-A in the cases analysed here can be considered negligible in engineering terms, *i.e.* the areas of non-coverage predicted by the two models are essentially the same. Therefore, TBI has no influence in the two bridges studied in this paper. Further research is required to establish whether this relevant finding could be retained as a general conclusion.

Keywords: Load models, high-speed train, dynamics of railway bridges.

1 Introduction

Comparison of the dynamic effects of new railway vehicles against current and previous load models is essential to guarantee that the existing bridges initially designed according to such load models will preserve their security and functionality. However, the bases for carrying out the comparison are often susceptible to debate because a part of the response predicted by mathematical models can of course be model-dependent. Therefore, there is no unambiguous response to the question of what should be the preferred method for comparing the dynamic effects of railway vehicles and load models.

This work investigates the influence of track-bridge interaction (TBI) in the comparison of railway vehicles vs. load models. Considering two representative examples of simply supported (S-S) structures, two high-speed trains that exceed the dynamic effects of load model HSLM-A from EN1991-2 are analysed. Their levels of *exceedance* and *required speed increase* are computed following the methodologies established by Museros et al. [1]. The comparative analysis is first carried out with a simple beam model; subsequently the comparison is repeated using a more sophisticated Finite Element (FE) model that includes the ballast, track and sleepers. Such coupled model has been developed in the framework of an ongoing research project: **Grant RTI2018-093621-B-I00**, "*Simulación integrada no lineal del comportamiento estructural de puentes ferroviarios de fábrica ante acciones dinámicas y nuevos requerimientos de tráfico*", **funded by MCIN/AEI/ 10.13039/501100011033 and by "ERDF A way of making Europe"**. The authors want to express their gratitude to the financial support provided by the aforementioned entities.

From the two representative examples analysed here, it will be investigated whether the mathematical model of the bridge plays a significant role in the conclusions obtained from the comparison of the railway vehicles vs. HSLM-A load model. While some discrepancies can be expected between a basic model of a S-S beam and a considerably more elaborated physical representation of the bridge, we will try to elucidate whether such discrepancies are large enough as to imply the need to consider TBI when comparing actual trains against load models.

2 Methods

Two mathematical models are considered. Both include the vehicle as a series of moving loads of constant value. First, a S-S, Bernoulli-Euler beam model where TBI is ignored is considered as a reference; this is the simplest possible model (only one d.o.f., first bending mode).

Secondly, a bi-dimensional (2D) FE model idealises the bridge as a solid slab of constant, rectangular cross-section, with the ballasted track represented according to Fig. 1. The track model is a three-layer 2D discrete model, as proposed by Zhai et al. [2]. It features a continuous beam that represents the two rails, and a set of discrete spring-dashpot and concentrated masses equally spaced along the track at the sleeper's positions, which represent the platform elements: rail pads (K_p, C_p), sleepers (M_s), ballast vibrating masses below the sleepers (M_b) with their corresponding stiffness/damping (K_b, C_b) and subgrade layer (K_f, C_f). Also, additional shear dissipation and stiffness between ballast vibrating masses are included (K_w, C_w). A track extension of 6 m exists before/after the beam (10 × sleeper distance). The equations of motion of the complete system are integrated in time domain applying Newmark's constant acceleration scheme.

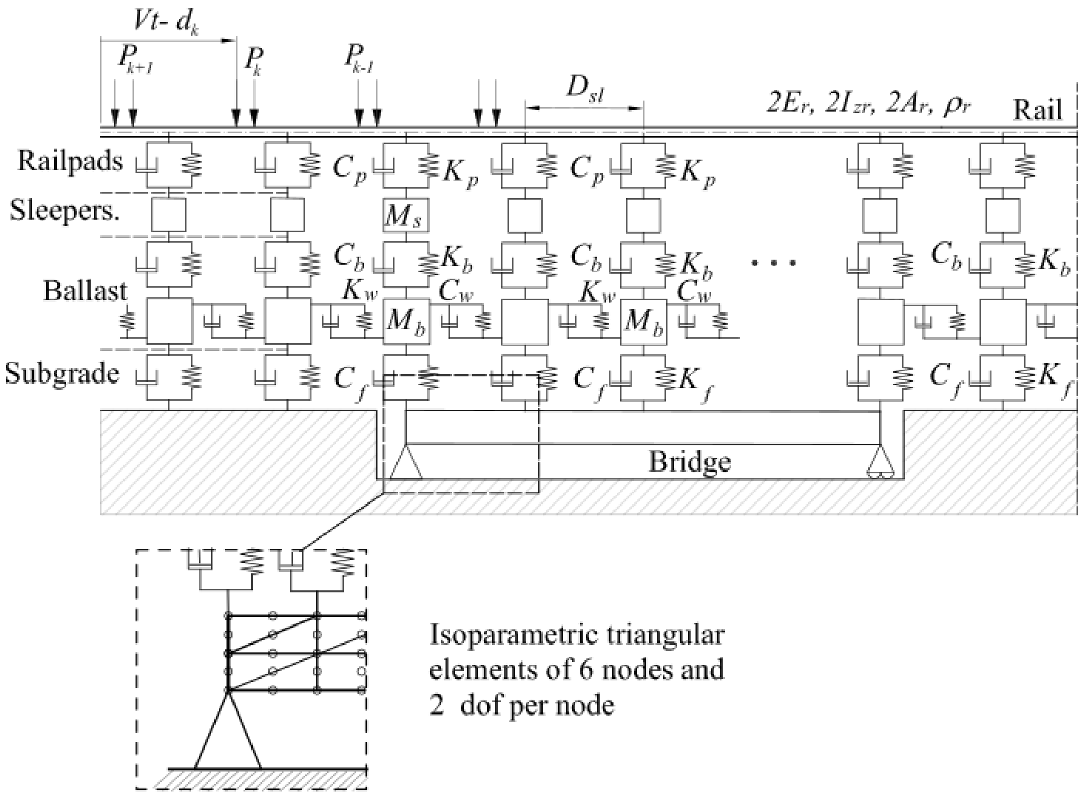

Figure 1. Bi-dimensional FE model of the track and bridge (TBI model).

The approach followed here consists in a comparison of dynamic effects (vertical displacement and acceleration at mid-span) at the bridge mid-span in two representative cases (Cases A and B), considering the two above mentioned models in each case. Those two cases have been selected after verifying that two particular trains are not well covered by HSLM-A at some speeds. Those trains are called "real" trains (RTs) in what follows. The parameters used to detect whether there is a lack of coverage or not are the *exceedance* in amplitude, and the *required speed increase* for the HSLM-A to actually cover the RTs, as they were introduced in previous works from the authors [3].

The RTs considered here comply with Annex E in EN1991-2 [4]; we follow notation and symbols in there. The axle distances in their (leading and trailing) power cars have the exact same distances of HSLM-A. For Case A we use a regular train with a total of 46 axles; wheelbases are D=11.0 m, D_{ic}=11.0 m; e_c=9.0 m. For Case B we use a conventional train with 56 axles; wheelbases are D=26.0 m, d_{BA}=2.5 m; d_{BS}=8.6 m. Axle loads are 170 kN in all cases.

3 Results

The two cases analysed are summarised first. Case A is described [and data for case B follow in square brackets]: Case A[B] is a short [medium span] bridge of span L=10.8 [25.2] m, rectangular cross section of 5×0.6023 [5×1.413] m^2, with mechanical properties: E=35GPa, ν=0.2, ρ=2710 [2590] kg/m^3. Density is slightly higher than for reinforced concrete, in order for the mass of ballast not included in the dynamic track model to be considered as dead load. Damping for the FE mesh is computed via Rayleigh's approach, assigning 1.64% [0.5%] to the first two theoretical frequencies of the bare beam (7.41 [3.49] and 29.6 [13.96] Hz). Total linear mass (used in the 1-DOF models) is 10030 [20160] kg/m; the first natural frequency of the FE model is 8.84 [3.77] Hz. Properties of the track are found in [5], except for M_b=752 kg.

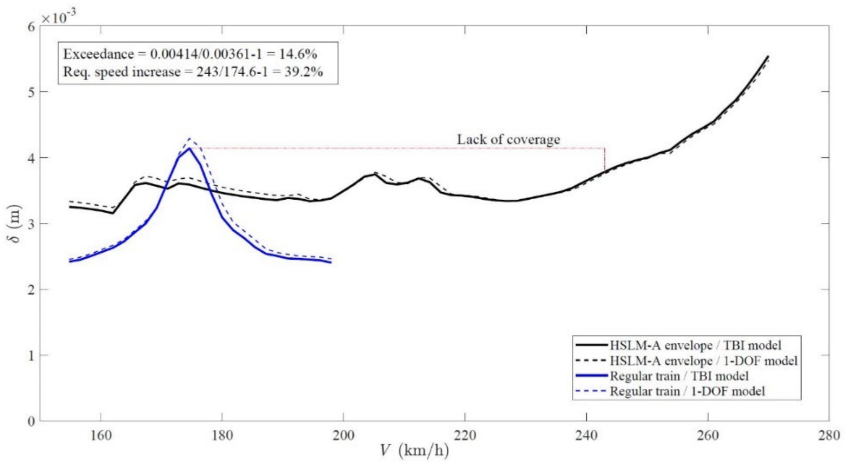

Figure 2. Vertical displacements at mid-span of the L=10.8 m bridge. Comparison of responses, and exceedance/speed increase values.

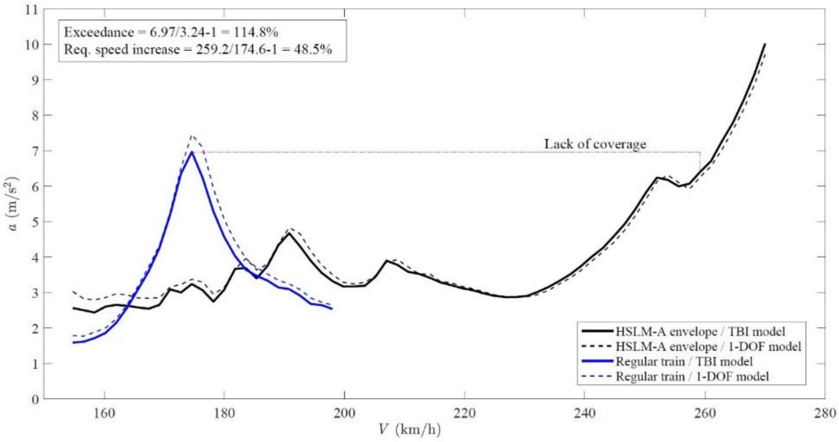

Figure 3. Vertical accelerations at mid-span of the L=10.8 m bridge. Comparison of responses, and exceedance/speed increase values.

Figs. 2 and 3 compare the response of HSLM-A vs the regular train for Case A, for both the 1-DOF model and the FE model. It is worth to remark that damping in the 1-DOF is assigned so as to reproduce the same free-vibration, logarithmic decrement than in the FE model. The agreement between both models is excellent, with slightly higher response in the 1-DOF model, most likely due to the lack of load distribution through sleepers and ballast. Clear ranges of non-coverage are visible and very similar in both models, which is the important result that try to elucidate here: Case A shows that for S-S spans around 10 m, TBI will not have a significant influence in the comparison of this RT vs. HSLM-A.

Fig 4. leads to the same conclusions for the medium-span Case B, limited to the acceleration (vertical displacements have almost null exceedance in this case). Case B is somewhat more theoretical, since a high mass is considered in the deck along with small damping; however, the results would probably be not too different with a lower mass and higher damping (a voided slab or beam bridge).

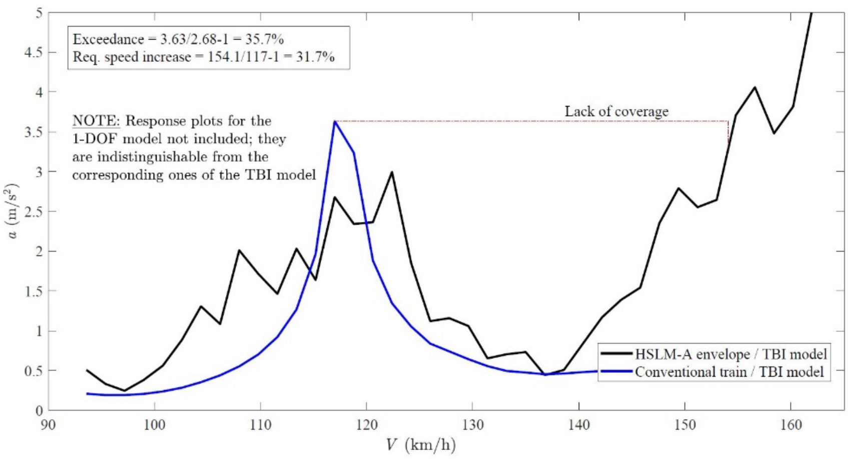

Figure 4. Vertical accelerations at mid-span of the L=25.2 m bridge. Comparison of responses, and exceedance/speed increase values.

4 Conclusions and Contributions

The main novelty of this paper is the investigation of the influence of track-bridge interaction (TBI) for the comparison of the dynamic effects of constant, concentrated load models of high-speed railway vehicles travelling over simply-supported bridges.

The general methodology for such comparisons was presented previously by the authors in [3], where two variables named *exceedance* (in amplitude) and *required speed increase* were used to analyse whether a real train is or is not duly covered by a model from a standard. Tolerance 10% is accepted for the exceedance, as explained in [3], which determinates the length of the areas of non-coverage. Particular cases are analysed here of one regular and one conventional train, both compliant with Annex E in EN1991-2, which are compared vs model HSLM-A in two slab-type bridges of spans 10.8 and 25.2 m.

The frequencies of the bridges are selected intentionally low, close to the limit of the frequency band defined in EN1991-2, section 6.4.4; therefore, it is of interest to note that the resonant speeds are also low, particularly for the second example. The mass of such second example could probably be lower that considered here, and its frequency therefore somewhat higher (a voided slab, or beam bridge); therefore, a low damping value has been assigned to it in order to partially compensate such effect.

The main conclusions of this work are two. First, the difference between the response predicted at mid-span by a simple beam model (only 1-DOF) and a much more elaborated FE model with TBI is very small (in the second example, totally negligible). The reason behind it is that the resonances of the fundamental mode prevail in the envelope response at mid-span of these two examples, as in the majority of simply-supported bridges. The remarkable similitude is obtained particularly because damping is derived from free vibration in the FE model, and subsequently assigned to the 1-DOF model.

Secondly, since the responses obtained with both models are so similar, their influence in the comparison of real trains vs. HSLM-A in the cases analysed here can be considered negligible in engineering terms, *i.e.* the areas of non-coverage predicted by the two models are essentially the same. Therefore, TBI has no influence in the two bridges studied in this paper. Further research is required to establish whether this relevant finding could be retained as a general conclusion.

Acknowledgements

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