MODAL IDENTIFICATION
OF
SINGLE-SPAN RAILWAY VIADUCTS

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Abstract

The paper reports on the results of the modal identification of several small to medium span railway viaducts. The identification was based mainly on the free vibration of each structure immediately after the train passages. The non-linear behavior of some components, namely the ballast and the aged bearing supports, conditioned the evolution of the free vibration signals, which showed increasing values for the eigenfrequencies and lower values for the damping as far as the vibration amplitude decreased. An estimate for the ballast shear stiffness and for the influence of the superstructure (track plus ballast) in the overall dynamic behaviour could also be obtained from the measurements evaluation.

1 Introduction

Due to the rapid increase of train speeds and/or transported loads, existing railway bridges and viaducts have to be structurally re-evaluated in order to maintain adequate safety levels under the new traffic conditions. This re-evaluation must be done according to standards (for instance Eurocode 1) whose clauses apply mainly to new structures. Therefore, for existing bridges, this poses an additional difficulty, since very often they exhibit a dynamic behavior concerning eigenfrequencies and structural damping different from those assumed in the original project, and this may decisively condition the need for strengthening/replacing of the structure.

The response of railway bridges to the passage of trains depends on one side on the type of train and on its traveling speed and on the other side on the type of bridge, specially its dynamic characteristics expressed through the eigenfrequencies and respective damping. As it is well known, the situation of resonance imposed by the dynamic loads generated during the periodic passage of the wheels or groups of wheels over the bridge can induce very high accelerations in the structure.

Recently, several existing small to medium span bridges in the track Linz-Wels (Austria) were reevaluated in order to permit the increase of train speeds. In a preliminary numerical evaluation, considering conservative values for the dynamic parameters, very high vertical accelerations were computed for some of those structures. Resonance under train speeds between 200km/h and 240 km/h lead to maximum accelerations of up to 20 m/s^2 in some cases.

An experimental program was thus carried out in order to get a better estimation of the dynamic behavior of the bridges, concerning mainly the first vertical eigenfrequency and the corresponding viscous damping. However, the amount of measurements and the different structural types of the bridges allowed some further conclusions about the contribution of the rails and the ballast to the overall dynamic behaviour of the bridges.

2 Measurement setup

2.1 Structural layout

Two types of structures were to be measured. One set was composed by several reinforced small span monolithic frames and another set contained several reinforced, prestressed and mixed single-span simply supported slabs, whith spans from 5.75 meters to 23.5 meters. Only results for this last set are here presented and discussed.

The viaducts, built in the 1970’s (see Figures 1 and 2) are composed of two twin decks laying side by side, one for each track. The ballast depth has an average of 0.60 m, varying between 0.55 m and 0.65 m depending on the slab thickness, and spans over the entire width of the two decks.

The structural layout of the prestressed concrete decks (Bridges 1, 3, 8 and 12) corresponds to one-span simply supported slab with slightly variable depth. In the case of Bridge 7 the simply supported slab is made of HEB360 steel bars filled with concrete. Bridge 11 is also simply supported and made of reinforced concrete.

The support conditions, although defined generally as simple supports, are of two types. In Bridges 1, 3, 8 and 12, the bearing supports (Figure 3), two at each extremity of the deck, are made of steel pots filled with elastomeer and can be considered free to rotate. For the other structures no specific apparatus have been used and the slab lies directly on the abutment top.

There is no continuity of the slab over the supports to the abutments, except the one materialized by the superstructure composed of ballast, sleepers and rails. It is also to stress the fact that in Bridges 1, 3 and 8 the line of supports is not collinear when considering both decks as illustrated in Figure 4 and that the line of supports in Bridge 12 is skew relatively to the axis of the bridge. Table 1 in conjunction with Figure 5 summarizes the structural characteristics of the bridges.

2.2 Measurement procedures

Two methods were considered for the identification of eigenfrequencies, eigenforms and damping:

i) free vibration after train passages

ii) natural ambient vibration

Since the traffic is intense during the daylight the second method could not be used with all its potentialities, that is, we could not use a set of reference sensors and move the other set of sensors across the structure. As a consequence the number of measurement points was limited to the number of channels available. It was decided to capture and record the signal during enough time in order to obtain a significant number of train passages, corresponding to two to five hours of continuous signal recording with at least 10 train passages.

Figure 1 – Bridge 12 (skew): general view

Figure 2–Bridge7 (filled beam): general view

Figure 3 – Bearing supports

Figure 4 – Bridge 8: view of the abutments area

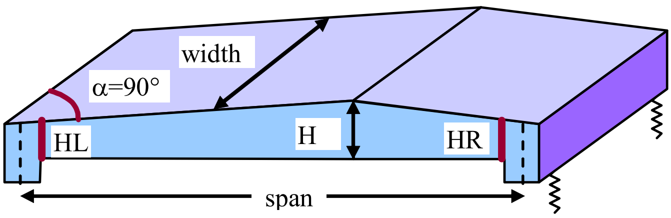

Figure 5 –Structural layout

Table 1 – geometrical characteristics of the bridges according to the structural layout

Bridge	**Span [m]**	**Width [m]**	**HL [m]**	**H [m]**	**HR [m]**	**α [º]**	**Type**
1	23.50	5.14	0.92	1.14	0.91	90º	Prestressed
3	19.50	6.49	0.97	1.10	0.84	90º	Prestressed
7	9.00	4.52	0.43	0.43	0.43	90º	Mixed
8	21.00	4.23	1.05	1.15	1.05	90º	Prestressed
11	5.75	4.44	0.40	0.40	0.40	90º	Reinforced
12	11.44	4.54	0.7	0.90	0.70	63.9º	Reinforced

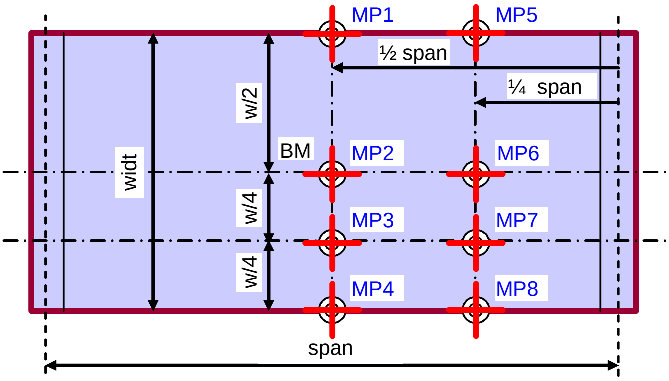

Figure 6 – General scheme of the measurement locations

Table 2 – Measurement points

MP	Bridge 1	Bridge 3	Bridge 7	Bridge 8	Bridge 11	Bridge 12
1		●		●	●	●
2	●	●	●	●	●	●
3		●		●		
4	●	●	●	●	●	●
5	●	●		●	●	
6	●	●	●	●	●	●
7		●				
8	●	●		●	●	

According to the expected eigenforms and to the symmetry conditions only one half-side of each deck was instrumented. Although a maximum of eight channels were available only a number of them were used to capture the vertical accelerations of the deck at mid-span and at ¼ of the span (see Figure 6 and Table 2). Nevertheless, the setup used in each bridge allowed the lower eigenforms, either flexural or torsional, to be clearly identified.

The response data was acquired using the Brüel & Kjær PULSE® multi analyser platform and recorded for post processing. In addition, observation files were manually filled with the type of train, number of carriages and velocity, measured with a speedometer.

During the post processing the signal was inspected and analyzed in such a way that the power spectra corresponding to the following situations could be obtained:

i) the free vibration immediately after the train leaves the bridge,

ii) the ambient free vibration caused by unidentified natural excitation during the interval of train passages and

iii) the individual response to each train passage.

Since the maximum signal peak input had to be set up for the expected maximum acceleration during the train passages, the signal to noise ratio for the ambient free vibration was poor and, in addition to relatively short time histories, conditioned the results obtained from this identification method.

On the other hand, the free vibration immediately after the train passages showed to be very useful. However, attention must be paid to the effective time after which there is no more excitation on the bridge in order to prevent the distortion of the results. In the present research the visual inspection of the time records was usually sufficient to ensure this, since the signal always became smoother after the train left the bridge.

3 Measurement results

3.1 Free vibration analysis

Time histories with a fixed time length were considered, starting from the instant when each train leaves the bridge. Considering the longest eigenperiod expected for the structures, that time length should be enough for the free response to decay significantly as it can be seen in Figure 7, where typical free response time histories are shown. Considering the sampling time interval used, a frequency definition of 0.125Hz was achieved.

Taking into account for each structure the entire sequence of all the time histories described above, it is possible to use a method for the parameter identification in the frequency domain such as the Peak Picking Method. The Enhanced Peak Picking Such methods implemented in the software ARTeMIS® were here used to extract the eigenmodes, frequencies and damping.

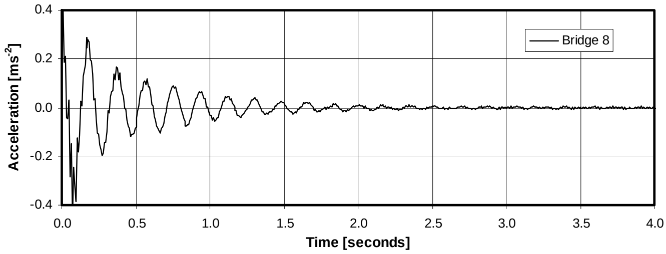

Figure 7 – Typical free vibration decay at the mid-span of the bridges

To assess the contribution of probable non-linear effects provided mainly by the track, ballast and supports, time shifts of up to 1,5 seconds were introduced in the starting point of the time histories, maintaining the same total time length. In this way significantly lower signal amplitudes are considered in those shifted time series when compared to the initial time considered before. In Table 3 the results from both shifted and non-shifted time series are presented.

Table 3 –Eigenfrequencies and damping considering non-shifted and shifted time series

Bridge	Mode	Non-shifted time series		Shifted time series		Mode type
		Frequency [Hz]	Damping Ratio [%]	Frequency [Hz]	Damping Ratio [%]	
1	1st	4.8	7.8	5.2	5.3	1st Bending
	2nd	13.3	4.9	14.9	2.8	2nd Bending
	3rd	16.9	2.0	17.0	1.3	1st Torsion
	4th	27.7	1.9	30.1	0.7	3rd Bending
3	1st	6.4	9.3	8.5	7.2	1st Bending
	2nd	14.7	4.5	16.0	2.6	1st Torsion
	3rd	18.5	2.5	22.2	1.9	2nd Bending
	4th	33.7	2.8	n.a.	n.a.	3rd Bending
7	1st	16.9	4.7	n.a.	n.a.	1st Bending
	2nd	25.5	0.9	n.a.	n.a.	1st Torsion
	3rd	49.0	0.7	n.a.	n.a.	2nd Bending
8	1st	5.4	6.1	5.8	5.2	1st Bending
	2nd	18.8	2.8	18.7	0.8	2nd Bending
	3rd	19.2	1.8	19.7	1.3	1st Torsion
	4th	37.4	1.2	n.a.	n.a.	3rd Bending
11	1st	15.0	2.0	n.a.	n.a.	1st Bending
	2nd	36.4	1.4	n.a.	n.a.	1st Torsion
12	1st–2nd	13.7 – 16.5	7.7 – 4.7	13.2 – 16.9	6.4 – 3.6	Bending
	3rd–4th	26.2 – 29.4	3.0 – 1.5	26.3 – 31.8	2.3 – 2.1	Torsion
	5th–6th	41.4 – 43.6	2.1 – 2.2	41.6 – 44.4	1.0 – 1.6	Bending
	7th–8th	50.7 – 51.8	0.2 – 0.2	50.7 – 51.8	0.2 – 0.3	Bending

When the modal parameters obtained from the time shifted response histories are compared with those from the non-shifted histories two tendencies can be recognized: (i) the frequency increases and (ii) the damping factor decreases. These results are more evident in the lower modes and must be, therefore, related to the amplitudes of vibration. In fact the higher amplitudes in the lower modes mobilize mechanisms of energy dissipation related to friction in the supports and internal friction in the ballast in a stronger way than they are mobilized in the higher modes, for which the amplitudes of vibration are much lower. This also stresses the idea that the friction forces, probably mainly inside the ballast, become more important for lower amplitudes having an effect of increasing the overall stiffness of the structure. Again, this effect is less important for the higher modes.

Bridge 12 is skew and, therefore, the influence of the interaction between both twin plates is shown in the duplication of the frequency peaks corresponding to symmetrical and non-symmetrical shapes with reference to the contact plan between the plates. This effect allows the quantification of the ballast shear stiffness and is further exploited in the item concerning parameter identification.

3.2 Ambient vibration

Ambient vibration methods are based on the assumption that all the dynamic excitation in the structure is of such a random type that all modes in the relevant range are excited in the same way. Although this is not a typical problem for the application of such methods, the length of the time histories corresponding to the free vibration considered before can be extended for several minutes, depending on the railway traffic, so that the effect of the free vibration induced by the train passage is dimmed.

The results are summarized in Table 4. Because of the insufficient total measuring time and the fact that the maximum signal peak input had to be set up for the expected maximum acceleration during the train passages, conditioning the goodness of the signal, the results are limited to the first natural frequency and to the longer span bridges. Comparing these results with those given in Table 3, the difference is to be found in the damping, resulting from the ambient vibration much lower damping values. The frequencies coincide with those given in Table 3, as expected.

Table 4 – Eigenfrequencies, eigenmodes and damping for Bridges 1, 3, 8 using ambient vibration

Bridge	Mode	Frequency [Hz]	Damping Ratio [%]	Mode type
1	1st	5.2	2.0	1st Bending
3	1st	8.6	1.9	1st Bending
8	1st	5.8	2.7	1st Bending

4 Modal identification

4.1 Structural models

In order to identify the stiffness distribution simple structural models were considered for each of the bridges. The discretization was made using common plate finite elements and the supports were considered free to rotate but not free to slip, where a spring of stiffness k_s was considered (see Figure 8). The continuity of the superstructure (ballast+sleepers+rails) was modeled using a single spring at the rails level with stiffness k_r. In the case of the skew slab the shear stiffness G_b of the ballast is modeled through elastic springs k_b.

The stiffness k_r can be estimated taking into account only the rails over an extension corresponding to about the interval length of the sleepers (0.60 meters). The stiffness k_s is unknown and have to be estimated by comparison with the measured frequencies. The shear stiffness of the ballast G_b is obtained directly from the comparison of the measured pairs of eigenfrequencies with those computed in the model for Bridge 12. The mass was estimated according to the mean values for the density of the concrete (2,5 ton/m^3) and of the superstructure (1,53 ton/m^2)

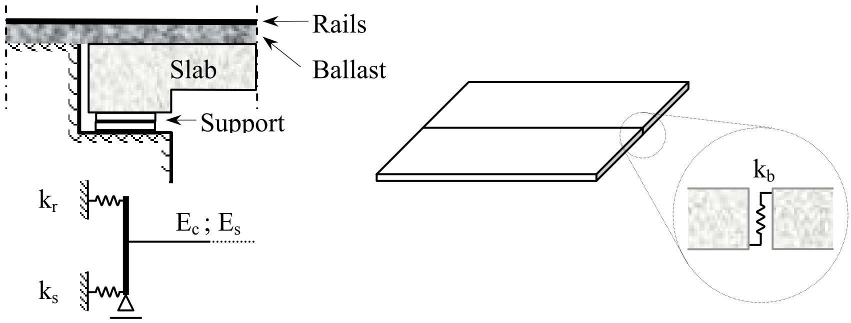

Figure 8 – Model for the support area and for interaction between twin skew plates

4.2 *Parameter identification*

As stated before only a small number of parameters were considered for identification. Table 5 summarizes the computed eigenfrequencies and the values considered for those parameters.

Table 5 – Results from FE models and parameter values used

Bridge	Mode	Frequency [Hz]	Mode type	Parameters
1	1st	4.9	1st Bending	E_c=39 GPa
	2nd	14.0	2nd Bending	K_s=2.6 GN/m
	3rd	18.4	1st Torsion	K_r=2.6 GN/m
	4th	28.0	3rd Bending	
3	1st	5.8	1st Bending	E_c=39 GPa
	2nd	14.9	1st Torsion	K_s=2.6 GN/m
	3rd	16.1	2nd Bending	K_r=2.6 GN/m
	4th	32.0	3rd Bending	
7	1st	16.1	1st Bending	E_c=30 GPa
	2nd	28.0	1st Torsion	E_s=200 GPa
	3rd	45.3	2nd Bending	K_s=2.6 GN/m
				K_r=2.6 GN/m
8	1st	5.5	1st Bending	E_c=39 GPa
	2nd	15.8	1st Torsion	K_s=2.6 GN/m
	3rd	22.2	2nd Bending	K_r=2.6 GN/m
	4th	32.3	3rd Bending	
11	1st	14.9	1st Bending	E_c=30 GPa
	2nd	30.1	1st Torsion	K_s=0.25 GN/m
				K_r=0.25 GN/m
12 Twin plates	1st – 2nd	14.0 – 16.5	Bending	E_c=39 GPa
	3rd – 4th	26.3 – 30.8	Bending + Torsion	K_s=2.6 GN/m
	5th – 6th	37.4 – 44.0	Bending + Torsion	K_r=2.6 GN/m
	7th – 8th	57.0 – 59.0	Bending + Torsion	G_b=35 MPa

5 Final remarks

A set of single span railway bridges were measured in order to characterize their dynamic behaviour. The free vibration after train passages showed to be very effective for the model identification.

In order to identify the stiffness distribution simple structural models were considered for each of the Bridges. In those models the main parameters were the modulus of elasticity of the materials (concrete or mixed concrete-steel), and the slab end-restriction at the supports. The stiffness of those springs showed to be much similar to the axial stiffness of the rails, considering the length of the sleeper interval.

Since a skew slab was also measured, it was possible to estimate the shear stiffness of the ballast analyzing the difference between pairs of eigenfrequencies, which depends only on that stiffness. A value of G_b=35 MPa for the shear modulus was found.

6 Aknowledgements

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