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Numerical evaluation of the mid-span assumption in the calculation of total load effects in railway bridges

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ABSTRACT

Maximum load effects in simply supported railway bridges traversed by trains are generally investigated at the mid-span section. However, this assumption is not necessarily correct. The true maximum load effect might occur at some other bridge section and its magnitude could be significantly greater. This paper quantifies the underestimation of the load effects as a result of exclusively considering the middle section, with special emphasis on resonant situations. A 2D numerical model of a vehicle–track–bridge system was used to evaluate different vehicle velocities, bridge properties and track irregularity conditions. The error due to the mid-span assumption depends on the particular case considered but can be related to the relative energy content of the higher modes of vibration. The results show that the error is greatest for accelerations, smaller for bending moments and is almost negligible for displacements.

1. Introduction

In bridge engineering, the analysis of the bridge response due to traffic loading is one of the key aspects to consider at the design stage or during the assessment of existing structures. It is generally assumed that the mid-span section features maximum displacements, bending moments and accelerations, in the case of single span structures. This assumption is not necessarily correct. Each of these Load Effects (LE) are the result of the combined contribution of static and dynamic components. First, regarding the static contribution, it is well known that for vehicles with an asymmetrical axle load distribution the maximum LE may be near but not necessarily at mid-span [1]. This can be calculated invoking the Müller-Breslau principle [2]; an example of its application can be found in [3]. In addition, when considering also the dynamic effects, the location where the actual maximum LE occurs might be far apart from the mid-span region because of the contribution of higher modes of vibration. As a result, the true maximum LE experienced by the bridge might be significantly larger than its mid-span counterpart.

Bridge design codes prescribe that the total LE (i.e. static plus dynamic) should be calculated from the static design load and the result factored by a Dynamic Amplification Factor (DAF) or equivalent factor. These factors have been calibrated from extensive measurement campaigns. Generally, measurements were performed only at the locations where the extreme LEs were expected. It is not feasible to monitor every section of the bridge, due to budget constraints, limited number of sensors and other technical questions. On the other hand, structures should be designed to withstand the maximum LE that it will be subjected to, regardless its location. Further, as specified in the Eurocode [4] the maximum dynamic response should be considered at any particular point in the structural element. Thus, the assumption that maxima at mid-span are comparable to the actual maximum LEs anywhere on the structure needs to be assessed.

From a theoretical perspective, the mid-span assumption can readily be checked using available closed form solutions of the moving load over beam problem. For instance, evaluating the load effects along the whole bridge using the formulation derived by Frýba [5] it is possible to see that the maximum oscillates around the mid-span section [6]. In [7], the analytical expressions of the maximum vertical displacements are derived, concluding that the mid-span assumption leads to errors that are particularly small for the critical speeds. These results refer only to beam displacements and do not mention possible errors in other load effects such as bending moments or accelerations. Esmailzadeha and Jalili [8] study the problem for the particular case of a 6-DOF vehicle traversing the bridge. They found that the maximum dynamic deflection occurs at the vicinity of the bridge centre, whereas the

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maximum bending moment was found to be at ±20% of the mid-span.

For the particular case of road bridges, several recent studies have evaluated the mid-span assumption and its consequences on the calculation of total maximum bridge response. A new factor is proposed in [9], namely the FDAF that stands for Full bridge length DAF, which extends the definition of DAF to any section of the bridge. Also [10] introduces the Dynamic Increment Factor (DIF) that quantifies the maximum difference between static and dynamic responses throughout the whole bridge. The study in [9] suggests that for typical 5-axle trucks on medium span bridges the increase in bending moment is up to 5%. The analysis of vehicle meeting events on the bridge [11] shows that even though large differences are observed, the values decrease considerably with increasing static load. Thus, for characteristic loading events, like those studied in [12], it is estimated that the mid-span bending moment should be increased by 2% to account for the whole bridge length. On the other hand, several studies [13,14] have shown that amplification factors prescribed by design codes for road bridges tend to be over-conservative. Thus, it can be concluded that for the particular case of road bridges the small load effect underestimation due to the mid-span assumption is covered by the inherent safety coefficients in the design process.

On the other hand, the behaviour of railway bridges due to the passage of trains is not comparable to the responses of road bridges. In general, the dynamic effects in railway bridges can be considerably greater than those of road bridges, in particular for high-speed railway lines. Furthermore, the regular load configuration (long trains composed of wagons with identical axle spacings) results in loading frequencies that can lead to resonant behaviour for certain critical speeds. This is a fundamental difference from road bridges, which are loaded by traffic with a mixture of axle configurations and separated by arbitrary gaps between vehicles. Even multiple vehicle events in road bridges do not produce resonant behaviour to the same extent as in railway bridges. In addition, railway bridge design codes, like the Eurocode [4], prescribe maximum values of deck acceleration to avoid ballast instability or derailment of the traversing train. It has been shown that the maximum deck acceleration criterion will in most cases be the decisive design factor [15].

Therefore, it is necessary to check the mid-span assumption on railway bridges separately. The conclusions drawn from road bridge studies are not applicable because of the higher dynamics in the response, the possibility of resonance and special attention needed to maximum deck accelerations. There exist only few studies that evaluate the maximum LE along the whole bridge. The analytical investigation of a beam traversed by successive loads in [16–18] show that the maximum vertical accelerations is not necessarily at mid-span indicating that higher modes cannot be neglected. In [19] a series of existing portal frame bridges are modelled obtaining maximum accelerations up to 30% compared to those of mid-span. Thus, the effect of the mid-span assumption has not been sufficiently investigated for railway bridges in general, and at resonance in particular.

This paper aims to quantify the underestimation of total LEs that results from exclusively considering the mid-span section of the structure. This is achieved by means of parametric studies and Monte Carlo analysis with a 2D numerical model that describes the behaviour of the vehicle, track, ballast and bridge. Three LEs are investigated, namely displacements, bending moments and accelerations for simply supported railway bridges, with special emphasis on resonant responses. Representative bridge properties are chosen to describe medium span bridges (10–40 m) in general. Moreover, the study not only examines responses during forced vibration, but also during free vibration, since maximum LEs might occur after the train has left the bridge [17].

The document starts with the description of the numerical model used in the study. Then it continues with the analysis of one particular example that highlights the error of the mid-span assumption. Next, a parametric study of the traversing speed and bridge properties and a Monte Carlo analysis of the track irregularities shows the consequences of the mid-span assumption and the influences of these factors. The last section is a discussion where the authors try to identify the circumstances where big errors are obtained when assuming that the maximum LE is at mid-span.

2. Model description

The numerical model used in this study incorporates the behaviour of train, ballasted track and bridge (Fig. 1). The train has been represented as a succession of individual vehicles each consisting of a multibody system with 10 Degrees Of Freedom (DOF). In each vehicle, the main body and the two bogies are represented as rigid bars and the four wheels as lumped masses. The primary and secondary suspensions are modelled as spring and dashpot systems linking the wheels to the bogies and the bogies to the main body respectively. This vehicle model is extensively used in related literature, which correctly describes the main components. The track is modelled as a beam resting on three layers of periodically spaced sprung mass systems as recommended by the UIC in [15] to check the design requirements of railway bridges for dynamic traffic loads. The beam represents the rail, whereas the sleepers and ballast are modelled as lumped masses. These elements are connected to each other by spring and dashpot systems representing the viscoelastic behaviour of the pad, ballast and sub-ballast. The bridge is modelled as an Euler–Bernoulli beam using a Finite Element Model (FEM) discretization with two elements between consecutive sleepers and proportional damping (Rayleigh) such that the desired damping ratio is defined at the first and second frequencies of the structure.

The three sub-systems (vehicle, track and bridge) are coupled together, which is achieved by updating the coupling terms of the equations of motion at every time step. This correctly accounts for the Vehicle–Bridge Interaction (VBI) and the inertial, centripetal and Coriolis forces that develop as the wheel masses move over the rail. The model does not account for wheel–rail detachment and impact and imposes permanent contact with the rail. Additionally, the presence of track irregularities are included in the model as well. The numerical integration is accomplished with the Newmark-β method and a sufficiently small time step. It is important to note that the planar model presented here is valid only for the analysis of bridges having beam-like behaviour, since no torsional effects or 3D loads can be included. The reader can refer to [20] for a complete description of the numerical model together with its validation. In addition, similar models have been described in literature before, being good examples [21,22].

The vehicle configuration adopted in this study is the ICE 2 train, which is composed of 1 locomotive at each end of the convoy and 8 passenger wagons. This high-speed train has been investigated for speeds up to 400 km/h. The dimensions and mechanical properties for the train have been taken from [23]. The track is longer than the bridge in order to include an approach and exit distance for the travelling vehicle. In particular, the approach is 100 m long to achieve dynamic equilibrium of the vehicle before it enters the bridge. The exit distance is different for every bridge and speed combination and is such that the free vibration response includes at least two cycles of the first mode of vibration of the bridge. The particular mechanical properties of the track can be found in [24]. The rail is modelled as a standard UIC60 rail for properties found in [25] and sleeper spacing of 0.6 m. Thus the bridge mesh consists of 0.3 m long beam elements. The bridge damping is