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Experimental Analysis of the Vertical Track-Bridge Interaction in Railway Bridges with Ballasted Track

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Abstract

Realistic vibration predictions of railway bridges during high-speed train crossings require reliable dynamic input parameters for the applied calculation model. In particular, the dynamic stiffness and damping properties of the ballast superstructure for the mathematical consideration of the vertical track-bridge interaction significantly influence the generated calculation results. However, due to a striking scattering of model-related characteristic values available in literature, adequate and realistic consideration of the dynamic properties of the vertical track-bridge interaction in vibration predictions is associated with considerable uncertainties. This uncertainness illustrates the need to determine experimental-based and reliable characteristic values. For targeted and isolated research of dynamic characteristics of vertical track-bridge interaction, a unique large-scale test facility was developed at the Institute of Structural Engineering at TU Wien, which replicates a section of ballast superstructure on a railway bridge on a scale of 1:1 and excited to vertical movements. This contribution presents substantial results and findings from the experiments, focussing on the determination of the ballast superstructure's dynamic stiffness and damping values, which can further be implemented in practical applications for vibration calculations. In addition, the experiments are used to assess the currently applicable international and national standardised permissible structural accelerations for railway bridges during high-speed train crossings.

Keywords: railway bridges, dynamics, track-bridge interaction, experiments, damping, model design.

1 Introduction

A realistic and thus economical assessment of the compatibility of operating high-speed trains and bridges as critical points of the rail infrastructure requires a suitable calculation model with reliable model-related input parameters and, in addition, profound knowledge of the limit states that occur. Concerning mechanical bridge modelling, the considered stiffness and damping parameters of the ballast superstructure fundamentally influence the generated calculation results of vibration predictions. However, a large number of available models of the ballast superstructure and a striking bandwidth of related characteristic values make it difficult to consider its properties in vibration predictions for railway bridges reliably. Furthermore, EN 1990/A1 [1] prescribes limit values for the permissible vertical structural acceleration due to train crossing as a serviceability criterion, whereby a rather conservative level of 3.50 m/s² must not be exceeded. In contrast, the Austrian standard B 4008-2 [2] allows a significantly higher acceleration limit of 6.0 m/s² for existing bridges. Building on this background, this contribution addresses the experimental-based modelling of railway bridges, focusing on the ballast superstructure and the experimental identification of limit states occurring in it due to vertical movements. For this purpose, a special large-scale test facility was developed at TU Wien, which enables targeted and isolated research of the dynamic behaviour of the ballast superstructure. Before introducing the test facility (Section 2) and presenting the main results and findings (Section 3), the relevant background to this contribution is briefly discussed below.

Figure 1 shows the results of a dynamic calculation for one exemplary railway bridge (maximum acceleration in the centre of the bridge depending on the trains' crossing speed) using the simplest mechanical models for the passing train and the bridge (Fig. 1 left). The train is idealised as a series of moving loads representing the axle loads (moving load model - MLM) and the bridge as a shear-rigid Euler-Bernoulli beam (EB beam), characterised by the properties of span L, bending stiffness EI, mass per unit length μ and damping factor ζ. The EB beam summarises all damping properties of the supporting structure and the ballast superstructure in one value ζ. The exemplary dynamic calculation in Figure 1 uses three different damping factors (1.0 %, 2.5 % and 5.0 %) to illustrate the significant influence of the damping properties on the results.

Furthermore, the acceleration limits according to EN 1990/A1 [1] and B 4008-2 [2] are shown in Figure 1 (dashed in red). Concerning the background of the limit value of EN 1990/A1 [1] of 3.50 m/s², this limit is based on investigations of the settlement behaviour of the cyclically vertically loaded track on a rigid base (see [3] and [4]). This limit value thus represents the situation of the vertically loaded ballast superstructure in the open field and not the ballast superstructure located on the bridge and excited by the structure's movements. Concerning the vertically vibrating superstructure on the bridge, there are no specific investigations so far regarding critical limit accelerations at which destabilisation processes occur in the ballast bed. This circumstance emphasises the need for targeted experimental investigations of structure-induced vibrations of the ballasted track to verify the currently applicable limit values.

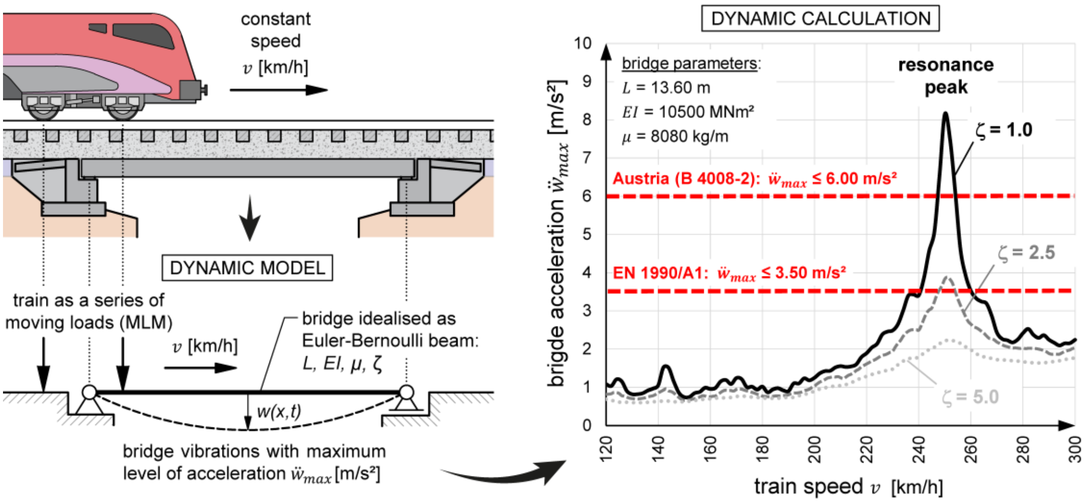

Figure 1: Example of dynamic calculation of a railway bridge and related acceleration limits according to international and national standard.

As mentioned above, the EB beam is the simplest mechanical model of the bridge that summarises the properties of the supporting structure and the ballast superstructure in one single beam. More complex models with separate consideration of supporting structure and ballast superstructure and, thus, increased modelling depth can provide more realistic results. In this respect, on the other hand, a large number of mechanical models with different levels of detail are available for dynamic calculations. Figure 2 shows three different vertically orientated two-dimensional models of the ballast superstructure with two, three or four layers. The vertical dynamic stiffness and damping properties of different components are considered by means of discrete spring-damper elements with the sleeper distance e_S in between them (related values for rail pad: k_{rp}, c_{rp} – ballast bed: k_{ba}, c_{ba} – sub-ballast: k_{sb}, c_{sb}). In addition to the models depicted in Figure 2, three-dimensional models may also be applied (e. g. in [5]), but these are only suitable for individual case analyses. In principle, increasing the modelling depth achieves more realistic results but also reduces the computational efficiency of the mechanical model used, making calculations for a large number of bridges – for example, necessary for the approval of new trains – uneconomical.

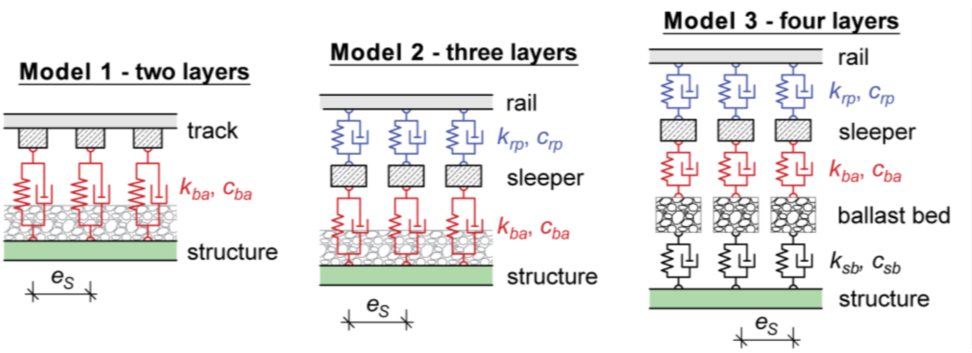

Figure 2: Models of the ballasted track with different levels of detail for consideration of vertical track-bridge interaction in the dynamic calculation.

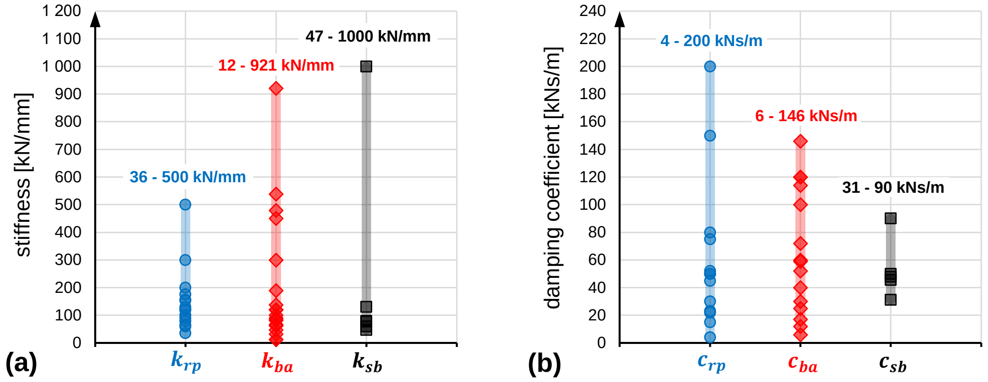

Figure 3: Model-related stiffness and damping values: (a) stiffness characteristics and (b) damping coefficients related to different models according to Figure 2.

However, the reliable, practical application of the models shown in Figure 2 is hampered by many model-related characteristic values to be found in the related literature, which are characterised by considerable scatter. A compilation of characteristic values found in the literature is given in [6] and [7]. The dynamic stiffness and damping characteristic values in [6] and [7] associated with the models shown in Figure 2 are illustrated in Figure 3. Figure 3 (a) contains the stiffness parameters, and Figure 3 (b) contains the damping parameters (discrete values in each case). Depending on the source used, the characteristic values scatter by a factor of 3 to 75, demonstrating the existing uncertainty when using more detailed models.

The scattering of the dynamic stiffness and damping parameters in Figure 3 illustrates the need for experimental-based dynamic parameters. In line with these requirements, the research with the large-scale test facility at TU Wien presented in this contribution aims to determine dynamic parameters to describe the vertical TBI based on experiments. Furthermore, the focus is also on the experimental identification of occurring destabilisation processes in the ballast superstructure due to structure-induced vertical vibrations. The investigation intends to verify the normative limit values for the permissible vertical acceleration due to train crossings.

In the following sections, the test facility is first presented (Section 2), followed by the main results and findings from the experiments on the dynamic behaviour of the vertical TBI (Section 3.1) and on the determination of model-related dynamic stiffness and damping parameters (Section 3.2). The investigations presented deal exclusively with the dynamic characteristics of the vertical TBI. Concerning the horizontal TBI, the authors of this contribution also carried out extensive experiments on a specialised large-scale test facility; the results for the horizontal TBI are presented in [8].

2 Large-scale test facility for targeted investigation of vertical track-bridge interaction

The test facility was developed as part of a research project focusing on investigating the dynamic behaviour of the vertical track-bridge interaction (TBI) and the related energy dissipation mechanisms. The facility, the operating principle, the experiments carried out, and the topics covered in this contribution are presented below.

2.1 Experimental setup and operating principle

The test facility (shown in Figure 4) consists of a 6.60 m long and 2.70 m wide steel trough, in which a section of ballasted track is installed over a length of 2.40 m on a scale of 1:1. In terms of its construction, the test facility corresponds to a typical single-track steel railway bridge with two main girders as the primary supporting structure, a deck plate in-between and transversely oriented girders (cross girders) in the area of the installed ballast superstructure. In terms of width, however, only half the cross-section of a single-track bridge is replicated, which means that the installed ballasted track (see Fig. 4 right) consists of one rail, four half concrete sleepers, 55 cm ballast bed and sub-ballast mat. The smaller superstructure section with only one rail axis was chosen in the design process because of the lower mass. The dynamic excitation takes place in a vertical direction, with hydraulic presses positioned below each of the two main girders of the steel trough and exciting the system with a pre-set combination of displacement amplitude and frequency. Due to the smaller mass of the installed ballasted track to be dynamically excited, the power capacity of the hydraulic excitation can be better utilised, allowing frequencies up to 25 Hz. On one end, the steel trough is supported on a fixed support (see Fig. 4, left), enabling rotary movement. Further, Figure 4 (left in blue) shows a pre-loading device that can be forced down on the track at two points and applies a static load on the track in the vertical direction. Thus, the pre-loading device enables the investigation of the vertically loaded track in addition to the situation of the unloaded track. The static axle load that can be applied is 125 kN, corresponding to half the axle load of an Austrian Railjet.

In addition, Figure 5 shows a longitudinal section of the test facility with the core elements and the linearised displacement and acceleration field of the steel trough and the rail below. The distance between the ballasted track and the fixed support is sufficiently large so that uniform vertical movement kinematics of the superstructure can be assumed in the oscillation state. Thus, the test facility simulates a representative section of a railway bridge that is dynamically excited on a 1:1 scale. The two hydraulic presses of the test facility are located in the symmetry axis of the integrated ballasted track (see Fig. 5). The steel trough is designed so that it behaves like a rigid body during the test and, regarding its movement kinematics, only performs a rotational movement around the fixed support.

Figure 4: Large-scale test facility for isolated research of vertical track-bridge interaction: overview (left) and integrated section of ballasted track (right).

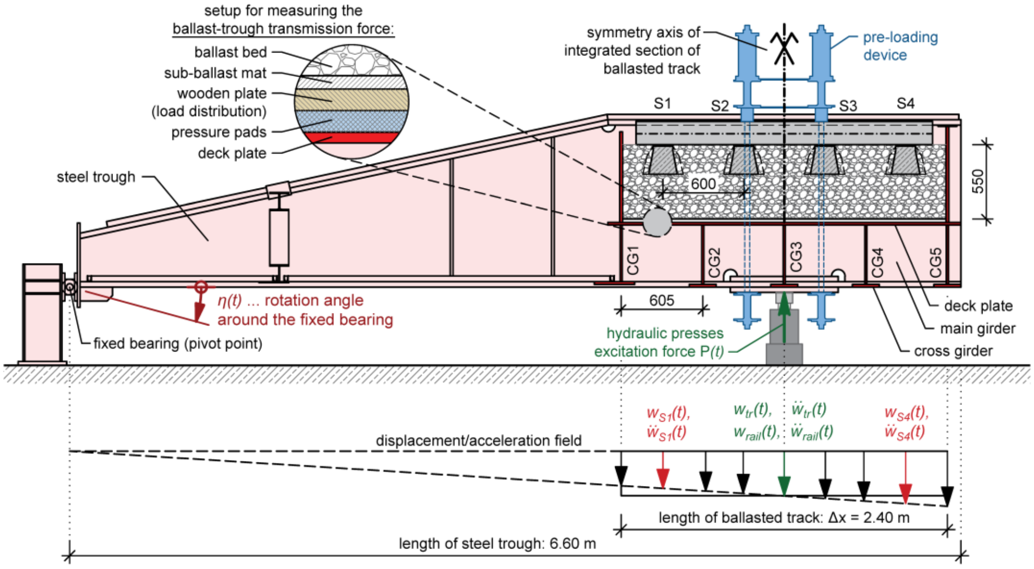

Figure 5: Longitudinal section of the test facility.

An essential subject of the experimental analysis of the vertical TBI using the test facility is the determination of model-related dynamic stiffness and damping parameters of the ballasted track for practical applications in bridge vibration predictions. Concerning this, it is crucial to know the transmission force between the ballast superstructure and the steel trough (or rather the deck plate) to determine characteristic values based on the experiments. Four water-filled pressure pads were installed between the ballast superstructure and the deck plate (see Fig. 6 left) to enable this transmission force to be measured during dynamic excitation of the test facility. In addition, to allow for better load distribution on the pressure pads, a wooden plate was installed between the pressure pads and the sub-ballast mat. In addition to Figure 6, Figure 5 schematically shows the setup for measuring the transmission force. The influence of the load-distributing wooden plate on the dynamic behaviour can be considered negligible due to its low mass. All tests were carried out both with and without installed pressure pads to quantify the influence of the installed water-filled pressure pads on the dynamic behaviour of the vertical TBI.

Figure 6: Setup for measuring the transmission force between ballast bed and deck plate: water-filled pressure pads (left) and sub-ballast mat before integration of the ballast superstructure (right).

2.2 Discussed topics and conducted test series

During the tests carried out with path-controlled dynamic excitation, the trough's displacement amplitude in the ballast superstructure's symmetry axis (see Fig. 5) and the related excitation frequency are precisely controlled. The test facility is in a stationary vibration state, whereby the achievable test spectrum covers frequencies between 2 and 25 Hz and vertical acceleration amplitudes of the trough of up to 10 m/s². The total number of all tests amounts to 1250, whereby tests were carried out under four different constellations depending on the load (loaded/unloaded track) and the installed pressure pads (with/without pressure pads):

- Constellation 1 (C1): with installed pressure pads, unloaded track
- Constellation 2 (C2): with installed pressure pads, loaded track
- Constellation 3 (C3): without installed pressure pads, unloaded track
- Constellation 4 (C4): without installed pressure pads, loaded track

Each of the four constellations includes tests between 2 and 25 Hz, so the tests primarily represent the frequency range of existing railway bridges (see [9]). Further, a considerable range of acceleration levels for each frequency is investigated with acceleration amplitudes of the trough of up to 10 m/s². In the following, two essential topics regarding the vertical TBI are discussed:

- Identification of destabilisation processes that may occur in the ballast superstructure as a result of vertical vibrations induced by the supporting structure
- Determination of model-related dynamic stiffness and damping parameters for the realistic implementation of the vertical TBI in mechanical bridge models

The identification of destabilisation processes is based on the tests without installed pressure pads (constellations 3 and 4); the tests with installed pressure pads (constellations 1 and 2) serve as a basis for determining the dynamic parameters, whereat the values are derived from experiments for an adaption of mechanical model 1 with two layers according to Figure 2.

3 Results

This section presents the main results and findings derived from the experiments conducted using the test facility subjected to dynamic excitation. First, the displacement and acceleration characteristics of the track and supporting structure and their interaction dynamics are discussed (Section 3.1), followed by the determination of dynamic stiffness and damping parameters of the ballasted track (Section 3.2).

3.1 Displacement and acceleration behaviour of vertical TBI

The basis for analysing the characteristics of vertical track-bridge interaction (TBI) is the tests without installed pressure pads (constellations 3 and 4). Figure 7 shows the results of the displacement and acceleration behaviour of the vertical TBI: Figure 7 (a) shows the amplitude of the relative displacement between the track (rail) and the steel trough $w_{rel,max}$ as a function of the acceleration amplitude of the steel trough $\ddot{w}_{tr,max}$.

Both amplitudes relate to the symmetry axis of the ballast superstructure (see Fig. 5). Figure 7 (b) also shows the relative displacement amplitude, in this case, as a function of the acceleration amplitude of the rail $\ddot{w}_{rail,max}$. Each point in the diagram represents one stationary vibration state defined by excitation frequency f and displacement/acceleration amplitude. The green markings represent the unloaded track, and the blue squares represent the loaded track, each containing the results with excitation frequencies between 2 and 25 Hz.

For both cases (loaded and unloaded track), the results show that the relative movements increase with increasing acceleration $\ddot{w}_{tr,max}$ and $\ddot{w}_{rail,max}$, with an approximately frequency-independent behaviour (low scatter along the regressions). Concerning the condition of the unloaded track, a disproportionate increase in relative displacements only occurs at accelerations above 7 m/s². However, there is no abrupt increase in the relative displacements but rather a continuous transition to larger relative displacements.

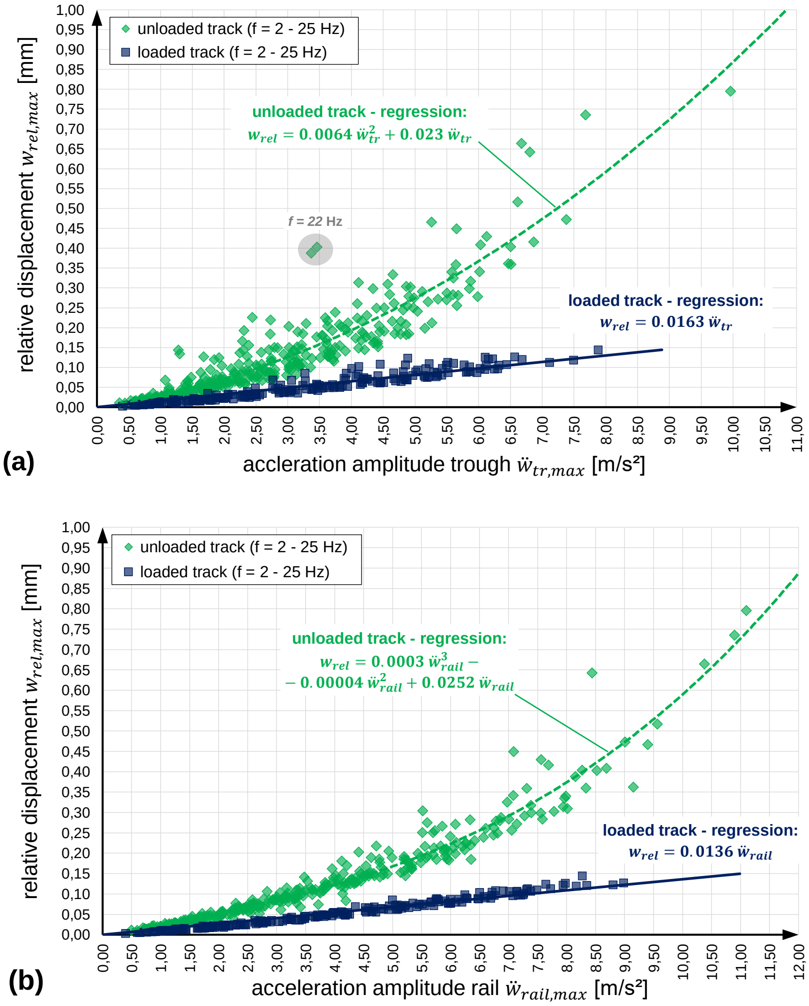

Figure 7: Results of the displacement and acceleration behaviour of the vertical TBI for the loaded and unloaded track: (a) relative displacement depending on the trough acceleration and (b) relative displacement depending on the rail acceleration.

In particular, the results in Figure 7 (b) for the unloaded track show a distinct course with a low scatter, approximated using a cubic regression. With regard to the situation of the loaded track, a frequency-independent (negligible scatter along the regression) and clearly linear relation between relative displacement and accelerations of rail and trough is shown for the entire test spectrum (blue marks and regression line in Fig. 7 (a) and (b)). Concerning the relation between the acceleration amplitudes of trough and rail, the acceleration amplitude of the rail $\ddot{w}_{rail,max}$ is consistently higher than the acceleration amplitude of the steel trough $\ddot{w}_{tr,max}$. Hence, the ballast superstructure has a vibration-amplifying effect in the analysed frequency spectrum of 2-25 Hz.

The essential finding from Figures 7 (a) and 7 (b) is that no clearly identifiable destabilisation process occurs in the ballasted track as a result of the vertical vibrations induced. The stationary vibration state during the individual tests is only of short duration (approx. 30 - 60 s). Conversely, concerning railway bridges, this means that temporary or short-term vibration peaks of the structure due to high-speed train crossings – which last several seconds – with amplitudes of up to 6 m/s² and beyond do not pose a hazard to the ballast superstructure.

3.2 Determination of characteristic values of the vertical TBI

This subsection addresses the determination of model-related dynamic characteristic values based on the conducted vibration tests. Here, the tests carried out with integrated pressure pads between the deck plate and sub-ballast mat to measure the transmission force between the trough and superstructure (see Fig. 5) serve as the basis (constellations 1 and 2). To determine the dynamic characteristic values, the ballast superstructure is considered an isolated and thus independent vibration system with the relative movements $w_{rel}(t)$ in the superstructure as the only degree of freedom (single-degree-of-freedom system).

Figure 8 shows a longitudinal section of the steel trough and a schematic illustration of the isolated vibration system. The superstructure is detached as a separate vibration system, whereby the deck plate is considered a rigid base for the isolated system. Thus, the absolute movements of the steel trough are entirely excluded from analysis. For the isolated vibration system, linear system properties are assumed for each considered stationary vibration state but not for the entire test spectrum. For the mechanical modelling of the ballast superstructure, an adaption of model 1 with two layers is used (see Fig. 2), whereby all effects of the vertical TBI are summarised in the characteristic values $\bar{k}_{ba,tr}$ and $\bar{c}_{ba,tr}$. In contrast to model 1 in Figure 2, continuous rather than discrete spring-damper elements are used and the characteristic stiffness and damping values $\bar{k}_{ba,tr}$ and $\bar{c}_{ba,tr}$ are per meter and for the installed half cross section of ballasted track. Concerning the overall system (see Fig. 8 right), the continuous values are summarised in an energy-equivalent manner, considering the track length $L_{bt} = 2.4\ m$, to form an overall value:

$$k_{ba,tr} = \bar{k}_{ba,tr}\, L_{bt} \quad \text{and} \quad c_{ba,tr} = \bar{c}_{ba,tr}\, L_{bt} \tag{1}$$

The values $k_{ba,tr}$ and $c_{ba,tr}$ therefore apply for the isolated SDOF system.

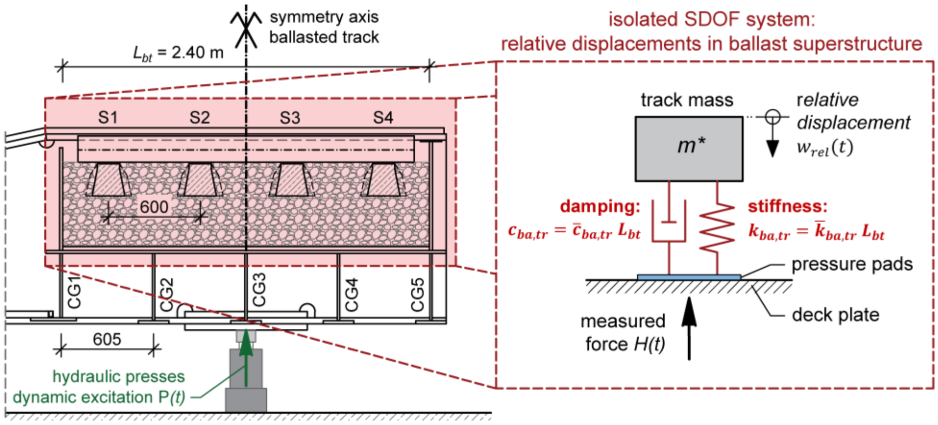

Figure 8: Idealisation of relative displacements in the ballast superstructure as a single-degree-of-freedom (SDOF) system and related dynamic characteristic values.

From the transmission force $H(t)$ between the superstructure and the deck plate, which represents the dynamic excitation force of the ballasted track, and the relative displacement between the rail and the steel trough $w_{rel}(t)$, hysteresis loops can subsequently be generated, which form the basis for determining the dynamic characteristic values. Figure 9 shows the measured displacement curve (Fig. 9 (a)), the measured force curve (Fig. 9 (b)) and the resulting hysteresis loop (Fig. 9 (c)) for an exemplary test with unloaded track and an excitation frequency of 10 Hz. The measured force and displacement curves in Figure 9 do not strictly correspond to a harmonic curve, which is why these curves are approximated by area-equivalent sinusoidal functions for further test evaluation (dashed lines in Fig. 9 (a) and 9 (b)). As a result, the evaluation is standardised for all tests, whereby the hysteresis loop always corresponds to an ellipse (see Fig. 9 (c)).

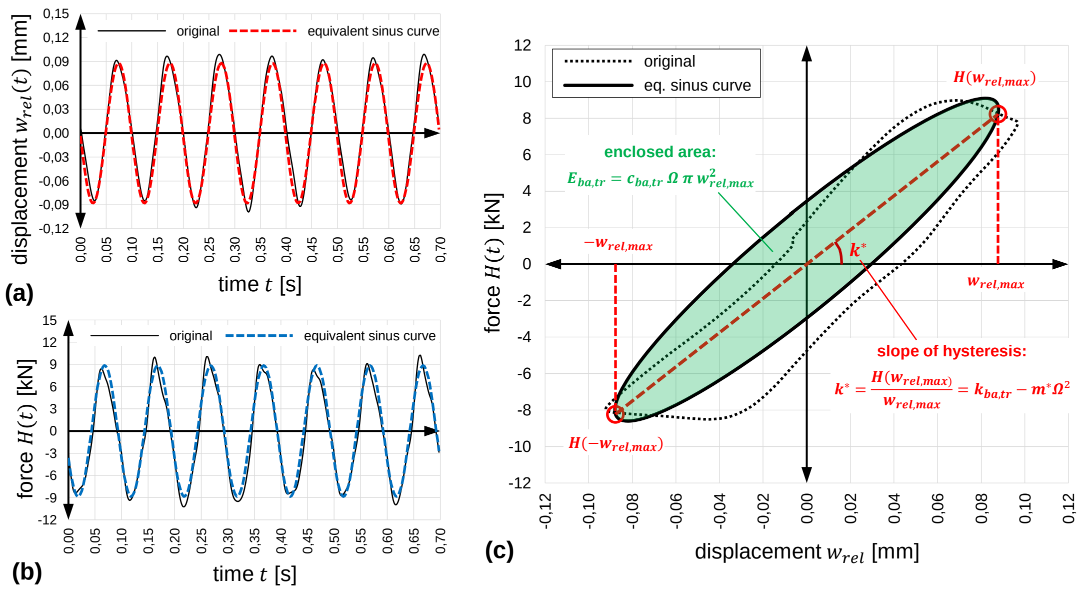

Figure 9: Measured force and displacement curves for one exemplary test with 10 Hz excitation frequency and unloaded track: (a) vertical relative displacement (b) transmission force and (c) from measurement generated hysteresis loop.

The generated hysteresis loops are an efficient tool for determining the dynamic characteristic values, as they can be used to determine the system's stiffness and damping properties. The stiffness $k_{ba,tr}$ can be determined from the inclination k^* of the hysteresis loop (red dashed line in Fig. 9 (c)) and the damping coefficient $c_{ba,tr}$ from the area enclosed by the hysteresis loop (green area in Fig. 9 (c)). The stiffness results from the inclination as well as the track mass m^* (mass of rail and sleepers) and the angular excitation frequency $\Omega = 2\pi f$ as follows (for theoretical background, see [10] and [11]):

$$k_{ba,tr} = k^* + m^* \Omega^2 = \frac{H(w_{rel,max})}{w_{rel,max}} + m^* \Omega^2 \tag{2}$$

The area enclosed by the hysteresis loop corresponds to the energy $E_{ba,tr}$ dissipated in the isolated oscillating system or in the damping element, which is defined as:

$$E_{ba,tr} = c_{ba,tr}\, \Omega\, \pi\, w_{rel,max}^2 \tag{3}$$

By using the area-equivalent sinusoidal functions for the measured curves, the area of the elliptical hysteresis also corresponds very closely to that of the original hysteresis (dotted line in Fig. 9(c)). The damping coefficient $c_{ba,tr}$ is thus calculated by transforming equation (3) as follows:

$$c_{ba,tr} = \frac{E_{ba,tr}}{\Omega\, \pi\, w_{rel,max}^2} \tag{4}$$

The conversion of the characteristic values $k_{ba,tr}$ and $c_{ba,tr}$ determined from the hysteresis loops to a mechanical bridge model with continuous spring-damper elements is carried out considering the track length L_{bt} and multiplication by a factor of 2 (conversion from half to a full cross-section) as follows:

$$\bar{c}_{ba} = 2\frac{c_{ba,tr}}{L_{bt}} \quad and \quad \bar{k}_{ba} = 2\frac{k_{ba,tr}}{L_{bt}} \tag{5}$$

The dynamic characteristic values $\bar{k}_{ba}$ and $\bar{c}_{ba}$ are determined using equations (2), (4) and (5). However, the tests have shown that the integrated pressure pads influence the dynamic behaviour in the frequency range above 10 Hz in a non-neglectable manner. Therefore, a reliable determination of dynamic characteristic values based on measured hysteresis loops is only possible in the frequency range between 2 and 10 Hz. The results presented therefore apply to the frequency range of 2-10 Hz.

Figure 10 shows the results for the vertical stiffness $\bar{k}_{ba}$ as a function of the relative displacement amplitude $w_{rel,max}$. As mentioned above, the characteristic values shown describe the continuous stiffness for a full cross-section. Figure 10 (a) shows an approximately exclusive displacement-dependent stiffness behaviour for both the unloaded (green) and the loaded track (blue), which is approximated by logarithmic regression functions (see definitions in Fig. 10 (a)). This non-linear stiffness behaviour could be determined qualitatively in the same way for the horizontal TBI, see [8]. To sum it up, the same non-linear displacement-dependent and approximately frequency-independent stiffness behaviour applies qualitatively for both the vertical and the horizontal TBI.

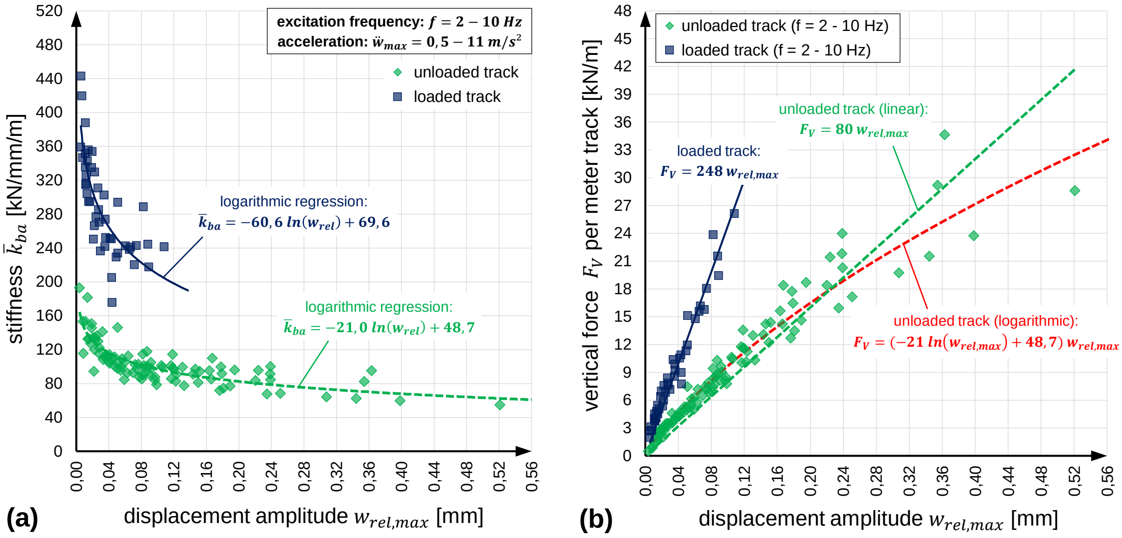

Figure 10: Experimental results for stiffness characteristics: (a) stiffness $\bar{k}_{ba}$ depending on the displacement amplitude and (b) vertical force in the track depending on the displacement amplitude.

Furthermore, Figure 10 (b) shows the vertical force F_v in the ballast superstructure

$$F_v = \bar{k}_{ba}\, w_{rel,max} \tag{6}$$

per meter track as a function of the relative displacement. A linear correlation is apparent for the loaded track's situation; the unloaded track's stiffness behaviour can be approximated linearly (green) or logarithmically (red). A simplified analysis of the unloaded track using a linear regression results in a ballast stiffness $\bar{k}_{ba}$ of 80 kN/mm/m.

Finally, Figure 11 shows the results for the damping coefficient $\bar{c}_{ba}$ as a function of the excitation frequency f, whereby the value again applies to a full cross-section of a bridge and per meter track. Figure 11 (a) shows the damping values for the unloaded track, with a colour-coded distinction between different acceleration levels (see legend). The damping values $\bar{c}_{ba}$ show both a frequency dependency and a dependency on the acceleration amplitude (as a scatter along the ordinate), which makes it challenging to specify a regression function reliably. In addition to the characteristic values for the unloaded track (green), Figure 11 (b) also contains the characteristic values for the loaded track (blue), which are significantly higher than for the situation of the unloaded track but are related to very low displacement amplitudes (< 0.10 mm), which is why these values are considered as somewhat uncertain.

Concerning a practical application, the authors recommend using the lower limit value of $\bar{c}_{ba} = 87$ kNs/m² for the unloaded track at this research stage. Investigations of the influence of the damping coefficient $\bar{c}_{ba}$ on the computational bridge vibrations in [12] show that the variation of the characteristic value $\bar{c}_{ba}$ influences the vibration behaviour to a negligible extent, which means that the damping coefficient plays a subordinate role.

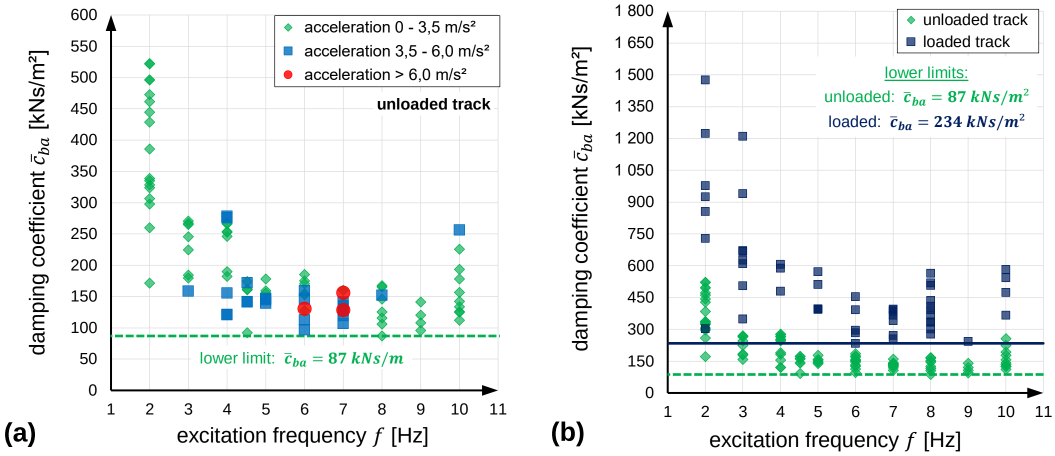

Figure 11: Experimental results for damping characteristics: (a) damping coefficient $\bar{c}_{ba}$ for the unloaded track and different acceleration levels and (b) damping coefficient $\bar{c}_{ba}$ for loaded and unloaded track.

4 Conclusions and Contributions

The experiments carried out using a special large-scale test facility for targeted and isolated research into vertical track-bridge interaction (TBI) have provided essential findings on the dynamic behaviour of the ballast superstructure. With frequencies between 2 and 25 Hz and vertical acceleration amplitudes of up to 10 m/s², the test spectrum broadly covers the spectrum of existing railway bridges. Based on the tests, significant findings could be derived concerning possibly occurring limit states due to vertical vibrations induced by the supporting structure. Furthermore, reliable test-based dynamic parameters were determined for use in mechanical bridge models for vibration predictions.

Concerning the characteristics of the vertical TBI in terms of the displacement and acceleration behaviour, the tests have shown that a disproportionate increase in vertical relative movements only occurs at accelerations above approx. 7 m/s² and that no abrupt destabilisation of the ballast bed occurs. Therefore, the limit value of 6 m/s² for the structural acceleration due to train crossing, according to [2], valid at the national level in Austria, can be seen as experimentally verified.

With regard to the determination of dynamic stiffness and damping parameters, there is significant uncertainty regarding a realistic application due to the striking bandwidth of characteristic values available in the literature. The test-based characteristic values presented in this contribution are an essential reference to considerably limit the range of characteristic values and, by specifying reliable values, enable a practical application corresponding to reality and, thus, economical and safe vibration predictions of railway bridges.

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References

[1] EN 1990/A1 Eurocode: Basis of structural design – Amendment 1: Application for bridges (consolidated version). Edition 2013-03-15.

[2] ÖNORM B 4008-2: Assessment of load capacity of existing structures – Part 2: Bridge construction. Edition: 2019-11-15 (in German).

[3] M. Zacher, M. Baeßler, "Dynamic behaviour of ballast on railway bridges", in "Dynamics of High-Speed Railway Bridges", R. Calcada, R. Goicolea, M. Gabaldon, (Editors), CRC Press, London, United Kingdom, 14 p., 2005.

[4] Baeßler, M. "Lageveränderungen des Schottergleises durch zyklische und dynamische Beanspruchungen", Doctoral Thesis, TU Berlin, 2009.

[5] J.C. Sanchez-Quesada, E. Moliner, A. Romero, P. Galvin, M. Martinez-Rodrigo, "Ballasted track interaction effects in railway bridges with simply supported spans composed by adjacent twin single-track decks", Engineering Structures 247, 113062, 2021. doi: 10.1016/j.engstruct.2021.113062

[6] A. Stollwitzer, "Developing an Approach for the Mathematical Calculation of the Damping Value of Railway Bridges with Ballasted Track", Doctoral Thesis, TU Wien, 2021.

[7] A. Stollwitzer, J. Fink, "Damping parameters of ballasted track on railway bridges – part II: energy dissipation in the ballasted track and related calculation model", Bautechnik 98(8), 552-562, 2021. doi: 10.1002/bate.202000100

[8] A. Stollwitzer, L. Bettinelli, J. Fink, „The longitudinal track-bridge interaction of ballasted track in railway bridges: Experimental analysis of dynamic stiffness and damping characteristics", Engineering Structures 274, 115115, 20 p., 2023, doi: 10.1016/j.engstruct.2022.115115

[9] L. Fryba, "Dynamics of railway bridges", Thomas Telford, London, 1996.

[10] A.K. Chopra, "Dynamics of structures: theory and applications to earthquake engineering", Prentice Hall, New Jersey, USA, 2011.

[11] C. Petersen, "Dynamik der Baukonstruktionen", Springer Vieweg, Wiesbaden, Germany, 2018.

[12] L. Bettinelli, A. Stollwitzer, J. Fink, „Numerical Study on the Influence of Coupling Beam Modeling on Structural Accelerations during High-speed Train Crossings", Applied Sciences 13(15), 8748, 46 p., 2023. doi: 10.3390/app13158746