



Title	Detection of Bridge Dynamic Parameters Using an Instrumented Vehicle
Authors(s)	González, Arturo, O'Brien, Eugene J., McGetrick, P.
Publication date	2010
Publication information	González, Arturo, Eugene J. O'Brien, and P. McGetrick. "Detection of Bridge Dynamic Parameters Using an Instrumented Vehicle." World Conference on Structural Control and Monitoring, 2010.
Conference details	5th World Conference on Structural Control and Monitoring, 12th-14th July, Tokyo, Japan
Publisher	World Conference on Structural Control and Monitoring
Item record/more information	http://hdl.handle.net/10197/4122
Notes	Proceedings for the conference are available online as a .zip file from http://www.bridge.t.u-tokyo.ac.jp/WCSCM5/5WCSCM_Paper.zip

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DETECTION OF BRIDGE DYNAMIC PARAMETERS USING AN INSTRUMENTED VEHICLE

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Abstract

Highway structures such as bridges are subject to continuous degradation primarily due to ageing and environmental factors. A rational transport policy requires the monitoring of this transport infrastructure to provide adequate maintenance and guarantee the required levels of transport service and safety. In Europe, this is now a legal requirement - a European Directive requires all member states of the European Union to implement a Bridge Management System. However, the process is expensive, requiring the installation of sensing equipment and data acquisition electronics on the bridge. This paper investigates the use of an instrumented vehicle fitted with accelerometers on its axles to monitor the dynamic behaviour of bridges as an indicator of its structural condition. This approach eliminates the need for any on-site installation of measurement equipment. A simplified half-car vehicle-bridge interaction model is used in theoretical simulations to test the possibility of extracting the dynamic parameters of the bridge from the spectra of the vehicle accelerations. The effect of vehicle speed, vehicle mass and bridge span length on the detection of the bridge dynamic parameters are investigated. The algorithm is highly sensitive to the condition of the road profile and simulations are carried out for both smooth and rough profiles.

Introduction

There has been a worldwide increase over recent years in the number of larger bridges that are being instrumented and monitored on an ongoing basis – so called Structural Health Monitoring (Farrar and Worden, 2007). Given the very large number of bridges that are not instrumented, some alternative method is needed to detect any change in behaviour of the structure which might be an indicator of some form of damage. This paper investigates the use of a vehicle fitted with accelerometers on its axles to monitor the dynamic behaviour of bridges. This approach eliminates the need for any sensing equipment or data acquisition electronics to be installed on the bridge and it is aimed at allowing the assessment of bridge condition to become a simplified and much less laborious and time consuming process. It would facilitate more effective and widespread monitoring of the condition of existing bridge structures in a transport network while its development would enable maintenance to be carried out at an earlier stage in degradation, which generally results in more economical repairs.

The feasibility of extracting bridge dynamic parameters from the dynamic response of a vehicle passing over a bridge has been verified theoretically by Yang *et al* (2004). The method studied by Yang *et al* would only require a vehicle instrumented with accelerometers acting as a ‘message carrier’ of the dynamic parameters of the bridge. The bridge was assumed to be a simply supported beam while a sprung mass represented the vehicle. The bridge natural frequency was extracted from the vehicle acceleration spectrum of the sprung mass and they found that the magnitude of the peak response at the bridge natural frequency increased with vehicle speed but decreased with increasing bridge damping ratio.

Lin and Yang (2005) carried out an experimental validation of this method. A two wheeled cart towed by a four wheel commercial light truck was driven across one simply supported 30m span of a six span bridge. The truck of this tractor-trailer system served as the exciter of the bridge into vibration and the cart was fitted with an accelerometer, acting as the receiver of the bridge vibrations. Their field tests verify that the bridge natural frequency can be easily identified from the dynamic response of the cart for vehicle speeds less than 40 km/h (11.11 m/s). They note that at higher speeds the bridge frequency becomes blurred by high frequency components from pavement roughness and cart structure. They also found that ongoing traffic had a beneficial effect as it increased the vehicle response.

Oshima *et al* (2008) also carried out an experimental investigation of an estimation method for the eigenfrequencies of a bridge. A heavy truck with an excitation system was driven across a 30m simply supported steel bridge at different speeds and acted as an exciter of the bridge while another vehicle equipped with accelerometers travelled behind it and acted as a receiver of the vibrations. The system did not detect the bridge frequency without forced vibration from the truck excitation system. However, the bridge frequency was detected with forced vibration at excitation frequencies close to the bridge frequency; the estimated frequency from the vibrations of the monitoring vehicle compared well with the value recorded directly on the bridge. Several bridge crossings were required to obtain all of the low-order eigenfrequencies which were extracted using a Bode diagram.

A further numerical and experimental analysis of this technique was carried out by González *et al* (2008). They tested the approach numerically for various speeds, road roughness, damping levels and traffic conditions using a 3-D FEM vehicle-bridge interaction (VBI) model. The experimental analysis consisted of a field test on a main route near Oviedo, Northern Spain. For this test a vehicle instrumented with accelerometers and GPS was driven over a long-span bridge (9 spans of lengths between 41 and 50 m, and a total length of 423.5 m) to obtain its frequencies. González *et al* concluded that an accurate determination of the bridge frequency is only feasible at low vehicle speeds and when the dynamic excitation of the bridge is high enough. Therefore, the interference of the road surface profile frequencies corrupts the spectrum and prevents the identification of the bridge natural frequency. They note that lower vehicle speeds and the presence of heavy traffic on the bridge would have improved the ability to detect the bridge frequency as the bridge deflection would have been more significant compared to the height of the road surface irregularities.

McGetrick *et al* (2009) performed a theoretical investigation of the identification of bridge dynamic parameters using a moving vehicle fitted with accelerometers on its axles. The instrumented vehicle was used to identify not only bridge frequencies of vibration but also a change in the bridge's structural damping. A simplified VBI model was used for theoretical simulations using MATLAB. The vehicle was represented by a 2-degree-of-freedom wheel/suspension system (quarter-car). McGetrick *et al* conclude that the bridge's frequency of vibration and structural damping can be detected easily from the dynamic response of the quarter car for a perfectly smooth road profile. Introducing a road profile resulted in the bridge frequency and structural damping becoming difficult to detect as the road profile had a greater influence than the bridge deflection on the vehicle's dynamic response which is similar to the conclusion reached by González *et al*.

This paper aims to extend the analysis carried out by McGetrick *et al* to other test conditions. For this theoretical investigation, a VBI simulation model is created in MATLAB. The vehicle is modelled as a 4-degree-of-freedom half-car and the bridge is modelled as an Euler-Bernoulli beam. The aim is to detect

the natural frequency of vibration of the bridge and also identify changes in bridge structural damping from the dynamic response of the vehicle. This investigation involves analysis of the frequency spectra of vertical vehicle accelerations obtained from the dynamic response of the half-car as it travels across the bridge. The frequencies of vibration are identified from the vertical acceleration spectra and compared to the true corresponding bridge or half-car frequencies. Simulations are carried out for simply supported bridge spans of 15, 25 and 35 metres, vehicle velocities of 5 m/s to 25 m/s and for smooth and rough road surface profiles of varying ISO class. As structural damping has been shown to be damage sensitive (Curadelli *et al*, 2008; Modena *et al*, 1999), bridge structural damping is also varied from 0% to 5% (in steps of 1%) to investigate its effect. 0% bridge damping is not a realistic value, but it is used here as a reference point. The effect of the vehicle mass on the dynamic response of the vehicle is considered by using gross vehicle weights of 4.8 tonnes, 9 tonnes and 18 tonnes representing an empty, half-loaded and fully loaded 2-axle truck respectively. The results will indicate the conditions in which this method can be used to detect bridge dynamic parameters with a reasonable degree of accuracy.

Vehicle – Bridge Interaction Model

A theoretical half-car model is used to represent the behaviour of the vehicle (Figure 1). Although a half car is a simplified version of a vehicle, its response still illustrates many of the important characteristics of dynamic tyre forces (Cebon, 1999). The half-car is a 4-degree-of-freedom suspension model. The four degrees of freedom account for axle hop displacements, sprung mass bounce displacement and sprung mass pitch rotation. The body of the vehicle is represented by the sprung mass, m_s , and the front and rear axle components are represented by unsprung masses, m_{u1} and m_{u2} respectively. The axle mass connects to the road surface via a spring of stiffness K_t , while the body mass is connected to the tyre by a spring of stiffness K_s in combination with a viscous damper of value C_s modelling the suspension. Tyre damping is assumed to be negligible and thus is omitted. Other properties the model accounts for are the sprung mass moment of inertia, I_s , and the distance of each axle to the vehicle's centre of gravity (o), i.e., D_1 and D_2 in Figure 1 ($D_1 + D_2$ is equal to the axle spacing). For these simulations, the centre of gravity of the vehicle (o) is taken to be equidistant from each axle ($D_1 = D_2$). The half-car property values are listed in Table 1 and are based on values obtained from work by Harris *et al* (2007) and Cebon (1999).

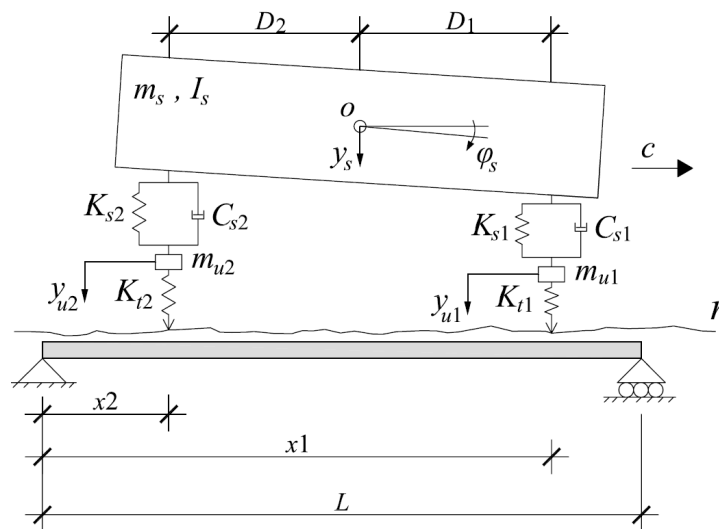


Figure 1 Half car – beam interaction model

The half-car travels at constant speed, c , over a simply supported Euler-Bernoulli beam which has constant cross section and mass per unit length, μ . It has span L , modulus of elasticity E , second moment

of area J and structural damping ζ . The properties of the 3 bridge spans used in these simulations are given in Table 2. The first and second natural frequencies of each bridge, $f_{bridge1}$ and $f_{bridge2}$, are also given for future reference.

Table 1 Half car properties

<i>Property</i>	<i>Unit</i>	<i>Symbol (Figure 1)</i>	<i>Model 1 (18 tonnes)</i>	<i>Model 2 (9 tonnes)</i>	<i>Model 3 (4.8 tonnes)</i>
<i>Body mass</i>	<i>kg</i>	m_s	16600	7600	3400
<i>Axle mass</i>	<i>kg</i>	m_{u1}	700	700	700
		m_{u2}	700	700	700
<i>Suspension Stiffness</i>	<i>N/m</i>	K_{s1}	4×10^5	4×10^5	4×10^5
		K_{s2}	4×10^5	4×10^5	4×10^5
<i>Suspension Damping</i>	<i>Ns/m</i>	C_{s1}	10×10^3	10×10^3	10×10^3
		C_{s2}	10×10^3	10×10^3	10×10^3
<i>Tyre Stiffness</i>	<i>N/m</i>	K_{t1}	3.5×10^6	3.5×10^6	3.5×10^6
		K_{t2}	3.5×10^6	3.5×10^6	3.5×10^6
<i>Moment of Inertia</i>	<i>kg m²</i>	I_s	64598	29575	13231
<i>Distance of axle to centre of gravity</i>	<i>m</i>	D_1	1.875	1.875	1.875
		D_2	1.875	1.875	1.875
<i>Body mass frequency of vibration</i>	<i>Hz</i>	f_{bounce}	1.05	1.55	2.31
		f_{pitch}	1	1.47	2.19
<i>Axle mass frequency of vibration</i>	<i>Hz</i>	f_{axle1}	11.9	11.9	11.9
		f_{axle2}	11.9	11.9	11.9

Table 2 Bridge properties

<i>Span Length, L (m)</i>	<i>Modulus of elasticity, E (N/m²)</i>	<i>Second moment of area, J (m⁴)</i>	<i>Mass per unit length, μ (kg/m)</i>	<i>Structural damping, ζ</i>	<i>1st natural frequency of vibration, $f_{bridge1}$ (Hz)</i>	<i>2nd natural frequency of vibration, $f_{bridge2}$ (Hz)</i>
15	3.5×10^{10}	0.5273	28125	1% to 5%	5.66	22.62
25	3.5×10^{10}	1.3901	18358	1% to 5%	4.09	16.37
35	3.5×10^{10}	3.4162	21752	1% to 5%	3.01	12.03

The simulation of the half-car crossing the beam is based on the approach proposed by Frýba (1999) whereby the half car – beam interaction model is described by a system of coupled differential equations. These can be solved using standard numerical techniques. In this case, the system is solved using the

Wilson-Theta integration scheme (Bathe and Wilson, 1976; Tedesco *et al*, 1999). The value of θ used is 1.420815, which is the optimum value (Weaver and Johnston, 1987).

Smooth Profile Simulation Results for a Single Axle

The following simulations are carried out using the VBI model outlined in the previous section with a smooth road surface profile. The properties which will be varied in these simulations are the bridge span and structural damping, and the half-car mass and velocity. The theoretical frequencies of the bridge and half-car given in Tables 1 and 2 will be located in the acceleration spectra and compared to the frequencies estimated from the vehicle accelerations. The scanning frequency used in all simulations is 8192 Hz. The results presented in the following sections are for a half-car mass of 18 tonnes and velocity of 20 m/s.

15-metre Simulation Results

Figure 2 shows an example of the power spectrum of processed accelerations obtained from the front axle of the half-car as it crosses the 15 metre bridge. All simulated structural damping levels are represented on this figure. The resolution of the acceleration spectra is ± 0.25 Hz. A small peak exists close to 1 Hz which can be attributed to the frequencies of vibration of the half-car body mass for the 18 tonne model, f_{bounce} and f_{pitch} respectively (Table 1). A clear peak is visible at 6 Hz which corresponds to the first natural frequency of the bridge, $f_{bridge1}$ (Table 2). The peak deviates slightly from the dashed line representing the bridge frequency. Driving the vehicle lower velocity would improve the accuracy of this peak as more measurements would result into a higher frequency resolution. At this peak, it can be seen that there is a decrease in Power Spectral Density (PSD) magnitude for increasing structural damping level. The sensitivity of this decrease to a 1% change in damping is greater for changes between lower levels of damping. For example, for a change from 0% to 1% damping, the sensitivity of the PSD is 20% and from 1% to 2% the sensitivity is 19%. This sensitivity is obtained by calculating the difference between the PSD peaks and dividing it by the PSD peak magnitude at the lower damping level e.g. for the change from 0% to 1%, the difference between the PSD peaks is divided by the PSD peak magnitude for 0%. This trend also existed for other velocities and vehicle masses investigated.

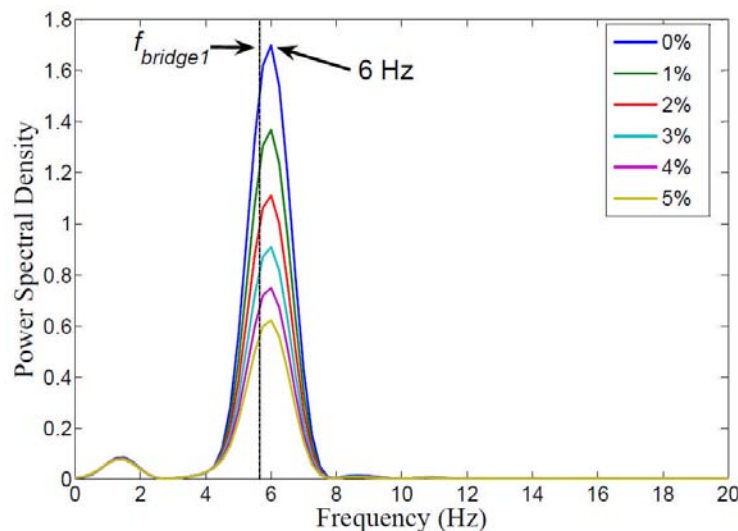


Figure 2. Acceleration spectra (PSD in m^2/s^3) for front axle of 18 tonne half-car travelling across 15-m bridge with smooth road profile. Damping varies from 0% to 5% and vehicle velocity is 20 m/s.

25-metre Simulation Results

The results obtained for the 25 metre bridge were similar to those obtained for the 15 metre bridge. Figure 3 shows the spectra of front axle accelerations obtained at 20 m/s for the 18 tonne vehicle. The resolution of the acceleration spectra is ± 0.125 Hz. A minor peak can be seen close to 1 Hz which corresponds to the frequencies of vibration of the half-car body mass. Once again a clear peak is visible in the spectra; this time at 4.25 Hz which corresponds to the first natural frequency of the 25 metre bridge. There is a slight inaccuracy at the peak similar to the one seen in Figure 2. There is also a decrease in PSD magnitude for increasing structural damping level at the bridge frequency peak. Once more, the sensitivity of this decrease in PSD to a 1% change in damping is greater for changes between lower levels of damping. For example, for a change from 0% to 1% damping, the sensitivity of the PSD is 24%, from 1% to 2% the sensitivity is 23% and from 4% to 5% the sensitivity is 21%. It is observed that the sensitivities are greater for this span than the 15 metre bridge, although it may also be taken into account that the absolute value of PSD is smaller and the final accuracy in detecting changes will also depend on the resolution of the accelerometers.

Two secondary peaks occur in the spectra in Figure 3 in the region of the second natural frequency, $f_{bridge2}$, of the 25 metre bridge. The decrease in PSD magnitude for increasing structural damping level is also seen at these two higher frequency peaks.

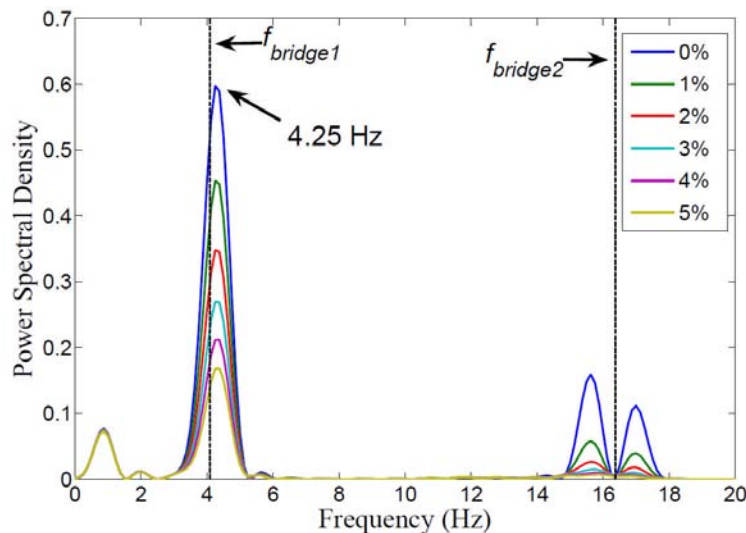


Figure 3. Acceleration spectra (PSD in m^2/s^3) for front axle of 18 tonne half-car travelling across 25-m bridge with smooth road profile. Damping varies from 0% to 5% and vehicle velocity is 20 m/s.

35-metre Simulation Results

Extending the discussion to the 35 metre bridge, there are some differences in the results compared to those obtained for the 15 and 25 metre bridges. Figure 4 shows the spectra of front axle accelerations obtained at 20 m/s for the 18 tonne vehicle. The resolution of the acceleration spectra is ± 0.125 Hz. Minor peaks can be seen close to 1 Hz and 3 Hz and these correspond to the frequencies of vibration of the half-car body mass and the first natural frequency of the 35 metre bridge respectively. Two clear peaks are visible in the spectra again, however, this time the peaks occur close to the second natural frequency of the 35 metre bridge; at 11.5 Hz and 12.5 Hz respectively, and they are much more significant than the peaks occurring at lower frequencies. They can be attributed to resonance which

occurs due to the second natural frequency of the 35 metre bridge (12.03 Hz) being almost equal to the axle hop frequency of the front axle of the half car (11.9 Hz). The decrease in PSD magnitude for increasing structural damping level is identifiable at these two frequency peaks too. Analysing the larger of the two peaks (at 11.5 Hz) there is a similar sensitivity trend to that seen earlier in results for the 15 and 25 metre bridges. For a change from 0% to 1% damping, the sensitivity of the PSD at 11.5 Hz is 65%, from 1% to 2% the sensitivity is 55% and from 4% to 5% the sensitivity is 31%. The sensitivities noted here are greater for this span than those for the 15 and 25 metre bridges. For this particular bridge, resonance is shown to cause a beneficial effect on vehicle response, amplifying some of the bridge properties being sought.

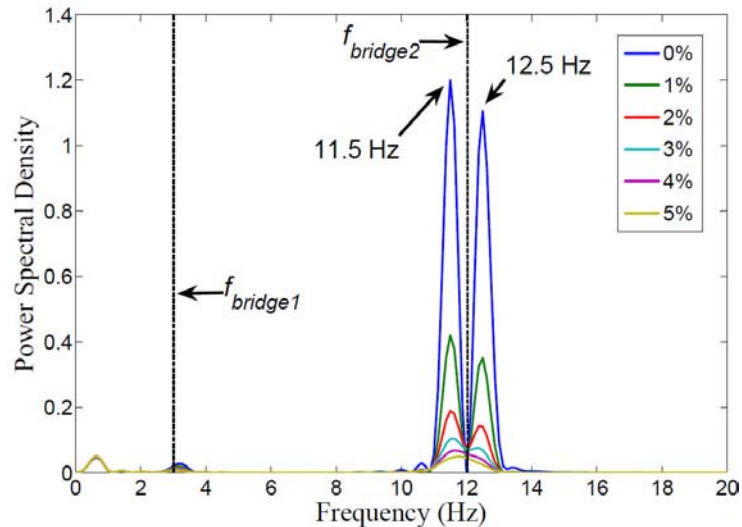


Figure 4. Acceleration spectra (PSD in m^2/s^3) for front axle of 18 tonne half-car travelling across 35-m bridge with smooth road profile. Damping varies from 0% to 5% and vehicle velocity is 20 m/s.

Summary of Smooth Profile Simulation Results

The results for the smooth profile simulations were in general positive. A peak was obtained corresponding to the first natural frequency of the 15 and 25 metre bridges and a peak also occurred at the second natural frequency for the 35 metre bridge. The accuracy of these frequency peaks depended on the velocity of the vehicle and the length of the bridge span as generally the deviation of the location of the highest peak in the frequency domain with respect to the true value occurred at greater velocities and shorter spans due to the reduction in time for the vehicle to record data as it crosses the bridge, i.e., poorer spectral resolution at higher velocities. For the 35 metre bridge, a resonance effect due to the axle hop frequency of the vehicle matching the second natural frequency of the bridge was found to favour the identification of a change in damping properties.

For all the frequency peaks in the spectra shown in Figures 2 to 4, a decrease in peak PSD magnitude exists for an increase in structural damping of the bridge. The percentage decreases in peak PSD with damping are very sensitive for the smooth profile simulations and suggest that the detection of bridge structural damping is possible through periodic measurements. The peak PSD is more sensitive to changes in damping between lower levels, i.e. larger changes in peak PSD occur between 0% and 1% damping than between 1% and 2% etc. In the results presented the 35 metre bridge provided the maximum sensitivity of 65% to a 1% change in damping at 20 m/s. Similar results were obtained in the

spectra of accelerations for the other vehicle masses of 4.8 tonnes and 9 tonnes. An illustration of the primary difference between the results for varying vehicle mass is shown in Figure 5. Only the spectrum of axle accelerations for a 15 metre bridge with 3% damping is plotted but this is representative of the trend occurring for other spans and damping levels. The magnitude of PSD at the bridge peak for smooth profile increases with increasing vehicle mass, being easier to measure by an accelerometer. This is a consequence of a heavier vehicle increasing the bridge deflections, and in turn, the vehicle vibration. However, the relative sensitivities of the PSD at the bridge peaks to changes in damping do not vary with vehicle mass significantly. It should be observed that results are shown for a vehicle velocity of 20 m/s which corresponds to highway speeds, i.e., a travelling condition where the instrumented vehicle would not cause traffic disruption while monitoring a bridge. Despite the encouraging results discussed here, a smooth profile is not sufficiently realistic as it does not account for the effect of a road profile roughness on the vibration of the vehicle, which will be discussed in the section that follows.

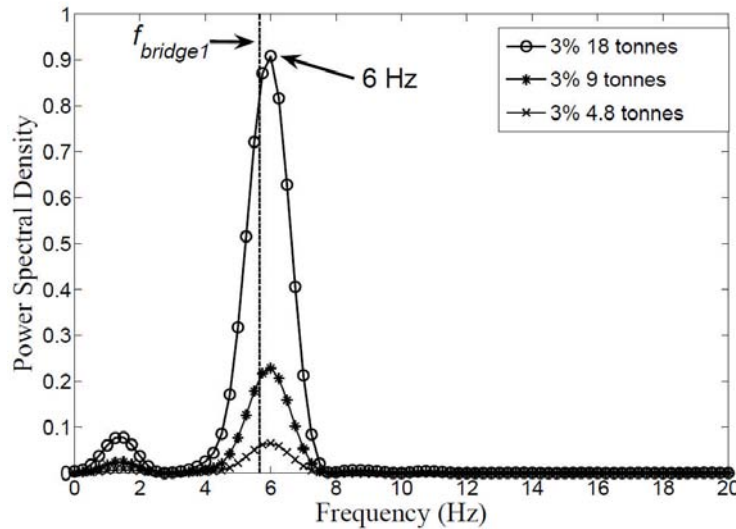


Figure 5. Acceleration spectra (PSD in m^2/s^3) for front axle of half-car travelling across 15-m bridge with smooth road profile. Damping is 3% and vehicle velocity is 20 m/s.

Rough Road Profile Simulation Results for a Single Axle

The importance of the road surface profile in the dynamic interaction between the vehicle and bridge has been highlighted by González *et al* (2008) and McGetrick *et al* (2009). Therefore, a rough road profile is now included in simulations to investigate its effect. Three rough road profiles are used in simulations. The road irregularities of each profile are randomly generated according to ISO (1995); the first two are ‘very good’ profiles falling within road class A, and the third is a ‘good’ profile defined as road class B. Due to similarity between results obtained for each road profile and bridge span, only the results from the simulations for the first class A road profile and 15 metre bridge are presented in the following sections unless otherwise stated. As for the smooth profile simulations, the properties varied in these simulations are the bridge span, bridge structural damping and the half-car mass and velocity. The scanning frequency used in all simulations is 8192 Hz. The results presented in this section are for a half-car mass of 18 tonnes and velocity of 20 m/s; spectra obtained for the 4.8 tonnes and 9 tonnes model gave similar results and are therefore not included.

15-metre Simulation Results

The spectra of accelerations recorded on the front axle of the half-car in the class A road profile simulation is shown in Figure 6. The resolution of the acceleration spectra is ± 0.25 Hz. In this figure the

clear peak with the largest magnitude occurs at 11.75 Hz which corresponds to the axle hop frequency of the half car, f_{axle1} . No peak is visible in the region around the first natural frequency of the 15 metre bridge. Also, the spectra for the damping levels are indistinguishable from each other for the scale shown in the figure. The rough road profile interferes with the ability to detect bridge frequency and changes in structural damping due to its considerable influence on the dynamic response of the vehicle. By referring back to the PSD magnitudes of Figure 2, it can be seen that this ISO class A road profile produces a dynamic vehicle response which is approximately 1000 times larger than the response due to a perfectly smooth road profile.

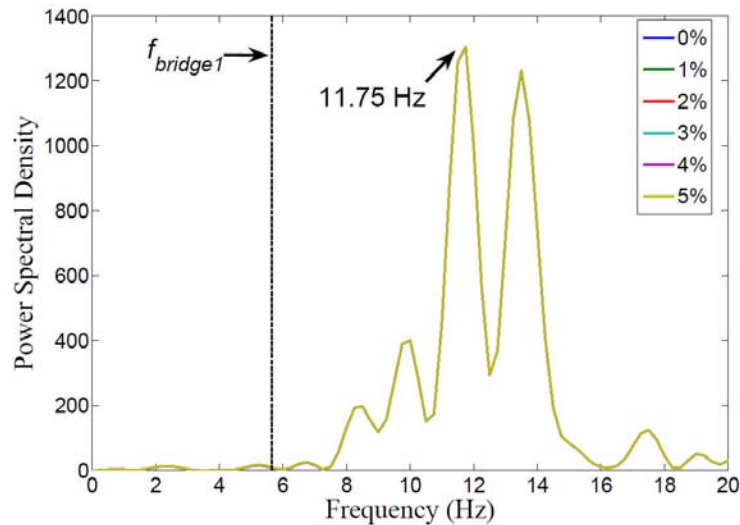


Figure 6. Acceleration spectra (PSD in m^2/s^3) for front axle of 18 tonne half-car travelling across 15-m bridge with ISO Class A road profile. Damping varies from 0% to 5% and vehicle velocity is 20m/s.

However, if the sensitivity of the PSD at all frequencies to 1% changes in damping level is calculated, it can be shown in Figure 7 that in the region of the first natural frequency of the bridge, the PSD is still quite sensitive to changes in damping, despite the lack of a significant peak at this frequency in Figure 6.

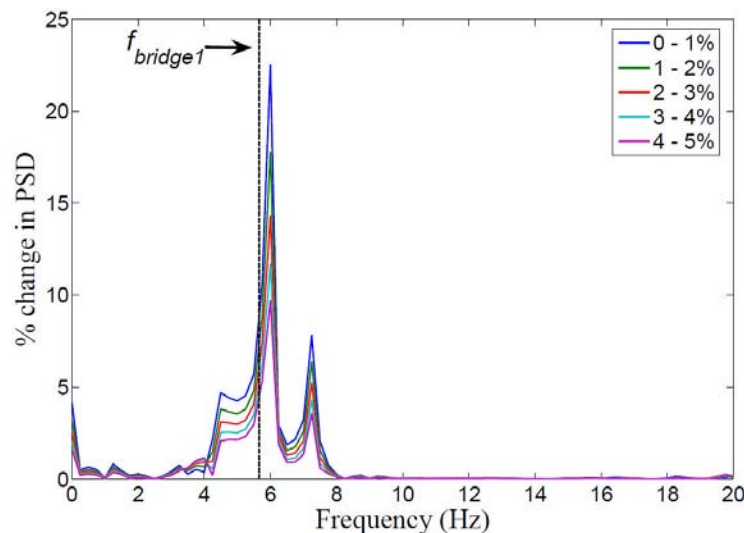


Figure 7. Sensitivity of overall PSD (in m^2/s^3) of accelerations for front axle of 18 tonne half-car travelling across 15-m bridge with ISO Class A road profile, vehicle velocity is 20 m/s.

Summary of Rough Road Profile Results

The results for the rough profile simulations have only been shown for the 15 metre bridge as similar conclusions arose for the 25 metre and 35 metre bridges. The axle hop vibration of the vehicle dominates all of the spectra due to the presence of a rough road profile. This is due to the ratio of the height of road irregularities to bridge deflections being too large which prevents the bridge having a more significant influence on the vehicle. Using a heavier vehicle than the relatively low weight vehicles used in this investigation would increase the bridge contribution to the vehicle vibration and improve results. However, by analysing the difference in the PSD of the entire spectrum between the different damping levels, it is clear that the vehicle is still sensitive to changes in damping in the region of the bridge frequency, even if the bridge frequency peak does not appear to be that relevant compared to other peaks in the spectrum. The relationship between vehicle mass and peak PSD magnitude in simulations with rough road profile is illustrated in Figure 8. As for the smooth road profile, the spectrum of axle accelerations for a 15 metre bridge with 3% damping is plotted and similarly, the peak PSD magnitude increases for increasing vehicle mass.

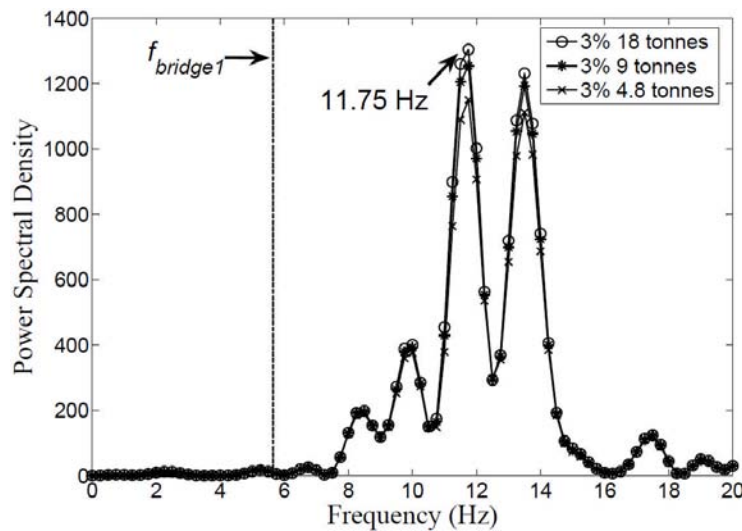


Figure 8. Acceleration spectra (PSD in m^2/s^3) for front axle of half-car travelling across 15-m bridge with 'very good' road profile. Damping is 3% and vehicle velocity is 20 m/s.

Characterisation of relationship between PSD and road profile roughness

It is obvious that the road profile roughness is a crucial element in the VBI and any attempt to extract bridge properties from vehicle measurements. Therefore, it is useful to characterise the effect of the road profile on the ability to detect bridge frequency and changes in the structural damping of the bridge. The following figures summarise the effect of the road profile roughness on the dynamic response of the vehicle for all 4 profiles investigated (Smooth, ISO class A-1, ISO class A-2 and ISO class B). Results are presented for the vehicle mass of 18 tonnes, vehicle velocity of 20 m/s and bridge span of 15 metres only. Similar conclusions were obtained for other vehicle masses, velocities and bridge spans. Figure 9 illustrates the relationship between road profile roughness and the absolute difference in PSD magnitudes at the first natural frequency bridge peak. The road profile roughness is represented here by the roughness coefficient, G_d (m^3/cycle), of each profile. The PSD differences are calculated at the first natural frequency of the bridge in the spectra of accelerations from simulations for each road profile. For example, for the smooth profile, $G_d=0$, and the difference in PSD is calculated from the corresponding

peak at $f_{bridge1}$ in Figure 2. Likewise, for the first ISO class A ‘very good’ road profile, $G_d = 5.233 \times 10^{-6} \text{ m}^3/\text{cycle}$ and the difference in PSD is calculated at $f_{bridge1}$ in Figure 6, even though there is not a clear significant peak at this location. For the ISO class A-2 ‘very good’ road profile, $G_d = 9.045 \times 10^{-6} \text{ m}^3/\text{cycle}$ and for the ISO class B ‘good’ road profile, $G_d = 50.97 \times 10^{-6} \text{ m}^3/\text{cycle}$. Figure 9 shows that the magnitude of the difference in PSD between different levels of damping is larger between the lower levels regardless of road roughness, i.e., the difference in PSD for 0% to 1% is greater than the difference for 1% to 2% etc.

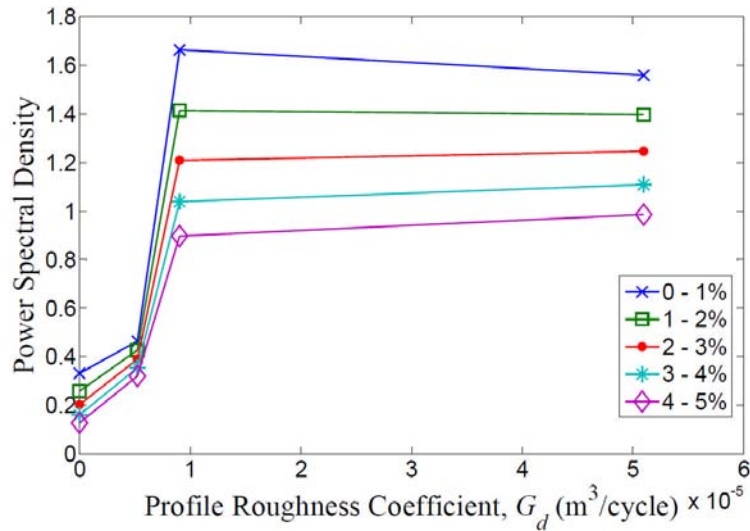


Figure 9. Difference in PSD of accelerations at the first natural frequency of 15-m bridge vs. road profile roughness (PSD is in m^2/s^3)

Figure 10 illustrates the magnitude of the maximum PSD at the first natural frequency of the 15 metre bridge for different road roughness relative to the magnitude of the differences shown in Figure 9. Changes in damping at the bridge frequency will be easier to detect when the difference in PSD is comparatively large relative to the maximum PSD to avoid interference due to changes in the road profile, noise, changes in lateral position of the vehicle etc. Figure 10 shows how as the road roughness increases, the difference in PSD becomes smaller relative to the maximum PSD. Therefore the greater the road profile roughness, the more difficult it will become to detect changes in damping as the maximum PSD magnitude increases at a much greater rate relative to the difference in PSD magnitude. This reiterates the need to remove or minimise the influence of road profile on the vibrations of the vehicle before a spectrum analysis on the region of the bridge frequency can be carried out. The identification of a change in damping could be facilitated through frequency matching between bridge and vehicle (i.e. at the second bridge frequency in Figure 4) or a higher dynamic excitation of the bridge due to other sources (i.e. caused by a very heavy truck, a number of trucks or a bump prior to the bridge).

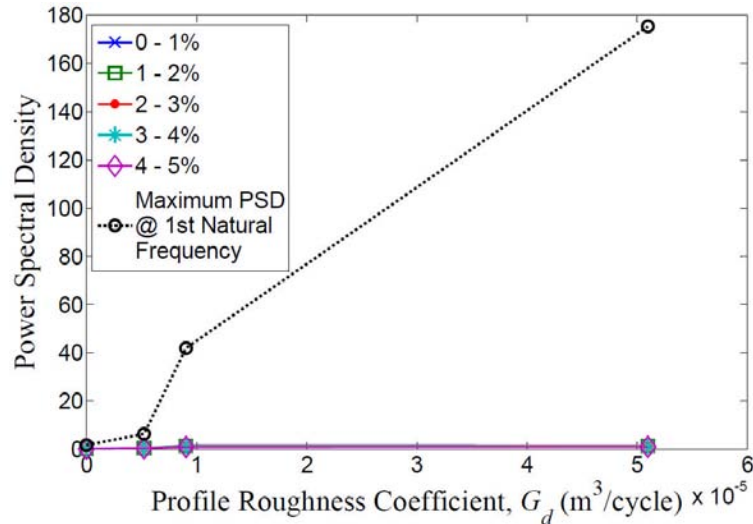


Figure 10. Difference in PSD magnitude and Maximum PSD magnitude at the first natural frequency of 15-m bridge vs. road profile roughness (PSD is in m^2/s^3)

Conclusions

This paper has investigated the feasibility of using an instrumented vehicle to detect the natural frequency and structural damping of a bridge. In past studies, it has been shown that it is possible to obtain the bridge frequency for smooth road profiles from the vehicle vibration but it is more difficult to do so with rougher road profiles. The results presented in this paper are found in agreement with those studies. Using a heavier vehicle would increase the bridge deflection and consequently increase the bridge influence on the vehicle vibration improving results. Also, frequency matching between the axle hop of the vehicle and the natural frequency of the bridge is beneficial for the detection of that bridge frequency. This paper has shown that the magnitude of PSD at the bridge frequency decreases with increasing bridge damping. This decrease is easy to obtain and quantify for a smooth profile due to the presence of a dominant bridge frequency peak. For a rough road profile there is no dominant bridge frequency peak but by analysing the whole spectrum in the region of the bridge frequency, a lower PSD also appears in this region as a result of a decrease in damping. The magnitude of the PSD for an axle of the half-car was found to increase for both increasing vehicle mass and road profile roughness. Overall, it was found that the greater the road profile roughness, the more difficult it becomes to detect changes in damping as the effect of a change in damping in the PSD becomes relatively negligible compared to the effect of a change in the road profile. The development of an instrumented vehicle as a sensitive low-cost method for the monitoring of the dynamic behaviour of short to medium span bridges requires to successfully identify bridge parameters in the presence of different degrees of road roughness. Further research is needed for the removal or minimisation of the influence of the road profile roughness on the vehicle vibration and enable a better definition of the region around the bridge frequency within the vehicle spectrum.

Acknowledgements

The authors wish to express their gratitude for the financial support received from the 7th European Framework ASSET Project towards this investigation.

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