Damping from bearing hysteresis in railway bridges

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ABSTRACT: The hysteretic behavior of bridge bearings contributes significantly to the overall damping of certain bridges. In railway bridge dynamics, the ability to dissipate energy is a key property, governing the resonant amplitudes of vibration. However, the efficiency of these damping mechanisms is coupled to parameters such as construction height and support stiffness. This paper presents an analysis of three different railway bridges, where the hysteretic behavior of the bearing mechanisms is modelled using the classical Bouc-Wen model. Both sliding and roller bearings are assumed to have similar backbone curves, the parameters of which have been chosen on basis of simple reasoning. Experimentally determined backbone curves for these mechanical subsystems are not available today. Instead, the performed analysis is validated by means of measurements of vertical acceleration in the bridge decks. The results show that the hysteretic behavior of the bridge bearings could explain the non-linear effects which can be seen in the frequency and amplitude modulation of the free vibrations in the studied bridges.

KEY WORDS: Railway bridges; Dynamics; Bearings; Damping; Hysteresis.

1 INTRODUCTION

The damping of common railway bridges originates from several different sources, i.e. material, radiation and frictional damping. Material damping is quite well understood, but constitutes only a small fraction of the total structural damping. Radiation damping is caused by soil-structure interaction and is essentially the result of energy being transported away from the structure in the form of elastic waves in the surrounding soil materials. Many different sources of friction exist in a railway bridge. For example, friction may arise in bearings, joints and between the components of the track superstructure and the amount of energy which is dissipated by these mechanisms is highly dependent on the amplitude of vibration.

Three beam-like railway bridges from which measurements of free vibrations after train passages were available have been studied. The continuous wavelet transform (CWT) was used to determine the variation in the natural frequency and modal damping ratio with amplitude of vibration from the measured free vibrations of the three bridges using the methodology presented in reference [1]. This revealed that the natural frequencies decreased with increasing amplitude of vibration while the corresponding modal damping ratios increased. An explanation to this observation is suggested in reference [2], where the Bouc-Wen (BW) model was used to model the rate-independent hysteresis of bridge roller bearings and the longitudinal track resistance of the case 1 bridge (see section 3). It was found that the bearing hysteresis can give rise to a large contribution to the modal damping ratio at certain amplitudes of vibration.

Although considerable efforts were made to reduce the uncertainties in the modelling presented in reference [2], some open questions remain, mainly with respect to the modelling of the bearings. In the author’s opinion, these issues can only be resolved by means of full-scale tests with amplitudes of vibration comparable to a state of train-bridge resonance. Nevertheless, in this study, the available data has been exploited as far as possible in order to approximately assess the damping capacity of bridge bearings.

This paper presents a qualitative, theoretical study of the energy dissipated by bridge bearings in freely vibrating beam-like railway bridges. The bridge superstructures were modelled using Euler-Bernoulli finite elements, the foundations were modelled using linear springs and viscous dashpots and the bearings were modelled using macro-elements based on the BW model. The BW model parameters were estimated based on the available literature and adjustments of parameters against empirical bridge deck accelerations. The analysis is performed assuming that the environmental variables are constant and have no influence on the dynamic response. This is stressed because in the cold climate of Sweden for example, the dynamic properties of railway bridges may vary considerably due to the seasonal variation in the environmental variables, see references [3] and [4] and the references therein.

The paper is organised as follows. Section 2 gives a brief overview of the most common bridge bearings in the Swedish common practice and a short introduction to the BW model. In section 3 a theoretical study of the influence of bridge bearings on the free vibrations of three typical Swedish railway bridges is described and section 4 holds the results of that study. It is shown that for some bridges, the damping caused by the bearings can give a considerable contribution to the total structural damping and that it seems to be possible to design bridges so that the dissipation of energy at the bearings is exploited in an optimal way. These implications are discussed in section 5, together with some suggestions regarding the continued research within this field.
2 BRIDGE BEARINGS

In the present context, two types of bridge bearings are commonly used: roller bearings and pot bearings. Roller bearings are mechanical devices which allow an almost linear motion in one direction, for very small displacements. Essentially, they consist of a roller; guided between two plates. Such a bearing is shown in Figure 1, where the roller is light grey, the plates are green and the red parts are guiding devices.

Pot bearings (see Figure 2) are more complicated devices, but they can be used to obtain almost perfectly linear guides in one or two directions. Also, the rotation is approximately free in two directions, enabling very flexible designs for optimal constraints between the sub- and superstructure. A pot bearing consists of a piston and a pot, in which a rubber plate is confined. The confined rubber is almost incompressible and therefore, it behaves much like fluid when it is pressurized.

This enables the rotations of the bearing. On top of the piston, a plate is mounted, which can be fixed to the piston, or movable in one or two directions. The translational motion is unconstrained, apart from the friction behaviour between a polytetrafluoroethylene (PTFE) sheet and a polished steel surface. The frictional properties of such contacts have been studied by Dolce et al. [5], among others, in the context of seismic isolation. Their study does not comprise a rotational mechanism in the bearing as in the present case, but nevertheless provides extensive studies of the most important factors affecting the frictional properties of steel-PTFE contacts.

In both rolling and sliding contacts, an initial resistance to motion exist. In rolling contacts, this is sometimes referred to as “pre-rolling resistance” [6] and in sliding contacts it is referred to as “micro-slip” [7]. In common for both types of contact mechanisms is that they give rise to rate-independent hysteresis. In reference [2], a qualitative study is presented which shows that due to this initial resistance, the modes of vibration of certain railway bridges have two different states; (1) at very small amplitudes of vibration, the movable bearings are fixed and (2) at somewhat larger amplitudes of vibration the bearings are movable. In between these states, a transition zone exists, where the natural frequency and the modal damping ratio varies smoothly. The natural frequencies of these two states are rather simple to estimate since they correspond to either fixed or movable bearings, but the damping ratio varies considerably during this transition zone.

2.1 Modelling of bridge bearings by means of Bouc-Wen models

Clearly, a detailed modelling of the bearing mechanisms leads to a rather complicated computational task. In the case of roller bearings, one important issue lies in the modelling of the contact stresses which need to be accurately modelled in order to obtain a correct behaviour. However, plastic deformations on a very small scale as well as other phenomena such as wear and corrosion may also have a certain influence on the rolling resistance of such bearings.

For pot bearings, the contact between their different parts needs to be accurately modelled and in addition to the highly non-linear contact mechanisms, non-linear hyper-elastic material models are needed in order to capture the moment resistance of the bearings in an appropriate manner. Such details are not relevant for global dynamic bridge analyses. Instead, macro models of various forms can be used to model the bearing mechanisms in a phenomenological sense. An example of such a model can be found in reference [2], where the Bouc-Wen model was used to approximate the rolling resistance of the roller bearings of the case 1 bridge (see section 3).

The BW model was first suggested by Bouc [8] and later refined by Wen [9] and has been successfully used in various applications involving rate-independent hysteresis. In its classical, most simple form, the BW model consists of an elastic spring and a hysteretic element in parallel (see Figure 3). The response of the BW model is governed by the following force-displacement relation

\[ F(t) = ak_0u(t) + (1 - a)Dk_pz(t) \]  

where \( a \) is a model parameter relating the initial stiffness \( k_0 \) to the stiffness in the fully plasticised system \( k_p = ak_0 \), \( D > 0 \) is referred to as the plastic limit and the so called
hysteretic variable \( z(t) \), which is an internal variable, is governed by the differential equation

\[
\ddot{z} = \frac{\dot{u}}{\gamma} (1 - (\beta + \gamma \cdot \text{sgn}(u \dot{z})) |\dot{z}|^n)
\]

(2)

Here, \( \beta + \gamma \neq 0 \) and \( n > 0 \) are model parameters and the sign function is defined as

\[
\text{sgn}(x) = \begin{cases} 
-1 & x < 0 \\
0 & x = 0 \\
1 & x > 0 
\end{cases}
\]

(3)

The classical Bouc-Wen model was implemented as a user-defined element in the commercial finite element (FE) code ABAQUS.

Figure 3. The classical Bouc-Wen model.

3 CASE STUDIES

The CWT can be used to determine the variation in natural frequency and modal damping ratio from free vibration data for a given mode of vibration [1]. The result is two functions of the amplitude of vibration, hereafter referred to as frequency and damping functions. Such functions have been determined for the fundamental mode of vibration of each of the three studied bridges (see Figure 6b, d and f). Only comparatively small amplitudes of vibration (about 0.1 m/s²) were present in free vibration data obtained from regular train traffic on the studied bridges. However, all three cases show the same trend, i.e. decreasing frequency and increasing damping with increasing amplitude of vibration.

The case 1 bridge (see Figure 6a) is a steel-concrete composite bridge carrying one ballasted track. It has a span length of 36 m. Details regarding this bridge can be found in references [2] and [1]. The frequency and damping functions of the free vibrations measured on this bridge are shown in Figure 4b. The case 2 bridge is a post-tensioned concrete beam bridge in two spans of 24 m each, carrying 2 ballasted tracks. The case 3 bridge is a post-tensioned concrete beam bridge in three spans of length 18.5 m, 26.0 m and 18.5 m. The bridge carries one ballasted track. Photos of the case 2 and 3 bridges are shown in Figures 6c and 6e. Further details regarding these bridges can be found in reference [10].

3.1 Finite element modelling of the bridges

In reference [2], the case 1 bridge was studied assuming that the source of the nonlinearities evidently present in the frequency and damping functions of the fundamental mode of vibration was either the track superstructure or the bearings (or a combination of the two). One conclusion from that study was that the bearing mechanism gave a much larger contribution to the overall damping than the track superstructure. Also, the variation in natural frequency was found to be caused almost exclusively by the bearings. Figure 4 shows a conceptual sketch of the FE model of the case 1 bridge. The same modelling features were used to model the case 2 and 3 bridges. The models consist of Euler-Bernoulli beam elements representing the bridge superstructure in which the mass of the ballast was included as non-structural mass. The eccentricity between the neutral axis of the superstructure and the support points was modelled using rigid links. The supports were modelled by linear springs and dashpots which include an approximation of the flexibility of the foundations and the substructures as well as an approximation of the equivalent viscous damping corresponding to the radiation damping at the frequency in question, i.e. near the natural frequency. It should be noted that the material damping did not give a modal damping ratio equal to that estimated from the measurements. A considerable amount of radiation damping was needed to match the damping ratios estimated at very small amplitudes of vibration.

Figure 4. A conceptual sketch of the FE model of the case 1 bridge.

As mentioned in section 2.1, some research efforts are needed in order to determine the values of the Bouc-Wen model parameters for the bearing mechanisms more precisely. Thus, in this study, these parameters have been chosen so as to match the measurements reasonably well. Hence, the obtained results are of a qualitative nature, and should be considered as indications of what to expect from these nonlinear modes of vibration.

Figure 5. A sketch of the support points of the case 3 bridge.

The model parameters which are expected to have the largest effect on the coupling between the support point and the bridge superstructure via the bearing mechanisms are the longitudinal support stiffness and the eccentricity between the neutral axis of the superstructure and the support point. If the longitudinal support stiffness is weaker than the initial stiffness of the bearings, the bearing resistance to sliding or rolling will not be exceeded and an elastic displacement of the foundation occurs instead of non-linear bearing motion.
Figure 6. Photographs of the studied bridges and the frequency and damping functions of the fundamental modes of vibration of the respective bridges.
The case 2 and 3 bridges have integrated abutments. This feature was modelled by linear springs connected to the bridge deck ends forming, elastic constraints. The spring stiffness for the embankments was fixed at an estimated value of 0.1 GN/m in the transversal direction and 1.0 GN/m in the vertical and longitudinal directions. The radiation damping at the embankments was ignored. However, this constraint was found to have a large influence on the free vibration response of the case 3 bridge (see Figure 8).

4 RESULT

Figure 7 shows the theoretical results for the three bridges in terms of the natural frequency and the modal damping ratio of the first vertical bending mode, as function of the amplitude of vibration. The natural frequencies decrease monotonically while the damping ratios increase drastically up to certain amplitudes of vibration and then decrease, apparently back towards the values obtained at very small amplitudes of vibration.

Clearly, the case 1 bridge, which has the largest variation in the damping ratio in the CWT analysis, rapidly decreases towards approximately 0.5% as the amplitude of vibration increases. Thus, in a state of train-bridge resonance, it is likely that the contribution from the bearing hysterisis is quite small for this bridge. The case 3 bridge however, did not show a very large variation in its damping ratio from the experimental results, but the theoretical model predicts a substantial increase in the damping ratio, which prevails over a much wider range of amplitudes of vibration.

5 CONCLUSION

It has been shown, in a qualitative sense, that the bearing mechanisms of three different railway bridges give rise to non-linear modes of vibration which vary between two states; (1) fixed bearings and (2) movable bearings. The transition zone between these two states consists of a fairly smooth variation of the natural frequency and the modal damping ratio of these modes of vibration. The damping ratio increases significantly during this transition and it appears as if one could design the support points of the bridge in such a way that the dissipated energy is maximized.

The results presented in this paper imply that certain railway bridges have a much larger damping capacity than that predicted using linear theories. Therefore, further research should be conducted in order to utilize this in cases where existing bridges are subjected to increased train speeds and to obtain guidelines for designers of railway bridges for high speed traffic.

A considerable research effort remains in order to be able to give practical guidelines regarding these mechanisms. Primarily, the model parameters for the Bouc-Wen models need to be determined in laboratory tests and the Bouc-Wen model used needs some further refinements to be useful in transient simulations of passing trains. Ultimately, full scale tests with amplitudes of vibration reaching at least 3 m/s² are needed to verify the theoretical modelling and give better input for the theoretical modelling of this behavior.

Since only the natural frequencies of the first 2-3 modes of vibration were available, all the model parameters could not be updated in a rigorous manner. Ideally, one would use operational modal analysis to determine the dynamic properties of several modes of vibration at small amplitudes of vibration and update all the linear components of the bridges using this data, assuming the bearings to be fixed. Then, the main uncertainties would be related to the non-linear bearing mechanisms alone.
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