Title	Theoretical investigation of the use of a moving vehicle to identify bridge dynamic parameters
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Publication date	2009-08
Publication information	McGetrick, P., Arturo González, and Eugene J. O'Brien. "Theoretical Investigation of the Use of a Moving Vehicle to Identify Bridge Dynamic Parameters" 51, no. 8 (August, 2009).
Publisher	British Institute of Non-Destructive Testing
Item record/more information	http://hdl.handle.net/10197/2552
Publisher's version (DOI)	10.1784/insi.2009.51.8.433

 DOI: 10.1784/insi.2009.51.8.433

Theoretical investigation of the use of a moving vehicle to identify bridge dynamic parameters

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Paper presented at CM 2009-MFPT 2009, the Sixth International Conference on Condition Monitoring and Machinery Failure Prevention Technologies, Dublin, Ireland, June 2009*

Pavements and bridges are subject to a continuous degradation due to traffic aggressiveness, ageing and environmental factors. A rational transport policy requires the monitoring of this transport infrastructure in order to provide adequate maintenance and guarantee the required levels of transport service and safety. This paper investigates the use of an instrumented vehicle fitted with accelerometers on its axles to monitor the dynamics of bridges. A simplified quarter car-bridge interaction model is used in theoretical simulations and the natural frequency of the bridge is extracted from the spectra of the vehicle accelerations. The accuracy is better at lower speeds and for smooth road profiles. The structural damping of the bridge was also monitored for smooth and rough road profiles. The magnitude of peaks in the power spectral density of the vehicle accelerations decreased with increasing bridge damping and this decrease was easier to detect the smoother the road profile.

1. Introduction

Increasingly in recent years, larger bridges are being instrumented and monitored on an ongoing basis. However, the installations are expensive, requiring installation of equipment on the bridge, typically to measure bridge modes and frequencies of vibration. This paper investigates the use of a moving vehicle fitted with accelerometers on its axles to monitor the dynamic behaviour of bridges. This approach eliminates the need for on-site installation of measurement equipment and is aimed at allowing the assessment of infrastructure condition to become a simplified and much less onerous proposition. It would enable more effective and widespread monitoring of existing bridge structures' condition while its development would allow maintenance to be instigated at an earlier stage in degradation, which (usually) results in less costly repairs.

The feasibility of extracting bridge frequencies from the dynamic response of a vehicle passing over a bridge has been verified theoretically by Yang *et al*(1). The method Yang *et al* investigated would only require an instrumented vehicle acting as a 'message carrier' of the dynamic properties of the bridge. The bridge was assumed to be a simply supported beam while the vehicle was modelled as a sprung mass. They extracted the bridge natural frequency from the vehicle acceleration spectrum of the sprung mass and found that the magnitude of the peak response increased with vehicle speed but decreased with increasing bridge damping ratio. They concluded that the bridge natural frequency should be close to the vehicle frequency so that the vehicle can be excited to resonance through adjustment of its speed.

This technique was verified experimentally by Lin and Yang(2). They highlighted the fact that a measured drop in frequency of vibration indicates a drop in stiffness of the bridge, due to damage or ageing. This allowed them to conclude that a bridge's condition or structural integrity can be assessed by measurement of a bridge's frequency of vibration and also that 'the methodology developed for the first time herein is simple and efficient, and can be applied to a wide range of bridge problems'. They used a small tractor-trailer system and travelled across one simply supported 30 m span of a six-span bridge. The tractor was a four-wheel commercial light truck and served as the exciter of the bridge into vibration. The trailer was fitted with an accelerometer, serving as the receiver of the motions of the already excited bridge. Their field tests verify that bridge frequency can be easily identified from trailer response for vehicle speeds less than 40 km/h (11.11 m/s). At larger speeds the bridge frequencies become blurred by high frequency components from pavement roughness and trailer structure. They also found that ongoing traffic had a beneficial effect on results as the vehicle response increased.

González *et al*(3) extended the analysis to other test conditions. They built a 3-D FEM vehicle-bridge interaction model to test the approach numerically for various speeds, road roughness, damping levels and traffic conditions. They also carried out an experimental test in a main route near Oviedo, Northern Spain, where 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. They concluded that an accurate determination of the bridge frequency is only feasible for low speeds and when the degree of dynamic excitation of the bridge is high enough. Therefore, the interference of the road profile frequencies corrupts the spectrum and prevents the identification of the bridge natural frequency. The inability to detect the bridge frequency at the experimental test was attributed to the relatively high vehicle speed and the lack of heavy traffic on the bridge that would ensure the bridge deflection to be significant compared to the height of the road irregularities.

In this paper, a vehicle-bridge interaction model is created for theoretical simulations using MATLAB. The vehicle is modelled as a quarter car and the bridge is modelled as an Euler-Bernoulli beam. This investigation involves analysis of the frequency spectra of vertical vehicle accelerations obtained from the dynamic response of the quarter 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 quarter 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, gross vehicle weights of 10 and 20 tonnes and for smooth and rough road surface profiles. Structural damping has been shown to be damage sensitive(4), therefore 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 is used as a reference.

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**This paper won the Len Gelman Award for the best paper presented at the conference by a person in the early stage of their career. Five referees (including Professor Gelman) selected the paper for this Award.*

This is the first study known to the authors that uses an instrumented vehicle to identify not only bridge frequencies but also changes in bridge damping. The results will indicate the conditions in which this method can be used to estimate bridge dynamic parameters with a reasonable degree of accuracy.

2. Theoretical simulation

2.1 Vehicle-bridge interaction model

Although a quarter car is a simplified version of a vehicle, its response still illustrates many of the important characteristics of dynamic tyre forces[5]. The quarter car (Figure 1) is a 2-degree of freedom suspension model which consists of a sprung mass, m_s, and an unsprung mass, m_u, representing the body and tyre masses of the vehicle system, respectively. The tyre 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. This combination represents the suspension of the vehicle system. Its properties are shown in Table 1 and are based on values obtained from work by Cebon[5].

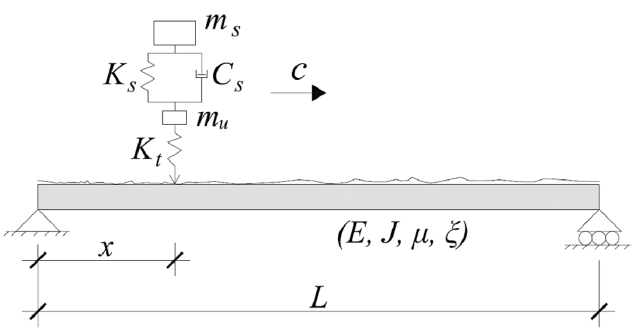

Figure 1. Quarter car – beam interaction model

Table 1. Quarter car properties

Property	Model 1 (10 tonnes)	Model 2 (20 tonnes)
Body mass, m_s	9000 kg	19,000 kg
Tyre mass, m_u	1000 kg	1000 kg
Suspension stiffness, K_s	8e4 N/m	8e4 N/m
Suspension damping, C_s	10e3 Ns/m	10e3 Ns/m
Tyre stiffness, K_t	2e6 N/m	2e6 N/m
Body mass frequency of vibration, f_{body}	0.47 Hz	0.32 Hz
Tyre mass frequency of vibration, f_{tyre}	7.26 Hz	7.26 Hz

The quarter 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 length L, modulus of elasticity E, second moment of area J and structural damping ξ. The properties of the three bridge spans used in these simulations are given in Table 2. The first natural frequency of each bridge is given also for future reference.

Table 2. Bridge properties

Span length, L (m)	Modulus of elasticity, E (N/m^2)	Second moment of area, J (m^4)	Mass per unit length, μ (kg/m)	Structural damping, ξ	1st natural frequency of vibration, f_b (Hz)
15	3.50E+10	0.5273	28125	1% to 5%	5.66
25	3.50E+10	1.3901	18358	1% to 5%	4.09
35	3.50E+10	3.4162	21752	1% to 5%	3.01

2.2 Smooth road profile simulations

The following simulations are carried out using the models outlined in Section 2.1 with a smooth road profile. The properties which will be varied in these simulations are the bridge span, bridge structural damping and the quarter car mass and velocity. Having already obtained the bridge and quarter car frequencies (Tables 1 and 2), we can locate and differentiate between them in the acceleration spectra and compare recorded frequencies to the tabulated values. The scanning frequency used for all simulations is 10,000 Hz. It should be noted that results presented in this section are for a quarter car mass of 10 tonnes and velocity of 20 m/s; spectra obtained for the 20 tonne model gave similar results and are therefore omitted.

2.2.1 15 m bridge span results

An example of the quarter car tyre mass accelerations obtained for the 15 m bridge span and 3% structural damping is shown in Figure 2.

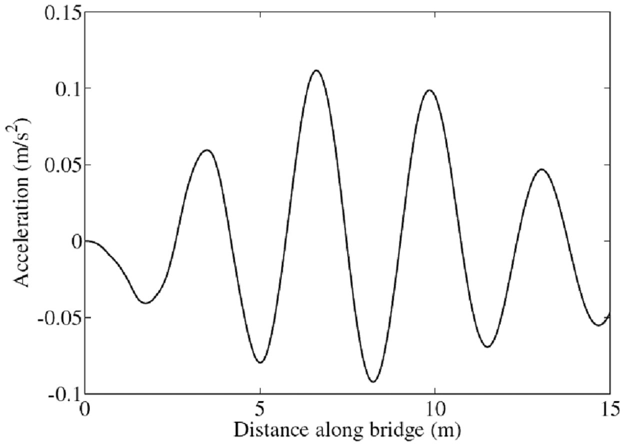

Figure 2. Vertical accelerations of tyre mass travelling across 15 m bridge at 20 m/s. Structural damping is 3%, smooth profile

The corresponding power spectrum of the processed accelerations is shown in Figure 3. The spectra of accelerations obtained at all the simulated structural damping levels are also illustrated in the Figure. A clear peak is visible at 6.1 Hz which corresponds to the first natural frequency of the bridge. The inaccuracy is due to the resolution of the spectra (± 1.22 Hz) which can be improved by driving the vehicle at slower speed. Also, it is clearly visible that the magnitude of the peak decreases for higher levels of damping.

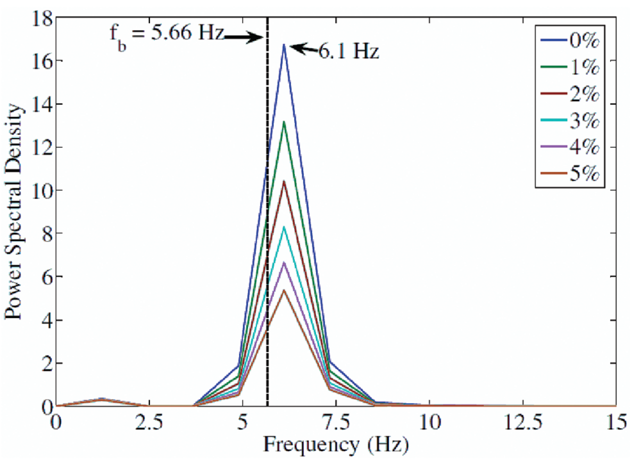

Figure 3. Acceleration spectra for tyre mass at 20 m/s on 15 m bridge. Damping varies from 0% to 5%, quarter car mass is 10 tonnes, smooth profile

Figure 4 shows the sensitivity of the peak power spectral density (PSD) of the tyre mass accelerations to a 1% change in structural damping, between 0% and 5%. Values for all velocities are included here. For example, at 5 m/s the percentage decrease in peak PSD between 0% and 1% damping levels is represented by the dark blue bar and has a value of 62%. At the same velocity, the percentage decrease in peak PSD between 2% and 3% damping levels is represented by the green bar and has a value of 52%. The percentage decrease in peak PSD is 44% between damping levels of 4% to 5% and is represented by the red bar. From this a clear trend can be seen for a velocity of 5 m/s; the PSD is more sensitive to changes in damping between lower levels. This trend also exists for the other velocities investigated as can be seen in Figure 4. Further, comparing sensitivities between velocities, it is clear from Figure 4 that changes in the peak PSD are easier to see for lower velocities. The overall highest percentage decrease (62%) occurred between 0% and 1% for a velocity of 5 m/s and the lowest decrease (16%) occurred between 4% and 5% for a velocity of 25 m/s. The 20 tonne quarter car model produced similar results.

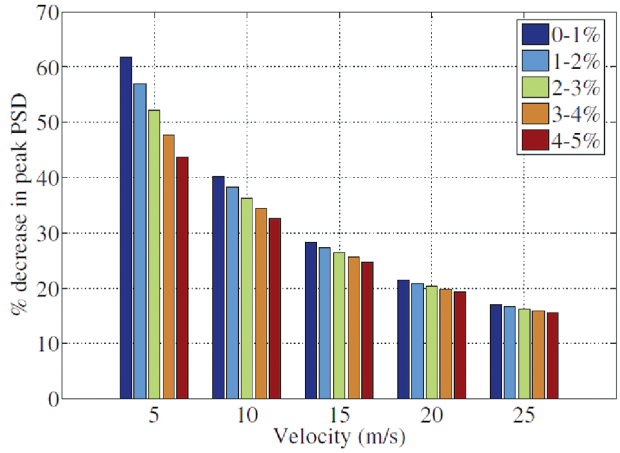

Figure 4. Peak PSD-damping trends at bridge frequency peak for tyre mass on 15 m bridge

2.2.2 25 m bridge span results

The simulations for the 25 m bridge provided similar trends to those for the 15 m bridge. Figure 5 shows the spectra obtained for accelerations at 20 m/s. The resolution of the spectra is ± 0.61 Hz; it is better than the resolution from the 15 m bridge results as more data was obtained due to the longer length of the bridge in this simulation. Once again, a clear peak is visible, this time at 4.27 Hz, which corresponds to the first natural frequency of the bridge. The resolution of the spectra has caused this slight inaccuracy.

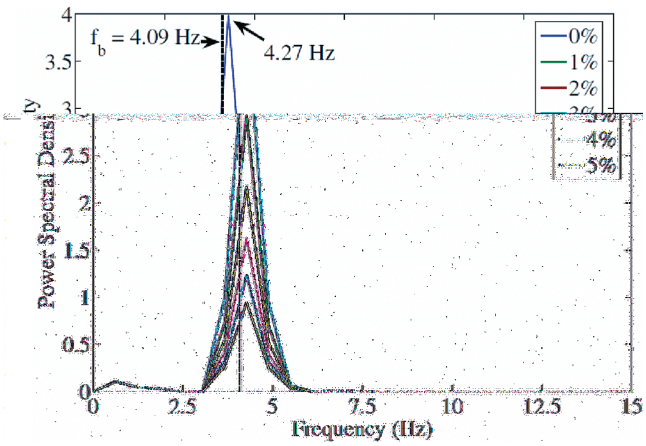

Figure 5. Acceleration spectra for tyre mass at 20 m/s on 25 m bridge. Damping varies from 0% to 5%, quarter car mass is 10 tonnes, smooth profile

Figure 6 shows the sensitivity of the peak PSD of the tyre mass accelerations to a 1% change in structural damping, between 0% and 5%, for the 25 m bridge. Values for all velocities are included here. A similar trend to that found for the 15 m bridge is visible; for each velocity the PSD is more sensitive to changes in damping between lower levels.

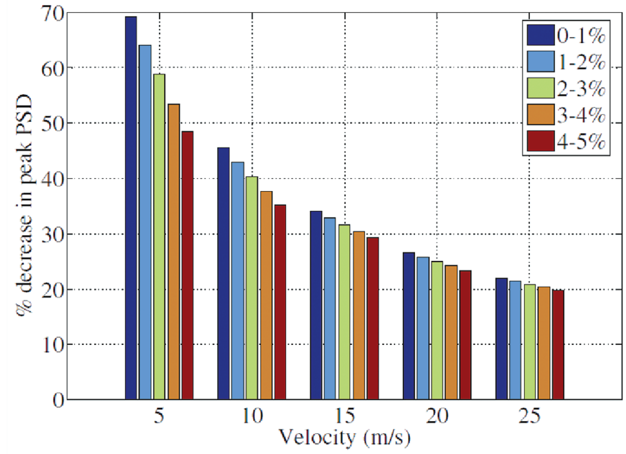

Figure 6. Peak PSD-damping trends at bridge frequency peak for tyre mass on 25 m bridge

Also, Figure 6 illustrates that changes in the peak PSD are easier to see for lower velocities. While both these trends are similar to those found for the 15 m bridge, the magnitudes of the decreases for the 25 m bridge are slightly more significant. The maximum percentage decrease is 69% which occurred between 0% and 1% for a velocity of 5 m/s; this value is greater than the maximum decrease of 62% for the 15 m bridge. The lowest decrease is 20% and this occurred between 4% and 5% for a velocity of 25 m/s and is greater than the minimum of 16% obtained for the 15 m bridge. This suggests it is easier to detect damping in the 25 m bridge. The 20 tonne quarter car model produced similar results.

2.2.3 35 m bridge span results

The simulations for the 35 m bridge gave similar trends to those seen in Sections 2.2.1 and 2.2.2. Figure 7 shows the spectra obtained for accelerations at 20 m/s. A clear peak is visible again at 3.05 Hz, which corresponds to the first natural frequency of the bridge. The resolution of the spectra in Figure 6 is ± 0.31 Hz and the bridge frequency can be identified more accurately than in the 15 m and 25 m bridges.

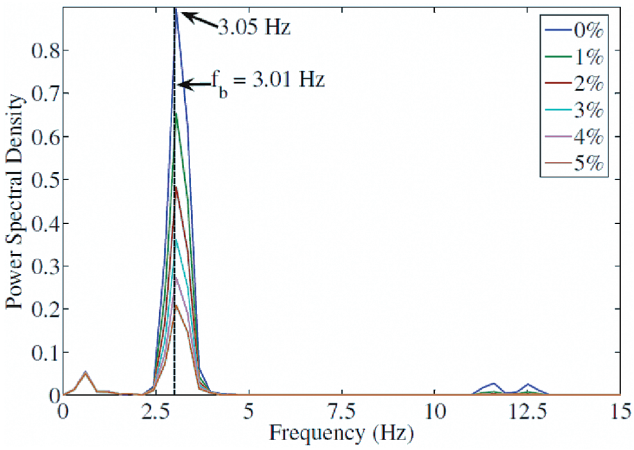

Figure 7. Acceleration spectra for tyre mass at 20 m/s on 35 m bridge. Damping varies from 0% to 5%, quarter car mass is 10 tonnes, smooth profile

Figure 8 shows the peak PSD-damping trend for the 35 m bridge, which illustrates a similar trend seen for the 15 m and 25 m bridges. It is easier to detect changes in damping between lower levels and also the changes are easier to detect at lower velocities. The maximum percentage decrease is 71% which occurred between

0% and 1% for a velocity of 5 m/s; this value is greater than the maximum decreases of 62% and 69% for the 15 m and 25 m bridges, respectively. The lowest decrease is 20% and this occurred between 4% and 5% for a velocity of 25 m/s and is equal to the minimum of 20% obtained for the 25 m bridge. These results suggest that it is easier to detect changes in damping in the 35 m bridge. The 20 tonne quarter car model once again produced similar results for the 35 m bridge.

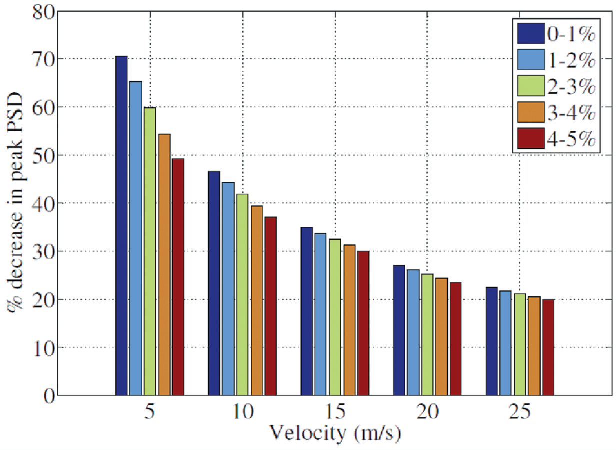

Figure 8. Peak PSD-damping trends at bridge frequency peak for tyre mass on 35 m bridge

2.2.4 Summary of smooth road profile simulations

For all the simulations with the smooth profile a peak was obtained corresponding to the natural frequency of the bridge. Figure 9 shows the obtained frequencies and the actual bridge frequencies for each span length and velocity of the quarter car model. The 10 and 20 tonne quarter car models both give the results shown in Figure 9. There is generally higher inaccuracy at greater velocities due to the reduction in time for the vehicle to record data as it crosses the bridge, *ie* poorer spectral resolution at higher velocities. However, Figure 9 also suggests that at velocities of 15, 20 and 25 m/s for the 15 m bridge and at 10, 15, 20 and 25 m/s for the 25 m and 35 m bridges the accuracy of the frequency peak is the same. This has occurred as the resolution at these velocities resulted in the same data point occurring close to the bridge frequency for each bridge length. If the velocity was further increased, the higher inaccuracy should be visible. The frequency peaks were most accurate for the 35 m bridge and least accurate for the 15 m bridge.

It has been noted that for all the frequency peaks in the spectra shown in this section, a decrease in peak magnitude exists for an increase in structural damping of the bridge. The percentage

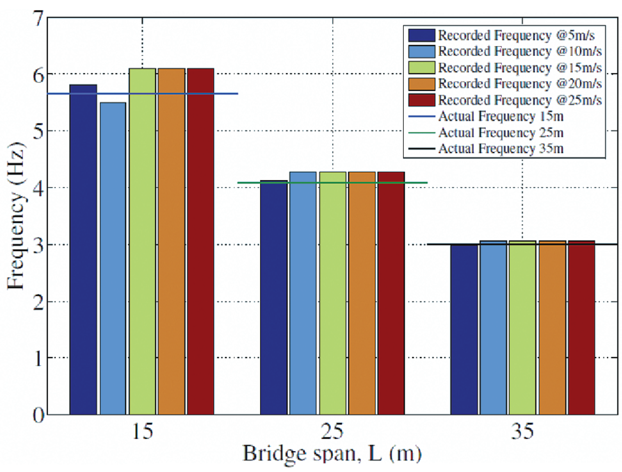

Figure 9. Estimated and true bridge frequency for all bridge spans and velocities

decreases in peak PSD with damping are very sensitive for the smooth profile simulations and suggest that it is possible to monitor damping through periodic measurements. It is more sensitive to changes in damping between lower levels, *ie* larger changes in peak PSD occur between 0% and 1% damping than between 2% and 3%. The peak PSD appears to be more sensitive at lower velocities but is still quite sensitive at higher velocities of 20 m/s and 25 m/s, which would correspond to highway speeds. These changes in damping were also easier to see as the bridge length increased; the 35 m bridge provided the maximum sensitivity of 71% to a 1% change in damping at 5 m/s. However, a smooth profile is not sufficiently realistic as it does not account for the effect of a road profile on the vibration of the vehicle.

2.3 Rough road profile simulations

The following simulations are carried out using the model outlined in Section 2.1 with a rough road profile. The road irregularities are randomly generated; defined as a class A (geometric spatial mean of 4e-06 m^3/cycle), 'very good' road profile according to ISO[6]. As in Section 2.2, the properties which will be varied in these simulations are the bridge span, bridge structural damping and the quarter car mass and velocity. The scanning frequency used for all simulations is 10,000 Hz once more. The results presented in this section are for a quarter car mass of 10 tonnes and velocity of 20 m/s unless otherwise stated; spectra obtained for the 20 tonne model gave similar results once more and can therefore be omitted.

2.3.1 15 m bridge span

Figure 10 shows the accelerations of the 10 tonne quarter car model travelling at 20 m/s over the 15 m bridge. The bridge damping is 3%. In comparison with Figure 2, the effect of the road profile is very noticeable and the accelerations have changed considerably.

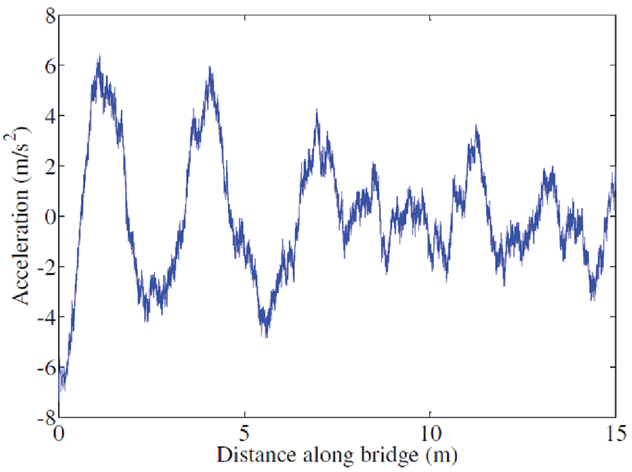

Figure 10. Vertical accelerations of tyre mass travelling across 15 m bridge at 20 m/s. Bridge damping is 3%, 'very good' profile

The power spectrum of these accelerations is shown in Figure 11. The spectra of accelerations obtained at all the simulated bridge damping levels are also included. The resolution of the spectra is ± 1.22 Hz. There is a peak at 7.32 Hz which corresponds to the frequency of the quarter car tyre mass; there is no peak corresponding to the bridge frequency. This is due to the dominating influence of the road profile on the vehicle vibration. However, at this peak it can still be seen that the peak magnitude decreases with increasing bridge damping.

For the rough road profile simulations a bridge frequency peak was only found only at a velocity of 5 m/s. Figure 12 shows the corresponding acceleration spectra. The resolution is ± 0.31 Hz. Two clear peaks are visible, one at 6.1 Hz, corresponding to the natural frequency of the bridge, and one at 7.32 Hz, corresponding

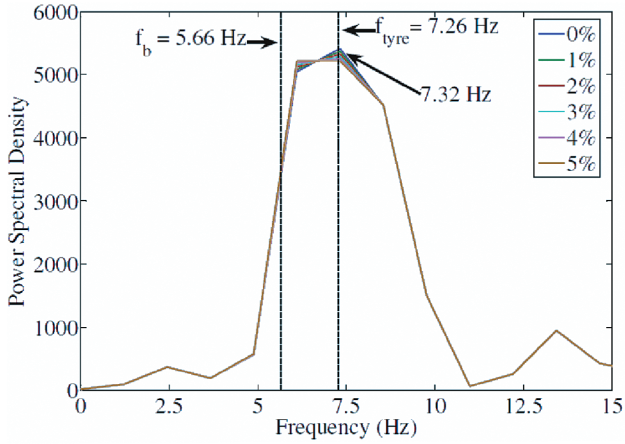

Figure 11. Acceleration spectra for tyre mass at 20 m/s on 15 m bridge. Damping varies from 0% to 5%, quarter car mass is 10 tonnes, 'very good' profile

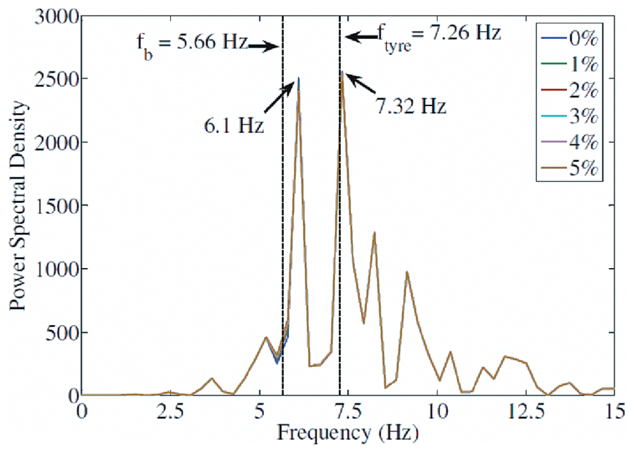

Figure 12. Acceleration spectra for tyre mass at 5 m/s on 15 m bridge. Damping varies from 0% to 5%, quarter car mass is 10 tonnes, 'very good' profile

to the tyre mass. On inspection of the bridge frequency peak, the peak PSD decreases with damping from 0% to 4%, similar to the trends seen in Section 2.2.

Figure 13 shows the sensitivity of this peak PSD to a 1% change in bridge damping, between 0% and 5%. Values for both quarter car masses are included here. A clear trend can be seen for each mass; the PSD is more sensitive to changes in damping between lower levels but between 4% and 5% the peak PSD actually increases. As this increase is so small, it is most likely due to noise from the road profile.

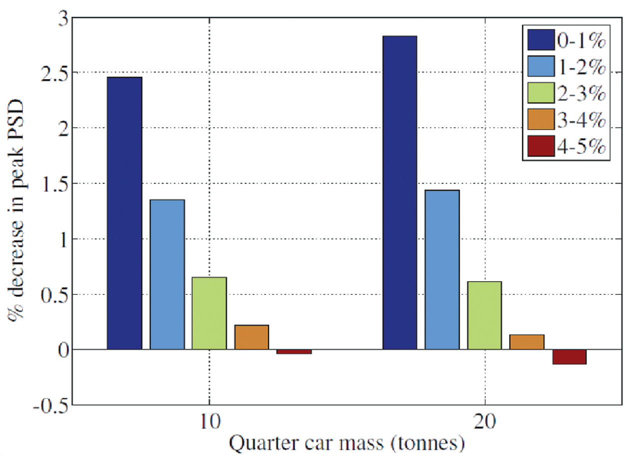

Figure 13. Peak PSD-damping trends at bridge frequency peak for tyre mass on 15 m bridge. Velocity is 5 m/s

At the tyre mass frequency peak of 7.32 Hz, the peak PSD is seen to decrease with increased bridge damping once again. This trend can be seen in Figure 14 for both quarter car masses but the sensitivity is quite low (maximum 0.35%). This illustrates that the magnitude of a vehicle frequency peak also varies with bridge damping.

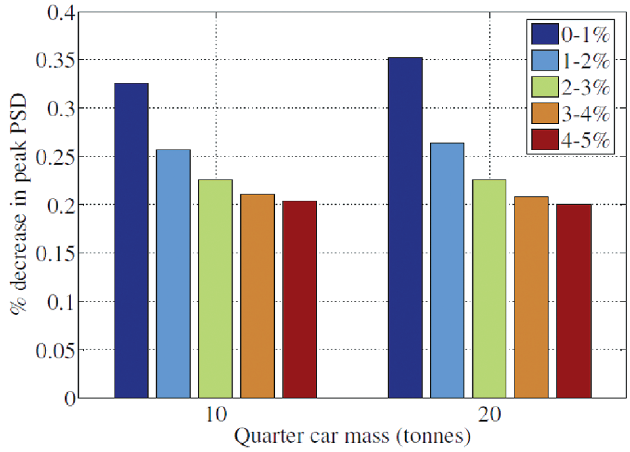

Figure 14. Peak PSD-damping trends at vehicle frequency peak for tyre mass on 15 m bridge. Velocity is 5 m/s

2.3.4 Summary of rough road profile simulations

The rough road profile simulation results are shown only for the 15 m bridge as those for the 25 m and 35 m provided similar conclusions. The vibration of the vehicle dominates all of the spectra due to the rough road profile. This is because the ratio of the height of road irregularities to bridge deflections is too large for the bridge to have an influence on the vehicle, mainly due to the low vehicle weight used in the simulations. However, at the obtained quarter car frequency peaks, the peak PSD magnitude decreases with bridge damping level. This suggests that even if a bridge frequency peak was not found, changes in bridge damping levels can be detected by analysing vehicle frequency peaks.

3. Conclusions

This paper has investigated the feasibility of using an instrumented vehicle to monitor the natural frequency and structural damping of a bridge. In past studies, it has been shown that it is feasible to obtain the bridge frequency at low vehicle speeds. This paper has confirmed that it is possible to obtain the bridge frequency at any speed with a smooth profile, but with a 'very good' road profile, the bridge frequency was only accurately obtained at 5 m/s. At higher speeds, the road profile's influence on the vehicle vibration dominates the spectra, hiding the bridge frequency. Use of a heavier vehicle should increase the bridge deflection, thus increasing the bridge influence on the vehicle vibration. This paper has shown that the magnitude of PSD at bridge and vehicle frequency peaks decreases with increasing bridge damping. This decrease is significant for a smooth profile but small for a 'very good' road profile. By analysing this decrease, changes in bridge damping could be efficiently monitored using the proposed instrumented vehicle. Further research is orientated to the removal or minimisation of the effect of the road profile on the spectra and if successful, the use of an instrumented vehicle could effectively become a sensitive low-cost method to be applied to the monitoring of short to medium span bridges.

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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