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Using Instrumented Quarter-Cars for ‘Drive By’ Bridge Inspection

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Summary

This paper investigates the concept of ‘drive by’ bridge inspection, a low cost alternative to Structural Health Monitoring (SHM), involving no sensors on the bridge. The concept may be of particular value after an extreme event, such as an earthquake or a flood, where a rapid indication of bridge condition is needed. Vehicle/bridge dynamic interaction is modelled to test the effectiveness of the approach. Damage is simulated here as a change in the bridge damping ratio. Two quarter-cars are simulated crossing the bridge with accelerometers on board. A frequency domain analysis then illustrates changes in the Power Spectral Density of the accelerations as the bridge becomes damaged. The time-lagged difference in the accelerations is found to be effective in detecting damage. Results are compared to those with sensors on the bridge and found to be similar.

Keywords: Bridge, damage, damping, dynamics, vehicle-bridge interaction, damage detection, drive-by.

1. Introduction

Bridges form a critical link in modern transport networks. However, in common with other Civil Engineering infrastructure, structures age and deteriorate over time. Traditionally, the task of detecting damage in bridges consists of visual inspections, which are labour intensive and are often an unreliable way of determining the true condition. With the improvement in sensor technologies and signal processing capacity, there has been a move towards sensor based analysis of bridge condition. Existing monitoring techniques generally involve the direct instrumentation of the structure – commonly referred to as Structural Health Monitoring (SHM) [1–3]. This requires the installation and maintenance of sensors and data acquisition electronics on the entire bridge stock which is expensive and time consuming. More recently, a small number of authors have shifted to the instrumentation of a vehicle, rather than the bridge, in order to assess bridge condition. This approach, referred to as ‘drive-by’ bridge inspection [4], has potential advantages in terms of reduced cost and ease of implementation.

Sensor based approaches are predicated on the assumption that changes in the physical properties of a bridge (indicating damage) cause changes in its modal properties. The feasibility of detecting changes in frequencies from the dynamic response of an instrumented vehicle passing over a bridge has been verified theoretically in [5]. This method was later tested in field trials [6,7] and laboratory investigations have also been carried out [4, 8, 9]. As an alternative to detecting changes in frequency, Yabe and Miyamoto [10] use the mean displacement of the rear axle of a city bus passing over a bridge a large number of times as a damage indicator. Kim et al [11] construct scaled Vehicle Bridge Interaction (VBI) laboratory experiments and consider the use of autoregressive coefficients as a damage indicator. The analysis of damping has been considered to a lesser extent in the field of damage detection [12]. However, recent evidence suggests that damping is quite sensitive to damage in structural elements and in some cases, more sensitive than natural frequencies [13, 14]. Furthermore, many researchers note that damping can be a useful damage sensitive feature and is highly indicative of the amount of damage that a structure has undergone during its lifetime [15–17].

This paper investigates the feasibility of ‘drive by’ bridge inspection. Damage is simulated here as a change in the bridge damping ratio. Firstly, two quarter-cars are simulated crossing a simply supported bridge where accelerometers are simulated at the mid span, measuring vertical acceleration. Separately accelerometers are placed on the axles of the vehicles and the time-lagged difference in the bridge deflections experienced by each quarter-car is found. A frequency domain analysis of both scenarios then illustrates changes in the Power Spectral Density (PSD) of the accelerations as the bridge becomes damaged. Results indicate that damage can be detected using sensors on the vehicle with a similar level of accuracy as using sensors on the bridge.

2. Vehicle-Bridge Interaction Model

Fig. 1: Two identical quarter-cars

VBI is modelled here as a coupled system, so the solution is given at each time step and no iteration is required in the computational process. The vehicle is represented by a pair of 2-degree-of-freedom unconnected quarter-cars (with identical properties), as illustrated in Fig. 1. The spacing between each quarter-car is 2 m. They travel at a constant speed of 20 m s^{-1} , giving a constant spacing. The properties of the two identical quarter-cars are listed in Table 1. All property values are based on values gathered from the literature [18–20].

Table 1: Quarter-Car Properties

Property	Symbol	Value	Unit
Body masses	$m_{s,1}, m_{s,2}$	9,300	kg
Axle masses	$m_{u,1}, m_{u,2}$	700	kg
Suspension stiffness	$K_{s,1}, K_{s,2}$	4×10^5	N m^{-1}
Suspension damping	$C_{s,1}, C_{s,2}$	10×10^3	Ns m^{-1}
Tyre stiffness	$K_{t,1}, K_{t,2}$	1.75×10^6	N m^{-1}
Moment of inertia	$I_{s,1}, I_{s,2}$	0.5365	kg m^2
Body bounce frequency	$f_{body,1}, f_{body,2}$	0.9405	Hz
Axle hop frequency	$f_{axle,1}, f_{axle,2}$	8.832	Hz

The bridge model used here is a simply supported 15 m Finite Element beam that consists of twenty discretized beam elements with four degrees of freedom. The beam therefore has a total of $n = 42$ degrees of freedom. It has a constant modulus of elasticity $E = 3.5 \times 10^{10} \text{ N m}^{-2}$, mass per unit length, $\mu = 28 \text{ 125 kg m}^{-1}$ and second moment of area, $J = 0.5273 \text{ m}^4$. The first natural frequency of the beam is 5.65 Hz.

The irregularities of the road profile are randomly generated according to the ISO standard [21] – Fig. 2. A Class ‘A’ road (very good profile, as expected in a well maintained highway), is considered which has a geometric spatial mean of $16 \times 10^{-6} (\text{m}^3 \text{ cycle}^{-1})$. A 100 m approach length is included in the road profile prior to the bridge.

Fig. 2: ISO Class 'A' Profile

The equations of motion of the vehicle are obtained by imposing equilibrium of all forces and moments acting on the vehicle and expressing them in terms of the degrees of freedom. They are given by,

$$\mathbf{M}_v \ddot{\mathbf{y}}_v + \mathbf{C}_v \dot{\mathbf{y}}_v + \mathbf{K}_v \mathbf{y}_v = \mathbf{f}_v \quad (1)$$

where $\mathbf{M}_v$, $\mathbf{C}_v$ and $\mathbf{K}_v$ are the mass, damping and stiffness matrices of the vehicle respectively. The $(n \times 1)$ vectors, $\mathbf{y}_v$, $\dot{\mathbf{y}}_v$ and $\ddot{\mathbf{y}}_v$ are vehicle displacements, their

velocities and accelerations respectively. The displacement vector of the vehicle is $\mathbf{y}_v = \{y_{s,1}, y_{s,2}, y_{u,1}, y_{u,2}\}^T$. The vector $\mathbf{f}_v$ contains the time varying interaction forces applied by the two quarter-cars to the bridge, $\mathbf{f}_v = \{0 \quad -F_{t,1} \quad 0 \quad -F_{t,2}\}^T$. The term $F_{t,i}$ represents the dynamic interaction force at wheel i :

$$F_{t,i} = K_{t,i}(y_{u,i} - y_{br,i} - r_i); \quad i = 1, 2 \quad (2)$$

It follows from Table 1 that the static axle loads of the vehicle are $P_1 = P_2 = 98,100$ N. The equations of motion of the VBI model are shown below. The four degrees of freedom correspond to body bounce (Eq. (3)) and axle hop for each quarter-car (Eq. (4)).

$$m_{s,i} \ddot{y}_{s,i} + C_{s,i}(\dot{y}_{s,i} - \dot{y}_{u,i}) + K_{s,i}(y_{s,i} - y_{u,i}) = 0; \quad i = 1, 2 \quad (3)$$

$$m_{u,i} \ddot{y}_{u,i} - C_{s,i}(\dot{y}_{s,i} - \dot{y}_{u,i}) - K_{s,i}(y_{s,i} - y_{u,i}) - P_i + F_{t,i} = 0; \quad i = 1, 2 \quad (4)$$

The response of a discretized beam model to a series of moving time-varying forces is given by the system of equations:

$$\mathbf{M}_b \ddot{\mathbf{y}}_b + \mathbf{C}_b \dot{\mathbf{y}}_b + \mathbf{K}_b \mathbf{y}_b = \mathbf{N}_b \mathbf{f}_{\text{int}} \quad (5)$$

where $\mathbf{M}_b$, $\mathbf{C}_b$ and $\mathbf{K}_b$ are the $(n \times n)$ global mass, damping and stiffness matrices of the beam model respectively and $\mathbf{y}_b$, $\dot{\mathbf{y}}_b$ and $\ddot{\mathbf{y}}_b$ are the $(n \times 1)$ global vectors of nodal bridge displacements and rotations, their velocities and accelerations respectively. The product $\mathbf{N}_b \mathbf{f}_{\text{int}}$ is the $(n \times 1)$ global vector of forces applied to the bridge nodes. The vector $\mathbf{f}_{\text{int}}$ contains the interaction forces between the vehicle and the bridge and is described as follows:

$$\mathbf{f}_{\text{int}} = \mathbf{P} + \mathbf{F}_t \quad (6)$$

where $\mathbf{P}$ is the static axle load vector and $\mathbf{F}_t$ contains the dynamic wheel contact forces of each axle. The matrix $\mathbf{N}_b$ is a $(n \times n_f)$ location matrix that distributes the n_f applied interaction forces on beam elements to equivalent forces acting on nodes. This location matrix can be used to calculate bridge displacement under each wheel, $\mathbf{y}_{br}$:

$$\mathbf{y}_{br} = \mathbf{N}_b^T \mathbf{y}_b \quad (7)$$

The damping ratio of the bridge, ξ , is varied in simulations to assess the system's potential as an indicator of changes in damping. Although complex damping mechanisms may be present in the structure, viscous damping is typically used for bridge structures and deemed to be sufficient to reproduce the bridge response accurately. Therefore, Rayleigh damping is adopted here to model viscous damping:

$$\mathbf{C}_b = \alpha \mathbf{M}_b + \beta \mathbf{K}_b \quad (8)$$

where α and β are constants. The damping ratio is assumed to be the same for the first two modes [22] and α and β are obtained from $\alpha = 2 \xi \omega_1 \omega_2 / (\omega_1 + \omega_2)$ and $\beta = 2 \xi / (\omega_1 + \omega_2)$ where ω_1 and ω_2 are the first two natural frequencies of the bridge [23].

The dynamic interaction between the vehicle and the bridge is implemented in Matlab. The vehicle and the bridge are coupled at the tyre contact points via the interaction force vector, $\mathbf{f}_{\text{int}}$. Combining Eq. (1) and Eq. (5), the coupled equation of motion is formed as,

$$\mathbf{M}_g \ddot{\mathbf{u}} + \mathbf{C}_g \dot{\mathbf{u}} + \mathbf{K}_g \mathbf{u} = \mathbf{F} \quad (9)$$

where $\mathbf{M}_g$ and $\mathbf{C}_g$ are the combined system mass and damping matrices respectively, $\mathbf{K}_g$ is the coupled time-varying system stiffness matrix and $\mathbf{F}$ is the system force vector. The displacement vector of the system is, $\mathbf{u} = \{\mathbf{y}_v, \mathbf{y}_b\}^T$. The equations for the coupled system are solved using the Wilson-Theta integration scheme [24, 25]. The optimal value of the parameter $\theta = 1.420815$ is used for unconditional stability in the integration schemes [26]. The scanning frequency used for all simulations is 1000 Hz.

3. Bridge Response – sensors on the bridge

Accelerometers are simulated placed at the mid span of the bridge. The two quarter-cars (Fig. 1) are simulated crossing a 100 m approach length followed by a 15 m simply supported bridge, both containing the Class ‘A’ road profile [21] of Fig. 2. This is repeated six times, once for each level of

bridge damping (from 0% to 5%). The resulting bridge accelerations for each level of bridge damping ratio can be seen in Fig. 3. The figure includes the last two metres of approach length before the 15 m bridge.

Fig. 3: Bridge Mid Span Accelerations

The bridge accelerations are transformed from the time domain into the frequency domain using the Fast Fourier Transform. The six different PSD curves are plotted on the same graph and can be seen in Fig. 4. Peaks in the acceleration spectra can be seen near the bridge frequency (5.89 Hz). This is close to the actual bridge frequency (5.65 Hz) – the small inaccuracy is due to the resolution of the

spectra which can be improved by increasing the scanning frequency. A pronounced decrease in PSD peak can be seen as bridge damping increases. This demonstrates that changes in bridge damping, indicating that the bridge might be damaged, can be detected.

Fig. 4: Power Spectral Density of Bridge Accelerations

4. Bridge Response – sensors on the vehicle

Accelerometers are also placed on the axles of the two quarter-cars, which are simulated travelling at 20 m s^{-1} over the 15 m bridge. It is found initially that when the acceleration response of one quarter-car is examined, no conclusions can be drawn as the influence of the road profile dominates the spectra, and bridge frequencies cannot be detected. This is because the ratio of height of road irregularities to bridge displacements is too large for the bridge to have a significant influence on the vehicle.

The excitation of the vehicle at any point in time consists of the bridge deflection under the axle and the road profile height. It is proposed here to subtract the axle accelerations of both quarter-cars from one another, allowing for the time shift. This has the effect of removing the road profile heights and leaves the difference in accelerations, as can be seen in Fig. 5.

Fig. 5: Difference in accelerations experienced by each quarter-car

Once again, the acceleration signal is transformed into the frequency domain using the Fast Fourier Transform. The PSD plots for each level of bridge damping can be seen in Fig. 6. As with the earlier simulations with the sensors on the bridge, a peak can be detected at the bridge frequency, and the magnitude of the peak decreases for higher levels of damping. This suggests that a pair of

Fig. 6: PSD of the difference in accelerations experienced by each quarter-car

identical instrumented vehicles have the potential to be a practical method of detecting changes in PSD which may then be used as an indicator of changes in bridge damping. These results indicate that instrumentation of the vehicle can be of similar accuracy to results found by instrumenting the bridge.

5. Discussion and Conclusion

This paper investigates the feasibility of ‘drive by’ bridge inspection. Results from simulating sensors on the bridge indicate that the bridge frequency can be detected when the bridge is excited by two quarter-cars. Separately, when the accelerometers are placed on the axles of the vehicles and the time-lagged difference in the bridge deflections experienced by each quarter-car is found, results indicate that the bridge frequency can be detected in this way also. A frequency domain analysis

illustrates that changes in the PSD of the accelerations as the bridge becomes damaged, can be detected – confirming that bridge damping can be detected. Results also indicate that damage can be detected using sensors on the vehicle at a similar level of accuracy as using sensors on the bridge.

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7. References

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