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Measuring displacements of a railroad bridge using DIC and accelerometers

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Abstract. Railroad bridges in North America are an integral but aging part of the railroad network and are typically only monitored using visual inspections. When quantitative information is required for assessment, railroads often monitor bridges using accelerometers. However without a sensor to directly measure displacements, it is difficult to interpret these results as they relate to bridge performance. Digital Image Correlation (DIC) is a non-contact sensor technology capable of directly measuring the displacement of any visible bridge component. In this research, a railroad bridge was monitored under load using DIC and accelerometers. DIC measurements are directly compared to serviceability limits and it is observed that the bridge is compliant. The accelerometer data is also used to calculate displacements which are compared to the DIC measurements to assess the accuracy of the accelerometer measurements. These measurements compared well for zero-mean lateral data, providing measurement redundancy and validation. The lateral displacements from both the accelerometers and DIC at the supports were then used to determine the source of lateral displacements within the support system.

Keywords: steel bridges; railroad bridges; field tests; instrumentation; dynamic loads; imaging techniques

1. Introduction

North American railroads expect to exceed their traffic volume capacities over the next 20 years at many locations and need to prepare their infrastructure accordingly (Cambridge Systematics 2007). Innovations in freight cars and locomotives, have resulted in a doubling of the average tons hauled per freight train (Weatherford *et al*. 2008). According to Unsworth (2010), the weight/car ratio has increased rapidly in the last few decades and the capacities of older bridges are being exceeded. Additionally, of the 100,000 railroad bridges in North America, the US Department of Transportation reports that more than half were built before 1920 (AREMA 2003). As such, many rail bridges are beyond their original design life.

Given that rail bridges are more heavily loaded and aging, railroad companies need to continuously assess the structural condition of their bridges to ensure the safety and operational performance of rail networks. American freight railroads invest billions of dollars to annually inspect more than 60,000 bridges (AAR 2016). Unexpected bridge closures can result in large expenses for railroads as well as affecting the movement of people and goods, and so must be avoided if possible. As such there is a need for monitoring technologies that can provide critical information for engineers to help to ensure that these bridges are still fit for purpose. Measuring bridge displacements under dynamic live loads can be used to evaluate railroad bridges (Moreu and LaFave 2012) and new technologies are continually being developed for this purpose.

The current industry practice for the maintenance and assessment of railroad bridges is to conduct periodic visual inspections (AREMA 2015). Current bridge inspection practices recommend observing and reporting "excessive deflection, settlement" (AREMA 2015). Measuring displacements using displacement transducers is difficult without a fixed reference point. When long-term or in-depth monitoring of a bridge is required, common industry practice is to install accelerometers and inclinometers on a bridge. These sensors can be used to determine the natural frequency of bridge components, and can establish a baseline behaviour for long term monitoring purposes. Ideally, accelerometer measurements can also be used to calculate displacements to evaluate the stiffness of the bridge and its response to train loading, however measurement noise can make this difficult. As such, engineers tasked with managing railroad bridge

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infrastructure still require a practical way to directly measure displacements without the need for a fixed-reference point.

Digital Image Correlation (DIC) is a developing technology that can be used as a non-contact sensor to directly measure displacement. DIC uses digital images to measure a 2-dimenstional displacement field. It overcomes many of the limitations of conventional displacement sensors, such as the need for a fixed reference point, and can be rapidly deployed in the field. By strategically positioning cameras around a bridge, DIC can be used to measure midspan and support displacements in the vertical, lateral, and longitudinal directions, as well as strains (Lee *et al.* 2011, Hoult *et al.* 2013), crack widths (McCormick and Lord 2010; Nonis *et al*. 2013, Hoult *et al*. 2016), settlement (Take *et al*. 2005), or spalling (Nonis *et al*. 2013). By combining DIC with conventional sensors, measurement redundancy and a more comprehensive evaluation of a bridge can be achieved.

The objectives of this research are to: (i) investigate the displacement response of a bridge to dynamic loading using DIC and accelerometer measurements and (ii) use the DIC and accelerometer measurements from this monitoring campaign to assess the bridge performance.

The following sections present a review of common bridge monitoring technologies as well as DIC. The bridge site is introduced and the experimental procedure will be described. The DIC displacement results from the dynamic monitoring of the bridge under service loads are presented, followed by the accelerometer results. The advantages and disadvantages of using both DIC and accelerometers for railroad bridge monitoring are discussed. DIC measurements are compared to those estimated from accelerations and the results from both measurement technologies are used to evaluate the bridge.

2. Background

2.1 Current bridge monitoring approaches

The most common form of structural monitoring of railroad bridges is to conduct visual inspections. Typically, railroad bridges need to be inspected at least once every year, with more in-depth inspections at least every ten years (CN 2016). Visual inspections are considered the industry standard but are subjective and often inaccurate (Graybeal *et al*. 2002, Phares *et al.* 2004). A study by Graybeal *et al.* (2002) revealed that at most, 81% of condition ratings were assigned correctly. Furthermore, Phares *et al.* (2004) tested the accuracy of visual inspections, revealing that at least 48% of individual condition ratings were incorrect. As such these inspections should be supplemented with quantitative data that can be used to more accurately determine the condition of these bridges.

In a survey-based study of structural engineers conducted by Moreu and LaFave (2012), measuring railroad bridge deflections under live service loads was identified as the current top research interest. They indicate that measuring real-time deflections under live loading can be beneficial both in terms of railroad bridge management and railroad bridge replacement prioritization.

There are several monitoring technologies that are capable of measuring dynamic bridge displacements. The most common of these are Linear Variable Differential Transformers (LVDTs). The reason LVDTs are not often used in the field for bridge monitoring is that they require a stationary reference point in the immediate vicinity from which to measure displacements. Moreu *et al.* (2015) were able to overcome this by constructing scaffolding beneath a bridge to the level of the bridge deck but this method is not feasible for many bridges that span across active roads or bodies of water and so a different approach is required.

There are sensors that do not require a fixed reference point in the immediate vicinity and can be used to calculate displacements, including accelerometers and inclinometers, however they do not measure displacement directly. A comprehensive investigation into wireless sensor networks by Hu *et al*. (2013) combined accelerometer measurements with strain and temperature measurements in order to estimate bridge displacements. In this study, the proposed measurement system was capable of estimating displacements and mode shapes but the measurements were compared only to a finite element computer model and not to other displacement sensors. Park *et al*. (2014) also demonstrated that a series of accelerometers can be used in combination with strain gages to estimate displacements with an error of less than 1% when compared to laser displacement sensors. In order to estimate displacement, the accelerations must be filtered and manipulated using a sophisticated algorithm that combines the accelerometer and strain data sets (Park *et al.* 2014). Both of these studies were capable of accurately estimating displacements, however both required access to the bridge to install the sensors.

Moreu *et al.* (2015) conducted field monitoring of several bridges using reference-free estimations from accelerations collected with Wireless Smart Sensors (WSS). In this work, actual displacements measured from a timber bridge trestle pile bent were compared with reference-free displacements under different traffic conditions. With the exception of trains at slow speeds or under harmonic roll, reference-free transverse displacements at a critical bridge location (as identified by the railroad operator) were consistently measured accurately using accelerations collected by WSS. Further improvements in the accuracy of the WSS can be attained by incorporating various different measurements from multiple sensors that can capture pseudo-static responses of bridges under train loading. However, they still need to be installed on the superstructure, and the displacement estimations are still obtained indirectly.

Sousa *et al*. (2013) calculated displacements of a bridge by measuring strain and inclinations and using curve fitting to approximate the curvature, and thereby calculate the displacements. This was done for highway bridges in the field with approximately 5% error. A study by He *et al.* (2014) demonstrated that a series of inclinometers can be used as stand-alone sensors to estimate displacements in a steel arch railroad bridge with a maximum error of 7%.

These studies show that inclinometers can be used to estimate displacements without requiring a fixed datum, but may lack the accuracy required for monitoring the small displacements of short-span railroad bridges.

There are several new technologies that have been developed in order to make non-contact displacement measurements. These include Global Positioning Systems (GPS), laser trackers, and laser scanners. GPS receivers lack the sensitivity of laser trackers and scanners and are therefore generally only used to track the large displacements of long-span suspension bridges (Roberts *et al.* 2004, Yi *et al.* 2013). Laser trackers measure the coordinates of targets with a high accuracy, whereas laser scanners sample 3-dimensional points on surfaces surrounding the scanner (Attanayake *et al*. 2011). Laser trackers can achieve a measurement resolution up to 0.32 micrometers but access to the bridge must be available to place targets, and the system can only track those points on the bridge (Attanayake *et al*. 2011). Laser scanners do not require access to the bridge and can construct 3-dimensional surfaces with an accuracy of 2 mm (Attanayake *et al*. 2011).

Digital image analysis techniques (other than DIC) have been developed and used for bridge monitoring. Lee and Shinozuka (2006) used images taken with a digital video camera with a telescopic lens of targets placed on the structure coupled with target recognition algorithms to measure displacements. They confirmed the accuracy of their method by comparing the measurements from their technique against linear variable differential transformer (LVDT) measurements during a shake table test. They then used their system to measure the displacements of a steel box girder in the field. One disadvantage of this technique is the need for targets on the structure. Fukuda *et al*. (2013) used Orientation Code Matching (OCM) coupled with video images taken at 60 frames per second to measure displacements. Unlike DIC, which matches pixel intensity, OCM matches gradient information between images to enable displacements to be calculated. They also verified their results against LVDT measurements taken during shake table tests. They then used their technique to measure the midspan displacements of a suspension bridge.

The main current bridge monitoring approaches are listed, along with their main advantages and disadvantages in Table 1.

2.2 Digital image correlation

DIC uses digital images of an object to measure two-dimensional displacement fields, and as a result overcomes many of the disadvantages of commonly used sensors, such as the need for a fixed reference point in the immediate vicinity of the bridge and the need for access to the bridge.

The first image in a series of images is referred to as the reference image. Areas of interest, known as subsets, can be identified in the reference image and tracked through the image series. DIC can track the natural surface of an object, provided there is a minimum level of texture, eliminating the need for targets or markings on the bridge. Any object of known dimensions within the image can be used to create a ratio of millimeters to pixels, known as a scale factor. This is used to convert the DIC output from image space measurements, in pixels, to physical space measurements, in millimeters for example. By using a high-speed camera system, dynamic displacements of a bridge can be calculated with the level of accuracy depending on the camera hardware, system setup, and DIC software. Cameras can be strategically positioned to measure lateral, longitudinal, or vertical displacements, at the midspan, the abutments, or any visible portion of a bridge. This research uses DIC to monitor only the superstructure of the bridge, however the substructure could also be monitored if visible and desired.

Stephen *et al*. (1993) first investigated the use of DIC for bridge monitoring. At the time, the technology was only sensitive enough to monitor long span bridges with large displacements; the Humber Bridge was selected for this study. Yoneyama *et al.* (2007) made static measurements using DIC before and after applying load to a short concrete girder bridge. More recently, DIC has been used to dynamically measure bridge displacements. Cigada et al. (2013) investigated the effect of using targets for DIC tracking on dynamic bridge measurements. It was determined that the quality of the natural texture of the bridge strongly impacted measurement accuracy. Murray *et al.* (2015) monitored a reinforced concrete highway bridge during static and dynamic load tests.

Table 1 List of main bridge monitoring technologies along with their advantages and disadvantages

Monitoring Technology	**Advantages**	**Disadvantages**
Visual Inspection	- Industry standard - Global picture of structure	- Highly subjective and variable - Surface only
Displacement Transducers	- High accuracy and repeatability - Common technology	- Requires contact with structure
Accelerometers	- No fixed reference required - Can be wireless	- Measurement drift due to noise when used for displacements
Inclinometers	- No fixed reference required - Can be wireless	- Multiple locations or other data required for displacements
Strain gauges	- No fixed reference required - Can be wireless	- Multiple locations or other data required for displacements
GPS	- No fixed reference required - Can be wireless	- Low accuracy - Bridge access required
Laser Trackers	- High accuracy	- Expensive - Bridge access required
Laser Scanners	- Non-contact	- Low accuracy
Digital Image Techniques	- Non-contact - High accuracy	- Requires line of sight - Requires scale factor

This research proposed using stationary objects in the image to reduce measurement errors for a bridge that experiences small displacements. McCormick *et al.* (2014) and Hoag *et al*. (2015) both compared dynamic DIC measurements to measurements taken by conventional linear potentiometers.

The DIC software package used in the current research is called GeoPIV and was developed by White et al. (2003) and modified by Stanier *et al*. (2016). GeoPIV initially measured displacement with an accuracy of 0.1 pixels (White *et al*. 2003) but with recent improvements in the sub-pixel interpolation scheme the accuracy has been increased to approximately 0.001 pixels (Lee *et al.* 2011). Murray *et al.* (2015) demonstrated the accuracy of GeoPIV for use in high speed displacement measurement by comparing DIC measurements to displacement transducer measurements of a reinforced concrete bridge under dynamic vehicle loading.

2.3 Displacement estimation using accelerations

Fig. 1 illustrates estimating displacements using accelerations from a time window using an algorithm that can be used to estimate reference-free displacements in the transverse direction using acceleration measurements. To eliminate the need for information about double integration and unknown constants of integration, Lee *et al.* (2010) proposed minimizing the difference between the double derivative of the displacement and the acceleration within a finite time interval. The objective function to be minimized is given in Eq. (1).

$$\min_u \Pi = \frac{1}{2}\left\| L - (\Delta t)^2 L_a \bar{a} \right\|_2^2 + \frac{\lambda^2}{2}\|u\|_2^2 \tag{1}$$

Where u, Δt, $\bar{a}$, L_a, L, $|\cdot|_2$, and λ are the estimated displacement, time increment, measured acceleration, integrator operator and diagonal weighting matrix, 2-norm of a vector, and optimal regularization factor, respectively.

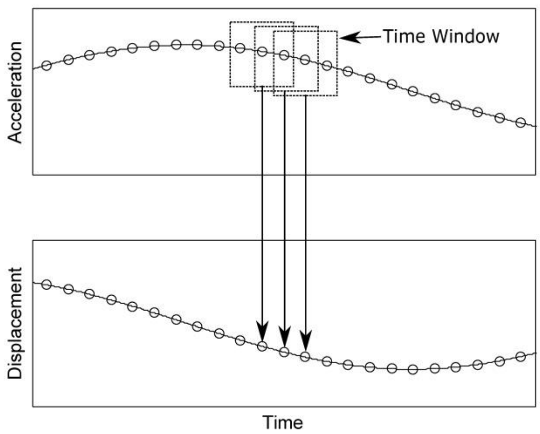

Fig. 1 Schematic of the estimation of displacements from acceleration data, showing the process developed by Lee *et al.* (2010)

The optimal regularization factor λ is presented in Eq. (2), and it depends on the number of data points in the time window (N).

$$\lambda = 46.81 \cdot N^{-1.95} \tag{2}$$

The size of the time window is usually two or three times the longest estimated period of the target structure. Using the measured acceleration and Eq. (1), the estimated displacement (u) can be calculated using Eq. (3).

$$u = \left(L^T + \lambda^2 I\right)^{-1} L^T L_a \bar{a}(\Delta t)^2 = C\bar{a}(\Delta t)^2 \tag{3}$$

I is the identity matrix and C becomes the coefficient matrix for the displacement reconstruction.

Park et al. (2013) programmed this algorithm into Wireless Smart Sensors (WSS) and conducted laboratory tests to demonstrate the potential of this approach. The validity of the proposed method was experimentally demonstrated on a three-story shear building during free vibrations. This algorithm will be employed here for direct estimation of railroad bridge deflections under actual train loadings using accelerometer data.

2.4 Halton 26.36 bridge test site

The bridge known as Halton 26.36 lies along the main CN Rail line between Canada and the US (43^{0}37’18.7”N, -79^{0}55’54.9”W). Halton 26.36 is a steel Deck Plate Girder (DPG) railroad bridge comprised of six spans with one track. Fig. 2 shows a schematic of the bridge, as well as an image of the bridge along with the sign convention used for the midspan displacement measurements. Each span is simply supported, approximately 30 meters long, and supported on tall masonry piers. The original lattice girders of this bridge were replaced with shallower deck plate girder spans in 1910. Since the masonry piers were not replaced at that time, the newer, shallower, spans were elevated above the piers on large steel castings. Fig. 2 shows the labelled spans of Halton 26.36 with a detail of a pier at a larger scale to illustrate the castings which act as the bearings for the bridge. The operator reported that this bridge experienced larger than expected lateral displacements based on visual observations during routine periodic inspections. For this reason, the bridge was monitored using accelerometers as well as cameras for DIC, so that the measurement techniques could be compared and the behaviour of the bridge could be quantified. It is worth noting that one of the advantages of DIC is that the displacement measurement accuracy can be improved by adjusting the scale factor. The more pixels that a given physical movement is divided into, the greater the accuracy of the measurement. The scale factor can be increased by using a camera with a larger sensor (in terms of number of pixels) or by using a lens with a larger focal length. Thus if smaller lateral displacements were expected, the scale factor could be modified accordingly.

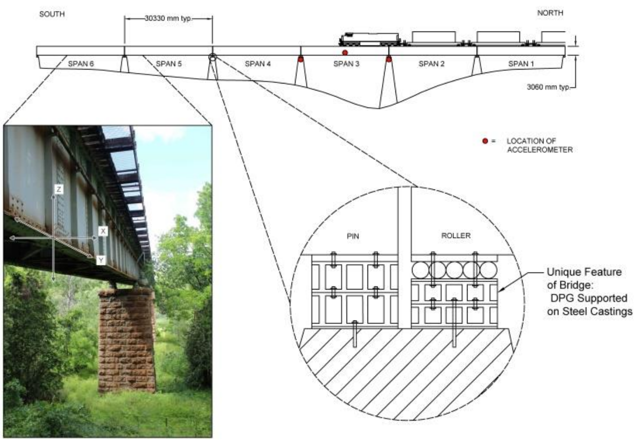

Fig. 2 Schematic of Halton 26.36 showing typical dimensions. The enlarged areas highlight the pin and roller connections of these spans as well as Span 6. The location of the accelerometers are shown

3. Experimental setup

Allied Vision Technologies (AVT) GX1050 8-bit monochrome 1 megapixel (MP) high speed cameras with 85 mm lenses were used for the DIC monitoring of this bridge.

These cameras recorded images at 100 Hz to capture the dynamic effects of the loading. Spans 3, 4, and 6 were each monitored using DIC under multiple freight train loadings. Cameras were positioned with a view of the bridge at midspan and at the girder immediately above the bridge piers to monitor displacements at these locations. At each location along the span, cameras adjacent to the bridge were used to measure vertical displacements and cameras below the span, aiming up at the bottom of the DPG, were used to measure lateral displacements. Fig. 3(a) shows a typical setup for two cameras monitoring vertical displacements at the midspan and pier of Span 3, respectively. Fig. 3(a) shows a typical Field Of View (FOV) for a camera aimed at the side of the DPG to capture the vertical displacement at midspan. Fig. 3(b) shows a typical camera setup for the remaining three cameras setup to monitor the lateral displacements of Span 3 at midspan and both piers.

Tri-axial CX1 accelerometers from SENSR were used to record 3-dimensional accelerations at the midspan and piers of Span 3 at 250 Hz (SENSR 2016). The locations of the accelerometers as well as an image of one are shown in Fig. 3b. The locomotive types and weights are summarized in Table 2.

4. Monitoring results

This section will discuss the DIC and accelerometer measurements, respectively. In some cases, only the results from the freight train locomotives are shown for clarity, although the displacements due to the whole train were measured. Time zero is defined as when the train first enters the span being monitored.

For comparison purposes, unless otherwise stated, all the monitoring results shown are from Span 3 under the dynamic loading of 'Freight 4' which was led by three locomotives: SD70M-2 (210 tons), C44-9W (195 tons), and C40-8M (197 tons). All of the DIC results shown are filtered using a low-pass filter to remove the effects of camera and tripod vibrations. A cutoff frequency of 7 Hz was selected as these equipment vibrations have a frequency of 10 Hz and above, whereas the dominant frequencies for the bridge displacements measured by DIC had a frequency less than 3.5 Hz.

A summary of the vertical and lateral displacements of all three spans under full freight train loading is shown in Table 2, which shows the peak vertical and lateral displacements under the seven freight trains that were monitored. Peak vertical displacements of the bridge spans are caused by locomotives and heavily loaded freight cars, however lateral displacements usually have largely consistent magnitudes throughout the passage of the train.

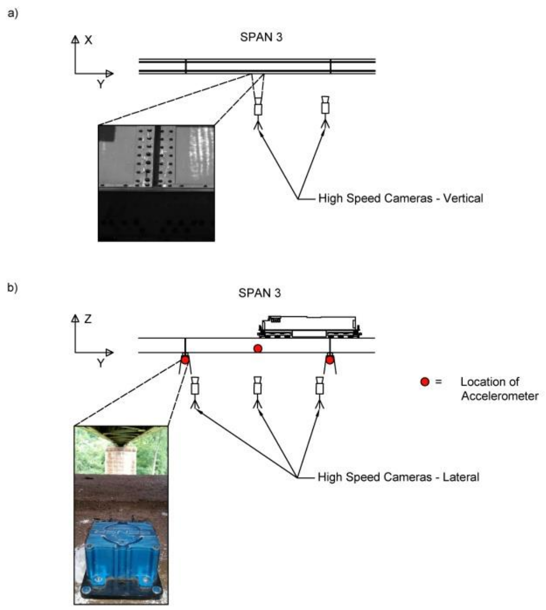

Fig. 3 Schematic of Span 3 showing typical locations of cameras as well as the accelerometer location: (a) Plan view of the span. A sample image from the midspan camera is shown for reference and (b) Side view of Span 3. An image of the accelerometer on the South Pier is shown for reference

Table 2 Summary of the recorded freight train crossings and the maximum vertical displacements and maximum and minimum lateral displacements measured using DIC

Span	Freight #	Locomotive Mass (ton)			Vertical Displacement Maximum (mm)	Lateral Displacement Maximum (mm)	Lateral Displacement Minimum (mm)
Span 3	3	195		193	16.81	2.91	3.51
	4	210	195	197	16.99	2.12	2.95
	5	210	210	210	17.01	2.80	3.90
Span 4	6	197		210	14.92	2.12	2.30
	7	206		197	14.46	4.33	3.40
Span 6	1	210		194	15.23	1.45	1.50
	2	210		197	15.42	2.79	2.79

4.1 DIC Displacement measurements

Fig. 4 shows the midspan vertical displacements of Span 3 under a) the full loading of Freight 4 and b) the locomotive loading. The span experiences a maximum displacement of 17.01 mm as seen in Fig. 4(a). The first large displacement occurs when the first truck of a locomotive is directly at midspan. The subsequent peak displacement occurs when the second truck of the first locomotive and the first truck of the second locomotive are centered near midspan. The small rebound between the large displacements occurs due to the bridge being under less load between the trucks of a locomotive. At approximately 17 seconds, the locomotives have left the span and empty freight cars (with the occasional loaded freight car) are the only masses loading the span. These cars are much lighter than the locomotives and therefore produce much smaller displacements.

Fig. 5 shows the midspan lateral displacements of Span 3 under the same freight train loading as Fig. 4. Fig. 5(a) shows the displacements under the full freight train, with the displacements due to the locomotives only shown in Fig. 5(b). It can be seen in Fig. 5(b) that, prior to the locomotives crossing the span, the bridge begins to oscillate about its initial position as the lateral displacements start to increase. Since the bridge is not skewed or curved, the displacements oscillate about 0 mm. Unlike the vertical displacements seen in Fig. 4(a), the lateral displacements do not decrease once the locomotives have left the span. Thus it appears that the magnitude of lateral displacements is not directly related to the weight of the train car. The peak lateral displacements occur near the middle of the train with a maximum and minimum value of 2.95 mm and -2.12 mm respectively. These displacements are much larger than those under the locomotives and are likely the result of dynamic effects related to particular train cars.

Fig. 6 shows the vertical displacements of Spans 3, 4, and 6, due to the locomotive crossings of separate freight train loadings.

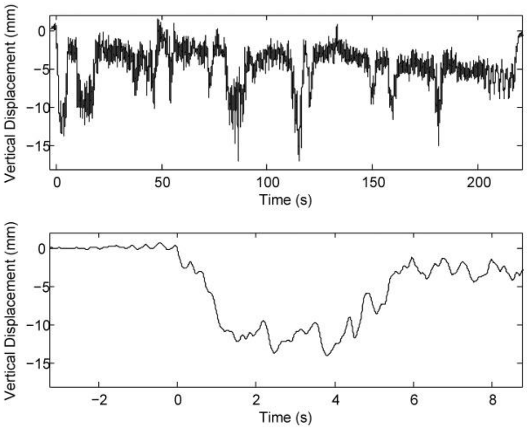

Fig. 4 Vertical Displacement at midspan of Span 3 measured using DIC. (a) the displacements under the full freight train crossing, and (b) an enlarged detail showing only the effects of the locomotives

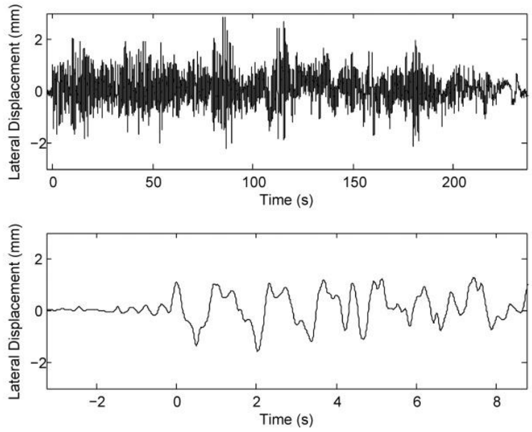

Fig. 5 Lateral Displacement at midspan of Span 3 measured using DIC, showing (a) the full freight train crossing and (b) an enlarged view of the locomotives crossing. Positive values represent displacements to the East

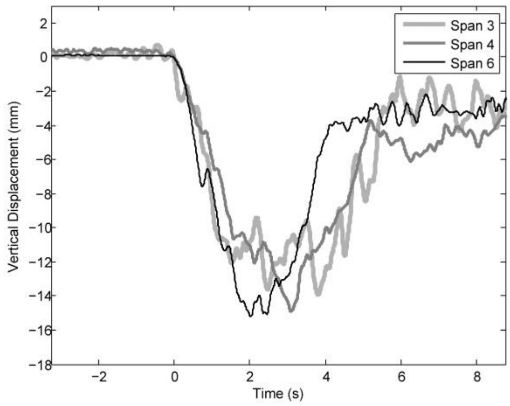

Fig. 6 A comparison of the vertical displacements of three different spans measured using DIC

The maximum displacements caused by the locomotives crossing are 13.92 mm, 14.92 mm, and 15.23 mm for Spans 3, 4, and 6 respectively. It should be noted that the trains that were recorded travelling across spans 4 and 6 each had only two locomotives, which results in fewer displacement peaks in Fig. 6 for these spans. It should also be noted that Span 6 is the last span of the bridge and so is supported by an abutment, rather than a pier on one side. Thus, no displacements are observed prior to the freight train locomotives entering the span (i.e., before time = 0 seconds on the plot).

The comparison of lateral displacements for these spans is shown in Fig. 7. The displacements of these three spans were recorded under different freight trains at different times. It can be seen that the displacements of Span 6 are generally smaller than the displacements of the other two spans. This is potentially again due to the abutment on one side, which is significantly stiffer than the piers and provides more lateral support. For all three spans, the peaks in the displacements due to the locomotives are generally between 1.5 mm and -1.5 mm.

4.2 Accelerations

Fig. 8 shows the accelerometer data, measured in g, from the midspan of Span 3 during the passage of the locomotives of Freight 4. The accelerations in the vertical (z), lateral (x), and longitudinal (y) directions are shown for comparison. It should be noted that these accelerations have been filtered using a low-pass filter with a cutoff frequency of 30 Hz to remove high-frequency noise. It can be seen that the vertical accelerations are an order of magnitude larger than the lateral and longitudinal accelerations, with a minimum of -0.53 g. The lateral and longitudinal accelerations oscillate about zero without tending towards one direction. The lateral acceleration peaks are between 0.05 g and -0.05 g and the longitudinal peaks are between 0.01 g and -0.01 g. Similar to the vertical displacements shown in Fig. 4, the vertical accelerations shown in Fig. 8 are largest under the heavy locomotives and are smaller under the lighter freight cars. The lateral and longitudinal accelerations remain fairly constant throughout the passing of the train, unaffected by freight car mass, which is similar to the lateral displacement measurements.

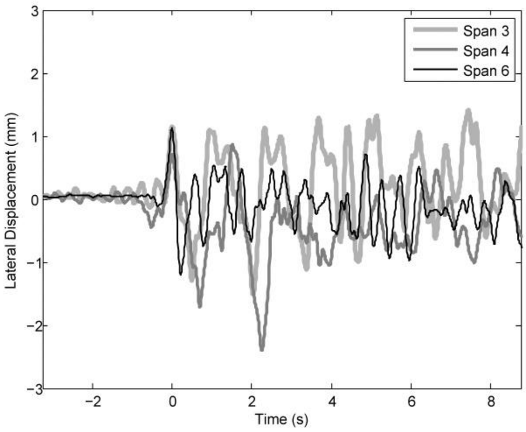

Fig. 7 A comparison of the lateral displacements of three different spans measured using DIC

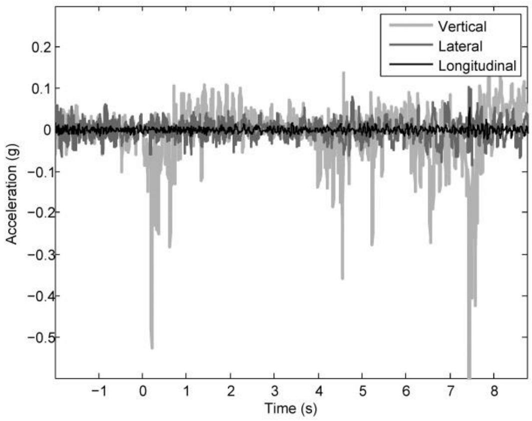

Fig. 8 Raw acceleration data of the midspan of Span 3 recorded using a tri-axial accelerometer

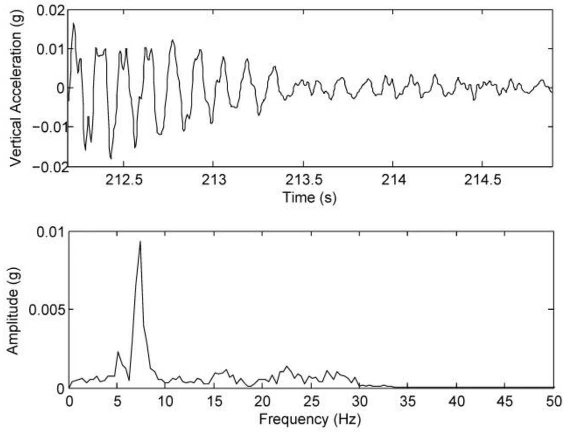

Fig. 9 Free vibration accelerations from accelerometers at the midspan of Span 3, plotted in (a) the time domain and (b) the frequency domain

One of the principal uses of acceleration measurements is to calculate the natural frequencies of the bridge and to establish a baseline for long term monitoring. An example of using accelerometer data to calculate natural frequency is shown in Fig. 9. The vertical accelerations of Span 3 after the freight exited the span are shown in Fig. 9(a). These accelerations represent the free vibrations of the bridge. The natural frequencies of these vibrations are shown in Fig. 9(b), with the accelerations plotted in the frequency domain. The frequency content of the acceleration signal was determined by using a Fast Fourier Transform (FFT) on the acceleration data. It can be seen that the first modal frequency of Span 3 is 7.40 Hz.

This result can be compared to the predicted frequency obtained using Eq. (4) from Unsworth (2010)

$$f = \frac{680}{L} = \frac{680}{99.5\,ft} = 6.83Hz \tag{4}$$

Eq. (4) can be used to estimate the fundamental frequency (f) of an open deck girder span based on the span length (L) in feet (ft). The natural frequency of 6.83 Hz, estimated using Eq. (4) can be compared with the measured natural frequency of 7.40 Hz. It should be noted that Eq. (4) is an empirical equation designed to give a first order approximation of frequency and it does not account for a number of variables such as train speed. As such, given the empirical and general nature of Eq. (4), it is difficult to tell whether the difference between these two values represents a significant issue.

4.3 Comparison of measurement techniques

For all measurements other than the vertical displacements (which have a mean of -3.25 mm), the mean measurement is approximately zero. This is true over the course of the entire train crossing event, however this would not hold true during short time-windows. The vertical displacements have a negative mean, as expected for a span being loaded in the negative direction.

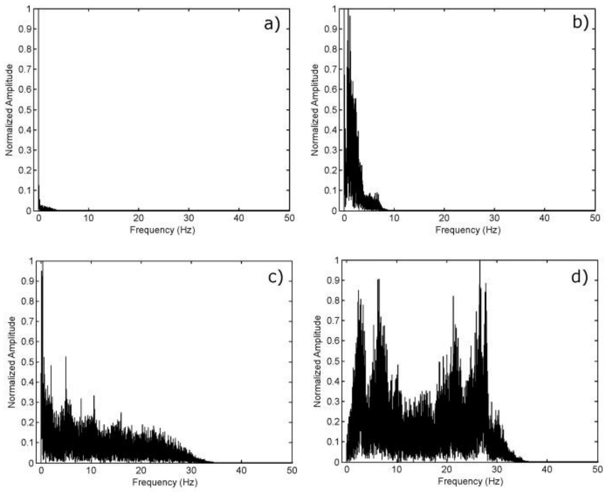

Fig. 10 Comparison of Frequency content between (a) Vertical DIC, (b) Lateral DIC, (c) Vertical Accelerations and (d) Lateral Accelerations. The FFT amplitudes have be normalized by the largest amplitude of each respective frequency analysis

Fig. 10 shows a comparison of the frequency content of the DIC and accelerometer measurements under the full train crossing event. The FFT amplitudes in Fig. 10 have been normalized by the largest amplitude in each respective plot to facilitate a direct comparison between each of the measurements. These frequencies represent the forced frequency of vibration of the bridge in the vertical and lateral directions. The vertical measurements of displacement and acceleration have dominant frequencies less than 1 Hz, whereas the lateral measurements have much higher frequency content. It should be noted that the same frequency filters were applied to vertical and horizontal measurements and thus do not affect this comparison.

This direct comparison of displacements and accelerations provides information, however it is difficult to use this data to evaluate the performance of the bridge. To further compare these two measurement systems, displacements can be calculated using the measured accelerations. These calculated displacements can be directly compared to the measured displacements and can be used to further investigate the response of the bridge. The process of calculating displacements from accelerations is described in the following section.

5. Comparison of displacements calculated using DIC and acceleration measurements

The lateral accelerations at midspan of Span 3 were used to estimate the lateral displacements as seen in Fig. 11. Fig. 11(a) shows the comparison for the entire train crossing, with good agreement between both sensor technologies, as highlighted for the locomotive crossing seen in Fig. 11(b).

Fig. 11 shows that accelerometers can reasonably estimate displacements for zero-mean data, such as these lateral accelerations. There are however, occasionally some large unexplained peaks that do not align with the DIC data as seen in Fig. 11(c). As discussed by Moreu *et al.* (2015), this shows the limitation of post-processing accelerations to estimate displacements. In general, the accelerometer estimates of the displacements align very closely with the DIC measurements as shown in Table 3, which compares the peak measurements from both monitoring technologies. Table 3 shows that the error in calculating peak displacements was 1 %, 5 %, and 22% for trains 3, 4, and 5 respectively, when compared to DIC.

The vertical accelerations at midspan were also used to calculate displacements and compared to DIC measurements as seen in Fig. 12. In order to facilitate a direct comparison between the two measurement systems, the DIC data was detrended, forcing the mean to be zero.

This was done in order to simulate a zero-mean data set, since the method of calculating displacements from accelerations presented earlier outputs zero-mean, peak-to-peak displacements. It can be seen in Fig. 12 that the calculated displacements often significantly overestimate the actual displacements of the span measured by DIC, and do not provide an accurate representation of vertical displacements.

These large errors are likely due mainly to a combination of four factors. First, the method described earlier for calculating displacements from accelerations functions with zero-mean data. The vertical acceleration measurements are zero-mean over a long window of time, i.e., over the course of a train crossing event, however in many short windows of time (such as when the locomotives first enter the span) the accelerations will not be zero-mean and this can lead to large calculation errors. Second, the microelectromechanical systems (MEMS) accelerometers used in this study are most accurate when monitoring high frequency vibrations. The vertical DIC and acceleration measurements have much lower dominant frequencies than in the lateral direction. This is illustrated in Fig. 8 which shows the frequency content of the a) vertical DIC measurements, b) lateral DIC measurements, c) vertical accelerations, and d) lateral accelerations. As seen in Fig. 8, the forced frequency of vibration was much lower in the vertical direction than in the lateral direction. These low-frequency vertical vibrations may be poorly measured by the accelerometers and thus lead to inaccuracies when calculating displacements. Third, there is intrinsic dynamic behaviour caused by impacts from the engines, axles, and cars on the bridge that alter the readings from the accelerometers in the vertical direction. Because of this, any attempt to estimate the vertical response should also include the moving mass component of the trains, which are larger than the mass of the bridge, and other researchers are using FE models to describe and monitor this behaviour (Kim *et al.* 2016). Finally, errors can also be introduced if the accelerometers are not properly attached to the bridge and there is differential movement between the two although the accurate estimation of the lateral displacements suggests that this is not the case, it is a potential source of error.

This technique of calculating displacements under a full train from accelerations may also lead to inaccuracies that do not reflect the entire passage of the train. This reinforces interest in using real time displacement measurements for bridge inspection, because some railroads are interested in the total dynamic movement under a train so that an engineer or competent person can quantify the movement being observed.

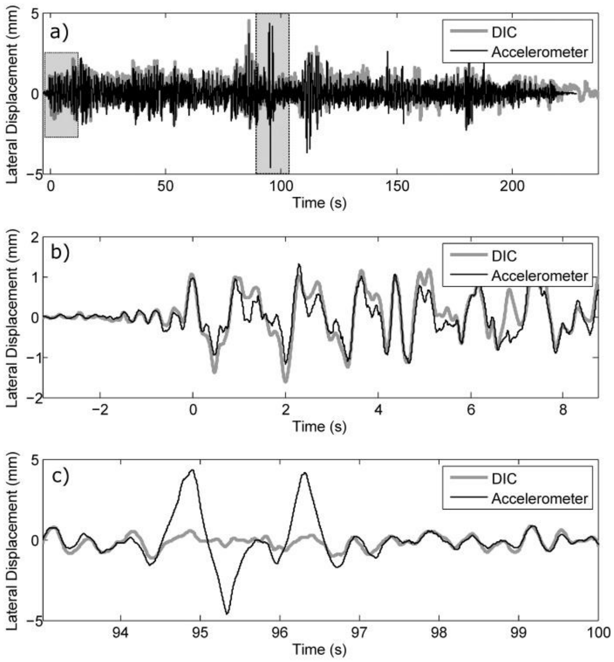

Fig. 11 Comparison of the midspan lateral displacements of Span 3 measured by DIC and calculated from accelerometer data. (a) Shows the comparison for the entire freight train crossing, (b) shows an enlarged view of the locomotive displacement comparisons and (c) shows large anomalous errors in the displacements calculated from accelerometer data

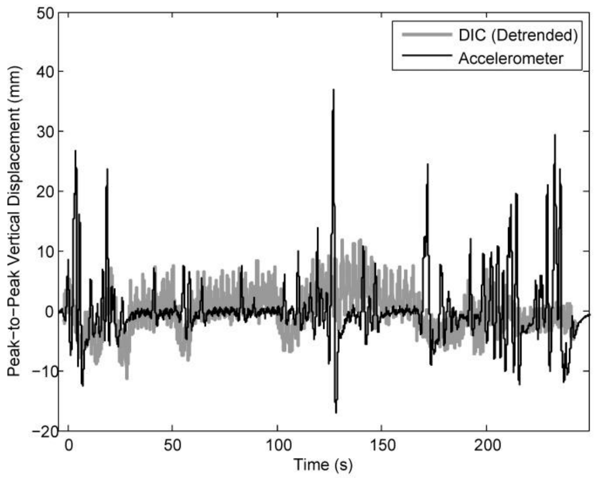

Fig. 12 Comparison of the midspan vertical displacements of Span 3 measured by DIC and calculated from accelerometer data

Table 3 Comparison of lateral displacements from DIC and calculated from accelerometers at midspan of Span 3 under three freight trains

Train	DIC Peak Lateral Displacement (mm) A	Peak Lateral Displacement Calculated from Accelerations (mm) B	Displacement Difference (mm) A-B	Error of Estimation (%) |A-B|/A×100
Freight 3	3.52	3.54	-0.02	0.57
Freight 4	4.38	4.6	-0.22	5.02
Freight 5	4.47	3.48	0.99	22.1

6. Bridge assessment using sensor data

The displacements found in Fig. 5 show that each of the spans monitored experienced comparable vertical displacements. This is expected for spans of the same age, made with the same materials, and designed with the same span and cross section. A difference in vertical displacements between the spans could be the result of a difference in stiffness between the spans and would most likely indicate faster decay of one particular span. This does not appear to be the case, with all spans demonstrating a similar stiffness. A lack of historical displacement data, as well as other unknown factors related to deterioration, including weather and fatigue, makes it impossible to determine if all the spans of the bridge are not deteriorating or if they are all deteriorating at the same rate. Fig. 5 does show however that Span 3 exhibits more dynamic vibrations than the other two spans. This is evident in the vibration response of the bridge prior to the train entering the span (e.g., before time = 0). Once the locomotives enter the span, Span 3 again shows more dynamic vertical excitation than the other spans.

A comparison of the maximum vertical displacements to span length is shown in Table 4. The span over displacement ratios are approximately L/2000. The theoretical vertical displacement of the span cannot be calculated without the use of a finite element model according to AREMA (2015), due to the dynamic nature of the loading. However, AREMA does limit the vertical displacements of steel railroad bridges to a maximum of L/640. CN limits the vertical displacement of their bridges to a stricter limit of L/750, however Halton 26.36 was not observed to experience displacements approaching either of these limits, reaching a maximum of 42.06% of the displacements allowed by CN. This indicates that the vertical displacements are not in excess of the industry standards and are not a concern.

AREMA (2015) suggests that the maximum lateral displacements of a bridge chord should not exceed 10 mm for tangent track. The maximum lateral displacement recorded of 4.33 mm is less than half of this limit. Therefore it can be concluded that the magnitudes of lateral displacement are not excessive and are not of concern.

Table 4 Comparison of the normalized vertical displacements measured with DIC between the three spans

Span	Length (m)	Maximum Vertical Displacement (mm)	Span to Displacement Ratio	AREMA Limit	CN Limit	Percent of CN Limit Reached (%)
Span 3	30.33	17.01	L/1783	L/640	L/750	42.06
Span 4	30.33	14.92	L/2033	L/640	L/750	36.89
Span 6	30.33	15.42	L/1967	L/640	L/750	38.13

However, Fig. 13 shows the lateral displacement of Span 3 at midspan compared to the lateral displacement at the piers, measured by DIC. It can be seen that the displacements of Span 3 at the South support are the same magnitude, and at times larger, than the displacements at midspan. This is unusual and could indicate an issue with lateral stiffness at the South support of the span since for a bridge with rigid supports, the lateral displacements at the supports of the span are anticipated to be small. Fig. 13 shows that for Span 3, the entire span moves laterally, although more so at one support than the other, potentially indicating that the piers are not as stiff as expected in the lateral direction. This is an unexpected displacement mechanism that could potentially be an indicator of deterioration of the lateral bracing system near the supports. Alternatively, these findings could indicate an issue with the bearing conditions of the bridge, related to the unusual steel casting bearings.

To further investigate these large lateral displacements near the South Pier, DIC measurements of the bridge girder near the pier were compared to displacements calculated from the accelerometer affixed directly to the top of the South Pier, as shown in Fig. 14. Fig. 14 indicates that the girder directly above the South Pier displaces significantly more than the pier itself. These findings indicate that the unexpected lateral displacements must originate in the steel casting bearings that support the girder above the pier. As such, it is suggested that these steel casting bearings should be the focus of any rehabilitation designed to reduce lateral displacements in this bridge based on the results of this monitoring campaign.

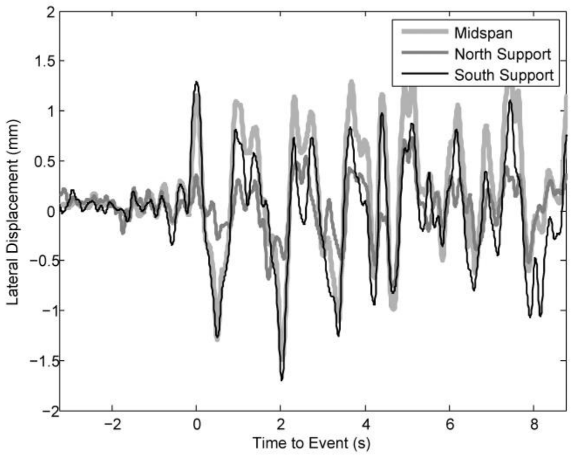

Fig. 13 Comparison of lateral displacement, measured by DIC, between midspan and the supports of Span 3 under dynamic freight train loading

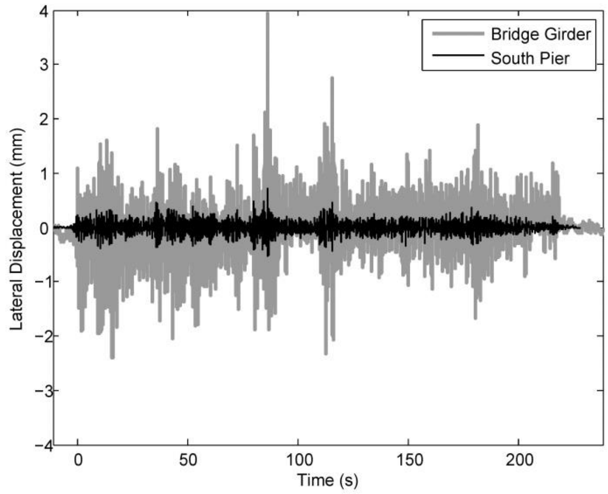

Fig. 14 Comparison of the lateral displacements of the Span 3 girder near the pier (measured by DIC), and of the South Pier (calculated from accelerometers)

Issues with lateral stiffness were identified in this bridge by using DIC measurements taken at midspan and at the supports of a span. This demonstrates the versatility of DIC as a bridge monitoring sensor, since displacement measurements can be taken at any visible part of the span. Accelerometer measurements were used to calculate the natural frequency of the bridge and in combination with DIC to determine in what part of the bridge the lateral displacements originate. This research shows that accelerometers and DIC are useful tools for evaluating railroad bridges and when used in combination, can be used for measurement validation and providing a more comprehensive evaluation of a structure.

7. Conclusions

A case study of a steel railroad bridge monitoring campaign has been presented in which DIC was used for monitoring vertical and lateral dynamic displacements at midspan and the supports of multiple spans. One span of the bridge was also monitored using accelerometers. A comparison between the displacements of three spans was presented, and it can be seen that all spans experience displacements of similar magnitude. The acceleration data has been presented and was converted to displacements through the use of a proven algorithm. For zero-mean data, such as the lateral accelerations, this process could be done for the entire data set, to estimate the lateral displacements under the whole train. The lateral displacement estimates from the accelerations were shown to be in agreeance with the DIC results, but with the occasional large erroneous peak due to equipment limitations. The calculation of vertical displacement from accelerometers showed many more errors when compared to DIC. This is likely due to the non-zero-mean nature of the vertical measurements. Also, the accelerometers are not optimized for low frequency acceleration measurement, which was the dominant behaviour in the vertical direction.

The DIC measurements of vertical displacement at midspan were compared to conventional limits and were shown not to be of concern. Using the lateral displacements measured by DIC at the midspan and at the supports of a span, it was shown that the displacements at the supports of the bridge were larger than expected, relative to the midspan displacements. By comparing measurements in the girder near the pier to measurements of the pier itself, it was shown that the pier does not exhibit the same lateral displacements as the girder. Therefore the lateral displacements must originate in the castings which support the bridge above the piers.

Based on the findings of this research, it can be concluded that DIC is an effective tool for bridge displacement monitoring. Direct measurements of displacement using DIC can be used to validate displacement estimates obtained from conventional sensors and provide measurement redundancy. The direct measurement of displacement that DIC can provide for any visible portion of the bridge is useful for identifying areas of concern with respect to stiffness and can be directly compared to serviceability limit states defined in various codes.

Accelerometer measurements can be used to calculate natural frequency. Displacements can be calculated using accelerations, however this works best for zero-mean data sets such as those in the lateral direction (although there may be cases where the lateral displacements are not zero mean in which case the same issues as seen with the vertical measurements may arise). Accelerometers can be used in combination with DIC for a more comprehensive evaluation of a bridge with measurement validation and to fully utilize the advantages of both systems.

Future work in this area will include obtaining DIC measurements of multiple bridges of different designs and span lengths. An area of particular interest is to compare total displacements of a span under a train travelling at different speeds. This could be used to inform the owners about how train speed affects the bridge response and what speed the traffic on the bridge should be limited to.

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