6th European Conference on Computational Mechanics (ECCM 6)
7th European Conference on Computational Fluid Dynamics (ECFD 7)
11 – 15 June 2018, Glasgow, UK

INDIRECT MONITORING OF RAILWAY BRIDGES BY DIRECT INTEGRATION

JENNIFER KEENAHAN [1], EUGENE J. OBRIEN [2] AND YIFEI REN[3]

[1] UCD School of Civil Engineering, University College Dublin, Newstead, Belfield, Dublin 4, Ireland, jennifer.keenahan@ucd.ie

[2] UCD School of Civil Engineering, University College Dublin, Newstead, Belfield, Dublin 4, Ireland, eugene.obrien@ucd.ie and http://www.ucd.ie/research/people/civilengineering/professoreugeneo'brien/

[3] UCD School of Civil Engineering, University College Dublin, Newstead, Belfield, Dublin 4, Ireland, ren.yifei@ucdconnect.ie

Key words: Railway, Bridges, SHM, Drive-by, Direct Integration, Apparent Profile, BDPD.

Abstract. Railway bridges are of importance as critical elements in transportation networks. Unfortunately, many railway bridges are old and these structures are subject to degradation over time. To monitor bridge structures, many methods are introduced. In recent years, indirect bridge monitoring methods have become more popular. These methods use passing vehicles to measure dynamic responses such as accelerations. In this paper, a new direct integration approach is introduced to directly calculate the apparent railway track profile (AP) that is consistent with the measured accelerations. An adaptation of the Newmark Beta numerical method is used for this purpose. Using AP, bridge displacement profile difference (BDPD) is calculated to monitor bridges. The BDPD is the difference between the baseline (healthy) profile and the apparent profile after damage and environmental effects. BDPD is sensitive to temperature change and bridge damage. It has its own frequency which is close to the bridge frequency.

1 INTRODUCTION

Railway bridges are critical elements of any railway network. With time, bridges may become damaged because of corrosion or bridge strikes. Such damage may have an adverse influence on the safe operation of the transportation network. In the event of serviceability limits being exceeded, or bridge collapse, railway infrastructure owners suffer the cost of both unscheduled maintenance/replacement and lost revenue during any period of closure. Therefore, Structural Health Monitoring (SHM) of bridges is an important area of research. SHM can help to detect damage and aid maintenance planning. At present, visual inspection is probably the most common method of monitoring bridges. However, this method requires a large number of inspectors which may result in significant cost. Furthermore, it is difficult to achieve consistency in monitoring due to the subjectivity of the inspectors. In the future, sensor based monitoring is likely to become more popular. This method can be divided into two types: direct and indirect. Direct methods involve the installation of a number of sensors on the bridge.

These sensors can measure the bridge response directly. This method requires the installation of sensors, data acquisition, storage and communication electronics and a power source on each bridge. Therefore, it tends to be more expensive than traditional monitoring. Also, such sensor systems cannot be reused in other bridges. What is more, a lot of data needs to be processed and the link between measurement data and bridge safety is still being researched.

To address these issues, the idea of an indirect or 'drive-by' method of bridge monitoring is proposed by Yang et al. [1]. In this method, the dynamic properties of bridge structures are extracted from the dynamic response of a passing vehicle. This dynamic vehicle response has the potential to be used to detect damage in the bridge. No sensors need to be deployed on the bridge itself – sensors are only needed on the passing vehicle. Compared to direct methods, the indirect method has its own advantages. It is easy to operate, economic, efficient, and mobile. While the instrumented vehicle may be expensive, one vehicle can monitor a very large number of bridges.

The concept of the apparent profile was developed by OBrien and Keenahan [2]. The apparent profile is the profile that, in the absence of a bridge, would have generated the recorded accelerations. It consists of a combination of bridge deflection and road profile, so it includes information related to bridge behaviour. Hence, information on the bridge condition can be extracted. To date, the apparent profile has been determined by an optimisation procedure. In this paper, a new direct integration approach is introduced to calculate the apparent profile directly from the vehicle accelerations. Using the apparent profile, Bridge Displacement Profile Difference (BDPD) is calculated and discussed. In this paper, BDPD will be used as an indicator of temperature change and bridge damage. The frequency of BDPD is also analysed.

2 DIRECT SOLUTION OF PROFILE CALCULATION

The railway vehicle is simplified here as a spring mass model, as shown in Figure 1. In this case, the mass and stiffness of the sprung mass system are 15 tonnes (i.e., 15,000 kg) and 3.5 × 10^6 N/m respectively.

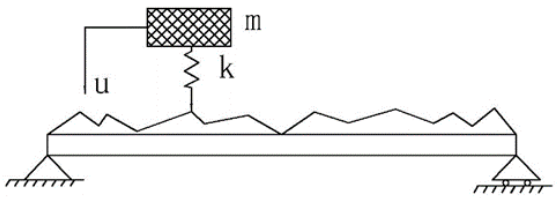

Figure 1: Spring mass model

The behaviour of dynamic systems can be described by ordinary differential equations. Numerical procedures are used to solve the differential equation and then calculate the profile. The general equation of a single degree of freedom dynamical system can be expressed as:

$$m\ddot{u} + c\dot{u} + ku = F \tag{1}$$

where m, c and k are the mass, damping and stiffness for the vehicle, respectively, $\ddot{u}$, $\dot{u}$ and u are the vehicle acceleration, velocity and displacement vectors, respectively, and F is the

applied force vector on the vehicle degree of freedom. F is related to apparent profile $y(t)$ and can be expressed as:

$$F = k \times y(t) \tag{2}$$

In this paper, the aim is to find a more efficient means of solving the inverse problem to determine the apparent profile $y(t)$ consistent with measured accelerations. All problems are solved here using the Newmark Beta integration scheme. The algorithm used to solve the inverse problem is presented in Figure 2. A MATLAB algorithm was used to implement this solution.

For each time step, acceleration of the mass is taken to be known. Then, the displacement and velocity of the mass can be calculated using the Newmark Beta integration scheme. Using the displacement, velocity and acceleration of the mass, the force being applied to the mass can be determined. Finally, the apparent profile is calculated.

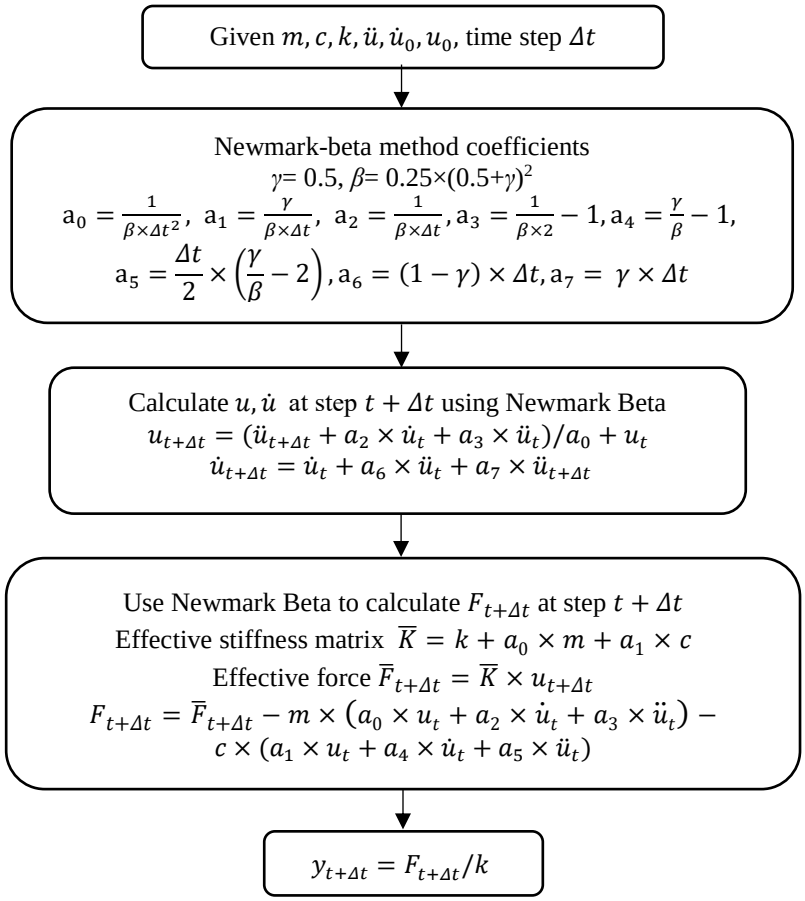

Figure 2: Apparent profile calculation algorithm

To test the concept, a general test profile is used in the forward problem to generate the 'simulated measured' acceleration signal. Using this simulated measured acceleration, the profile is recalculated by the direct integration approach. The recalculated profile matches well with the original profile assumed.

3 BRIDGE DISPLACEMENT PROFILE DIFFERENCE

According to Elhattab et al. [3], bridge displacement profile difference (BDPD), is sensitive to bridge damage. The bridge displacement profile difference is the difference between the baseline (healthy) and the damaged apparent profiles. In this section, the BDPD is used to find the effect of temperature change and bridge damage. It is hoped that the effect of damage can be separated from the natural influences of temperature.

The vehicle is simulated crossing over an approach length and a bridge. The approach before the bridge is 1 m. The bridge is simply supported bridge and its length is 20 m. The vehicle in this case is a spring-mass model. This vehicle crosses over the approach and bridge at 80 km/h (22.22 m/s). The road profile for the approach and the bridge is categorised as 'very good' (Class A) according to ISO [4]. This bridge is modelled using 20 Euler Bernoulli beam Finite Elements. The vehicle/bridge interaction model is simulated in MATLAB.

3.1 Effect of Temperature Change

Temperature change is simulated in the bridge to determine if it can be distinguished from damage in BDPD results. Temperature change affects the modulus of elasticity of concrete. Žnidarič et al. [5], citing Hill and Shimmin, 1961; Lee et al. 1988; Kassir et al. 1996; Downie, 2005; Bastami et al., 2010, have stated that the elastic modulus variation with temperature is linear:

$$E_k = E_0\,(1 + \beta \times \Delta T) \tag{3}$$

where,

E_k: Modulus of elasticity at temperature, T_k

E_0: Reference modulus at temperature, T_0

$\Delta T = T_k - T_0$

β: Thermal hardening coefficient = increase in E per unit change in temperature (°C^{-1})

According to Downie, $\beta = -0.0027$ and this value is used in this paper when the temperature is above 0°C [6]. The resulting relationships between E_k and E_0 under different ΔT conditions are shown in Table 1.

Table 1: Relationships between E_k and E_0 under different ΔT

ΔT(°C)	5	10	15	20
E_k	0.986 E_0	0.973 E_0	0.959 E_0	0.946 E_0

Temperature change is assumed here to vary from 5°C through 20°C in increments of 5°C. BDPD equals the difference between the AP after temperature change and the baseline one (no temperature change). Temperature change affects the bridge stiffness and hence its first natural frequency. This in turn influences the vehicle/bridge dynamic interaction as the vehicle crosses and the resulting acceleration in the sprung mass. BDPD is calculated from this acceleration and is influenced by the change in bridge temperature. Figure 3 shows the BDPDs due to temperature changes in the studied bridge. It shows that the BDPD is a little sensitive to temperature change in the structure. Also, the greater the temperature change, the bigger the BDPD.

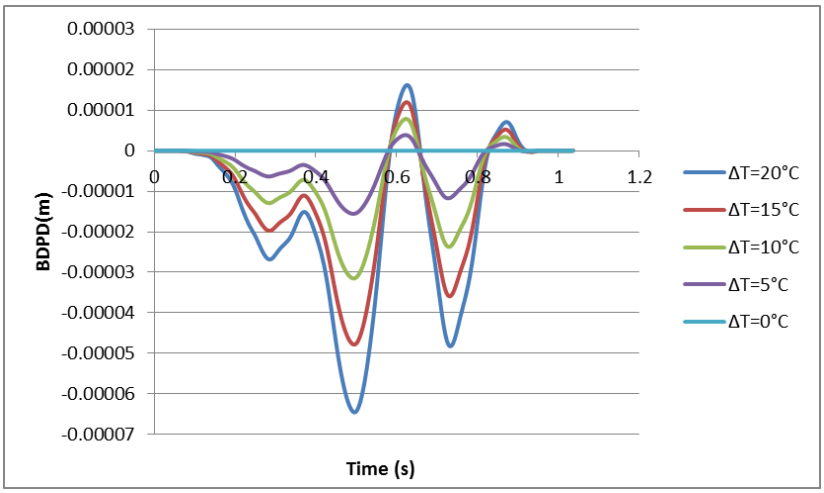

Figure 3: The BDPDs for the studied bridges

3.2 BDPD Frequency

In this section, the frequency of BDPD is investigated. For example, when ΔT=20°C, the frequency of BDPD is f_{BDPD}=3.89 Hz. This is does not correspond to the frequency of bridge (f_{bridge}=4.26Hz) or the frequency of the spring-mass model (f_{sm}=2.43Hz). However, the BDPD and bridge frequencies are close. To investigate the relationship between the frequency of BDPD and bridge, several examples are used. Different types of bridge which have different frequencies are simulated to find the relationship. These bridges have different length, L, mass per unit length, m, and second moment of area, J. In addition, different vehicle velocities are used (80 km/h and 20 km/h). The types of bridge are shown in Table 2. Figure 4 shows the relationship between the frequency of vehicle and the frequency of bridge for the two velocities. The frequency of BDPD is closer to the frequency of the bridge. As the bridge frequency is site-specific, the frequency of BDPD will be site-specific. It is the reason why BDPD is oscillating related to bridge frequency. When the vehicle velocity is low, the frequency of BDPD is much close to the frequency of the bridge.

Table 2: The parameters of different bridges

Type of bridge	A	B	C	D
Length (m)	20	19	15	9
Mass per unit length (kg/m)	37,500	35,625	28,125	16,875
Second moment of area (m^4)	1.2594	1.0717	0.5273	0.1139

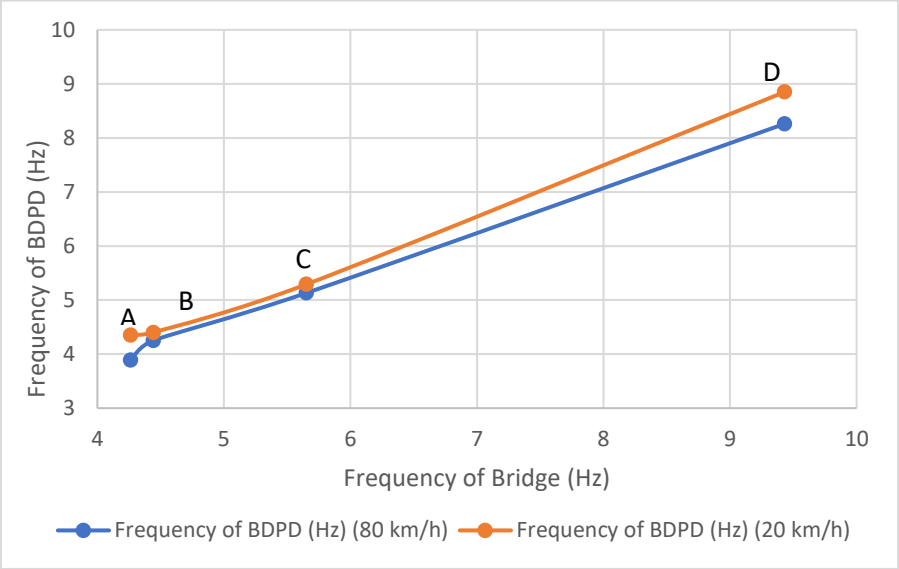

Figure 4: The relationship between the frequency of BDPD and the frequency of the bridge

3.3 Effect of Damage

In this section, the effect of damage is studied. For this study, the source of damage is assumed to be a bridge strike which causes a local change in flexural stiffness. In this example, the damage is simulated at Element 5 (0.225 s) of 20 elements. It means that there is a loss of stiffness in that element. The loss of stiffness varies from 30% through 60%. Figure 5 shows the BDPD for different levels of damage. It shows that the amplitude of the BDPD is sensitive to damage. This result is similar to that reported by Elhattab et al. [3]. They also stated that the difference increases for higher damage levels which can be used to indicate the level of bridge degradation.

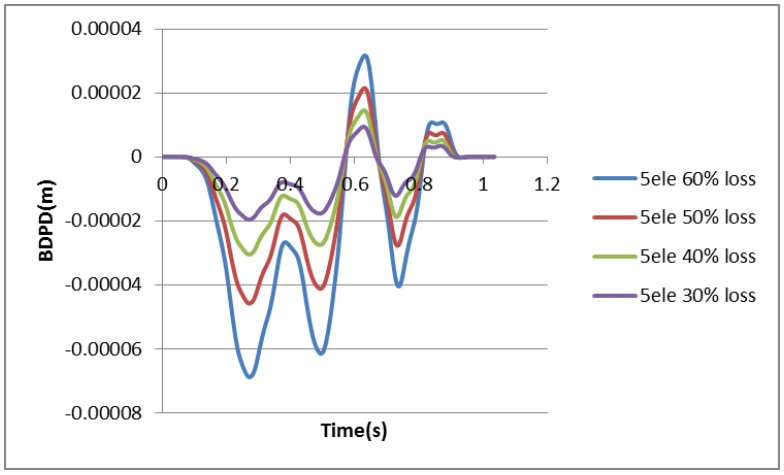

Figure 5: BDPD for different damage levels in Element 5

Figure 6 shows the BDPD for different damage locations. Four different locations are considered for this bridge: Element 5, Element 6, Element 7 and Element 8. The distances

corresponding to the centres of these element in Figure 6 is 5.5 m, 6.5 m, 7.5 m and 8.5 m. In the beginning, they have similar trends. Then the BDPDs change at the damaged element because of its damage.

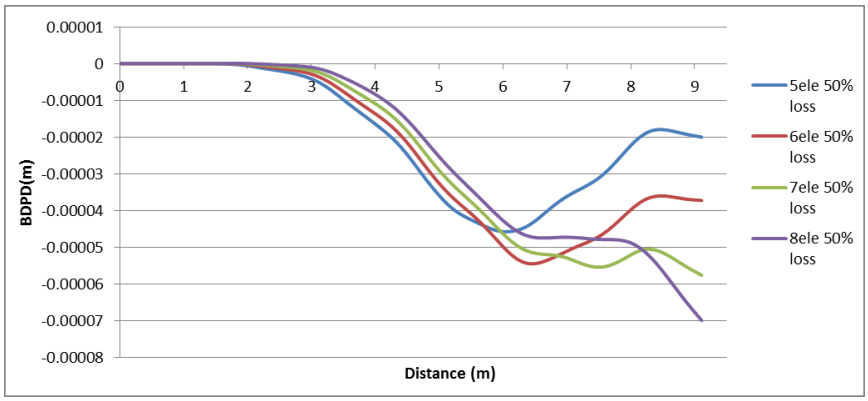

Figure 6: The BDPD for different damage locations (Element 5, 6, 7 and 8 of 20)

Compared to the temperature change effect, the effect of damage on BDPD is quite small. Figure 7 shows the BDPD due to bridge strike damage in one element and temperature change. In this example, ΔT=20°C which means that E_k= 0.946 E_0, i.e. 5.4% uniform loss of stiffness. At the same time, the effect of stiffness loss in Element 5 is shown for a 5.4% loss of stiffness which is the same as the temperature change effect. Clearly the number of elements subject to the change in stiffness has a significant influence.

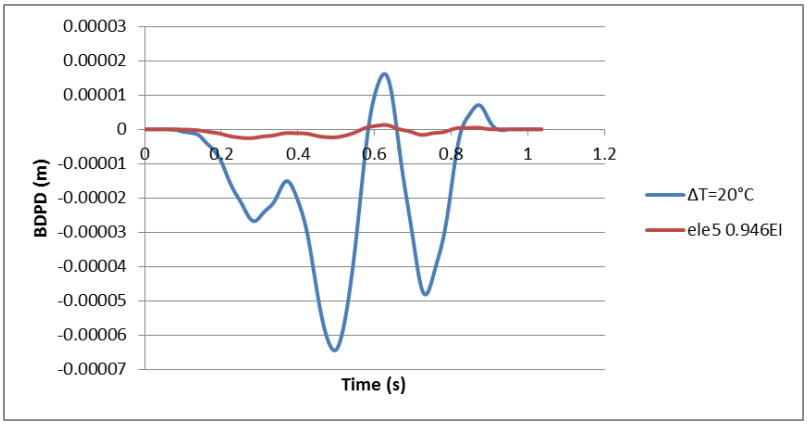

Figure 7: The BDPD because of damage effect and temperature change effect

4 CONCLUSIONS

This paper introduces a new direct integration approach to calculate the AP using accelerations. The acceleration histories are simulated using a vehicle/bridge dynamic interaction model. This approach is based on the Newmark Beta integration scheme. This AP calculation algorithm is highly efficient and the profile found is shown to match well with the profile originally used to generate the acceleration signal. It represents a most significant

advance as it makes it possible to find the AP in a tiny fraction of the computation time required previously.

Using AP, BDPD is calculated to monitor bridges. BDPD is the difference between the baseline (healthy) profile and apparent profiles after damage and temperature change. It is shown that BDPD is sensitive to damage and temperature change. Also, it is more sensitive to the temperature change for the levels of temperature and damage considered. BDPD tends to change suddenly at the damaged element. It has its own frequency which is close to that of the bridge.

ACKNOWLEDGEMENTS

Mr. Yifei Ren acknowledges the Ph.D. scholarship received jointly from University College Dublin and China Scholarship Council.

REFERENCES

[1] Yang, Y.B. and Lin, C.W. Vehicle-bridge interaction dynamics and potential applications. *Journal of Sound and Vibration* (2005) **284:** 205-226.
[2] OBrien, E.J. and Keenahan, J. Drive-by damage detection in bridges using the apparent profile. *Struct Control Health Monit* (2015) **22(5):** 813–825.
[3] Elhattab, A., Uddin, N. and OBrien, E.J. Drive-by bridge damage monitoring using bridge displacement profile difference. *J. Civil Struct. Health Monit* (2016) **6(5):**839–850.
[4] ISO 8608:, 1995. *Mechanical vibration-road surface profiles - reporting of measured data*
[5] Žnidarič, A., Kalin, J., Kreslin, M., Favai, P. and Kolakowski, P. Railway Bridge Weigh-in-Motion system. *6th Transport Research Arena April 18-21* (2016) **2016:**10.
[6] Downie, B. Effect of Moisture and Temperature on the Mechanical Properties of Concrete. *Master thesis* (2005) Morgantown, West Virginia.