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Exploratory investigation of monitoring data obtained from an in-service pre-stressed concrete bridge incorporating a distinct half-joint layout

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Abstract

Measurements obtained from a structural health monitoring (SHM) system currently installed on a three-span pre-stressed concrete bridge are analysed for condition assessment in this paper. The middle span of the bridge contains eight suspended I girders which are supported on half-joints, located at the ends of cantilevered lengths of the side girders. The half-joints also contain external strengthening. The objective of the work is to conduct a preliminary monitoring data based investigation to identify whether and what indices can be derived from the data which could be used to identify the condition or changes in the condition of the bridge.

1. **Introduction**

Structural health monitoring (SHM) of bridges is an area of continuing interest to many engineers and researchers. Even though such monitoring is mostly done for large iconic bridges, SHM systems have also been installed on short to medium span bridges over the years (Seo et al., 2016). Various indices have been chosen by different researchers, and changes of these indices, which are identified based on the measured data, are used to detect any damage occurring to the bridge (Li and Hao 2016, Li et al., 2014).

In vibration-based SHM, the measured values are usually the accelerations and the indices used are mostly the natural frequencies and mode shapes (Gomez, 2011, Brownjohn et al., 2003). In strain-based SHM, the measured values are strains of the structure at various locations. Strain-based SHM is usually carried out with known loads as the measured strain is dependent on the applied loading. These measured strains can be used to obtain the ‘best-fit’ parameters of a FE model, so that the measured strains agree with the analytical values (Sanayei et al., 2012). In addition to directly using the measured strains, many researchers have also considered strain based indices such as the neutral axis position, transverse distribution factors and unit influence lines (Wang, 2005, Gangone et al., 2011). In addition to model-based approaches for damage detection referred to above, purely data-based non-model approaches have also been published in the literature. For example Christopher et al. (2014) proposed a method in which the survival distribution functions of maximum measured strains, obtained from a duration of 6 months, were used together with the bootstrapping method to define a ‘bridge signature’ which was then used as a basis for damage detection.

The study described in this paper has been carried out as part of an ongoing body of work, where the objective is to use SHM data, obtained from a pre-stressed concrete bridge with a unique half-joint arrangement, to understand the behaviour and condition of the bridge and in particular the half-joint. The focus of this work is to use the data in a meaningful manner using simple and established data analysis techniques.

2. **Instrumentation system and data measurement methodology**

The bridge that has been considered in this paper is a three-span (14m-24m-14m) pre-stressed post-tensioned concrete I girder beam and slab bridge owned by Main Roads Western Australia

(MRWA). The central span of the bridge is made up of eight 17m long suspended I-girders supported on half-joints located at the ends of 3.5m long cantilevered lengths of the side girders. The girder section is solid at the half-joint locations. The half-joints also contain external vertical steel strengthening rods at either side of the half joint as well as horizontal strengthening rods joining the two sides of the joint. The schematic arrangements of the suspended girders of the central span and the half-joint are shown in Figure 1. The half-joint arrangement is different to the typical joint arrangement as there is no bearing between the suspended and supporting nibs while the joint is also post-tensioned by an internal tendon crossing the joint.

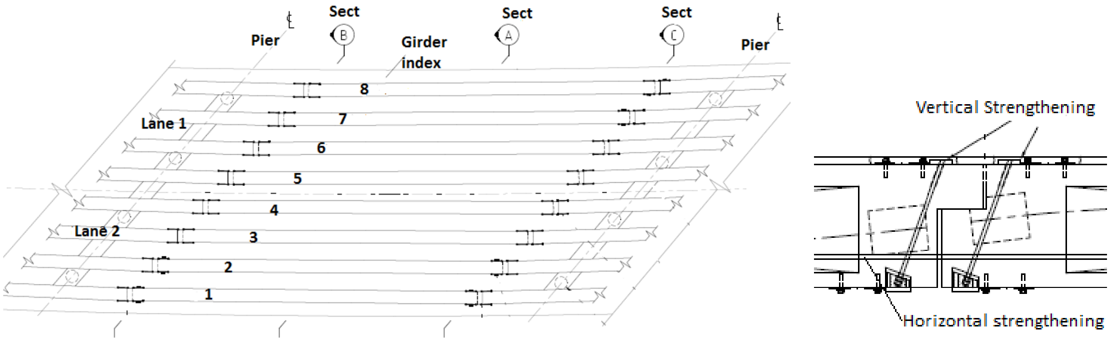

Figure 1. Schematic arrangement of half joint and suspended girders of central span
(Continuous internal pre-stressing tendon crossing the joint is not shown for clarity)

An instrumentation system was installed by MRWA on the afore-mentioned bridge in August 2014. The mid-span section of the girders (Section A in Figure 1) was instrumented with 16 strain rings (SR1-16) measuring the strain in the concrete at the lower faces of the top and bottom flanges, as shown in Figure 2. Two tri-axial accelerometers (A1, A2) were also installed on the soffits of girders 1 and 8 as shown at mid-span. In addition, at Sections B and C, as shown, eight strain gauges each (SG1-8, 9-16) were installed on the vertical strengthening rods of the half-joints (on the suspended girder side). This paper discusses the data measured through these strain rings/gauges and accelerometers. The system was setup so that when a vehicle passes over the bridge, if it causes a response above a pre-defined threshold in any of the strain rings (SR1-16), then a two-minute data window (approx. 1 min before to 1 min after the maximum) of all the instrumentation would be recorded. In addition, images taken from a fixed elevated camera showing the vehicle(s) causing the response would also be recorded. MRWA made available 123 such two-minute long ‘event data’ to the authors recorded during five different months (3, 29 and 1 event(s) in April, September and October 2015 & 40 and 50 events in February and March 2016 respectively).

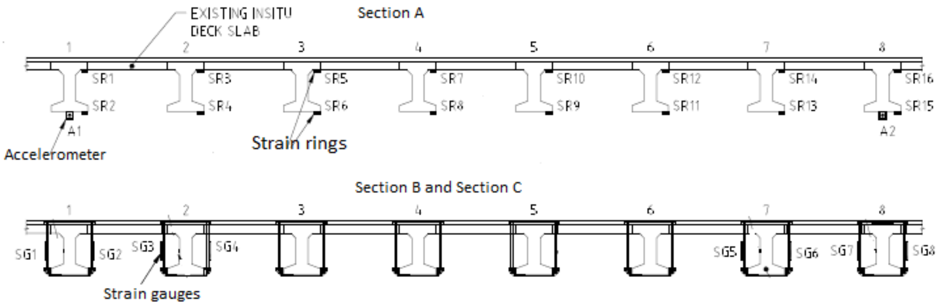

Figure 2. Locations of strain rings/gauges and accelerometers on bridge girders

These event data were recorded normalized with respect to the background measurements and hence contained responses due to the heavy vehicles alone. 18 event data were not used by the authors as those data were found to contain errors and hence this paper discusses the remaining 105 data sets. The data were recorded at a sampling frequency of 130Hz. When evaluating the measured strains, in order to extract the equivalent static response from the measured response, the measured strain signal was filtered by calculating the Fourier transform of the response, then zeroing out all components of the Fourier spectrum above a frequency of 1.25Hz and then taking the inverse Fourier transform to obtain the 'filtered' signal. The cut-off was determined by inspection of the spectrum in a similar manner to that presented by Zaurin and Catbas (2010). Figure 3 shows the unfiltered and filtered responses of the recorded strain signal of one of the sensors due to one event.

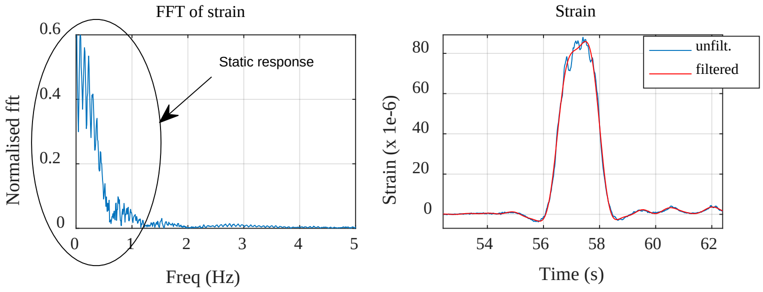

Figure 3. Filtering of measured strain signals to obtain static response

The maximum measured responses from each strain ring/gauge in sections A, B and C were considered in the evaluation. For section A, the tensile and compressive maximums were considered separately. For each event, at the instant when the maximum compressive strain was measured, the corresponding moments (and distribution factors) carried by each girder were calculated by considering the co-existing strains at that instant, assuming no-tension linear elastic behaviour for concrete and linear elastic behaviour for steel. The reinforcement and tendon arrangement were taken from the as-built drawings supplied by the bridge owner. The co-existing strains at the instant of compressive maximum were used so as to minimize any effect of measurement error. The frequency spectra of the vertical acceleration data (negligible in other directions) were obtained using the operational modal analysis (Brincker and Ventura, 2015) technique of Complex Mode Indicator Function (CMIF). In addition to these 'event data', the background response is also recorded by the system, together with environmental conditions such as temperature and wind speed, at 30min intervals. However that data were not used for the work described in this paper.

3. **Results and discussion**

Based on the measured strains at the mid-span section (Section A), the distributions of the maximum tensile and compressive strains were obtained and are shown in Figure 4. The equivalent static responses due to the passing vehicles were considered for this evaluation. As can be seen in Figure 4, which also differentiates the data by month of measurement, there is considerable scatter in the measured strains. The means and standard deviations of the data, for all the data as well as by month are listed in Table 1. When considering the statistics by month, the events of April and October 2015 were not considered due to inadequate data numbers.

Table 1. Primary statistical properties of measured maximum strains (in micro-strain)

Type	Property	All months	Month 2	Month 4	Month 5
Tensile	Mean	102	104	103	101
	Standard deviation	12.4	11.7	13.3	12.4
Compressive	Mean	-20	-20	-20	-19
	Standard deviation	3.7	3.6	4.0	3.4

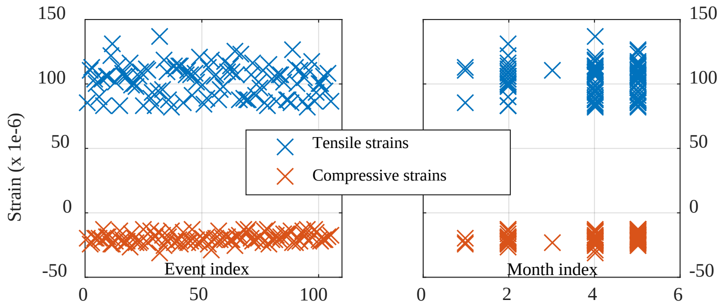

Figure 4. Distribution of maximum strains by event index and by month

There appears to be no significant difference in the primary statistical parameters of the maximum tensile and compressive strains, for the months considered above. Potentially these statistical properties and their variations with time could possibly be used as an indicator of any changes that may occur to the condition of the bridge, in a manner similar to that done by Christopher et al. (2014). However in order to be able to identify any variations, base values of these parameters would have to be established first using a much larger number of event data than the 105 events considered in this paper. Christopher et al. (2014) used strains caused by 1670 heavy vehicle events over a six-month period in order to establish a base-line survival distribution function, which formed the basis of their ‘bridge signature’ which was used for damage detection.

It should be noted that the maximum tensile and compressive strains did not necessarily always occur in the same girder. The maximum tensile and maximum compressive strains occurred either in the same girder or at girders adjacent to each other or spaced with a girder in between. If the load is primarily carried by bending one would expect the maximum compressive and tensile strains to occur in the same girder, but this apparent discrepancy could be due to differences in the location of the operative elastic neutral axis of each girder. This could be due to the effective operative cross-section, contributing to the stiffness, being different between the eight girders. This difference in effective cross section could be due to a multitude of reasons, such as the skewed nature of the bridge, differences in post-tensioning losses, localized damage occurring in some girders etc., which cannot be established purely based on the measured data. Establishing the reasons behind these differences would require the use of the data together with structural modelling and analysis.

The measured responses inherently depend on the type of the vehicle causing the response and especially on the weights and spacing of the vehicle axles. Since the vehicle type and axle distribution could be qualitatively obtained from the recorded images, it was judged that it would be more representative to consider the distribution of the maximum responses for each vehicle type separately. The distributions of the measured maximum strains by identified vehicle type are shown in Figure 5. Vehicle types for which there were only a very limited number of event data, specifically four and seven-axle mobile cranes and nine and eleven-axle double road trains, together with events for which no image data were available are not shown in Figure 5 (totalling 25 out of the 105 events).

In the absence of sufficient numbers of event data to satisfactorily define base values of the statistical properties of the maximum responses considered previously, it would potentially be possible to use vehicle-type specific maximum data as an indicator to detect any changes occurring to the bridge condition. When using the distribution of all the maximum responses as a condition indicator, the data will be sensitive to any changes in seasonal travel patterns of heavy vehicles, but vehicle type-specific distributions will not have that inherent sensitivity. For vehicle types such as mobile-cranes, which have limited variability in their axle weight and spacing configuration, considering the distribution of the maximum measured responses by vehicle type will also allow greater sensitivity in detecting potential changes. From the data given in Figure 5, it can also be observed that the maximum tensile or compressive strains did not occur in the same girder for all events. For example, it is clear that the majority of the maximum compressive strains were recorded in either girder 3 or girder 5, while most of the maximum tensile strains were recorded in girders 3 or 6.

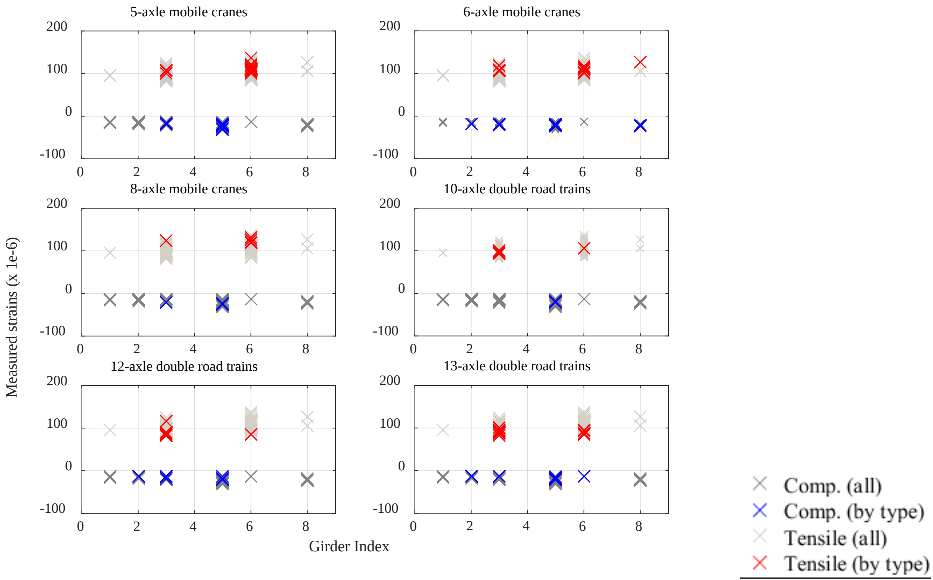

Figure 5. Maximum measured strains by vehicle type and measured girder

The strains measured at the mid-span were also used to calculate the transverse distribution of sectional moment in the girders as described in the methodology. The calculated distribution factors are shown in Figure 6. The distributions have been grouped based on the girder in which the largest measured tensile strain occurred, which is representative of the lane (or lanes) in which the vehicle (or vehicles in the case of two vehicles side by side) causing the response was (were) travelling in.

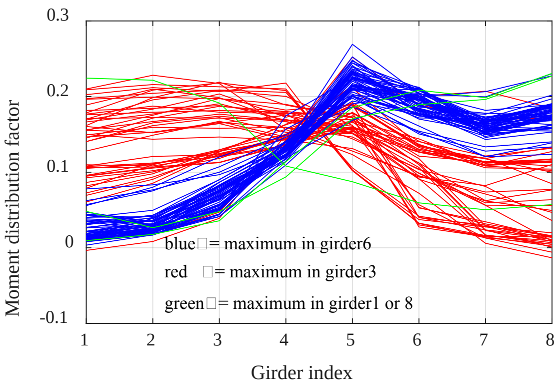

Figure 6. Transverse distribution of moment at mid-span section of suspended span

The transverse distribution of the load could also be used as an indicator of changes occurring to the bridge as has been previously considered by other researchers (Cardini and DeWolf, 2009, Gangone et al., 2011). As can be seen from Figure 6, there is a clearly identifiable general shape of the moment distribution which appears to be dependent on the lane in which the vehicles causing the response were travelling in. However as can be seen from the distributions given in Figure 6, the transverse distributions of moment show far greater variability between the different events. Even for vehicles travelling in the same lane, there is significant variations in the values of the transverse distribution factors. This suggests that the distribution of sectional moment at the mid-span section is quite sensitive to the transverse position of the vehicle within the carriageway lane and hence appears to be less viable as a change indicator.

The distributions of the largest responses measured by the strain gauges attached to the vertical strengthening rods at the half joints (SG1-16) were also considered and are shown in Figure 7. As can be seen from the magnitudes of the strains of the distributions, apart from the maximum (tensile) strains measured by gauge SG1, the measured strains are quite small. Compared with the maximum range of measurement of the strain gauges, which is a strain of 2% (20,000 micro-strain), these small strain values possibly fall within the measurement error of the gauges. Therefore it is demonstrated that it would not be possible to use the strains measured in the strengthening rods as indicators of any changes occurring to the bridge. The maximum strain measured in SG1 corresponds to a stress of 8.4MPa which is 6% of the tension capacity of the rod, which suggests that the utilization of the strengthening rods is low. This in turn suggests that the load-transfer through the half-joint is occurring without significant participation of the rods at present. However the fact that SG1 has non-negligible strains compared to the other gauges suggest that the half-joint corresponding to SG1 may be behaving differently to the other joints.

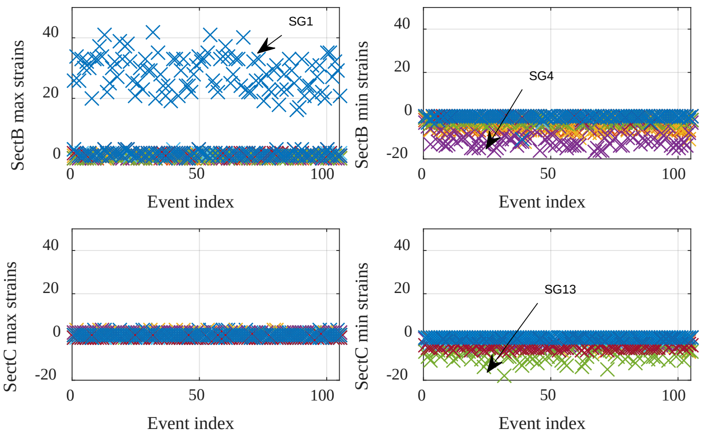

Figure 7. Distributions of maximum responses measured from gauges attached to strengthening rods (in micro-strain)

It is possible in this manner to obtain a qualitative understanding of the half-joint behaviour through the measured rod strains, though for further investigation analysis together with a structural model would be necessary. However, it can be expected that if there is some deterioration of the joint, larger strains would be measured in the gauges attached to the strengthening rods.

The frequency spectra obtained by analysing the measured vertical acceleration responses at the soffits of the edge girders at mid-span for the 105 events are shown in Figure 8. From the spectra as shown in Figure 8, it is observed that there exists a lowest natural frequency of the bridge at approximately 5.8Hz (a number of closely spaced peaks exists in this region as seen in the obtained spectrums) which is in the range of natural frequencies which are generally expected for concrete girder bridges with similar span lengths. For example Brownjohn et al. (2003) obtained through testing that the natural frequency of a 18m span pre-cast pre-tensioned inverted T-beam and slab simply supported bridge was 5.4Hz. If it is considered that the half-joint of the bridge under consideration is an effective simple support, then the suspended central girders can be considered as 17m long simply supported spans and hence the measured value of 5.8Hz compares well with the result obtained by Brownjohn et al. (2003).

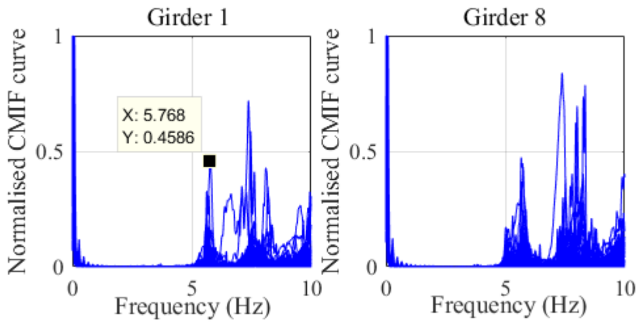

Figure 8. Frequency spectrums obtained from measured accelerations due to 105 events

The peak at approximately 5.8Hz was observed for all the events that have been considered in this paper. This suggests that there may have been no significant changes in the bridge condition, which would affect the vibration characteristics, i.e. frequencies of the bridge, during the time period corresponding to the data. Therefore, it is potentially possible to use the natural frequency of the bridge as a condition indicator, though it is not possible to deduct, using the measured data alone, what changes (and the corresponding sensitivities) in condition could be detected using the measured natural frequency From the work of previous researchers, such as that of Farrar and Doebling (1997), significant condition changes may need to occur for any noticeable changes to occur to the measured frequency spectrum. Since the condition of the half-joint is of particular interest in the bridge that has been considered in this paper, the sensitivity of the measured spectrum to the joint condition needs to be established through structural modelling of the bridge. The measured frequencies could also be used to aid in updating such a model of the bridge.

4. Limitations and Further work

It is also potentially possible to consider a control-chart based approach, as done by Brent et al. (2013) where the relationship between pairs of sensor measurements are considered to detect any anomalies corresponding to any 'changes'. However this would need a significantly more number of event data to establish the control charts than the 105 events used in this paper. A further index which could be used as a condition indicator is the unit influence line of the measurement. Influence lines and specifically changes in the shape of the influence line have been considered by a number of researchers (Wang, 2005, Catbas and Aktan, 2002, Zhu et al., 2014) for condition and damage detection of bridges, though these have mainly been for steel bridges. Influence line based methods (Chen et al., 2016) are attractive as it is dependent on the entire length of the bridge and hence can potentially detect changes which are not localized to the measurement location. However in order to derive the unit influence lines, knowledge of the axle weights and spacings of the specific vehicles causing the response as well as the respective speeds are required. However for the bridge discussed in this paper, complete information on such specifics is not available though it may be possible to use a vehicle-based influence line (which is essentially the measured pulse shaped response shown in Figure 3) as a condition indicator instead, based on the responses due to vehicle types with low variability, such as mobile cranes.

5. Conclusions

This paper has discussed the preliminary investigations carried out on how the measured strains and accelerations, of the suspended central span of a three-span pre-stressed post-tensioned concrete bridge with half-joints, could potentially be used to identify any changes occurring to the condition of the bridge. This work is the initial work of an ongoing overall effort, to use measurement data to better understand the behaviour of the half-joints of the bridge, which are of a unique arrangement. The distributions of the maximum measured girder strains, the moment distribution across the section of measurement, the measured strains in the strengthening rods and the frequency spectra obtained from the acceleration measurements were considered as condition indices. It was clear that except for the measurements obtained from the strengthening rods, the others could potentially be used as indicators to detect changes occurring to the bridge. In some cases, it was also possible to infer a qualitative understanding of the bridge behaviour using the measurements. However it was clearly ascertained that, using the measured data and corresponding data indices alone, it would not be possible to obtain the locations of any changes that would be detected nor would it be possible to obtain the sensitivity of the measured data to any changes that may occur. However, using the data together with a finite element model of the bridge is likely to provide a better physical understanding of the bridge condition and of the half-joints in particular.

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