The efficient service of bridges is essential if our transport networks are to contribute to economic growth and provide important social benefits. It is imperative that the bridges remain in an acceptable condition and are maintained throughout their life to prevent damage that may lead to major traffic disruptions and economic losses. Damage to bridge structures accumulates as a result of daily traffic loading. Therefore, continuous structural health monitoring of bridges is crucial and should be performed periodically. In this paper, an innovative methodology for monitoring the dynamic behaviour of bridges has been developed by using interferometric radar sensors, Weight-In-Motion (WIM) technology and Finite Element modelling. Using the Merlynston Creek Bridge in Melbourne, Australia. The findings demonstrated that the proposed methodology can both efficiently

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and accurately capture the real-time dynamic behaviour of the bridge under traffic loading as well as the dynamic characteristics of the bridge, such as natural frequencies which are sensitive indicators of structural integrity. The outcomes from this research could potentially enhance the durability of bridges which is an important component of the sustainability of transport infrastructure.

Keywords: dynamic performance; bridges; non-contact sensors; deflection; natural frequency.

1. Introduction

The performance of ageing road infrastructure with the increased demand of heavy vehicle traffic is a significant issue and a major interest for governments, road authorities and asset managers. In Australia, the overall freight task expressed in tonne-kilometres has had an annual growth rate of over 5% in the last 35 years and the average loads carried by articulated trucks have more than doubled in the same period. Moreover, articulated trucks contribute about 80% of the total freight task [Mitchell, 2010]. It is estimated that the number of articulated trucks required to carry the freight will increase by an average of 5% over the next couple of decades [Thompson et al., 2015]. Vibration induced by heavy traffic moving at high speed is a major issue for the service life of bridge structures. This dynamic effect can be more alarming if the bridge is old and subjected to an increase in both the magnitude and frequency of loading [Lilley and Winslade, 2014]. Such dynamic phenomenon induced by traffic loading can also result in accelerated creep and crack propagation in concrete [Koh et al., 1997, Zhang et al., 2013]. Since the dynamic effects as a result of moving traffic on bridges can be very large, leading to a reduced service life of bridge structures [Karoumi et al., 2005], it is essential to better understand the effects of traffic loads on the performance of the bridges, which could result in considerable economic benefits and extended service life of bridges.

The measurement of dynamic parameters under operational conditions is becoming more popular and hundreds of bridges and civil engineering structures have been tested worldwide [Gentile and Bernardini, 2010]. Such dynamic tests are generally performed using piezoelectric sensors or accelerometers which require hardwiring from transducer to a data acquisition system. These conventional methods of vibration measurement are particularly time consuming, laborious, requiring direct access to the structure, but are unable to provide a direct measurement of structural displacements [Pieraccini et al., 2008]. On the other hand, with the development of various non-contact sensors, laser-based systems have been introduced to measure the dynamic properties (natural frequencies, mode shapes and damping ratios) of the bridge structure and their applications have been reported in the literature [Cunha et al., 2001, Kaito et al., 2005]. However, current laser-based technology cannot detect the overall deformation of a bridge and the devices are often found to be impractical for a range of field conditions because they are very sensitive to dust and changes of environmental conditions [Pieraccini, Fratini et al., 2008]. Recently, interferometric radar sensors (e.g. IBIS-S) have been successfully implemented to measure the static or dynamic deflections of several points of a large structure with
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sub-millimetric accuracy [Gentile, 2010]. Furthermore, this newly developed IBIS-S technology overcomes the limitation of microwave beam sensors which are not capable of detecting range resolution (i.e. different targets in the scenario illuminated by the microwave beam) [Pieraccini et al., 2007, Gentile and Bernardini, 2008, Pieraccini, Fratini et al., 2008, Gentile and Bernardini, 2010]. The use of IBIS-S has several advantages in comparison to traditional vibration measurement techniques, such as dynamic monitoring from a distance up to 1km with an accuracy of 0.01mm, fast installation and operation, real-time simultaneous mapping of deformations, structural vibration sampling and analysis up to 200 Hz for model testing and operating day-night in all weather conditions [Gentile and Bernardini, 2008]. Most importantly, the dynamic characteristics of the structures can be monitored without disruption to traffic moving on bridges. A recent experimental study of dynamic properties of a concrete bridge undertaken by authors demonstrated the significant advantage of using the IBIS-S technology [Zhang et al., 2015].

Current studies that aim to identify damage on concrete bridges by monitoring their vibration responses are mainly dependent on conventional techniques and have difficulties in monitoring the overall bridge structures in an efficient and effective manner [Pandey et al., 1991, Cornwell et al., 1999]. Further, very few studies have studied the dynamic behaviour of bridges by integrating IBIS-S and numerical modelling. [Bayissa and Haritos, 2007] and [Bayissa et al., 2008] have proposed spectral strain energy as a vibration response parameter for damage identification of bridge structures. Moreover, the performance and robustness of this method was verified using experimental modal data obtained from a full-scale bridge structure. On the other hand, research carried out using recently developed radar sensors (IBIS-S) [Pieraccini, Parrini et al., 2007, Pieraccini, Fratini et al., 2008, Shimo et al., 2015] generally only provide experimental results and/or comparison of measurements using various types of instruments. There is very limited literature [Alani et al., 2014] in which complementary numerical analysis has been used to verify and interpret experimental data. Similarly, it is not always possible to employ such instrumentation in order to measure the dynamic characteristics of a bridge, especially in operational condition when the traffic movement is a significant issue. Hence, it is essential for the development of numerical models to be performed in conjunction with NDT techniques for continuous monitoring of large transport infrastructure (e.g. concrete bridges).

The purpose of this paper is to develop a non-contact dynamic testing technique and Finite Element model for monitoring the dynamic behaviour of concrete bridges using a concrete bridge in Melbourne, Australia as a case study.
2. Method

In collaboration with VicRoads, the State Road Authority in Victoria, Australia, we monitored the dynamic behaviour of the Merlynston Creek Bridge (M80 Ring Road, Melbourne) using non-contact sensors (IBIS-S). A detail description of the bridge is given in Section 2.1, while radar technology and working methodology of IBIS-S are described in Section 2.2. First, the dynamic behaviour of the bridge was captured through field testing using IBIS-S. The measured results were then compared with the simulation results from the FE model described in Section 2.3. Finally, the major findings are presented and discussed in Section 3.

2.1. Introduction to the Merlynston Creek Bridge

The bridge structure under investigation in this study is a three-span prestressed concrete bridge constructed in 1996, over the Merlynston creek on the Metropolitan Ring Road (M80) in Melbourne, Australia (‘the bridge’). This bridge was recently upgraded in 2012 to accommodate the increased amount of traffic in this section of freeway, and to reduce the traffic congestion during peak periods. After this extension, the bridge consists of four lanes in each direction and serves traffic with a maximum speed of 100 km/h (Figure 1). The bridge deck is supported by numbers of longitudinal span girders which are then supported on abutments at two extreme ends of the bridge and two rows of intermediate piers (four piers in each row) as shown in Figure 1. It can be seen that there are eight girders on top of two piers each (constructed in 1996) while only four girders are on both of the other two piers (recently constructed in 2012). It should be noted here that the size of the piers and girders are different due to the technology and the traffic demand at the time of construction.
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2.2. Dynamic monitoring of Merlynston Creek Bridge using interferometric radar technique

The IBIS-S consists of three components: (a) Sensor module; (b) Control laptop; and (c) Power supply. The sensor model is supported by a rotating head tripod with two horn antennas which automatically generate, transmit and receive electromagnetic waves (Figure 2). The sensor module, which has a USB interface, is connected to the control laptop for data acquisition, system management and overseeing the initial results in real time. The power supply is provided by 12 V battery units [Gentile et al., 2008], [Gentile and Bernardini, 2010].
Fig. 2. View of interferometric radar sensor

The IBIS-S system provides non-contact measurements of both the position and the displacement of target located at different distances from the sensor [Gentile and Bernardini, 2010] as it combines two well-known radar techniques, i.e. the stepped-frequency continuous wave technique (SF-CW) and the interferometry technique. It should be noted that the radar has only 1D imaging capabilities, i.e. different targets are only distinguished based on their distance from the sensor. Hence, measurement errors may arise from the multiplicity of contributions to the same range bin, coming from different points located at the same distance from the sensors [Gentile and Bernardini, 2010].

2.2.1. SF-CW technique

The SF-CW technique is used to detect the position of different targets located along the radar’s line of sight. It uses short time duration (τ) pulses to achieve high range resolution [Pieraccini, Parrini et al., 2007]. The minimum distance between two points on the structure which can be differentiated by SF-CW radar is given by following expression [Taylor, 2001]:

\[ \Delta R = \frac{c \tau}{2} \]  

(1)

where \( c \) is speed of light in free space and \( \tau = 1/B \). The frequency bandwidth (\( B \)) of the electromagnetic waves emitting by radar can be expressed as:

\[ B = (N - 1) \Delta f. \]  

(2)
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$N$ is a burst of electromagnetic pulses and $\Delta f$ is the constant frequency increment. It should be noted that large civil engineering structures (e.g. bridges, buildings and towers) normally have a typical vibration frequency not more than 10 Hz. Thus, an acquisition rate of 30 Hz is able to meet the requirements of frequency sampling for such dynamic testing, and the long-term phase stability is necessary in case of the static measurement. In addition, the range of the targets has to be restricted to the maximum measured distance ($R_{\text{max}}$), as given below, in order to obtain an unambiguous range measurement.

$$R_{\text{max}} = \frac{c}{2\Delta f}$$  \hspace{1cm} (3)

2.2.2. Interferometry technique

The interferometry technique measures the displacement of an object by comparing the phase differences of electromagnetic waves reflected by the objects at different time intervals. It should be noted that the interferometric technique offers the radial displacement ($d_r$) of the scatter objects of the structure illuminated by the antenna beam (Eq. 4). Hence, it is important to have knowledge of the direction of motion to evaluate the actual displacement of the target points. For a bridge structure, the displacement of various elements that are of particular interest under traffic loading are the vertical displacement and this can be easily obtained by making simple geometric projections (Figure 3).

$$d_r = -\frac{\lambda}{4\pi} \Delta \Theta.$$  \hspace{1cm} (4)

where $\lambda$ is the wavelength of the electromagnetic signal, and $\Delta \Theta$ is the phase shift.

\[\text{Diagram:}
\]
2.2.3. Experimental set up

The objective of this experiment is to capture the dynamic characteristics (displacement time history, frequency and mode shapes) of ‘the bridge’ under traffic loading. The IBIS-S radar was placed underneath the bridge in such a way that the sensor could illuminate the bridge girders and capture the displacement of the girders caused by traffic loading, in two stages of testing (Fig. 4a). In the first stage of testing (Test 1), to capture the dynamic properties of girders supporting the east-bound traffic loading (girders no. 8 and 9) as shown in Figure 4 (b), the sensor was configured to cover a horizontal distance of 15 m and a height of 5 m. The second stage of testing (Test 2) was dedicated for girders no. 12, 13 and 14 which collectively support the west-bound traffic loading from the deck (Figure 4c), so that configuration of the sensor was changed to illuminate the area covered by a horizontal distance of 40 m and a height of 5 m. In both of the test configurations, the position, range resolution (i.e. 0.75m) and sample frequency of the IBIS-S were kept the same. The sampling frequency, which depends on the intensity of the reflected signal in the radar field of view, was kept below 200 Hz [Gentile and Bernardini, 2010]. These configurations allow the capturing of dynamic deflections at the mid span of both types of girders between two piers, which support traffic of both directions.
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2.2.4. Results from radar interferometer (IBIS-S) test

The range profile scenario presents the relative distance from the sensor with several peaks Signal Noise Ratio (SNR) that can be used as displacement measurement points. Figure 5 shows the ground range profile of the two tests with several peaks (i.e. TP8-TP14) representing the points of discontinuity of a structure (e.g. corner zones of the bridge girders) with good electromagnetic reflectivity (i.e. SNR over 45dB). These points (TP8-TP14) are considered as locations where the displacements of the girders were captured with excellent accuracy. The IBIS-S sensor was set up at the middle of the two piers so that the shortest distance from the sensor to the bridge girders is at the mid-span of the girders. The distances from the sensor to the girders 8 & 9 in Test 1 and 13 & 14 in Test 2 were measured using a measuring tape and a laser (as reported in Table 1) so that the locations of girders can be compared and verified to the ground range obtained from IBIS-S measurement.

The sensor measurements taken during Tests 1 and 2 have been processed using the data processing IBISDV software and the dynamic parameters of the bridge girders (displacement time history at various target points, natural frequency and mode shapes) have been obtained. The typical displacement time history of TP8 and TP9 is shown in Figure 6 (Test 1), while Figure 7 shows the time-dependent displacement of TP13 and TP14 (Test 2). It should be noted that the maximum displacement time histories due to
heavy vehicles traffic moving in the bridge at the time of IBIS-S testing were highlighted and later validated using FE simulation. The displacement profiles are consistent to that of previous theoretical [Fryba, 1999] and experimental studies [Pieraccini, Parrini et al., 2007],[Gentile and Bernardini, 2010].

![Graph 1](image1.png)

**Fig. 5.** Thermal signal-to-noise ratio (SNR) as a function of range profile of the bridge

**Table 1.** Locations of target girders from IBIS-S sensor

<table>
<thead>
<tr>
<th>Girder number in Figure 4</th>
<th>Type of girder</th>
<th>Distance from sensor (m)</th>
<th>Supporting pier</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>Normal T</td>
<td>9.6</td>
<td>1</td>
</tr>
<tr>
<td>9</td>
<td>Super T</td>
<td>10.8</td>
<td>2</td>
</tr>
<tr>
<td>10</td>
<td>Super T</td>
<td>12.7</td>
<td>2</td>
</tr>
<tr>
<td>11</td>
<td>Super T</td>
<td>14.6</td>
<td>2</td>
</tr>
<tr>
<td>12</td>
<td>Super T</td>
<td>16.3</td>
<td>2</td>
</tr>
<tr>
<td>13</td>
<td>Normal T</td>
<td>17.95</td>
<td>3</td>
</tr>
<tr>
<td>14</td>
<td>Normal T</td>
<td>19.3</td>
<td>3</td>
</tr>
</tbody>
</table>
The maximum peak displacement points are most likely to be generated by heavy trucks while lower peaks are from light vehicles. As shown in Figure 6, the displacement patterns of TP8 and TP9 (i.e. girders 8 and 9 in Figure 4) have similar trend. Furthermore, the maximum displacements of all these girders occurred simultaneously (i.e. $t = 249.86$ sec.), indicating the same heavy truck passing over at that time. Similar trends of displacement history can also be seen in Figure 7 for TP13 and TP14 (i.e. girders 13 and 14 as shown in Figure 4). Interestingly, there are also similarities of the displacement trends between these two target points and the maximum displacements occur at same time of $134.8$ sec. This is because both of these girders are of the same
size, type, construction material and are assumed to have similar capacity. Furthermore, there are some noticeable smaller peaks in the displacement time histories in Figures 6 and 7, which are most likely generated by other heavy vehicles and/ or light vehicles. The observed maximum vertical displacement was generally below 2 mm at the test condition which is well below the allowable serviceability limit state requirements of the bridge (Australian Standard AS 3600-2009 Concrete Structures [2009]).

Fig. 7. Displacement time histories of girders 13 and 14 (Test 2)
Another important parameter that characterizes the dynamic behaviour of a bridge structure is its natural frequencies, which are sensitive to structural integrity. The abnormal loss of structural stiffness is inferred when measurements are substantially lower than expected (e.g. change by about 5%) [Salawu, 1997]. It has been found that the common causes of service failure of structures under dynamic loads is near-resonant vibration which in turn results in dangerously large fatigue stress and even failure of the structure [Lazan, 1954]. As shown in Figure 8, the natural frequencies of the bridge were obtained by processing data of both of the tests using frequency domain decomposition technique in terms of displacement spectrum. It is seen from Figure 8 that the peak displacements occur at same frequency for target points TP8 & TP9 and TP13 & TP14, respectively in Figures (a) and (b). It can be seen from Figure 8 that the fundamental frequency of the bridge is ~ 6.3 Hz and the second mode frequency is ~ 9.3 Hz. This result is consistent with previous studies which suggested that the fundamental natural frequency of the similar concrete bridges is within the range of 5-10 Hz [Menn, 1990, Twayana and Mori, 2014].
2.3. FE analysis of Merlynston Creek Bridge

2.3.1. Characterize traffic loading imposed on the bridge by analyzing WIM data

Weigh-in-motion (WIM) technologies can provide a variety of traffic data such as axle weights, axle spacing, gross vehicle weights, average daily truck traffic and speeds. Large quantities of data can be collected quickly and continuously at low cost for analyzing the loads applied to bridges [Miao and Chan, 2002], [Karoumi, Wiberg et al., 2005], [Miao and Chan, 2002, Karoumi et al., 2005, Thompson, 2014]. In this study, the bridge live loads imposed on the bridge were characterized by analyzing the WIM data provided by VicRoads, and used as input data for the proposed FEM model. The bridge live load models are different from country to country. For example, the Australian and USA models specify a truck for short spans and a uniformly distributed load plus a concentrated load for long spans (Standard Specifications for Highway Bridges, [2002]). The Ontario model proposed by Csagoly and Dorton, based on the equivalent base length, is considered as the most effective method for developing a bridge live load model. According to this method, individual loads of the wheels are treated as a uniformly distributed load of an equivalent base length calculated using the equal moment envelope concept [Csagoly and Dorton, 1973]. That is,

\[ B_m = \frac{A}{w} \sum_{i=1}^{N} |P_i x_i| - \frac{2(N-1)}{bNt} \left( \sum_{i=1}^{N} (P_i x_i) \right)^2. \]  

\( (5) \)
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where, $N =$ Total numbers of discrete loads  
$W =$ Total weight of discrete loads  
$P_i =$ Concentrated loads  
$x_i =$ Lengths between each concentrated load and the centre of gravity of loads

According to a statistical approach proposed by [Miao and Chan, 2002], the highway bridge live load could be derived using the extensive set of WIM data of Hong Kong collected over a period of time. However, this model is unable to capture the time-dependent traffic loading as a result of moving traffic.

To provide inputs for proposed FE model (Section 2.3.2), the traffic load on the bridge was obtained by converting the loading of a truck into a uniformly distributed load over an equivalent base length through analyzing the WIM data collected at the time of the testing. Figure 9 illustrates schematic of positions of different vehicles travelling in various lanes of the bridge in two different points of time when the tests were carried out.

![Diagram of bridge traffic lanes](image)

a) At time $t_1$ of Test 1
2.3.2. The development of FE model

Three-dimensional analytical model was developed using commercial FE software package ANSYS 14.5 [ANSYS, 2015]. The structural components of the bridges were treated as solid elements and their geometry and material properties were based on the “As built” construction drawings provided by VicRoads. As mentioned in Section 2.1, the bridge consists of two types of girders (Figure 10) and the differences in sectional characteristics were considered in the FE analysis. While the piers and abutments were assumed to be fully fixed at ground level, roller support was introduced near the abutment to allow displacement in the horizontal direction. The nominal concrete compressive strength of 40 MPa was considered as per the information provided by VicRoads. The ANSYS model elevation and section of the bridge are shown in Figures 11 and 12. Both Modal analysis and Transient analysis were conducted to obtain the modal frequencies and displacements of the bridge due to the traffic loads at the time of testing as presented in Section 2.3.1. The moving loads due to the vehicles were applied as time dependent loads in each lane.
3. Results and Discussions

The mode shapes and natural frequencies of the bridge were obtained from the full bridge model in ANSYS. Figure 13 shows the first two mode shapes of the bridge obtained from the analysis. The results of analysis show that the fundamental frequency of the bridge is 6.6 Hz, while the second mode frequency is 9.1 Hz. Table 2 presents a comparison of natural frequencies of the bridge obtained from the FE analysis and that from IBIS-S measurements. It can be seen that the results from FE analysis fit reasonable well with that from IBIS-S.

a) First mode
The deflection of bridge girders was analyzed by taking advantage of the symmetrical FE analysis to improve the computational efficiency. A typical snapshot of deflection pattern of the bridge girders from FE analysis is presented in Figure 14. The deflection time history of the selected bridge girders were obtained from the FE analysis and compared with the IBIS-S testing (Section 2.2). It should be noted that the time dependent deflection was measured for a period of 5 minutes in the experiments. For the purpose of comparison with FE results, the maximum deflection from experimental results was studied. Figure 15 shows comparison results for all four girders (i.e. T8, T9, T13 and T14). It shows that the FE predictions are consistent with that from experimental measurements, and thereby demonstrates the accuracy of modelling approach. Furthermore, the results from both the FE analysis and the IBIS-S measurements are consistent with other recent studies [Alani, Aboutalebi et al., 2014]. The validated model has the potential to quantify the dynamic behaviour of bridge structures, and therefore predict the service life of a bridge.
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Fig. 14. Typical view of displacement of girder 8 from FE modelling

a) Deflections at midpoint of girder no. 8 (TP 8)
b) Deflections at midpoint of girder no. 9 (TP 9)

c) Deflections at midpoint of girder no. 13 (TP 13)
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In addition, both the numerical predicted and IBIS-S measured results show that there is little difference in deflections between the two types of bridge girders, i.e. conventional T girders supported by piers 1 & 3, and Super-T girders supported by piers 2 & 4 (Figure 4). The deflection patterns and maximum values of girders 8 and 9 are quite similar despite each being of a different type and configuration, which implies that the dynamic performance of both types of girders is comparable under the real-time traffic loading. Interestingly, the maximum deflections of girders 13 and 14 (i.e. on top of pier 3) are relatively low in comparison to that of girders 8 and 9 due to the relatively low volume of traffic in west bound lanes supported by these two girders. Hence, it is concluded that the integration of new and existing bridge structures has little impact on overall dynamic performance of the bridge. Furthermore, the FE model presented in this study can be implemented to conduct the condition assessment of a bridge by monitoring its dynamic behaviour.

4. Conclusion

The dynamic performance of RC bridges under heavy traffic loading is a concern for all the stakeholders worldwide, especially for ageing bridges which are subject to increasing traffic loading. Use of modern Non-Destructive Testing (NDT) technique (i.e. IBIS-S) in conjunction with FE simulation provides a both efficient and accurate way of assessing the structural performance a bridge via monitoring its dynamic behaviour. By using Merlynston Creek Bridge as a case study, this study’s outcomes demonstrate that the real-time dynamic characteristics of the bridge (e.g. natural frequencies and maximum deflection) predicted by the FE model are consistent with that obtained from IBIS-S.
measurements. Furthermore, it was shown that although the bridge is composed of two components constructed at a different time with different configurations, there is no distinct difference in dynamic performance of these components under traffic loading. The methodology proposed in this study can be further extended to quantify the damage accumulation within a bridge, and thereby predict the service life of a bridge. The advancement in this area will allow rapid assessment of the structural health of bridges by detecting ongoing damage and so enhance the structural performance of bridges.

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