Vibration Investigation of Single Span Steel Bridges

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ABSTRACT: The influence of railway ballast on the dynamic behaviour of simply supported steel bridges was measured on site using accelerometers. The vertical eigenfrequencies and the corresponding viscous damping for various construction stages, including the before and after bridge deck installation and addition of ballast track, was investigated for two new steel railway bridges in Austria. The structural identification was based on infield acceleration measurements with the excitation being carried out getting several people to bounce up and down in unison and also by the trains travelling over the deck. The data collected from the free vibration of the structures, immediately after the end of the excitation, was particularly useful for both eigenfrequency and damping identification. This paper reports on the results of the experimental investigation, especially the good concordance between measurement and prediction, and the comparison of different analysis methods. Some of the ballast influences on the overall dynamic behaviour are also presented. A metrological highlight is shown as well as an idea on how to deal with such metrological problems.

1 INTRODUCTION

Nowadays public transport is gaining more and more in importance. Therefore the main interest of the Local Authorities is to improve the existing railway lines and build new lines in accordance with the European standards. In Austria some sections of the important connection between Vienna and Salzburg are currently being investigated to check their suitability for high performance trains. Many existing bridges were checked using a preliminary numerical evaluation, and dynamic measurements were then performed to verify the previously assumed parameters [11,13]. In general the results indicate that the real measured eigenfrequencies lie above the values of the numerical analysis. For concrete bridges the differences are higher than those for steel bridges.

In 2004 two new single-span steel bridges were installed on a light railway line near Graz, Austria, operated by the local railway company GKB. The main reason for this investigation was to find the causes for the differences found in the previous results. Does the railway ballast have a significant influence? What is the magnitude of this influence? In order to get an idea of the real behaviour of the ballast, these two steel bridges were investigated during different installation stages.

2 INVESTIGATED BRIDGES

Table 1 gives some properties of the bridges reported on in this paper. The two steel bridges are simply supported beams, consisting of two I-beams, connected with cross girders and an orthotropic plate as shown in Figure 1. The main I-beams of both bridges are strengthened to cope with static requirements.

Table 1. Major dimensions and damping coefficients for the investigated bridges.

#	Material	Spans L [m]	TL [m]	SL [m]	TB [m]	B [m]	H [m]	EC Damping [%]
1	steel	29.00	30.00	22.58	6.6	4.4	2.66	0.5
2	steel	18.32	19.14	11.92	6.4	4.4	2.19	0.72

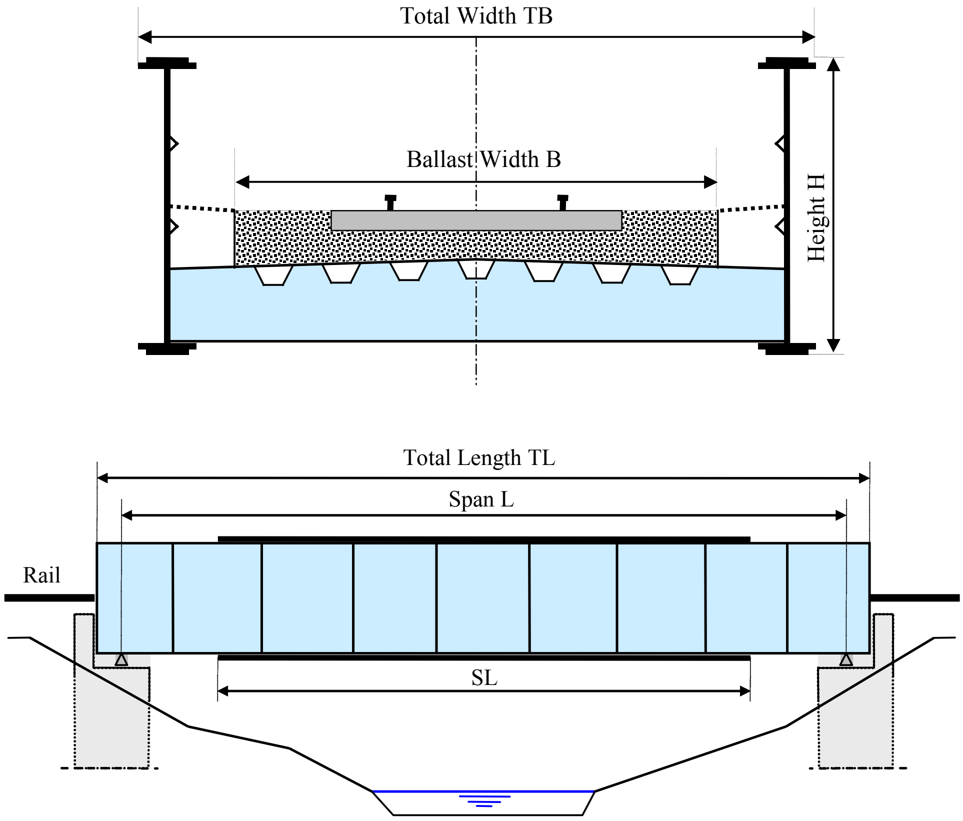

Figure 1. Cross section and elevation of investigated steel bridges

The bridges are located on a local light-railway-line, where no high-speed trains are being operated. The design of these bridges was completed by the Nipitsch/Heiden design office in Graz. Static design checks according to the Austrian regulations, were required by the owner [6]. The maximum line speed on this track is 100km/h and therefore, in accordance with Eurocode [8,9], no dynamic analysis is necessary.

3 MEASUREMENTS

3.1 *Setup*

The replacement of the two old steel-truss bridges with the new construction allowed dynamic acceleration measurements to be performed during different construction stages. Both bridges were prefabricated in the factory and assembled on site using cranes. See Figure 3. Five different installation configurations (CF1…5) were investigated for bridge #1and only two for bridge #2. Details are given in Table 2. The measurements were done before and after ballast installation.

Table 2. Schedule for measurements events

Bridge	CF	Date	Span [m]	Loading	Comment
#1	1	19.03.04	29.00	SW	temporary position
#1	2	20.03.04	30.00	SW	final position, no ballast
#1	3	20.03.04	30.00	SW+G2	40% ballast + sleepers + rail
#1	4	20.03.04	30.00	SW+G2	95% ballast + sleepers + rail
#1	5	07.04.04	30.00	complete	compressed ballast + welded rail
#2	1	16.10.04	18.32	SW	just self weight
#2	2	21.10.04	18.32	complete	compressed ballast + welded rail

The bridge deck was instrumented according to the expected eigenforms. Because of the symmetry of the structure, only half of the bridge was instrumented (One side). For the current investigation two channels were used simultaneously to capture the vertical accelerations of the deck at mid-span and at one quarter point. From this setup we may expect to identify the lower eigenfrequencies - either flexural or torsional. The accelerometers were mounted on top of one of the main I-girders to allow measurements to be made while the trains were passing.

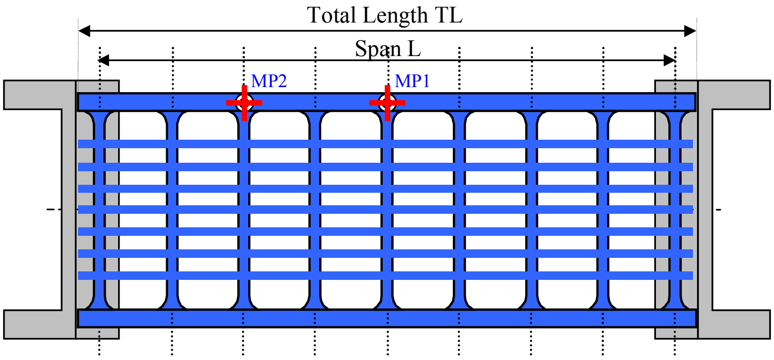

Figure 2. Schematic plan of the investigated bridges including instrument layout

3.2 *Equipment*

The Brüel&Kjaer PULSE® data acquisition system [1] was used to measure and record the vibration history for post processing. The Brüel&Kjaer front-end 2825 and two transducer 4507 were used for both bridges. The transducers employed for these vibration measurements were self-generating piezoelectric accelerometers with integrated preamplifiers for optimum dynamic range, requiring no additional power supply. The heart of a piezoelectric accelerometer is the slice of piezoelectric material, usually an artificially polarized ferroelectric ceramic, which exhibits the unique piezoelectric effect. When it is mechanically stressed, either in tension, compression or shear, it generates an electrical charge across its pole faces, which is proportional to the applied force [1].

3.3 *Mounting*

The method of mounting the accelerometer to the measuring point is one of the most critical factors in obtaining accurate results from practical vibration measurements. Sloppy or inaccurate mounting results in a reduction in the mounted resonant frequency, which can severely limit the useful frequency range of the accelerometer at higher frequencies [1]. The main interest is in the low frequencies of the steel bridges. So the ‘bad mounting’ which affects high resonant frequencies are of less importance.

A thin layer of bees-wax for sticking the accelerometer in place - a commonly used mounting method - was applied to all measurements. This method allows the accurate measurement of high resonant frequencies up to about 29kHz and the measurable frequency is only slightly reduced compared with other mounting methods. This method is restricted to about 40°C because bees-wax becomes soft at high temperatures. With clean surfaces, bees-wax fixing is usable for acceleration levels up to about 100 m/s² [1].

3.4 *Vibration excitation*

The excitation of the investigated steel bridges was carried out using people -from one person up to several people bouncing up and down in unison at mid span and at the quarter point as shown in Figure 3. Construction machines and trains travelling over the bridge deck were also used for causing the excitation after track installation was complete.

Figure 3. 7 people bouncing on bridge #1, 'CF1' and deck installation of bridge #2

3.5 *Data evaluation*

The recorded data was evaluated using two different methods. On the one hand the free vibration after the people-bouncing and the passing trains was used for identification of the first vertical eigenfrequency and the appropriate damping coefficient. On the other hand all recorded acceleration time histories were strung together for a ´complete signal´ of each installation configuration. Then a frequency spectrum was analyzed using the Welch's method [15], which estimates the power spectral density (PSD) of a signal. The method consists of dividing the time series data into (possibly overlapping) segments, computing a modified periodogram for each segment, and then averaging the PSD estimates. The result is the Welch's PSD estimate, which is implemented in the software MATLAB® [5]. The random stringing together of the time signals will produce some inaccuracies –steps– in the time-history recordings. The order of magnitude of these small steps was therefore investigated and found to be insignificant. As an example the vertical acceleration time history of one person bouncing up and down in mid span can be seen in the left part of Figure 4 and a Welch´s PSD frequency estimate of all measurements strung together for the installation 'CF5' of bridge #1 in the right part. The corresponding PSD spectrum for the time history (not shown in this paper) gives nearly the same results i.e. 4.5Hz for the first vertical eigenfrequency.

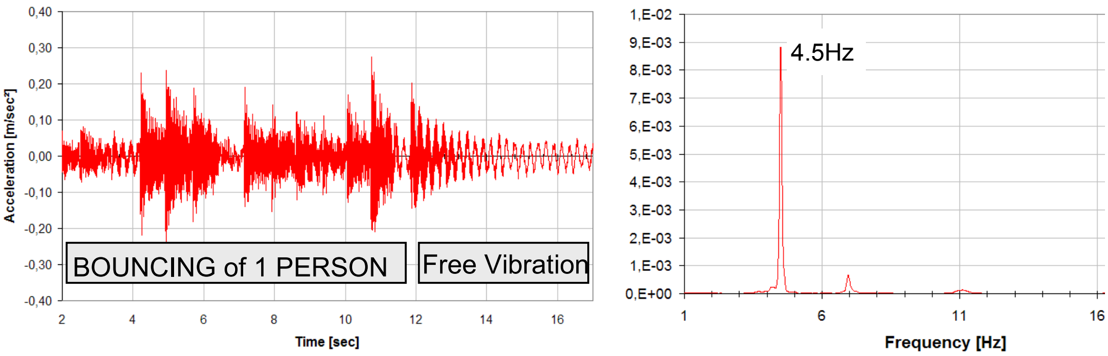

Figure 4. Vertical time-history for 1 person bouncing on bridge #1, 'CF5', and a PSD-spectrum

The free vibration after the people-bouncing and the passing trains could not be observed in all the recorded measurements – where the excitation frequency was close to a multiple of the eigenfrequency these effects could be easily seen. These time histories, where this vibration decay could be identified clearly, were investigated in more detail. First the vibration decay was separated from the rest of the signal. It should be noted that even the free vibration of a simply supported beam is often superposed with higher frequencies which are the results of secondary members and their connections. To smooth the free vibration, a modified robust baseline estimation according to [4,12] was applied to eliminate the higher frequency disturbances. This smoothed curve was the starting point for a least square fit investigation in order to find the optimum vibration decay. The amplitude, the eigenfrequency, the distortion of phase and the critical damping were modified by this method so that the squares of deviation were minimized resulting in reasonable values for the eigenfrequencies and corresponding damping. One example is demonstrated in Figure 5. On the left side the raw data from 7 people bouncing on bridge #1 (CF2) can be seen. The right side shows the smoothed curved and finally the optimized least square vibration decay function.

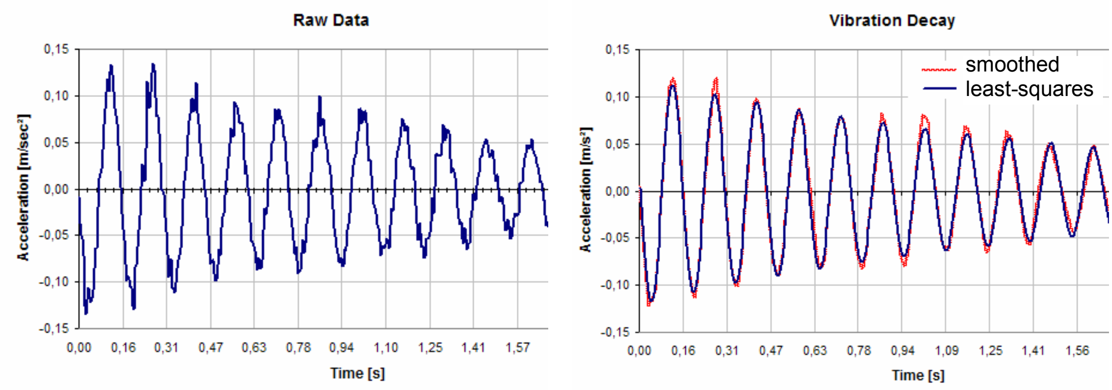

Figure 5. Parameter identification (eigenfrequency, damping…..) for vibration decay

As a result of the above optimisation in this example, the first eigenfrequency of 6.66Hz and a corresponding damping of 1.4% of the critical damping were found. These methods were applied to several measurement events. The comparison with the Welch´s PSD frequency showed good concordance.

4 RESULTS

The investigation of the steel bridges presented was focused on the determination of the first vertical eigenfrequencies during different installation stages to gain a dimension of the expected non-linear contribution of the track together with the ballast. Subsequently the results of the measured eigenfrequencies were compared with the results of a simple hand calculation as well as a more accurate eigenmode analysis using the program system RM2004 [14]. The vertical eigenfrequency of simple beams with different support conditions depends on 4 different parameters as can been seen from equation 1 below presented in [3].

$$\omega_m = \frac{\alpha_m^2}{L^2} \cdot \sqrt{\frac{EI}{\mu}} \qquad (1)$$

These parameters are: the support condition, the span length, the stiffness and the mass. In our case, α_m from equation 1 becomes π for a simply supported beam for the first eigenmode etc.

4.1 *Bridge #1*

The results of the first vertical eigenfrequencies for five different installation stages are specified in Table 3 below. It is obvious that, the eigenfrequency for later installation stages will decrease due to the increase of mass. As can been seen in Figure 1, the stiffness of the main girder is increased at mid span. For the hand analysis performed with equation 1, the increased stiffness of one I-girder was used as representative of the whole bridge. It is worth mentioning, that equation 1 neither considers shear deformation nor eccentricities of the supports. Because of the inaccurate assumptions in the hand calculation an additional analysis was carried out using the software system RM2004 [14]. The structure was modelled in the same way as it was built. The varying stiffness, see Figure 1, the vertical eccentricities at the supports, the contribution of the longitudinal stiffeners, the deck plate and the shear deformation were therefore correctly represented. The impact of the shear deformation on the vertical eigenfrequency is meaningful because the girder height is over 2m. The combined effects of the accurate modelling of the structure increase the overall stiffness at mid span by 13.9%.

Table 3. Comparison of first vertical eigenfrequency for different installation stages

CF	Span [m]	Mass μ [t/m]	Measurement [Hz]	Formula [Hz]	Factor [-]	RM-2004 [Hz]	Factor [-]	Damping [%]
1	29.0	3.18	7.1	7.11	1.00	7.08	1.00	---
2	30.0	3.18	6.7	6.64	1.01	6.62	1.02	0.9 to 1.9
3	30.0	5.10	5.3	5.24	1.01	5.22	1.01	---
4	30.0	7.11	4.5	4.44	1.01	4.42	1.02	---
5	30.0	7.15	4.5	4.43	1.02	4.41	1.02	0.2 to 1.0

The results of the analyses are based on certain assumptions. The uncertainty is mainly the mass. Interestingly, the simple analysis using equation 1 fits together quite well with the measured results. In this case equation 1 gives a good estimate although many of the above effects are not considered. The reason for the remaining differences is mainly the inaccuracy of the mass. Even the steel self-weight contains uncertainties, e.g. how much steel has been used for the welding etc., not to mention the real ballast thickness and compaction.

However, the tendency is the same everywhere. If, for example, we now look at 'CF1' above - it was found for a single I-girder configuration that if the shear deformation, the support eccentricities and the real distribution of the I-girder with the additional stiffening plates were considered, the eigenfrequency decreases by 0,48Hz. 'CF1' and 'CF2' just consist of the self-weight of the structure. The uncertainties in this configurations are the additional materials such as weld materials, size of connectors, the corrosion protection etc. 'CF3' to 'CF5' include the ballast but the real thickness and the specific weight are questionable. According to [10] the shaking density of the ballast used for these structures is about 16,6 kN/m³ compared with the specified value of 20,0 kN/m³ adopted in the static analysis. The shaking density complies with the specific weight of the compacted ballast. Previous investigations [13] did not consider this decrease in mass, and therefore preliminary findings assumed too much contribution from the ballast in the overall stiffness.
The results above show however, that there is still an influence from the ballast and track with respect to the overall stiffness but it is much smaller than previously thought.
Compared to the frequency results, the damping values are widely scattered. The lowest values for 'CF5' are below the given damping in the Eurocode [9] as can be seen in Table 1. The main problem is the variation depending on the size of amplitude of the free vibration. According to [7] there is an increase in damping for higher amplitudes. These effects could be confirmed and also explain the higher damping of 'CF2', where amplitudes of up to 0,12m/s² could be found compared to amplitudes of 0,05m/s² for 'CF5'.

4.2 *Bridge #2*

The results for bridge #2 show the same tendency as those for bridge #1 and are summarized in Table 4 below. An average value of the cross section properties was used in the hand analysis

using equation 1 to represent the effects of the stiffener plate only occurring over part of the beam. This was done differently for bridge #2 because the stiffener plates extends over a smaller distance compared with bridge #1. See Table 1.
The effect of the additional overall stiffness seems once again to be very small. The eigenfrequency of the accurate analysis is higher than the measured one. This indicates that the estimate of the self weight of the bridge #2 is too low as opposed to the overestimate of bridge #1.

Table 4. Comparison of first vertical eigenfrequency for different installation stages

CF	Span [m]	Mass μ [t/m]	Measurement [Hz]	Formula [Hz]	Factor [-]	RM-2004 [Hz]	Factor [-]	Damping [%]
1	18.32	2.80	12.2	11.99	1.02	12.51	0.97	---
2	18.32	6.63	8.0	7.78	1.02	8.00	0.98	0.2 to 0.9

5 PARTICULAR METROLOGICAL PROBLEM

It is hardly surprising that environmental influences can generate a significant influence known as cable noise. Nowadays piezoelectric accelerometers have a high output impedance, so problems can sometimes arise with noise signals induced in the connecting cable. These interferences can result from ground loops, tribo-electric noise or electromagnetic noise [1].
Ground loop currents can sometimes flow in the shield of the accelerometer cables due to the separate earthing of the accelerometers and measuring equipment. The ground loop can be broken by electrically isolating the accelerometer base and the conductive connector parts from the mounting surface. Tribo-electric noise is often induced into the accelerometer cable by mechanical motion of the cable itself and originates from local capacity and charge changes due to dynamic bending, compression and tension of the layers making up the cable. This problem can be avoided by using a proper graphited accelerometer cable or by taping and gluing it down to the deck as close to the accelerometer as possible. Electromagnetic Noise is often induced in the accelerometer cable when it lies in the vicinity of running machinery with strong electromagnetic fields [1]. High quality cables were used in this investigation as they minimizes the effects of tribo-electric and electromagnetic noise.
A typical 50Hz hum-noise occurs connecting two different potentials of electric ground with a signal cable, for example between the earthed power supply system on the one hand and the vibration transducers or connectors touching a different electric ground on the other hand. In this case the difference between the electrical potentials will be balanced over the signal ground wiring showing a significant 50-Hz-peak (providing a 50-Hz-power source) at the frequency response of the measurement.

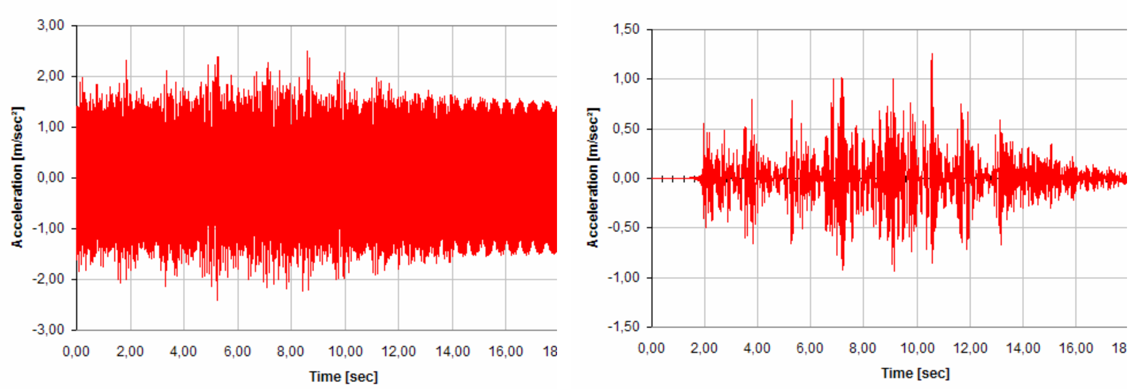

Figure 6. Vibration measurement of bridge #2 with and without ground loop effects.

A ground loop occurred during this investigation which affected the measurements for bridge #2, 'CF1'. The bridge #2 deck was installed on a rainy day, (the same day that the measurements were performed). The PULSE® data acquisition system [1] was connected with a power set (generator) on site. Additionally some welding works were carried out at the same time. A ground loop resulted from these wet conditions. On the left hand side in Figure 6 a time history of a digger excitation is shown superposed with a 50Hz hum-noise of the power set. The transformation into the frequency domain showed a dominating peak at 50.7Hz. After applying a notch filter around 50Hz an adjusted time signal is achieved as can be seen on the right hand side of Figure 6. This signal is now ready for further evaluations.

6 CONCLUSIONS

Two single span steel bridges with spans of 18.32 and 29.0m were investigated before and after deck installation. The experimental program consisted of acceleration measurements in order to quantify the dynamic influence of the ballast acting together with the track.
A good agreement between measurements and analysis were found, in spite of some inaccuracies especially of the predicted mass. On the whole, it seems safe to say that the overall dynamic stiffness is somehow affected by non-linear ballast effects as was also found in [2,11]. The influence is however very small for these investigated span lengths und much smaller than was assumed in previous investigations. This is especially true because the real shaking density lies about 17% below the common used density for the static analysis.
Furthermore it can be mentioned, that in spite of an occurring metrological problem, the measurement results can still be considered useful for further evaluation after applying suitable mathematical adjustments.

7 ACKNOWLEDGEMENTS

The authors would like to thank the local Styrian railway company GKB and the design office Nipitsch/Heiden for their technical support as well as the provision of the facilities required for carrying out the investigations.

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